

PHASE III DRAINAGE REPORT

FOR

DAWSON TRAILS BOULEVARD NORTH
TOWN OF CASTLE ROCK, COLORADO
CASE NO. 23-0020

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September 19, 2025

Engineer's Statement:

I affirm that this report and plan for the Phase III drainage design of Dawson Trails Boulevard North, was prepared by me (or under my direct supervision) in accordance with the provisions of the Town of Castle Rock Drainage Design and Technical Criteria for the owners thereof. I understand that the Town of Castle Rock does not and will not assume liability for drainage facilities designed by others.

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Drainage Maps

I. GENERAL LOCATION AND DESCRIPTION

A. Site Location

This drainage report supports the design of Dawson Trails Boulevard, a proposed 4-lane minor arterial roadway, between Plum Creek Parkway and the Dawson Trails planned development. The road alignment roughly parallels Interstate 25, approximately 0.2 miles west of the highway. The project spans 6000 linear feet between its northern and southern limits, with a proposed right-of-way width of 111 feet.

The initial phase of construction for Dawson Trails Boulevard will include right-of-way grading, storm drain and culvert construction, water main construction, detention ponds, and paving of the east half of the roadway. One travel lane in each direction will be accommodated within the initial pavement section. The western half of the road (southbound lanes) and median are anticipated to be constructed as a future phase. The drainage design herein plans for the full build-out roadway section, and all pipes and ponds are designed with capacity for the ultimate condition. Within the detention pond, the outlet structure has been designed for the interim condition. In addition, proposed roadway and drainage improvements that are specific to the interim phase are supported with separate drainage calculations where they vary from the ultimate condition.

The project area is contained within the eastern quarter sections of Sections 10 and 15, Township 8 South, Range 67 West of the Sixth Principal Meridian.

The project area and adjacent lands are generally unimproved and undeveloped. The north end of the project abuts Plum Creek Parkway, a four-lane arterial with Interstate highway access. The southern end of the project will connect to a separate segment of Dawson Trails Boulevard which is currently in design. The proposed road alignment intersects Yucca Hills road—an existing local unpaved road connecting multiple rural properties to the I-25 Frontage Road—and one private driveway.

The Dawson Trails planned development is directly south of the project limits. The Miller's landing development is north of the project limits on the opposite side of Plum Creek Parkway.

B. Description of Property

The right-of-way dedication for the project will encompass 15.35. Total disturbance area will be approximately 38 acres.

Ground cover in the project area is mostly prairie grass, with some small trees and shrubs lining gullies in the north part of the site. Site Topography is generally sloping west to east between 3% and 7% grade. The northern third of the site is steeper terrain with deep drainage gullies crossing west to east and slopes between 5:1 and 2:1.

Soils within the project area are detailed on the soils map in Appendix A. Most of the project area contains Hydrologic Soil Group B soils, with minor regions of A and C group soils in Basin A1.1 and A1.2 in the southern third of the project area.

The Westfield Tributary of East Plum Creek is the only drainageway with a tributary area greater than 130 acres that intersects the project site. Several minor drainages also cross the project area, conveying runoff from undeveloped, hilly terrain west of the site toward Plum Creek 0.25 miles east of the site.

The project area is contained within FIRM map panel 08035C0301G. A small area of shaded Zone X crosses into the site along the Westfield Tributary. The shaded Zone X region lies west of and upstream of the existing BNSF rail line that runs 0.2 miles east of the project site. Due to proposed grade changes in the shaded Zone X region, a Floodplain Modification Study will accompany this design (FD23-0009). A 2013 FHAD study for the Omni/Industrial/Westfield watersheds provides a detailed analysis of the Westfield Tributary's hydrology.

There are no irrigation canals within or near the project area. Some of the adjacent properties contain topographic remnants of historic private, on-site diversions, but none of these function as conveyances at present.

There are no notable geologic features within or near the site.

The sole use of the project is transportation; no other development use is proposed within the site.

II. DRAINAGE BASINS AND SUB-BASINS

A. Major Drainage Basins

The drainage design of this project is largely defined by existing topography. The proposed road alignment traverses multiple undeveloped drainage basins, and the roadway profile is designed to minimize extents of cut and fill by following existing topography. Consequently, the major onsite basins of this project are generally defined by the intersection of historic basins and the proposed road right-of-way.

On-site major basins

The project area is separated into five major basins.

Basin A1 covers the road right-of way between roadway high points at stations 67+46 and 89+97, and associated detention facilities. Basin A1 has an area of 6.30 acres and an overall imperviousness of 73.1%. Runoff from Basin A1 is detained and treated in Pond A1.

Basin A2 covers the road right-of way between roadway high points at stations 89+97 and 110+39, and associated detention facilities. Basin A2 is accounting for a future minor basin from the proposed development of Castle Meadow to the West per plans received from JR Engineering on May 27, 2025. Basin A2 has an

area of 5.74 acres and an overall imperviousness of 76.8%. Runoff from Basin A2 is detained and treated in Pond A2. See Appendix A of this report for calculations.

Basin A4 covers the road right-of way between the roadway high point at station 110+39 and the northern project limits at Plum Creek Parkway, as well as associated detention facilities. A4 also contains a significant area of earthen cut slope that drains to the roadway. Basin A4 has an area of 15.58 acres and an overall imperviousness of 18.6%. Runoff from Basin A4 is detained and treated in Pond A4.

Basin 7 covers the road right-of way between the southern project limits and the roadway high point at station 67+46. Basin 7 has an area of 1.34 acres and an overall imperviousness of 75.2%. Runoff from Basin 7 is accounted for in the concurrently designed segment of Dawson Trails Boulevard that abuts the southern project limits. The surface flows that cross the southern project limits will be intercepted in street inlets 450 feet further south and conveyed to a separately proposed detention pond (Pond F1) that will serve surrounding areas of the Dawson Trails planned development. Pond F1 has been sized to accommodate stormwater from Basin 7 that surface flows into the Dawson Trails development area from the proposed roadway. Excerpts from the Phase III Drainage Study for that project (currently in design) are included in Appendix D of this report to document the coordination of designs.

Off-site major drainage basins

Off-site drainage basins OS-8 through OS-13 consist of largely undeveloped, hilly terrain west of the project site. This area contains large-acreage rural lots with isolated structures and driveways. These off-site areas are analyzed with the rational method with imperviousness ranging between 2% and 12%.

Off-site Basin OS-8 **(368 ac; 8.5% I, Q100=877 cfs)** is the most significant of the drainage areas that affect this project. The basin consists of 368 acres of undeveloped, rural residential land and drains to the Westfield Tributary of East Plum Creek. This basin flow was calculated by the Westfield Tributary Floodplain Modification Study, Town Project #FD23-0009. A double 6' x 8' concrete box culvert is proposed to bypass OS-8 flows across the roadway. A FHAD study and master plan for the Westfield Tributary have been previously published by CH2MHill for the Town of Castle Rock. The FHAD study identifies a 100-year flow rate of 653 cfs in the reach that is traversed by Dawson Trails Boulevard. The FHAD and Master Plan documents, and their relevance to this project's drainage design, are further discussed in sections II.A and III.B of this report.

Offsite Basin OS-9 **(47.2 ac; 12% I, Q100=78.0 cfs)** contains the area that drains to the topographical low point at roadway station 83+70. A (2)-36" concrete pipe culvert is proposed to bypass OS-9 flows across the roadway.

Offsite Basin OS-10 **(10.7 ac; 12% I, Q100=19.8 cfs)** contains the area that drains to the topographical low point at roadway station 92+30. A 24" concrete pipe culvert is proposed to bypass OS-10 flows across the roadway.

Offsite Basin OS-11 **(27.2 ac; 5.2% I, Q100=45.4 cfs)** contains the area that drains to the topographical low point at roadway station 98+70. A 36" concrete pipe culvert is proposed to bypass OS-11 flows across the roadway.

Offsite Basin OS-12 **(50.1 ac; 2% I, Q100=78.8 cfs)** contains the area that drains to the topographical low point at roadway station 106+00. A 42" concrete pipe culvert is proposed to bypass OS-12 flows across the roadway.

Offsite Basin OS-13 **(20.0 ac; 2% I, Q100=40.1 cfs)** contains the area that drains to the topographical low point at roadway station 111+70. A 36" concrete pipe culvert is proposed to bypass OS-13 flows across the roadway.

The Westfield Tributary of East Plum Creek is analyzed in two previous drainageway studies:

- Omni, Industrial and Westfield Watershed Master Plan - Final Preliminary Design Report by CH2MHill, January 2011
- Omni, Industrial and Westfield Watersheds - Flood Hazard Area Delineation by CH2MHill, April 2013

The above studies identify potential flood areas, infrastructure constraints, and potential future mitigation of drainageway issues. Both plans show a conceptual alignment for a north-south roadway in the general vicinity of the Dawson Trails Boulevard crossing. However, neither of the plans integrate their analyses or conceptual improvements with a road crossing. The roadway improvements and box culvert crossing detailed in this report are designed to be independent from but compatible with any separate drainageway improvements. The box culvert design seeks to avoid adverse impacts to existing properties and to accommodate potential future drainageway improvements as outlined in the studies.

There are no formal drainage channels or any drainage paths with a base flow on or adjacent to the project area.

The northern and southern thirds of the proposed road alignment traverse existing zoned and incorporated properties. It is anticipated that these areas, if developed, will generally maintain existing drainage patterns, with detention and water quality treatment as necessary to stay compatible with surrounding drainage infrastructure. The properties abutting the middle third of the roadway are rural and unincorporated, with no current development plans.

B. Minor Drainage Basins

On-site sub-basins

On-site Basins A1, A2, and A4 contain all areas within the road right-of-way and any adjacent, directly tributary cut slopes. These are further delineated into sub-basins for each runoff interception point within the project site.

Sub-basins **A1.1 (2.85 ac; 71.0% I, Q100=12.8 cfs)** and **A1.2 (2.88 ac; 79.2% I, Q100=13.9 cfs)** contain the watershed between roadway high points at stations

67+46 and 89+97. A1.1 and A1.2 drain via street gutter to curb inlets A1.1 and A1.2, respectively, and are ultimately captured in Pond A1.

Sub-basin **A1.3 (0.57 ac; 53.3% I, Q100=3.4 cfs)** includes areas directly tributary to Pond A1 via surface flow. The 100-year water surface area is treated as impervious.

Sub-basins **A2.1 (0.91 ac; 79.7% I, Q100=6.2 cfs)** and **A2.2 (0.91 ac; 79.7% I, Q100=6.2 cfs)** contain the watershed between roadway high points at stations 89+97 and 103+26. A2.1 and A2.2 drain via street gutter to curb inlets A2.1 and A2.2, respectively, and are ultimately captured in Pond A2.

Sub-basins **A2.3 (1.69 ac; 79.7% I, Q100=10.1 cfs)** and **A2.4 (1.70 ac; 79.6% I, Q100=10.2 cfs)** contain the watershed between roadway high points at stations 103+26 and 110+39. A2.3 and A2.4 drain via street gutter to curb inlets A2.3 and A2.4, respectively, and are ultimately captured in Pond A2.

Sub-basin **A2.5 (0.44 ac; 50.8% I, Q100=2.6 cfs)** includes areas directly tributary to Pond A2 via surface flow. The 100-year water surface area is treated as impervious.

Sub-basin **A2.6 (2.37 ac; 63.0% I)** includes areas directly tributary to Pond A2 via storm sewer. A2.6 will be a future basin from the future development to the west. This basin has been included in the Pond A2 calculations.

Sub-basin **A4.1 (6.14 ac; 21.6% I, Q100=21.4 cfs)** contains the west half of the right-of-way between the roadway high point at station 110+39 to the northern project limits at Plum Creek Parkway, in addition to the cut slope immediately west of the road. As Dawson Trails Boulevard approaches Plum Creek Parkway from the south, the transition from a normal 2% crown to a cross-slope at the intersection will reduce conveyance capacity in the future west half street gutter. Inlet A4.1 is anticipated as a future sump inlet at road station 121+40 to intercept gutter flows. The storm sewer originating from the inlet location is sized to convey full capture of gutter flows. In the interim condition, before the west curb line is built out, a type C inlet is proposed to intercept flows from the unpaved western half of the right-of-way and the adjacent cut slope.

Sub-basin **A4.2 (3.82 ac; 38.0% I, Q100=16.5 cfs)** contains the eastern half of the right-of-way between the roadway high point at stations 110+39 to the northern project limits at Plum Creek Parkway, in addition to the cut slope immediately east of the road. Sub-basin A4.2 drains via street gutter to a sump curb inlet A4.2.

A small storm sewer network collects flows from Inlets A4.1 and A4.2 for conveyance to Pond A4. All pipes of the storm sewer network are design based on the hydrology of the ultimate road section.

Sub-basin **A4.3 (5.63 ac; 2.0% I, Q100=14.6 cfs)** includes areas directly tributary to Pond A4 via surface flow. The 100-year water surface area is treated as impervious.

Sub-basins **7.1 (0.69 ac; 69.5% I, Q100=4.1 cfs)** and **7.2 (0.65 ac; 81.3% I, Q100=4.2 cfs)** contain the watershed between roadway high points at stations 102+25 and 109+62. 71 and 72 drain via street gutter across the south project limits to curb inlets on a separately-designed road segment, respectively, and are ultimately captured in separately-designed Pond F1. The imperviousness, area, and runoff from sub-basins 7.1 and 7.2 are appropriately accounted for in the ongoing design of the adjoining segment of Dawson Trails Boulevard and Pond F1.

Sub-basins A1.2, A2.2, A2.4, A4.2, and A7.2 contain the eastern half of the arterial section and will have roadway and sidewalk paving in first phase of construction.

Sub-basins A4.1 will also have significant paving area in the western half of the right-of-way installed during the initial phase of construction.

Sub-basins A1.1, A2.1, A2.3, and 7.1 are expected to remain unpaved after the initial phase of road construction but are analyzed per the ultimate condition of a fully paved arterial roadway section.

Off-site sub-basins

10 Offsite sub-basins are delineated within their parent basins to quantify flow in different segments of the roadside swale, where natural flow paths are interrupted by the road alignment. These areas consist of cut slopes from road grading and offsite undeveloped areas above those cut slopes.

Off-site sub-basin **OS-7N (10.7 ac; 2.0% I, Q100=24.4 cfs)** consists of areas tributary to the western roadside swale between road stations 62+50 and 67+48. These flows are conveyed south in the roadside swale to Culvert 7N.

Off-site sub-basin **OS-8S (5.7 ac; 8.5% I, Q100=11.8 cfs)** consists of areas tributary to the western roadside swale between road stations 67+48 and 79+00. These flows are conveyed north in the roadside swale to Culvert 8 (14x6 Box).

Off-site sub-basin **OS-9S (10.2 ac; 12.0% I, Q100=19.2 cfs)** consists of areas tributary to the western roadside swale between road stations 80+30 and 83+70. These flows are conveyed north in the roadside swale to Culvert 9. Due to complex topography and drainage paths in Basin OS-9, sub-basin OS-9S and OS-9N extends a considerable distance west up the hillside from the road.

Off-site sub-basin **OS-9N (21.0 ac; 12.0% I, Q100=34.6 cfs)** consists of areas tributary to the western roadside swale between road stations 83+70 and 89+97. These flows are conveyed south in the roadside swale to Culvert 9.

Off-site sub-basin **OS-10S (0.5 ac; 12.0% I, Q100=1.6 cfs)** consists of areas tributary to the western roadside swale between road stations 89+97 and 92+50. These flows are conveyed north in the roadside swale to Culvert 10.

Off-site sub-basin **OS-11S (3.4 ac; 5.2% I, Q100=6.6 cfs)** consists of areas tributary to the western roadside swale between road stations 92+50 and 98+75. These flows are conveyed north in the roadside swale to Culvert 11.

Off-site sub-basin **OS-11N (5.0 ac; 5.2% I, Q100=10.6 cfs)** consists of areas tributary to the western roadside swale between road stations 98+75 and 103+25. These flows are conveyed south in the roadside swale to Culvert 11.

Off-site sub-basin **OS-12S (0.3 ac; 2.0% I, Q100=0.8 cfs)** consists of areas tributary to the western roadside swale between road stations 103+25 and 104+50. These flows are conveyed north in the roadside swale to Culvert 12.

Off-site sub-basin **OS-12N (0.6 ac; 2.0% I, Q100=1.5 cfs)** consists of areas tributary to the western roadside swale between road stations 110+20 and 106+75. These flows are conveyed south in the roadside swale to Culvert 12.

Off-site sub-basin **OS-13N (0.3 ac; 2.0% I, Q100=0.7 cfs)** consists of areas tributary to the western roadside swale between road stations 110+60 and 111+40. These flows are conveyed south in the roadside swale to Culvert 13.

No irrigation facilities will be impacted by the project.

Concentrated flow paths upstream and downstream of the project limits will see zero to minimal impact from the project. Flows from natural drainage paths will traverse the disturbance area through culverts and will be returned to historic drainage swales at non-erosive velocities. Offsite overland flows that directly tributary to the project will be collected in roadside swales and conveyed to culverts crossing where they will return to their ultimate historic flow paths.

III. DRAINAGE DESIGN CRITERIA

A. Regulations

The proposed stormwater facilities are designed to be complaint with Town of Castle Rock drainage criteria, specifically: design rainfall, allowable street conveyance, allowable inlet ponding, pipe velocities, surface flow velocity, culvert headwater, pond storage, release rates and freeboard.

B. Drainage Studies, Master Plans, Site Constraints

A concurrent drainage plan for the Dawson Trails planned development references the project area on its periphery and analyzes some of the off-site runoff that it tributary to this project site. However, there are no specific drainage improvements from that plan which will be implemented within this project nor affect water quality or detention in this project.

The southern portion of the project area overlaps the 2011 Omni, Industrial and Westfield Watershed Master Plan. While this project focuses on roadway construction, and not development of adjacent properties, the design does consider the conceptual improvements from the plan so as not to preclude their future implementation if the surrounding areas do develop.

The next, adjoining segment of Dawson Trails Boulevard to the south is being concurrently designed and its abutting drainage system is being coordinated with this project. That project's assigned reference number is: CD23-0056. There

are three specific drainage sub-basins that directly interact with the design to the south—OS-7.1, OS-7.2 and OS-7N. These drainage areas and their associated facilities are discussed in detail in sections II.B and IV.A of this report.

The proposed roadway and box culvert crossing at Westfield Tributary take into consideration the existing drainage concerns identified in the Omni/Industrial/Westfield Master Plan. The proposed box culvert and inlet/outlet transitions are designed to avoid and/or mitigate any adverse impacts to the existing conditions of the drainageway. This includes consideration of upstream backwater impacts and downstream flow properties. Because the drainageway crossing is surrounded by rural residential lots and is outside of the town's incorporation limits, the proposed infrastructure to support roadway construction is limited as closely as possible to the right-of-way dedication. The design provisions of the box culvert crossing are intended to not preclude future drainageway improvements as outlined in the master plan.

The stormwater management features of Basins A1 and A2 are largely unconstrained by topography and existing infrastructure. Pond A4 has topographic constraints posed by Plum Creek Parkway and the large cut slope next to the pond. These constraints require about 700 feet of piping to daylight the outfall in a suitable release location.

C. Hydrology

Rational method is used for on-site runoff calculations and for tributary cut slopes. On-site watersheds range from 0.9 to 9 acres in size. The rational method is also used for analysis of off-site runoff and for sizing of by-pass flow infrastructure. Off-site watershed areas range from 12 to 275 acres.

The 5-year mean recurrence interval is used for the minor drainage system design. The 100-year mean recurrence interval is used for the major drainage system design.

Design 1-hour rainfall amounts are 1.43 in. for the 5-year event and 2.60 in. for the 100-year event.

Full spectrum detention basins are designed to accommodate a total storage of 100-year detention volume + $\frac{1}{2}$ water quality capture volume. The MHFD-Detention_v4-05 spreadsheet is used for these pond storage and outflow calculations.

D. Hydraulics

Culvert performance is analyzed using HY-8 software. Inlet condition model a groove end of pipe with headwall for the pipe culverts and 45-degree wingwalls for the box culvert. Culverts are designed to pass the major storm flow with a headwater to pipe height ratio of 1.2 or less and with internal flow velocities less than 15 feet per second. The culvert performance properties during the major storm are also used as design parameters for low-tailwater basins and riprap aprons at the culvert outlets, as appropriate.

Hydraulic grade lines for on-site, detained storm sewers and pond outfall pipes are computed through StormCAD software, version 10.03.04.53. Headloss coefficients are applied in accordance with the technical publication "Modeling Hydraulic and Energy Gradients in Storm Sewers: A Comparison of Computational Methods" by UDFCD. Hydraulic grade lines within off-site flow bypass culverts are computed through HY-8 software version 7.80.0.2.

Open channel flow depths for roadside swales are computed with Hydraflow Express Extensions for Autodesk Civil 3D software, version 2026.

The MHFD-Detention_v4-05 spreadsheet is used for pond routing and outflow calculations.

E. Storage and Water Quality Enhancement

All stormwater detention is provided in full-spectrum on-site facilities. There are no existing or proposed sub-regional or regional detention ponds within the affected drainage basins. On-site full spectrum detention ponds are designed to drain WQCV and EURV capture volumes in 40 and 72 hours, respectively. Attenuated flows from the major storm are designed to be released at 0.9 of the historic major storm flow rates.

Adherence to the four-step process for water quality control is detailed in Section IV.C of this report.

IV. STORMWATER MANAGEMENT FACILITY DESIGN

A. Stormwater Conveyance Facilities

The drainage conveyance concept consists of two primary strategies:

1) conveyance of onsite flows from the road right-of-way and directly tributary cut slopes in curb/gutter to proposed detention ponds for peak flow attenuation.

2) bypass of off-site flows through culverts under the roadway at historic drainage path locations

Proposed extended detention basins A1 and A2 capture flows at design low points that are generally consistent with existing drainage paths. The outfall points for these ponds are adjacent to broad, moderate-grade, vegetated swales that serve as historic flows paths. The outfall for Pond A4 requires 700 feet of piping to daylight to an appropriate location in the historic drainage. Low trailwater basins for energy dissipation are provided at pipe outfalls from all four ponds, set 1 foot vertically above the existing swale flowline.

All bypass culvert and Pond Outfalls release to their historic drainage paths of their respective tributary areas. The natural flow paths toward the East Plum Creek major drainageway are interrupted by the BNSF railroad grade. Four historic flow paths across the railway are identified on the drainage plan as culverts E1 through E4. Since flows from all outfalls from the site will be equal to historic flowrates (for bypass culverts), or reduced to 0.9 of historic rates (for pond

releases), there will be no adverse impacts from additional flows with respect to historic conditions.

This project does not propose any primary drainage basin conveyance through open channels. Roadside swales are, however, proposed on the west side of Dawson Trails Boulevard to capture incidental runoff that is interrupted by the proposed roadway grading, which would otherwise surface drain to existing vegetated broad swales. The roadside swales are designed with a 4-foot bottom width, 2-foot depth and 5:1 side slopes. Swale flowline grades match road centerline profile and do not exceed 3%. All roadside swales can convey major storm runoff with depths under 12 in. and flow velocities less than 5 feet per second. Swales are designed with at least one foot of freeboard. Swales will be seeded and protection with erosion control blanket after grading to encourage vegetation growth. A grass lining is sufficient for most swale sections outside of culvert entrance areas. A riprap lining is required for swales OS-8S, OS-9N, and OS-11, and also at transitions to the entrances of Culverts 8, 9, 10 and 11. Swale stability assessment is based on Figure 8-22 of the MHFD Criteria Manual.

Dawson Trails Boulevard is designed as a 4-lane major arterial, which is a Type C drainage classification. Minor storm street capacities are controlled by the 6 inches of depth limitation at face of curb. Major storm flows for A1 and A2 basins are controlled by a maximum gutter flow depth of 10.9 inches before spillover at the right-of-way boundary. Gutter flow depths of up to 12 inches can be attained in Basin A4 due to flow confinement by cut slopes beyond the right-of-way.

Curb inlets A1.2, A2.2, A2.4 and A4.2 capture runoff from the eastern half of Dawson Trails Boulevard, which will be constructed in the initial phase.

Inlets A1.1, A2.1, A2.3, and A4.1 capture runoff from the western half of the roadway, which may be built out in a future phase. Inlet calculations for the interim configuration of these inlets is included in the report calculations to demonstrate that the drainage design will adequately accommodate the interim roadway buildout. Type C area inlets are proposed in the interim condition to collect runoff from the western, unpaved half of the right-of way. 1 foot depressions are provide around all Type C area inlets. All storm sewer are sized for the larger ultimate condition flows. The ultimate conditions flows are also reflected in the hydraulic grade line computations.

All three detention facilities proposed with this project will have vehicular maintenance access to the structural features in the pond bottoms. Since all pond tracts and adjacent to the proposed roadway, maintenance paths will be directly accessible from the road right-of-way.

The outfalls of Ponds A1, A2, and A4 release into existing flow paths at 0.9 of the historic rate. There is no need for improvements to the existing vegetated broad swales to effectively convey flows to the Plum Creek major drainageway 0.25 miles east.

There are no development sites within or adjacent to the project limits. Drainage patterns within existing rural lots will be largely maintained per historic precedent.

B. Stormwater Storage Facilities

Ponds A1 and A2 are located east of the proposed roadway and outfall to their existing drainage paths immediately downstream from where the roadway crosses the historic basins. Specifically, Pond A1 outfalls just below Culvert 9 and Pond A2 outfalls into culvert 11. These outfall locations have well-defined stream topography; therefore, rip rap has been provided for the outfalls of Ponds A1 and A2. Pond A4 is located in a significant hillside cut about 860 feet from the intersection of Dawson Trails Boulevard and Plum Creek Parkway, and is not directly adjacent to its historic receiving flowpath. The roadway low points and pond are also at a lower elevation than the historic basin due to the deep roadway cut. Consequently, the release flows from Pond A4 are piped 250 feet southeast to the nearest logical location and elevation where the flows can be discharged to their historic basin.

Emergency Spillways for Ponds A1, A2, and A4 drain to generally the same areas as the pond outfall pipes.

All ponds have been designed to release 97% of runoff from a 5-year or lesser storm within 72 hours, and to release 99% of runoff from a storm greater than a 5-year event within 120 hours.

Maintenance accommodations are designed for all three detention ponds constructed with this project. These accommodations include vehicle access paths to the ponds' forebays and outlet structures, compliant with Town criteria. Drainage easements will be dedicated over all drainage culverts, ponds, pond outfall pipes, and pipe outlets where they extend beyond the right-of-way.

C. Water Quality Enhancement Control Measures

Implementation of the four-step process of water quality control include the following features:

- 1) Detached walks disconnect 25% of the projects impervious area from the conveyance system with a landscape strip.
- 2) Full spectrum detention facilities are provided for all runoff within the proposed road right-of-way. Flow attenuation will be provided for major storm flows—with a maximum release rate 0.9 of historic flow—and for lesser magnitude, higher frequency events with and EURV capture volume. Additionally stormwater from regularly occurring rain events will be captured and slowly released over a 40-hour drain time to promote settling out of suspended pollutants.
- 3) All runoff flows—including both site-generated, attenuated flows and off-site bypassed flows—will have energy-dissipation features to accommodate release to historic flow paths at non-erosive velocities. Additionally, roadside swales for surface conveyance of adjacent off-site flows are designed for stability with a grass lining.

- 4) The proposed transportation facilities are multi-modal, with a 10' detached walk and a 4' dedicated lane for cycles. The transportation alternatives accommodated by the design encourage a reduction in motor vehicles that may introduce pollutants. Streets will be swept by the Town of Castle Rock as recommended by MHFD's best management practices for source controls.

Where practical, the flow paths through extended detention basins have been maximized to promote longer conveyance and infiltration times. Ponds A2 and A4 have a flow path that traverse most of the ponds' length between forebay and outlet structure.

The low-flow routing through Pond A1 is dictated by the pond's inflow pipe and outlet structure locations, which are both in the southern half of the pond. Since the roadway low point inlets and the pond's outfall location generally coincide with the existing drainage path near station 84+50, there is limited flexibility to attain separation between inlet and outlet. Additionally, the topographical constraints of the road low point and the outfall locations limit how much length can be added to the overall system's routing.

Since water quality treatment is integrated into the three extended detention basins of this project, the conveyance from outfall to major drainageway follows is the same as described in Section IV.B.

A water quality exclusion will be applied to portion of the offsite swale basins west of the roadway under Section 13.1.5.7, as they contain land disturbance to undeveloped land that will remain undeveloped. These basins include OS-7N, OS-8S, OS-9S, OS-9N, OS-10S, OS-11S, OS-11N, OS-12N, OS-12S, and OS-13S, for a total of +/- 9.0 acres. These offsite basins include swales that capture the existing offsite flows, and convey the flows through culverts under Dawson Trails Blvd, consistent with historic flow patterns. Land will be disturbed in these basins for the purpose of tying the roadway back into existing ground, with no proposed development within the basins excepting the swales.

D. Floodplain Modification

The proposed roadway traverses a segment of Shaded Zone X area on FIRM Panel 08035C0301G. The road's embankment will alter some of the topography within the shaded region.

The floodplain boundary information shown on the drainage map was obtained electronically through the FEMA Map Service Center and overlaid onto the plan using state plane coordinates. The digital flood zone limits for the 2016 FIRM revision are consistent with the 2013 FHAD study of the Westfield Tributary.

CORE Consultants, Inc. (CORE) has been hired by Westside Property Investment Company to complete a Floodplain Modification Study (FMS) for the Westfield Tributary in coordination with the proposed construction of Dawson Trails Blvd (part of the Prairie Hawk North Development). The modifications are within the

existing Town of Castle Rock (TOCR) regulated 100-year floodplain and Zone X of the Federal Emergency Management (FEMA) Flood Insurance Rate Map #08035C0301G. The Town project number for the Floodplain Modification Study is FD23-0009.

E. Geotechnical Investigation and Sub-surface Collection System Design

There are no significant sub-surface elements of that would expect impacts from the groundwater table. Considerations for groundwater behind wingwalls will be made as appropriate.

F. Operation and Maintenance

The proposed roadway improvements and associated drainage facilities will be public infrastructure that will be owned and maintained by the Town of Castle Rock.

Recommended maintenance practices for stormwater management facilities in the Town of Castle Rock are found in Section 4.6 of the Storm Drainage Design and Technical Criteria Manual, and further detailed in the Stormwater Management Facility Operation and Maintenance (O&M) General Guidelines.

Maintenance activities can be split into three categories:

1. Routine Work: activities that will be performed numerous times during the year, and may be completed without any prior notification to the Town.
 - a. Mowing, and trash/debris pickups during the growing season
 - b. Removal of material that may be clogging the outlet structure, well screens, and trash racks.
 - c. Weed control, mosquito treatment and algae treatment.
 - d. Regular maintenance of the stormwater facilities will include: Inspection and debris removal (twice annually), aeration (annually), and sediment removal from forebay and trickle channel (annually). All other work will be completed as needed, to be determined by inspection.
2. Minor Work: this work consists of a variety of isolated or small-scale maintenance and operational problems. Most of this work can be completed by a small crew, hand tools, and small equipment. These items require notification, at least seven days prior to the start of work to the Town's Stormwater staff. Occasional or as-needed maintenance activities will include:
 - a. Sediment removal from basin bottom—when sediment volume reaches 20% WQCV (~15 to 25 years)
 - b. Repair of eroded areas—as determined by inspection
3. Major Work: this work consists of larger maintenance/operational problems and failures within the stormwater management facilities. All of this work requires notification, at least 14 days prior to the start of work, to

the Town's Stormwater staff to ensure the proper maintenance is performed. This work requires that the engineering staff review the original design and construction drawings to assess the situation and assign the necessary maintenance. This work may also require more specialized maintenance equipment, design/details, surveying, or assistance through private contractors and consultants.

G. Additional Permitting Requirements

Aside from a potential floodplain development permit as described in Section IV.D, there are no attributes of this project, associated disturbance areas, or impacted drainages that are expected to trigger additional permitting requirements.

H. General

The following published tables and charts have been used in the drainage design and are referenced in the Appendices:

- a. *Castle Rock Storm Drainage Design and Technical Criteria*, Figure 7-7, Town of Castle Rock Street Capacity Chart: This chart is used to verify that street flows for all basins are within allowable limits.
- b. *Urban Storm Drainage Criteria Manual*, Volume 2, Figure 8-22, Swale Stability Chart: This chart is used to determine appropriate grass or riprap lining for roadside swales, based on flow and grade.
- c. *Castle Rock Storm Drainage Design and Technical Criteria*, Figure 13-7, Embankment Protection Details and Rock Sizing Chart: This chart is used to determine appropriate riprap protection on pond embankments below spillways.

V. CONCLUSIONS

A. Compliance with Standards

The drainage infrastructure that supports this segment of Dawson Trails Boulevard has been designed to adhere to Town of Castle Rock criteria and standards.

Stormwater conveyance and storage elements have been coordinated with currently proposed adjacent infrastructure projects in accordance with the overall vision of the Dawson Trail Master Drainage Plan. This drainage plan also acknowledges the previous analyses and conceptual improvements of the Westside Tributary, and will not preclude future implementation of that plan.

The project area and associated drainage areas are not within the Cherry Creek Reservoir Watershed.

The project area and associated drainage areas do lie within the Chatfield Reservoir Watershed. To date, the project has not been subjected to specific requirements regarding Regulation No. 73.

B. Variances

A variance is requested from Section 13.3.11 of the Town of Castle Rock Storm Drainage Design and Technical Criteria Manual. This section states, "Orifice sizing shall be a minimum of 2.5 inch diameter or 2-inch square. Smaller diameters will only be approved by variance." Increasing the size of the orifices on this site to 2.5-inch in diameter would reduce drain times in the WQCV event to under 10-hours which is significantly less than the minimum drain time for the WQCV event of 40-hours. Orifices will be sized to meet the drain time requirements shown in the SDDTCM. Effort will be made to have fewer orifice holes that are larger in size.

C. Drainage Concept

This drainage design detailed in this report effectively implements the stormwater management principles described below:

- a. Conveyance, capture and detention of on-site runoff
- b. Flow attenuation through full spectrum detention basin, including water quality treatment, and release to historic drainage basins.
- c. Effective conveyance of pass-through flows from off-site drainage areas along historic flow paths.

VI. REFERENCES

- A. Town of Castle Rock Storm Drainage Design and Technical Criteria Manual, February 2025.
- B. Mile High Flood District Urban Storm Drainage Criteria Manual, March 2024.
- C. Web Soil Survey, Soil Survey Staff (Natural Resources Conservation Service), United States Department of Agriculture. Available online at the following link:
<https://websoilsurvey.sc.egov.usda.gov/>. Accessed February 16, 2023.
- D. Flood Insurance Rate Map (FIRM) No. 08035C0301G, Federal Emergency Management Agency, Revised March 16, 2016. Available online at the following link:
<https://msc.fema.gov/portal/home>. Accessed March 14, 2023.
- E. Omni, Industrial and Westfield Watersheds - Flood Hazard Area Delineation by CH2MHill, April 2013.
- F. Omni, Industrial and Westfield Watershed Master Plan - Final Preliminary Design Report by CH2MHill, January 2011.
- G. Modeling Hydraulic and Energy Gradients in Storm Sewers: A Comparison of Computational Methods by AMEC, October 2009

Computer Modeling Programs:

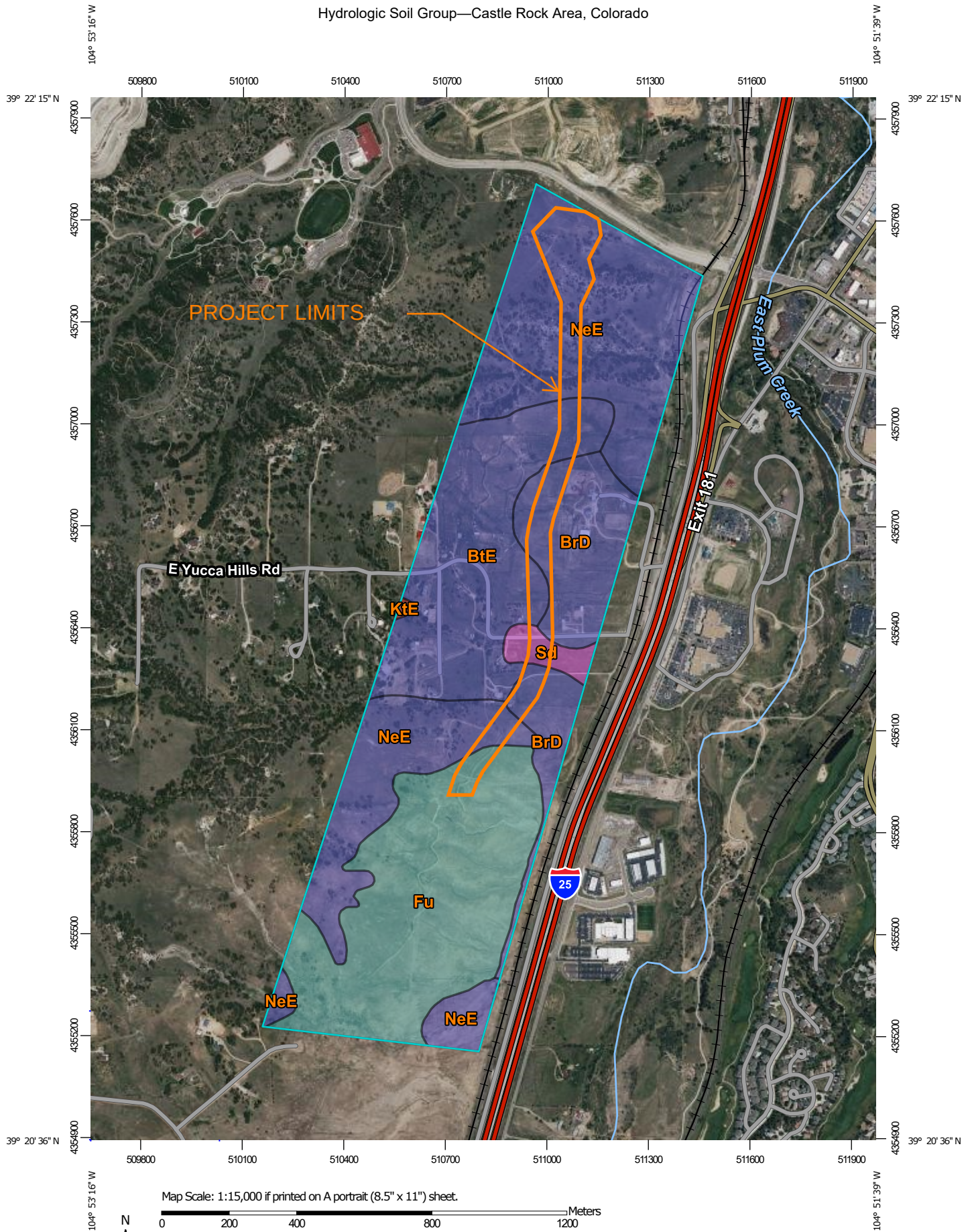
- A. StormCAD by Bentley Systems, Inc., Version 10.03.04.53.
- B. Hydraflow Express Extension for Autodesk Civil 3D by Autodesk, Version 2026.
- C. Detention Basin Design Workbook (MHFD-Detention v4.05.xlsm) by Mile High Flood District, Version 4.05, January 2022.
- D. Street and Inlet Hydraulics Workbook (MHFD-Detention v5.02.xlsm) by Mile High Flood District, Version 5.02, August 2022.
- E. HY-8 by Federal Highway Administration, Version 8.0.1.2, March 2025.

VII. APPENDICES

APPENDIX A

HYDROLOGIC COMPUTATIONS

Hydrologic Soil Group—Castle Rock Area, Colorado



MAP LEGEND

Area of Interest (AOI)

 Area of Interest (AOI)

Soils

Soil Rating Polygons

 A
 A/D
 B
 B/D
 C
 C/D
 D
 Not rated or not available

Soil Rating Lines

 A
 A/D
 B
 B/D
 C
 C/D
 D
 Not rated or not available

Soil Rating Points

 A
 A/D
 B
 B/D

 C
 C/D
 D
 Not rated or not available

Water Features

 Streams and Canals

Transportation

 Rails
 Interstate Highways
 US Routes
 Major Roads
 Local Roads

Background

 Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service

Web Soil Survey URL:

Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

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

Soil Survey Area: Castle Rock Area, Colorado

Survey Area Data: Version 15, Sep 1, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jun 9, 2021—Jun 12, 2021

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

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
BrD	Bresser sandy loam, cool, 5 to 9 percent slopes	B	48.2	13.2%
BtE	Bresser-Truckton sandy loams, 5 to 25 percent slopes	B	76.7	21.0%
Fu	Fondis-Kutch association	C	99.9	27.3%
KtE	Kutch sandy loam, 5 to 20 percent slopes	D	0.5	0.1%
NeE	Newlin gravelly sandy loam, 8 to 30 percent slopes	B	133.8	36.6%
Sd	Sandy alluvial land	A	6.2	1.7%
Totals for Area of Interest			365.3	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: Higher

NOTES TO USERS

This map is for use in administering the National Flood Insurance Program. It does not necessarily identify all areas subject to flooding, particularly from local drainage sources of small size. The community map repository should be consulted for possible updated or additional flood hazard information.

To obtain more detailed information in areas where **Base Flood Elevations (BFEs)** and/or **footprints** have been determined, users are encouraged to consult the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables shown on the FIRM. Users should be aware that BFEs shown on the FIRM represent rounded whole-foot elevations. These BFEs are intended for flood insurance rating purposes only and should not be used as the sole source of flood elevation information. Accordingly, flood elevation data presented in the Flood Profiles and Floodway Data and/or Summary of Stillwater Elevations tables should be utilized in conjunction with the FIRM for purposes of construction and/or floodplain management.

Boundaries of the floodways were computed at cross sections and interpolated between cross sections. The floodways were based on hydraulic considerations with regard to requirements of the National Flood Insurance Program. Floodway widths and other pertinent floodway data are provided in the Floodway Data table shown on the FIRM.

The **projection** used in the preparation of this map was Universal Transverse Mercator (UTM) zone 13. The **horizontal datum** was NAD 83, GRS 1980 spheroid. Differences in datum, spheroid, projection or UTM zones used in the production of FIRMs for adjacent jurisdictions may result in slight positional differences in map features across jurisdiction boundaries. These differences do not affect the accuracy of this FIRM.

Flood elevations on this map are referenced to the North American Vertical Datum of 1988. These flood elevations must be compared to structure and ground elevations referenced to the same vertical datum. For information regarding conversion between the National Geodetic Vertical Datum of 1929 and the North American Vertical Datum of 1988, visit the National Geodetic Survey website at <http://www.ngs.noaa.gov> or contact the National Geodetic Survey at the following address:

NGS Information Services
NOAA, NNGS12
National Geodetic Survey
SSMC-3, #9202
1315 East-West Highway
Silver Spring, Maryland 20910-3282
(301) 713-3242

To obtain current elevation, description, and/or location information for **bench marks** shown on this map, please contact the Information Services Branch of the National Geodetic Survey at (301) 713-3242, or visit its website at <http://www.ngs.noaa.gov>.

Base map information shown on this FIRM was provided by the Douglas County GIS Department and the Town of Castle Rock GIS Department. Additional input was provided by the City of Lone Tree and Town of Parker. These data are current as of 2010.

Certain areas not in Special Flood Hazard Areas may be protected by **flood control structures**. Refer to Section 2.4 "Flood Protection Measures" of the Flood Insurance Study report for information on flood control structures for the jurisdiction.

The **profile baselines** depicted on this map represent the hydraulic modeling baselines that match the flood profiles in the FIS report. As a result of improved topographic data, the profile baseline, in some cases, may deviate significantly from the channel centerline or appear outside the SFHA.

Based on updated topographic information this map reflects more detailed and up-to-date stream channel configurations and floodplain delineations than those shown on the previous FIRM for this jurisdiction. As a result, the Flood Profiles and Floodway Data tables for multiple streams in the Flood Insurance Study Report (which contains authoritative hydraulic data) may reflect stream channel distances that differ from what is shown on the map. Also, the road to floodplain relationships for unreviewed streams may differ from what is shown on previous maps.

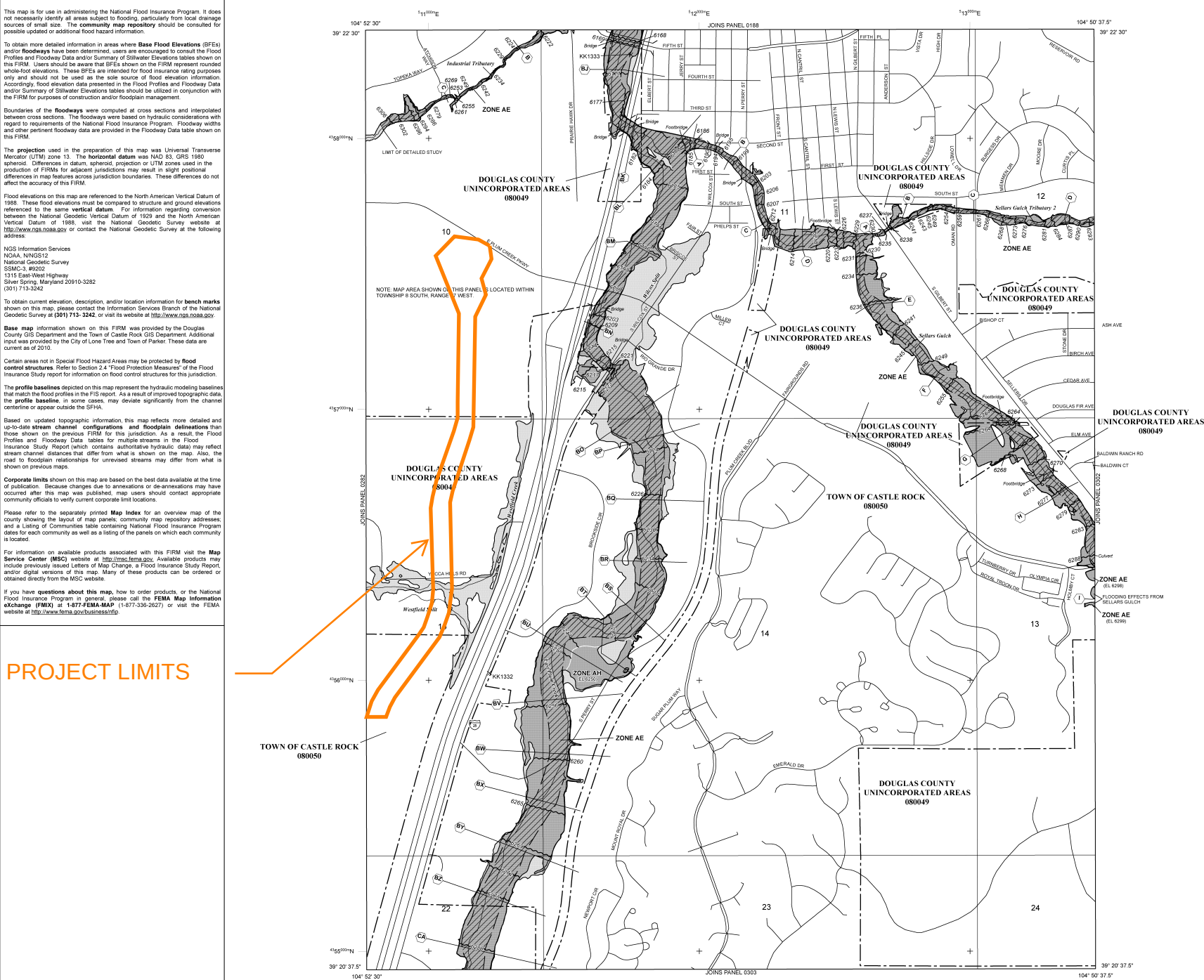
Corporate limits shown on this map are based on the best data available at the time of publication. Because changes due to annexations or de-annexations may have occurred after this map was published, map users should contact appropriate community officials to verify current corporate limit locations.

Please refer to the separately printed **Map Index** for an overview map of the county showing the layout of map panels, community map repository addresses and a listing of Communities table containing National Flood Insurance Program dates for each community as well as a listing of the panels on which each community is located.

For information on available products associated with this FIRM visit the **Map Service Center (MSC)** website at <http://msc.fema.gov>. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the MSC website.

If you have questions about this map, how to order products, or the National Flood Insurance Program in general, please call the **FEMA Map Information eXchange (FMIX)** at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA website at <http://www.fema.gov/businessmap>.

PROJECT LIMITS



LEGEND

SPECIAL FLOOD HAZARD AREAS (SFHAs) SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD
The 1% annual chance flood (50-year flood), also known as the **base flood**, is the flood that has a 1% chance of being equaled or exceeded in any given year. Flood hazard areas are delineated by the 1% annual chance flood. Areas of Special Flood Hazard include Zone A, AE, AH, AO, AR, AV, V, and VE. The **Base Flood Elevation** is the water-surface elevation of the 1% annual chance flood.

- ZONE A** No Base Flood Elevations determined.
- ZONE AE** Base Flood Elevations determined.
- ZONE AH** Flood depths of 1 to 3 feet (usually shear flow on sloping terrain); Base Flood Elevations determined.
- ZONE AO** Flood depths of 1 to 3 feet (usually shear flow on sloping terrain); average depths determined; areas of flood hazard areas are shown.
- ZONE AR** Special Flood Hazard Areas formerly protected from the 1% annual chance flood by a flood control system that was subsequently identified. Zone AR indicates that the former flood control system is being removed to provide protection from the 1% annual chance or greater flood.
- ZONE AV** Areas to be protected from the 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined.
- ZONE VE** Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined.

FLOODWAY AREAS IN ZONE AE
The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

- OTHER FLOOD AREAS**
- ZONE X** Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with average areas less than 1 square mile, and areas protected by levees from 1% annual chance flood.
- OTHER AREAS**
- ZONE D** Areas determined to be outside the 0.2% annual chance floodplain. Areas in which flood hazards are undetermined, but possible.
- COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS**
- OTHERWISE PROTECTED AREAS (OPAs)**

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

- 1% Annual Chance Floodplain Boundary
- Floodway boundary
- Zone D boundary
- CBRS and OPA boundary
- Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths, or flood velocities.
- Base Flood Elevation line and value: elevation in feet*
- Base Flood Elevation value where uniform within zone; elevation in feet*

*Referenced to the North American Vertical Datum of 1988

- (A)** Cross section line
- (B)** Transit line
- Geographic coordinates referenced to the North American Datum of 1983 (NAD 83) Western Hemisphere
- 3000-meter Universal Transverse Mercator grid values, zone 13
- MSM** N
- MSM** X
- Bench mark (see explanation in Notes to Users section of this FIRM panel)
- M** 1 S
- M** 1 S
- MAP REPOSITORIES
- Refer to Map Repositories list on Map Index

EFFECTIVE DATE OF COUNTRYWIDE FLOOD INSURANCE RATE MAP

SEPTEMBER 30, 2005

EFFECTIVE DATE(S) OF REVISION(S) TO THIS PANEL

MARCH 16, 2016: To update corporate limits, to change base flood elevations, to add base flood elevations, to add special flood hazard areas, to update map format, to add roads and road names, to reflect updated topographic information, to incorporate previously issued letters of map revision.

For community map revision history prior to countywide mapping, refer to the Community Map History table located in the Flood Insurance Study report for this jurisdiction.

To determine if flood insurance is available in this community, contact your insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Notice to User: The **Map Number** shown below should be used when placing map orders, the **Community Number** shown above should be used on insurance applications for the subject community.

MAP NUMBER
08035C0301G
MAP REVISED
MARCH 16, 2016

Federal Emergency Management Agency

NATIONAL FLOOD INSURANCE PROGRAM

PANEL 0301G

FIRM

FLOOD INSURANCE RATE MAP

DOUGLAS COUNTY, COLORADO AND INCORPORATED AREAS

PANEL 301 OF 495

(SEE MAP INDEX FOR FIRM PANEL LAYOUT)

CONTAINS:

COMMUNITY NUMBER PANEL SUFFIX

CASTLE ROCK, TOWN OF 080050 0301 G

DOUGLAS COUNTY 080049 0301 G

DOUGLAS COUNTY 080049 0301 G

DOUGLAS COUNTY 080049 0301 G

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DOUGLAS COUNTY 080049 0301 G

Dawson Trails Boulevard (North) - Future Condition

CORE Project #: 20-224

Prepared By: ACJ

IMPERVIOUS CALCULATIONS**-REFERENCE UDFCD Vol.1 RUNOFF Table 6-3**

		Residential & Commercial												Soil Type		
		Single Family			0.00									Soil Type A Area	Soil Type B Area	Soil Type C/D Area
		0.25 acres or less	0.25 - 0.75 acres	2.5 acres or larger	Park/ Landscape	Roof/ Concrete	Asphalt/ Ponded Water	Packed Gravel	2-7% Slope	>7% Slope	Historic					
		45.00%	30.00%	12.00%	10.00%	90.00%	100.00%	40.00%	2.00%	2.00%	2.00%					
% Imperv.	Design Point											Total Area (ac)	Percent Impervious			
BASIN		Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)			Area (ac)	Area (ac)	Area (ac)
A-1.1	A1.1	-	-	-	-	0.52	1.55	-	0.79	-	-	2.85	71.0%	0.571	1.43	0.86
A-1.2	A1.2	-	-	-	-	0.52	1.80	-	0.56	-	-	2.88	79.2%	0.576	1.44	0.86
A-1.3	A1.3	-	-	-	-	-	0.30	-	-	0.27	-	0.57	53.3%	-	0.57	-
A1	A1	-	-	-	-	1.03	3.65	-	1.35	0.27	-	6.30	73.1%	1.146	3.44	1.72
A-2.1	A2.1	-	-	-	-	0.16	0.57	-	0.17	-	-	0.91	79.7%	-	0.91	-
A-2.2	A2.2	-	-	-	-	0.16	0.57	-	0.17	-	-	0.91	79.7%	-	0.91	-
A-2.3	A2.3	-	-	-	-	0.30	1.07	-	0.32	-	-	1.69	79.7%	-	1.69	-
A-2.4	A2.4	-	-	-	-	0.31	1.07	-	0.32	-	-	1.70	79.6%	-	1.70	-
A-2.5	A2.5	-	-	-	-	-	0.26	-	0.28	-	-	0.53	48.9%	-	0.53	-
A2	A2	-	-	-	-	0.94	3.54	-	0.66	-	0.60	5.74	76.8%	-	5.74	-
A-4.1	A4.1	-	-	-	-	0.27	0.98	-	-	4.88	-	6.14	21.6%	-	6.13	-
A-4.2	A4.2	-	-	-	-	0.26	1.18	-	-	2.39	-0	3.82	38.0%	-	3.82	-
A-4.3	A4.3	-	-	-	-	-	-	-	-	5.63	-	5.63	2.0%	-	5.63	-
A4	A4	-	-	-	-	0.53	2.16	-	-	12.89	-	15.58	18.6%	-	15.58	-
		-	-	-	-	-	-	-	-	-	-	-		-	-	-
7.1	7.1	-	-	-	-	0.12	0.36	-	0.20	-	-	0.69	69.5%	-	-	0.69
7.2	7.2	-	-	-	-	0.12	0.42	-	0.11	-	-	0.65	81.3%	-	-	0.65
		-	-	-	-	-	-	-	-	-	-	-		-	-	-
OS-7.N	7N	-	-	-	-	-	-	-	-	1.60	9.07	10.67	2.0%	-	-	10.67
OS-8.S	8S	-	-	3.70	-	-	-	-	-	1.42	0.57	5.70	8.5%	-	2.85	2.85
OS-9.S	9S	-	-	10.24	-	-	-	-	-	-	-	10.24	12.0%	-	10.24	-
OS-9.N	9N	-	-	20.96	-	-	-	-	-	-	-	20.96	12.0%	-	20.96	-
OS-10.S	10S	-	-	0.45	-	-	-	-	-	-	-	0.45	12.0%	-	0.45	-
		-	-	-	-	-	-	-	-	-	-	-		-	-	-
OS-11.S	11S	-	-	1.09	-	-	-	-	-	-	2.31	3.39	5.2%	-	3.39	-
OS-11.N	11N	-	-	1.59	-	-	-	-	-	-	3.38	4.97	5.2%	-	4.97	-
OS-12.S	12S	-	-	-	-	-	-	-	-	-	0.31	0.31	2.0%	-	0.31	-
OS-12.N	12N	-	-	-	-	-	-	-	-	-	0.57	0.57	2.0%	-	0.57	-
OS-13.S	13S	-	-	-	-	-	-	-	-	-	0.26	0.26	2.0%	-	0.26	-

Basin ID	% Imperv.	<i>i</i>	Soil Type	Runoff Coefficients, C				Basin Area	Total Area	Weighted Runoff Coefficients, C			
				2-Year	5-Year	10-Year	100-Year			2-Year	5-Year	10-Year	100-Year
A-1.1	71.0%	0.71	A	0.54	0.56	0.57	0.66	0.57	2.85	0.56	0.59	0.63	0.75
			B	0.56	0.59	0.63	0.76	1.43					
			C or D	0.57	0.62	0.66	0.78	0.86					
A-1.2	79.2%	0.79	A	0.62	0.64	0.65	0.73	0.58	2.88	0.64	0.67	0.70	0.79
			B	0.64	0.67	0.70	0.80	1.44					
			C or D	0.64	0.68	0.72	0.81	0.86					
A-1.3	53.3%	0.53	A	0.37	0.39	0.40	0.53	0.57	0.57	0.40	0.43	0.49	0.68
			B	0.40	0.43	0.49	0.68						
			C or D	0.41	0.47	0.53	0.70						
A1	73.1%	0.73	A	0.56	0.58	0.59	0.68	1.15	6.30	0.58	0.61	0.65	0.76
			B	0.58	0.61	0.65	0.77	3.44					
			C or D	0.58	0.63	0.67	0.78	1.72					
A-2.1	79.7%	0.80	A	0.62	0.64	0.66	0.73	0.91	0.91	0.64	0.67	0.70	0.80
			B	0.64	0.67	0.70	0.80						
			C or D	0.64	0.69	0.72	0.81						
A-2.2	79.7%	0.80	A	0.62	0.64	0.66	0.73	0.91	0.91	0.64	0.67	0.70	0.80
			B	0.64	0.67	0.70	0.80						
			C or D	0.64	0.69	0.72	0.81						
A-2.3	79.7%	0.80	A	0.63	0.64	0.66	0.73	1.69	1.69	0.64	0.67	0.70	0.80
			B	0.64	0.67	0.70	0.80						
			C or D	0.64	0.69	0.72	0.81						
A-2.4	79.6%	0.80	A	0.62	0.64	0.66	0.73	1.70	1.70	0.64	0.67	0.70	0.80
			B	0.64	0.67	0.70	0.80						
			C or D	0.64	0.69	0.72	0.81						
A-2.5	48.9%	0.49	A	0.33	0.34	0.36	0.49	0.53	0.53	0.36	0.39	0.45	0.66
			B	0.36	0.39	0.45	0.66						
			C or D	0.37	0.44	0.49	0.68						
A2	76.8%	0.77	A	0.60	0.61	0.63	0.71	5.74	5.74	0.62	0.65	0.68	0.79
			B	0.62	0.65	0.68	0.79						
			C or D	0.62	0.66	0.70	0.80						
A-4.1	21.6%	0.22	A	0.11	0.12	0.13	0.28	6.13	6.13	0.14	0.16	0.23	0.53
			B	0.14	0.16	0.23	0.53						
			C or D	0.15	0.21	0.29	0.57						
A-4.2	38.0%	0.38	A	0.24	0.25	0.26	0.41	3.82	3.82	0.27	0.30	0.37	0.60
			B	0.27	0.30	0.37	0.60						
			C or D	0.28	0.35	0.41	0.64						
A-4.3	2.0%	0.02	A	0.01	0.01	0.01	0.13	5.63	5.63	0.01	0.01	0.07	0.44
			B	0.01	0.01	0.07	0.44						
			C or D	0.01	0.05	0.15	0.49						
A4	18.6%	0.19	A	0.09	0.10	0.11	0.25	15.58	15.58	0.12	0.14	0.21	0.51
			B	0.12	0.14	0.21	0.51						
			C or D	0.13	0.19	0.27	0.56						

Basin ID	% Imperv.	<i>i</i>	Soil Type	Runoff Coefficients, C				Basin Area	Total Area	Weighted Runoff Coefficients, C			
				2-Year	5-Year	10-Year	100-Year			2-Year	5-Year	10-Year	100-Year
										-	-	-	-
7.1	69.5%	0.69	A B C or D	0.52 0.55 0.55	0.54 0.58 0.60	0.56 0.62 0.65	0.65 0.75 0.77	0.69	0.69	0.55	0.60	0.65	0.77
7.2	81.3%	0.81	A B C or D	0.64 0.66 0.66	0.66 0.69 0.70	0.67 0.72 0.73	0.74 0.81 0.82	0.65	0.65	0.66	0.70	0.73	0.82
										-	-	-	-
OS-7.N	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	10.67	10.67	0.01	0.05	0.15	0.49
OS-8.S	8.5%	0.08	A B C or D	0.03 0.05 0.05	0.04 0.06 0.10	0.04 0.13 0.19	0.18 0.47 0.52	2.85 2.85	5.70	0.05	0.08	0.16	0.49
OS-9.S	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	10.24	10.24	0.07	0.09	0.15	0.48
OS-9.N	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	20.96	20.96	0.07	0.09	0.15	0.48
OS-10.S	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	0.45	0.45	0.07	0.09	0.15	0.48
										-	-	-	-
OS-11.S	5.2%	0.05	A B C or D	0.02 0.03 0.03	0.02 0.03 0.08	0.02 0.10 0.17	0.15 0.45 0.51	3.39	3.39	0.03	0.03	0.10	0.45
OS-11.N	5.2%	0.05	A B C or D	0.02 0.03 0.03	0.02 0.03 0.08	0.02 0.10 0.17	0.15 0.45 0.51	4.97	4.97	0.03	0.03	0.10	0.45
OS-12.S	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.31	0.31	0.01	0.01	0.07	0.44
OS-12.N	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.57	0.57	0.01	0.01	0.07	0.44
OS-13.S	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.26	0.26	0.01	0.01	0.07	0.44

Dawson Trails Boulevard (North) - Future Condition

CORE Project #: 20-224

Prepared By: ACJ

TIME OF CONCENTRATION CALCULATIONS | BASIN A

-REFERENCE UDFCD Vol.1 Section 2.4

NRCS Conveyance factors, K -REFERENCE UDFCD Vol.1 RUNOFF Table 6-2

SF-2

Heavy Meadow	3	Short Grass Pasture & Lawns	7	Grassed Waterway	15
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Tillage/field	5	Nearly Bare Ground	10	Paved Area & Shallow Gutter	20
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[illegible]

SUB-BASIN DATA			INITIAL / OVERLAND TIME			CHANNEL / TRAVEL TIME T(t)						T(c) CHECK (URBANIZED BASINS)		FINAL T(c) min.
DRAIN BASIN	AREA ac.	C(5)	Length ft.	Slope %	T(i) min	Length ft.	Slope %	Coeff.	Velocity fps	T(t) min.	COMP. T(c)	% IMPER-VIOUS	USDCM Eq . 6-5	
7.1	0.69	0.60	25	2.0	3.6	528	1.1	20	2.1	4.1	7.7	0.7	18.6	7.7
7.2	0.65	0.70	25	2.0	2.9	518	1.2	20	2.2	4.0	6.9	0.8	16.1	6.9
OS-7.N	10.67	0.05	500	8.2	21.1	383	10.4	7	2.3	2.8	24.0	0.0	27.8	24.0
OS-8.S	5.70	0.08	500	13.2	17.5	535	1.3	7	0.8	11.1	28.7	0.1	32.2	28.7
OS-9.S	10.24	0.09	500	5.8	22.9	1051	3.6	7	1.3	13.2	36.1	0.1	32.6	32.6
OS-9.N	20.96	0.09	500	8.3	20.3	2363	5.3	7	1.6	24.5	44.8	0.1	40.0	40.0
OS-10.S	0.45	0.09	100	15.0	7.5	100	1.5	7	0.9	1.9	9.4	0.1	25.2	9.4
				0.0			0.0							
OS-11.S	3.39	0.03	490	8.8	20.8	337	1.5	7	0.9	6.6	27.4	0.1	29.9	27.4
OS-11.N	4.97	0.03	500	11.4	19.3	354	4.9	7	1.6	3.8	23.1	0.1	27.8	23.1
OS-12.S	0.31	0.01	188	11.2	12.1	102	2.2	7	1.0	1.7	13.8	0.0	26.9	13.8
OS-12.N	0.57	0.01	200	15.0	11.4	180	2.2	7	1.0	2.9	14.2	0.0	27.8	14.2
OS-13.S	0.26	0.01	166	12.7	11.0	89	2.8	7	1.2	1.3	12.2	0.0	26.6	12.2

Dawson Trails Boulevard (North) - Future Condition

CORE Project #: 20-224

Prepared By: ACJ

RATIONAL METHOD PEAK RUNOFF

5-Year STORM Rainfall Depth-Duration-Frequency (1-hr) = 1.43

SF-3

-REFERENCE UDFCD Vol.1 EQ 5-1 & EQ 6-1

BASIN INFORMATION				DIRECT RUNOFF			
DESIGN POINT	DRAIN BASIN	AREA ac.	5yr Runoff COEFF	T(c) min	C x A	I in/hr	Q cfs
A1.1	A-1.1	2.85	0.59	14.4	1.69	3.31	5.60
A1.2	A-1.2	2.88	0.67	13.8	1.92	3.37	6.46
A1.3	A-1.3	0.57	0.43	5.0	0.25	4.85	1.21
A1	A1	6.30	0.61	5.0	3.86	4.85	18.70
A2.1	A-2.1	0.91	0.67	5.5	0.61	4.72	2.88
A2.2	A-2.2	0.91	0.67	5.5	0.61	4.72	2.88
A2.3	A-2.3	1.69	0.67	8.4	1.13	4.13	4.69
A2.4	A-2.4	1.70	0.67	8.4	1.14	4.13	4.72
A2.5	A-2.5	0.53	0.39	8.5	0.21	4.11	0.87
A2	A2	5.74	0.65	5.0	3.70	4.85	17.95
A4.1	A-4.1	6.14	0.16	11.6	1.00	3.64	3.63
A4.2	A-4.2	3.82	0.30	9.6	1.15	3.93	4.52
A4.3	A-4.3	5.63	0.01	14.7	0.07	3.28	0.22
A4	A4	15.58	0.14	5.0	2.15	4.85	10.41
7.1	7.1	0.69	0.60	7.7	0.41	4.26	1.77
7.2	7.2	0.65	0.70	6.9	0.45	4.42	2.01
7N	OS-7.N	10.67	0.05	24.0	0.55	2.55	1.40
8S	OS-8.S	5.70	0.08	28.7	0.47	2.30	1.07
9S	OS-9.S	10.24	0.09	32.6	0.88	2.14	1.87
9N	OS-9.N	20.96	0.09	40.0	1.79	1.88	3.38
10S	OS-10.S	0.45	0.09	9.4	0.04	3.96	0.15
11S	OS-11.S	3.39	0.03	27.4	0.12	2.37	0.28
11N	OS-11.N	4.97	0.03	23.1	0.17	2.61	0.45
12S	OS-12.S	0.31	0.01	13.8	0.00	3.37	0.01
12N	OS-12.N	0.57	0.01	14.2	0.01	3.33	0.02
13S	OS-13.S	0.26	0.01	12.2	0.00	3.56	0.01

Dawson Trails Boulevard (North) - Future Condition

CORE Project #: 20-224

Prepared By: ACJ

RATIONAL METHOD PEAK RUNOFF

100-YR STORM

SF-3 Rainfall Depth-Duration-Frequency (1-hr) = 2.60

-REFERENCE UDFCD Vol.1 EQ 5-1 & EQ 6-1

BASIN INFORMATON				DIRECT RUNOFF			
DESIGN POINT	DRAIN BASIN	AREA ac.	100YR RUNNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs
A1.1	A-1.1	2.85	0.75	14.4	2.13	6.02	12.80
A1.2	A-1.2	2.88	0.79	13.8	2.27	6.13	13.88
A1.3	A-1.3	0.57	0.68	5.0	0.39	8.82	3.42
A1	A1	6.30	0.76	5.0	4.77	8.82	42.10
A2.1	A-2.1	0.91	0.80	5.5	0.73	8.58	6.23
A2.2	A-2.2	0.91	0.80	5.5	0.73	8.58	6.23
A2.3	A-2.3	1.69	0.80	8.4	1.35	7.52	10.15
A2.4	A-2.4	1.70	0.80	8.4	1.36	7.52	10.23
A2.5	A-2.5	0.53	0.66	8.5	0.35	7.48	2.62
A2	A2	5.74	0.79	5.0	4.51	8.82	39.81
A4.1	A-4.1	6.14	0.53	11.6	3.24	6.61	21.40
A4.2	A-4.2	3.82	0.60	9.6	2.31	7.15	16.53
A4.3	A-4.3	5.63	0.44	14.7	2.45	5.96	14.60
A4	A4	15.58	0.51	5.0	8.00	8.82	70.54
7.1	7.1	0.69	0.77	7.7	0.53	7.75	4.09
7.2	7.2	0.65	0.82	6.9	0.53	8.04	4.25
7N	OS-7.N	10.67	0.49	24.0	5.25	4.64	24.37
8S	OS-8.S	5.70	0.49	28.7	2.81	4.19	11.76
9S	OS-9.S	10.24	0.48	32.6	4.94	3.88	19.18
9N	OS-9.N	20.96	0.48	40.0	10.11	3.42	34.62
10S	OS-10.S	0.45	0.48	9.4	0.22	7.19	1.56
11S	OS-11.S	3.39	0.45	27.4	1.53	4.30	6.58
11N	OS-11.N	4.97	0.45	23.1	2.24	4.74	10.62
12S	OS-12.S	0.31	0.44	13.8	0.13	6.14	0.82
12N	OS-12.N	0.57	0.44	14.2	0.25	6.05	1.50
13S	OS-13.S	0.26	0.44	12.2	0.11	6.48	0.73

RUNOFF SUMMARY TABLE				
DIRECT RUNOFF				
DESIGN POINT	BASIN	AREA (AC)	5-Year RUNOFF (CFS)	100-Year RUNOFF (CFS)
A1.1	A-1.1	2.9	5.6	12.8
A1.2	A-1.2	2.9	6.5	13.9
A1.3	A-1.3	0.6	1.2	3.4
A1	A1	6.3	13.3	30.1
A2.1	A-2.1	0.9	2.9	6.2
A2.2	A-2.2	0.9	2.9	6.2
A2.3	A-2.3	1.7	4.7	10.1
A2.4	A-2.4	1.7	4.7	10.2
A2.5	A-2.5	0.5	0.9	2.6
A2	A2	5.7	16.0	35.5
A4.1	A-4.1	6.1	3.6	21.4
A4.2	A-4.2	3.8	4.5	16.5
A4.3	A-4.3	5.6	0.2	14.6
A4	A4	15.6	8.4	52.5
7.1	7.1	0.7	1.8	4.1
7.2	7.2	0.6	2.0	4.2
7N	OS-7.N	10.7	1.4	24.4
8S	OS-8.S	5.7	1.1	11.8
9S	OS-9.S	10.2	1.9	19.2
9N	OS-9.N	21.0	3.4	34.6
10S	OS-10.S	0.5	0.2	1.6
11S	OS-11.S	3.4	0.3	6.6
11N	OS-11.N	5.0	0.4	10.6
12S	OS-12.S	0.3	0.01	0.8
12N	OS-12.N	0.6	0.02	1.5
13S	OS-13.S	0.3	0.01	0.7
Total		82.4	50.1	238.2

Dawson Trails Boulevard (North) - Interim Condition

CORE Project #: 20-224

Prepared By: ACJ

IMPERVIOUS CALCULATIONS**-REFERENCE UDFCD Vol.1 RUNOFF Table 6-3**

		Residential & Commercial												Soil Type		
		Single Family			0.00									Soil Type A Area	Soil Type B Area	Soil Type C/D Area
		0.25 acres or less	0.25 - 0.75 acres	2.5 acres or larger	Park/ Landscape	Roof/ Concrete	Asphalt/ Ponded Water	Packed Gravel	2-7% Slope	>7% Slope	Historic					
		45.00%	30.00%	12.00%	10.00%	90.00%	100.00%	40.00%	2.00%	2.00%	2.00%	Total Area (ac)	Percent Impervious			
% Imperv.	Design Point	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)			Area (ac)	Area (ac)	Area (ac)
BASIN																
A-1.1	A1.1	-	-	-	-	-	-	-	2.85	-	-	2.85	2.0%	0.571	1.43	0.86
A-1.2	A1.2	-	-	-	-	0.52	1.80	-	0.56	-	-	2.88	79.2%	0.576	1.44	0.86
A-1.3	A1.3	-	-	-	-	-	0.30	-	-	0.27	-	0.57	53.3%	-	0.57	-
A1	A1	-	-	-	-	0.52	2.10	-	3.41	0.27	-	6.30	41.9%	1.146	3.44	1.72
A-2.1	A2.1	-	-	-	-	-	-	-	0.91	-	-	0.91	2.0%	-	0.91	-
A-2.2	A2.2	-	-	-	-	0.16	0.57	-	0.17	-	-	0.91	79.7%	-	0.91	-
A-2.3	A2.3	-	-	-	-	-	-	-	1.69	-	-	1.69	2.0%	-	1.69	-
A-2.4	A2.4	-	-	-	-	0.31	1.07	-	0.32	-	-	1.70	79.6%	-	1.70	-
A-2.5	A2.5	-	-	-	-	-	0.26	-	0.28	-	-	0.53	48.9%	-	0.53	-
A2	A2	-	-	-	-	0.47	1.90	-	3.37	-	-	5.74	41.7%	-	5.74	-
A-4.1	A4.1	-	-	-	-	-	-	-	-	6.13	-	6.14	2.0%	-	6.13	-
A-4.2	A4.2	-	-	-	-	0.29	1.23	-	-	2.31	-	3.82	40.2%	-	3.82	-
A-4.3	A4.3	-	-	-	-	-	-	-	-	5.63	-	5.63	2.0%	-	5.63	-
A4	A4	-	-	-	-	0.29	1.23	-	-	14.07	-	15.58	11.4%	-	15.58	-
		-	-	-	-	-	-	-	-	-	-	-		-	-	-
7.1	7.1	-	-	-	-	-	-	-	0.69	-	-	0.69	2.0%	-	-	0.69
7.2	7.2	-	-	-	-	0.12	0.42	-	0.11	-	-	0.65	81.3%	-	-	0.65
		-	-	-	-	-	-	-	-	-	-	-		-	-	-
OS-7.N	7N	-	-	-	-	-	-	-	-	1.60	9.07	10.67	2.0%	-	-	10.67
OS-8.S	8S	-	-	3.70	-	-	-	-	-	1.42	0.57	5.70	8.5%	-	2.85	2.85
OS-9.S	9S	-	-	10.24	-	-	-	-	-	-	-	10.24	12.0%	-	10.24	-
OS-9.N	9N	-	-	20.96	-	-	-	-	-	-	-	20.96	12.0%	-	20.96	-
OS-10.S	10S	-	-	0.45	-	-	-	-	-	-	-	0.45	12.0%	-	0.45	-
		-	-	-	-	-	-	-	-	-	-	-		-	-	-
OS-11.S	11S	-	-	1.09	-	-	-	-	-	-	2.31	3.39	5.2%	-	3.39	-
OS-11.N	11N	-	-	1.59	-	-	-	-	-	-	3.38	4.97	5.2%	-	4.97	-
OS-12.S	12S	-	-	-	-	-	-	-	-	-	0.31	0.31	2.0%	-	0.31	-
OS-12.N	12N	-	-	-	-	-	-	-	-	-	0.57	0.57	2.0%	-	0.57	-
OS-13.S	13S	-	-	-	-	-	-	-	-	-	0.26	0.26	2.0%	-	0.26	-

Basin ID	% Imperv.	<i>i</i>	Soil Type	Runoff Coefficients, <i>C</i>				Basin Area	Total Area	Weighted Runoff Coefficients, <i>C</i>			
				2-Year	5-Year	10-Year	100-Year			2-Year	5-Year	10-Year	100-Year
A-1.1	2.0%	0.02	A	0.01	0.01	0.01	0.13	0.57	2.85	0.01	0.02	0.08	0.39
			B	0.01	0.01	0.07	0.44	1.43					
			C or D	0.01	0.05	0.15	0.49	0.86					
A-1.2	79.2%	0.79	A	0.62	0.64	0.65	0.73	0.58	2.88	0.64	0.67	0.70	0.79
			B	0.64	0.67	0.70	0.80	1.44					
			C or D	0.64	0.68	0.72	0.81	0.86					
A-1.3	53.3%	0.53	A	0.37	0.39	0.40	0.53	0.57	0.57	0.40	0.43	0.49	0.68
			B	0.40	0.43	0.49	0.68						
			C or D	0.41	0.47	0.53	0.70						
A1	41.9%	0.42	A	0.27	0.28	0.30	0.44	1.15	6.30	0.30	0.34	0.39	0.60
			B	0.30	0.33	0.40	0.62	3.44					
			C or D	0.31	0.38	0.44	0.66	1.72					
A-2.1	2.0%	0.02	A	0.01	0.01	0.01	0.13	0.91	0.91	0.01	0.01	0.07	0.44
			B	0.01	0.01	0.07	0.44						
			C or D	0.01	0.05	0.15	0.49						
A-2.2	79.7%	0.80	A	0.62	0.64	0.66	0.73	0.91	0.91	0.64	0.67	0.70	0.80
			B	0.64	0.67	0.70	0.80						
			C or D	0.64	0.69	0.72	0.81						
A-2.3	2.0%	0.02	A	0.01	0.01	0.01	0.13	1.69	1.69	0.01	0.01	0.07	0.44
			B	0.01	0.01	0.07	0.44						
			C or D	0.01	0.05	0.15	0.49						
A-2.4	79.6%	0.80	A	0.62	0.64	0.66	0.73	1.70	1.70	0.64	0.67	0.70	0.80
			B	0.64	0.67	0.70	0.80						
			C or D	0.64	0.69	0.72	0.81						
A-2.5	48.9%	0.49	A	0.33	0.34	0.36	0.49	0.53	0.53	0.36	0.39	0.45	0.66
			B	0.36	0.39	0.45	0.66						
			C or D	0.37	0.44	0.49	0.68						
A2	41.7%	0.42	A	0.27	0.28	0.30	0.43	5.74	5.74	0.30	0.33	0.39	0.62
			B	0.30	0.33	0.39	0.62						
			C or D	0.31	0.38	0.44	0.65						
A-4.1	2.0%	0.02	A	0.01	0.01	0.01	0.13	6.13	6.13	0.01	0.01	0.07	0.44
			B	0.01	0.01	0.07	0.44						
			C or D	0.01	0.05	0.15	0.49						
A-4.2	40.2%	0.40	A	0.26	0.27	0.28	0.42	3.82	3.82	0.29	0.32	0.38	0.61
			B	0.29	0.32	0.38	0.61						
			C or D	0.30	0.36	0.43	0.65						
A-4.3	2.0%	0.02	A	0.01	0.01	0.01	0.13	5.63	5.63	0.01	0.01	0.07	0.44
			B	0.01	0.01	0.07	0.44						
			C or D	0.01	0.05	0.15	0.49						
A4	11.4%	0.11	A	0.05	0.05	0.06	0.20	15.58	15.58	0.07	0.08	0.15	0.48
			B	0.07	0.08	0.15	0.48						
			C or D	0.07	0.13	0.22	0.53						

Basin ID	% Imperv.	<i>i</i>	Soil Type	Runoff Coefficients, C				Basin Area	Total Area	Weighted Runoff Coefficients, C			
				2-Year	5-Year	10-Year	100-Year			2-Year	5-Year	10-Year	100-Year
										-	-	-	-
7.1	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.69	0.69	0.01	0.05	0.15	0.49
7.2	81.3%	0.81	A B C or D	0.64 0.66 0.66	0.66 0.69 0.70	0.67 0.72 0.73	0.74 0.81 0.82	0.65	0.65	0.66	0.70	0.73	0.82
										-	-	-	-
OS-7.N	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	10.67	10.67	0.01	0.05	0.15	0.49
OS-8.S	8.5%	0.08	A B C or D	0.03 0.05 0.05	0.04 0.06 0.10	0.04 0.13 0.19	0.18 0.47 0.52	2.85 2.85	5.70	0.05	0.08	0.16	0.49
OS-9.S	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	10.24	10.24	0.07	0.09	0.15	0.48
OS-9.N	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	20.96	20.96	0.07	0.09	0.15	0.48
OS-10.S	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	0.45	0.45	0.07	0.09	0.15	0.48
										-	-	-	-
OS-11.S	5.2%	0.05	A B C or D	0.02 0.03 0.03	0.02 0.03 0.08	0.02 0.10 0.17	0.15 0.45 0.51	3.39	3.39	0.03	0.03	0.10	0.45
OS-11.N	5.2%	0.05	A B C or D	0.02 0.03 0.03	0.02 0.03 0.08	0.02 0.10 0.17	0.15 0.45 0.51	4.97	4.97	0.03	0.03	0.10	0.45
OS-12.S	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.31	0.31	0.01	0.01	0.07	0.44
OS-12.N	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.57	0.57	0.01	0.01	0.07	0.44
OS-13.S	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	0.26	0.26	0.01	0.01	0.07	0.44

Prepared By: ACJ

-REFERENCE UDFCD Vol.1 Section 2.4

NRCS Conveyance factors, K -REFERENCE UDFCD Vol.1 RUNOFF Table 6-2

SF-2	Heavy Meadow	3	Short Grass Pasture & Lawns	7	Grassed Waterway	15
	Tillage/field	5	Nearly Bare Ground	10	Paved Area & Shallow Gutter	20

[illegible]

SUB-BASIN DATA			INITIAL / OVERLAND TIME			CHANNEL / TRAVEL TIME T(t)						T(c) CHECK (URBANIZED BASINS)		FINAL T(c) min.
DRAIN BASIN	AREA ac.	C(5)	Length ft.	Slope %	T(i) min	Length ft.	Slope %	Coeff.	Velocity fps	T(t) min.	COMP. T(c)	% IMPER-VIOUS	USDCM Eq . 6-5	
7.1	0.69	0.05	25	2.0	7.5	528	1.1	20	2.1	4.1	11.7	0.0	34.6	11.7
7.2	0.65	0.70	25	2.0	2.9	518	1.2	20	2.2	4.0	6.9	0.8	16.1	6.9
OS-7.N	10.67	0.05	500	8.2	21.1	383	10.4	7	2.3	2.8	24.0	0.0	27.8	24.0
OS-8.S	5.70	0.08	500	13.2	17.5	535	1.3	7	0.8	11.1	28.7	0.1	32.2	28.7
OS-9.S	10.24	0.09	500	5.8	22.9	1051	3.6	7	1.3	13.2	36.1	0.1	32.6	32.6
OS-9.N	20.96	0.09	500	8.3	20.3	2363	5.3	7	1.6	24.5	44.8	0.1	40.0	40.0
OS-10.S	0.45	0.09	100	15.0	7.5	100	1.5	7	0.9	1.9	9.4	0.1	25.2	9.4
				0.0			0.0							
OS-11.S	3.39	0.03	490	8.8	20.8	337	1.5	7	0.9	6.6	27.4	0.1	29.9	27.4
OS-11.N	4.97	0.03	500	11.4	19.3	354	4.9	7	1.6	3.8	23.1	0.1	27.8	23.1
OS-12.S	0.31	0.01	188	11.2	12.1	102	2.2	7	1.0	1.7	13.8	0.0	26.9	13.8
OS-12.N	0.57	0.01	200	15.0	11.4	180	2.2	7	1.0	2.9	14.2	0.0	27.8	14.2
OS-13.S	0.26	0.01	166	12.7	11.0	89	2.8	7	1.2	1.3	12.2	0.0	26.6	12.2

Dawson Trails Boulevard (North) - Interim Condition

CORE Project #: 20-224

Prepared By: ACJ

RATIONAL METHOD PEAK RUNOFF

5-Year STORM Rainfall Depth-Duration-Frequency (1-hr) = **1.43**

SF-3

-REFERENCE UDFCD Vol.1 EQ 5-1 & EQ 6-1

BASIN INFORMATION				DIRECT RUNOFF			
DESIGN POINT	DRAIN BASIN	AREA ac.	5yr Runoff COEFF	T(c) min	C x A	I in/hr	Q cfs
A1.1	A-1.1	2.85	0.02	17.2	0.06	3.04	0.20
A1.2	A-1.2	2.88	0.67	13.8	1.92	3.37	6.46
A1.3	A-1.3	0.57	0.43	5.0	0.25	4.85	1.21
A1	A1	6.30	0.34	5.0	2.12	4.85	10.30
A2.1	A-2.1	0.91	0.01	8.9	0.01	4.04	0.04
A2.2	A-2.2	0.91	0.67	5.5	0.61	4.72	2.88
A2.3	A-2.3	1.69	0.01	11.5	0.02	3.65	0.08
A2.4	A-2.4	1.70	0.67	8.4	1.14	4.13	4.72
A2.5	A-2.5	0.53	0.39	8.5	0.21	4.11	0.87
A2	A2	5.74	0.33	5.0	1.90	4.85	9.23
A4.1	A-4.1	6.14	0.01	13.3	0.07	3.43	0.26
A4.2	A-4.2	3.82	0.32	9.4	1.22	3.96	4.82
A4.3	A-4.3	5.63	0.01	14.7	0.07	3.28	0.22
A4	A4	15.58	0.08	5.0	1.26	4.85	6.10
7.1	7.1	0.69	0.05	11.7	0.04	3.63	0.13
7.2	7.2	0.65	0.70	6.9	0.45	4.42	2.01
7N	OS-7.N	10.67	0.05	24.0	0.55	2.55	1.40
8S	OS-8.S	5.70	0.08	28.7	0.47	2.30	1.07
9S	OS-9.S	10.24	0.09	32.6	0.88	2.14	1.87
9N	OS-9.N	20.96	0.09	40.0	1.79	1.88	3.38
10S	OS-10.S	0.45	0.09	9.4	0.04	3.96	0.15
11S	OS-11.S	3.39	0.03	27.4	0.12	2.37	0.28
11N	OS-11.N	4.97	0.03	23.1	0.17	2.61	0.45
12S	OS-12.S	0.31	0.01	13.8	0.00	3.37	0.01
12N	OS-12.N	0.57	0.01	14.2	0.01	3.33	0.02
13S	OS-13.S	0.26	0.01	12.2	0.00	3.56	0.01

Dawson Trails Boulevard (North) - Interim Condition

CORE Project #: 20-224

Prepared By: ACJ

RATIONAL METHOD PEAK RUNOFF

100-YR STORM

SF-3 Rainfall Depth-Duration-Frequency (1-hr) = 2.60

-REFERENCE UDFCD Vol.1 EQ 5-1 & EQ 6-1

BASIN INFORMATON				DIRECT RUNOFF			
DESIGN POINT	DRAIN BASIN	AREA ac.	100YR RUNNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs
A1.1	A-1.1	2.85	0.39	17.2	1.11	5.53	6.16
A1.2	A-1.2	2.88	0.79	13.8	2.27	6.13	13.88
A1.3	A-1.3	0.57	0.68	5.0	0.39	8.82	3.42
A1	A1	6.30	0.60	5.0	3.77	8.82	33.25
A2.1	A-2.1	0.91	0.44	8.9	0.40	7.35	2.91
A2.2	A-2.2	0.91	0.80	5.5	0.73	8.58	6.23
A2.3	A-2.3	1.69	0.44	11.5	0.73	6.64	4.88
A2.4	A-2.4	1.70	0.80	8.4	1.36	7.52	10.23
A2.5	A-2.5	0.53	0.66	8.5	0.35	7.48	2.62
A2	A2	5.74	0.62	5.0	3.57	8.82	31.46
A4.1	A-4.1	6.14	0.44	13.3	2.67	6.24	16.67
A4.2	A-4.2	3.82	0.61	9.4	2.35	7.20	16.91
A4.3	A-4.3	5.63	0.44	14.7	2.45	5.96	14.60
A4	A4	15.58	0.48	5.0	7.47	8.82	65.89
7.1	7.1	0.69	0.49	11.7	0.34	6.61	2.23
7.2	7.2	0.65	0.82	6.9	0.53	8.04	4.25
7N	OS-7.N	10.67	0.49	24.0	5.25	4.64	24.37
8S	OS-8.S	5.70	0.49	28.7	2.81	4.19	11.76
9S	OS-9.S	10.24	0.48	32.6	4.94	3.88	19.18
9N	OS-9.N	20.96	0.48	40.0	10.11	3.42	34.62
10S	OS-10.S	0.45	0.48	9.4	0.22	7.19	1.56
11S	OS-11.S	3.39	0.45	27.4	1.53	4.30	6.58
11N	OS-11.N	4.97	0.45	23.1	2.24	4.74	10.62
12S	OS-12.S	0.31	0.44	13.8	0.13	6.14	0.82
12N	OS-12.N	0.57	0.44	14.2	0.25	6.05	1.50
13S	OS-13.S	0.26	0.44	12.2	0.11	6.48	0.73

RUNOFF SUMMARY TABLE				
DIRECT RUNOFF				
DESIGN POINT	BASIN	AREA (AC)	5-Year RUNOFF (CFS)	100-Year RUNOFF (CFS)
A1.1	A-1.1	2.9	0.2	6.2
A1.2	A-1.2	2.9	6.5	13.9
A1.3	A-1.3	0.6	1.2	3.4
A1	A1	6.3	7.9	23.5
A2.1	A-2.1	0.9	0.04	2.9
A2.2	A-2.2	0.9	2.9	6.2
A2.3	A-2.3	1.7	0.1	4.9
A2.4	A-2.4	1.7	4.7	10.2
A2.5	A-2.5	0.5	0.9	2.6
A2	A2	5.7	8.6	26.9
A4.1	A-4.1	6.1	0.3	16.7
A4.2	A-4.2	3.8	4.8	16.9
A4.3	A-4.3	5.6	0.2	14.6
A4	A4	15.6	5.3	48.2
7.1	7.1	0.7	0.1	2.2
7.2	7.2	0.6	2.0	4.2
7N	OS-7.N	10.7	1.4	24.4
8S	OS-8.S	5.7	1.1	11.8
9S	OS-9.S	10.2	1.9	19.2
9N	OS-9.N	21.0	3.4	34.6
10S	OS-10.S	0.5	0.2	1.6
11S	OS-11.S	3.4	0.3	6.6
11N	OS-11.N	5.0	0.4	10.6
12S	OS-12.S	0.3	0.01	0.8
12N	OS-12.N	0.6	0.02	1.5
13S	OS-13.S	0.3	0.01	0.7
Total		86.5	32.5	216.7

Dawson Trails Boulevard (North) - Future Offsite Basins

CORE Project #: 20-224

Prepared By: CORE

IMPERVIOUS CALCULATIONS**-REFERENCE UDFCD Vol.1 RUNOFF Table 6-3**

		Residential & Commercial												Soil Type				
		Single Family			0.00									Soil Type A Area	Soil Type B Area	Soil Type C/D Area		
		0.25 acres or less	0.25 - 0.75 acres	2.5 acres or larger	Park/ Landscape	Roof/ Concrete	Asphalt/ Ponded Water	Packed Gravel	2-7% Slope	>7% Slope	Historic							
% Imperv.	Design Point	45.00%	30.00%	12.00%	10.00%	90.00%	100.00%	40.00%	2.00%	2.00%	2.00%	Total Area (ac)	Percent Impervious					
BASIN																		
		Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)							
		-	-	-	-	-	-	-	-	-	-			-	-	-	-	-
OS-9, OS-9.N, OS-9.S		OS9	-	-	47.24	-	-	-	-	-	-			-	47.24	12.0%	-	47.24
OS-10, OS-10.S	OS10	-	-	10.71	-	-	-	-	-	-	-	10.72	12.0%	-	10.71	-		
OS-11, OS-11.N, OS-11.S	OS11	-	-	8.70	-	-	-	-	-	-	18.49	27.19	5.2%	-	27.19	-		
OS-12, OS-12.N, OS-12.S	OS12	-	-	-	-	-	-	-	-	-	50.13	50.13	2.0%	-	50.13	-		
OS-13, OS-13.S	OS13	-	-	-	-	-	-	-	-	-	19.99	19.99	2.0%	-	19.99	-		

Dawson Trails Boulevard (North) - Future Offsite Basins

CORE Project #: 20-224
Prepared By: CORE

Table 6-4. Runoff coefficient equations based on NRCS soil group and storm return period

NRCS Soil Group	Storm Return Period						
	2-Year	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
A	$C_A = 0.84i^{1.302}$	$C_A = 0.86i^{1.276}$	$C_A = 0.87i^{1.232}$	$C_A = 0.84i^{1.124}$	$C_A = 0.85i+0.025$	$C_A = 0.78i+0.110$	$C_A = 0.65i+0.254$
B	$C_B = 0.84i^{1.169}$	$C_B = 0.86i^{1.088}$	$C_B = 0.81i+0.057$	$C_B = 0.63i+0.249$	$C_B = 0.56i+0.328$	$C_B = 0.47i+0.426$	$C_B = 0.37i+0.536$
C/D	$C_{CD} = 0.83i^{1.122}$	$C_{CD} = 0.82i+0.035$	$C_{CD} = 0.74i+0.132$	$C_{CD} = 0.56i+0.319$	$C_{CD} = 0.49i+0.393$	$C_{CD} = 0.41i+0.484$	$C_{CD} = 0.32i+0.588$

i = % imperviousness/100 expressed as a decimal
 C_A = Runoff coefficient for NRCS HSG A soils
 C_B = Runoff coefficient for NRCS HSG B soils
 C_{CD} = Runoff coefficient for NRCS HSG C and D soils.
Natural Resource Conservation Service (NRCS)

Total Weighted Runoff Coefficients, C			
2-Year	5-Year	10-Year	100-Year
0.03	0.04	0.11	0.46

Basin ID	% Imperv.	i	Soil Type	Runoff Coefficients, C				Basin Area	Total Area	Weighted Runoff Coefficients, C			
				2-Year	5-Year	10-Year	100-Year			2-Year	5-Year	10-Year	100-Year
								- - -		-	-	-	-
OS-9, OS-9.N, OS-9.S	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	47.24	47.24	0.07	0.09	0.15	0.48
OS-10, OS-10.S	12.0%	0.12	A B C or D	0.05 0.07 0.08	0.06 0.09 0.13	0.06 0.15 0.22	0.20 0.48 0.53	10.71	10.71	0.07	0.09	0.15	0.48
OS-11, OS-11.N, OS-11.S	5.2%	0.05	A B C or D	0.02 0.03 0.03	0.02 0.03 0.08	0.02 0.10 0.17	0.15 0.45 0.51	27.19	27.19	0.03	0.03	0.10	0.45
OS-12, OS-12.N, OS-12.S	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	50.13	50.13	0.01	0.01	0.07	0.44
OS-13, OS-13.S	2.0%	0.02	A B C or D	0.01 0.01 0.01	0.01 0.01 0.05	0.01 0.07 0.15	0.13 0.44 0.49	19.99	19.99	0.01	0.01	0.07	0.44

Dawson Trails Boulevard (North) - Future Offsite Basins

CORE Project #: 20-224
Prepared By: CORE

TIME OF CONCENTRATION CALCULATIONS

-REFERENCE UDFCD Vol.1 Section 2.4

NRCS Conveyance factors, K -REFERENCE UDFCD Vol.1 RUNOFF Table 6-2

SF-2	Heavy Meadow	3	Short Grass Pasture & Lawns	7	Grassed Waterway	15
	Tillage/field	5	Nearly Bare Ground	10	Paved Area & Shallow Gutter	20

SUB-BASIN DATA			INITIAL / OVERLAND TIME			CHANNEL / TRAVEL TIME T(t)						T(c) CHECK (URBANIZED BASINS)		FINAL T(c)
DRAIN BASIN	AREA ac.	C(5)	Length ft.	Slope %	T(i) min	Length ft.	Slope %	Coeff.	Velocity fps	T(t) min.	COMP. T(c)	% IMPER-VIOUS	USDCM Eq . 6-5	min.
				0.0			0.0							
OS-9, OS-9.N, OS-9.S	47.24	0.09	500	8.3	20.3	2363	5.3	7	1.6	24.5	44.8	0.1	40.0	40.0
OS-10, OS-10.S	10.72	0.09	500	9.0	19.8	1600	7.0	7	1.9	14.4	34.2	0.1	33.4	33.4
OS-11, OS-11.N, OS-11.S	27.19	0.03	500	10.0	20.1	1900	9.0	7	2.1	15.1	35.2	0.1	36.0	35.2
OS-12, OS-12.N, OS-12.S	50.13	0.01	500	8.5	21.7	1900	9.0	7	2.1	15.1	36.8	0.0	37.0	36.8
OS-13, OS-13.S	19.99	0.01	500	20.0	16.3	1125	11.6	7	2.4	7.9	24.2	0.0	31.6	24.2

Dawson Trails Boulevard (North) - Future Offsite Basins

CORE Project #: 20-224

Prepared By: CORE

RATIONAL METHOD PEAK RUNOFF

5-Year STORM

Rainfall Depth-Duration-Frequency (1-hr) = 1.43

SF-3

-REFERENCE UDFCD Vol.1 EQ 5-1 & EQ 6-1

BASIN INFORMATON				DIRECT RUNOFF			
DESIGN POINT	DRAIN BASIN	AREA ac.	5yr Runoff COEFF	T(c) min	C x A	I in/hr	Q cfs
OS9	OS-9, OS-9.N, OS-9.S	47.24	0.09	40.0	4.05	1.88	7.62
OS10	OS-10, OS-10.S	10.72	0.09	33.4	0.92	2.10	1.93
OS11	OS-11, OS-11.N, OS-11.S	27.19	0.03	35.2	0.94	2.04	1.91
OS12	OS-12, OS-12.N, OS-12.S	50.13	0.01	36.8	0.61	1.98	1.21
OS13	OS-13, OS-13.S	19.99	0.01	24.2	0.24	2.54	0.62

Dawson Trails Boulevard (North) - Future Offsite Basins

CORE Project #: 20-224

Prepared By: CORE

RATIONAL METHOD PEAK RUNOFF

100-YR STORM

SF-3

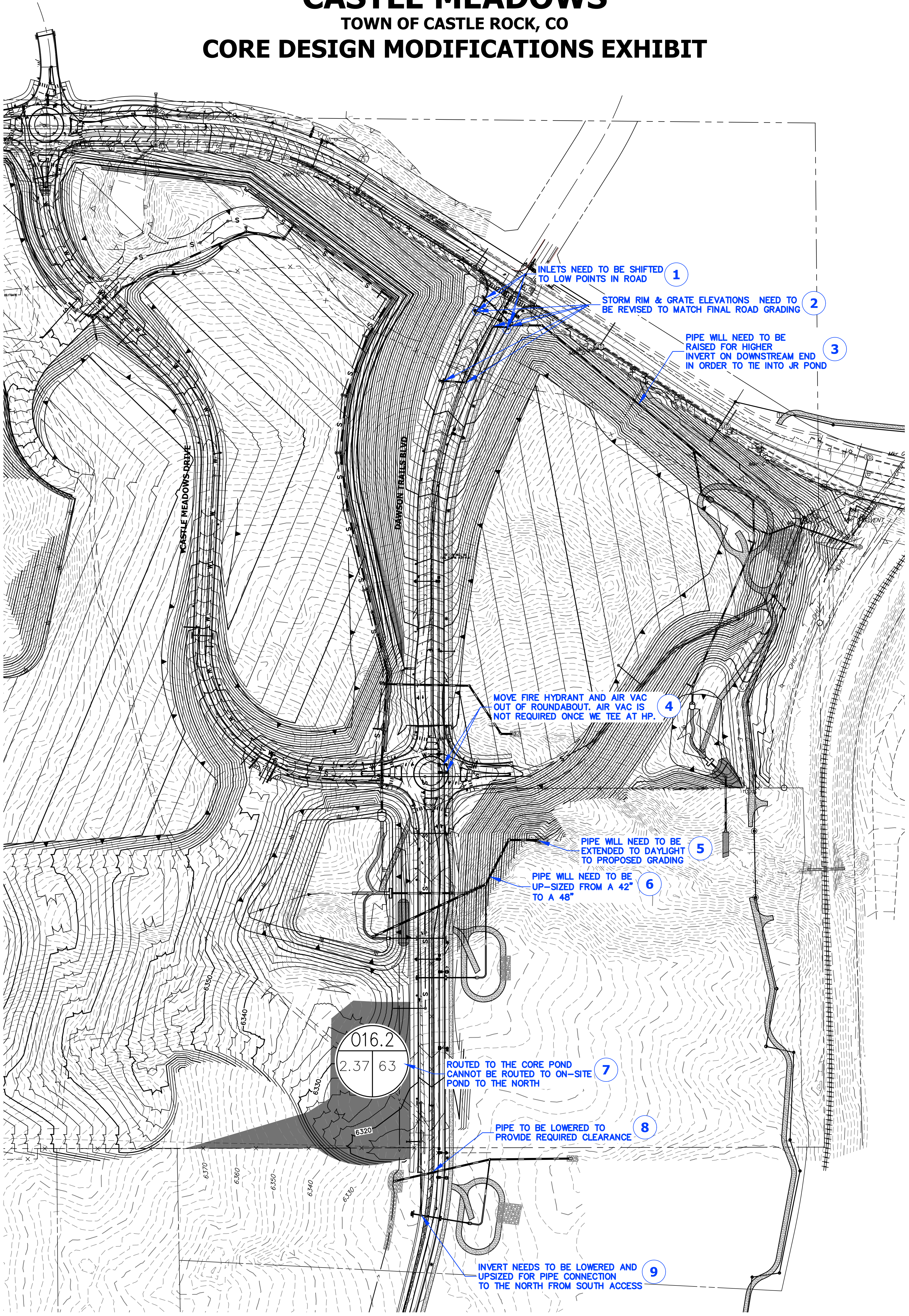
Rainfall Depth-Duration-Frequency (1-hr) = **2.60**

-REFERENCE UDFCD Vol.1 EQ 5-1 & EQ 6-1

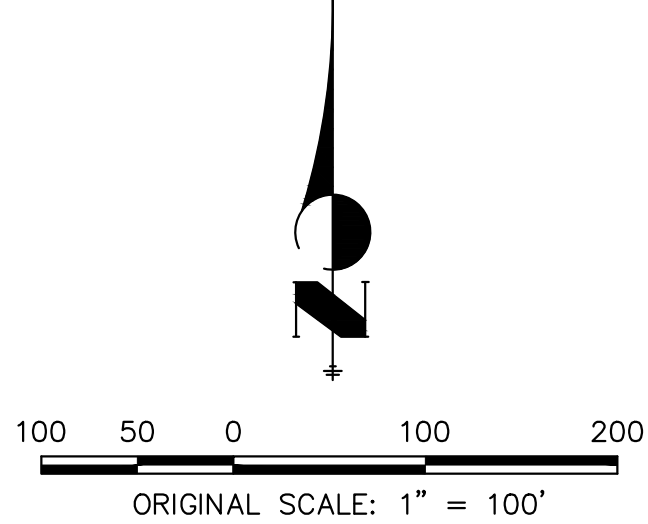
BASIN INFORMATON				DIRECT RUNOFF			
DESIGN POINT	DRAIN BASIN	AREA ac.	100YR RUNNOFF COEFF	T(c) min	C x A	I in/hr	Q cfs
OS9	OS-9, OS-9.N, OS-9.S	47.24	0.48	40.0	22.79	3.42	78.02
OS10	OS-10, OS-10.S	10.72	0.48	33.4	5.17	3.83	19.78
OS11	OS-11, OS-11.N, OS-11.S	27.19	0.45	35.2	12.25	3.71	45.38
OS12	OS-12, OS-12.N, OS-12.S	50.13	0.44	36.8	21.82	3.61	78.76
OS13	OS-13, OS-13.S	19.99	0.44	24.2	8.70	4.61	40.14

RUNOFF SUMMARY TABLE				
DIRECT RUNOFF				
DESIGN POINT	BASIN	AREA (AC)	5-Year RUNOFF (CFS)	100-Year RUNOFF (CFS)
OS9	OS-9, OS-9.N, OS-9.S	47.2	7.6	78.0
OS10	OS-10, OS-10.S	10.7	1.9	19.8
OS11	OS-11, OS-11.N, OS-11.S	27.2	9.5	45.4
OS12	OS-12, OS-12.N, OS-12.S	50.1	1.2	78.8
OS13	OS-13, OS-13.S	20.0	0.6	40.1

CASTLE MEADOWS
TOWN OF CASTLE ROCK, CO
CORE DESIGN MODIFICATIONS EXHIBIT



CORE: PLEASE NOTE, THIS SHEET IS PROVIDED TO SHOW THE FUTURE BASIN O16.2 LINEWORK FROM JR THAT IS INCORPORATED INTO POND A2 CALCULATIONS. SEE NEXT PAGE.



CORE DESIGN MODIFICATIONS EXHIBIT
CASTLE MEADOWS
JOB NO. 15856.25
MAY 21, 2025
SHEET 1 OF 1



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Fort Collins 970-491-9888 • www.jrengineering.com

Dawson Trails Boulevard (North) - Pond A2

CORE Project #: 20-224

Prepared By: ACJ

IMPERVIOUS CALCULATIONS**-REFERENCE UDFCD Vol.1 RUNOFF Table 6-3**

		Residential & Commercial												Soil Type			
		Single Family			Multi-Family									Soil Type A Area	Soil Type B Area	Soil Type C/D Area	
		0.25 acres or less	0.25 - 0.75 acres	2.5 acres or larger	Attached	Roof/ Concrete	Asphalt/ Ponded Water	Packed Gravel	2-7% Slope	>7% Slope	Historic						
		% Imperv.	Design Point	45.00%	30.00%	12.00%	75.00%	90.00%	100.00%	40.00%	2.00%			2.00%	2.00%		
BASIN		Design Point											Total Area (ac)	Percent Impervious			
			Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)	Area (ac)			Area (ac)	Area (ac)	Area (ac)
A-2.1	A2.1	-	-	-	-	0.16	0.57	-	0.17	-	-	0.91	79.7%	-	0.91	-	
A-2.2	A2.2	-	-	-	-	0.16	0.57	-	0.17	-	-	0.91	79.7%	-	0.91	-	
A-2.3	A2.3	-	-	-	-	0.30	1.07	-	0.32	-	-	1.69	79.7%	-	1.69	-	
A-2.4	A2.4	-	-	-	-	0.31	1.07	-	0.32	-	-	1.70	79.6%	-	1.70	-	
A-2.5	A2.5	-	-	-	-	-	0.26	-	0.28	-	-	0.53	48.9%	-	0.53	-	
O16.2	O16.2	-	-	-	-	-	1.48	-	-	-	0.90	2.37	63.0%	-	2.37	-	
Pond A2	Pond A2	-	-	-	-	0.94	5.01	-	1.26	-	0.90	8.11	72.8%	-	8.11	-	
TOTAL		0.00	0.00	0.00	0.00	1.87	10.02	0.00	2.53	0.00	1.79	16.21	72.8%				

APPENDIX B

HYDRAULIC COMPUTATIONS

INLET MANAGEMENT

Worksheet Protected

INLET NAME	Inlet A1-1 Swale	Inlet A1-2	Inlet A2-1 Swale
Site Type (Urban or Rural)	URBAN	URBAN	URBAN
Inlet Application (Street or Area)	AREA	STREET	AREA
Hydraulic Condition	Swale	In Sump	Swale
Inlet Type	CDOT Type C	CDOT Type R Curb Opening	CDOT Type C

USER-DEFINED INPUT**User-Defined Design Flows**

Minor Q_{Known} (cfs)	0.2	6.5	0.04
Major Q_{Known} (cfs)	6.2	13.9	2.9

Bypass (Carry-Over) Flow from Upstream [Inlets must be organized from upstream \(left\) to downstream \(right\) in order for bypass flows to be linked.](#)

Receive Bypass Flow from:	No Bypass Flow Received	No Bypass Flow Received	No Bypass Flow Received
Minor Bypass Flow Received, Q_b (cfs)	0.0	0.0	0.0
Major Bypass Flow Received, Q_b (cfs)	0.0	0.0	0.0

Watershed Characteristics

Subcatchment Area (acres)			
Percent Impervious			
NRCS Soil Type			

Watershed Profile

Overland Slope (ft/ft)			
Overland Length (ft)			
Channel Slope (ft/ft)			
Channel Length (ft)			

Minor Storm Rainfall Input

Design Storm Return Period, T_r (years)			
One-Hour Precipitation, P_1 (inches)			

Major Storm Rainfall Input

Design Storm Return Period, T_r (years)			
One-Hour Precipitation, P_1 (inches)			

CALCULATED OUTPUT

Minor Total Design Peak Flow, Q (cfs)	0.2	6.5	0.0
Major Total Design Peak Flow, Q (cfs)	6.2	13.9	2.9
Minor Flow Bypassed Downstream, Q_b (cfs)	0.0	N/A	0.0
Major Flow Bypassed Downstream, Q_b (cfs)	0.0	N/A	0.0

INLET MANAGEMENT

Worksheet Protected

INLET NAME	Inlet A2-2	Inlet A2-3 Swale	Inlet A2-4
Site Type (Urban or Rural)	URBAN	URBAN	URBAN
Inlet Application (Street or Area)	STREET	AREA	STREET
Hydraulic Condition	In Sump	Swale	In Sump
Inlet Type	CDOT Type R Curb Opening	CDOT Type C	CDOT Type R Curb Opening

USER-DEFINED INPUT**User-Defined Design Flows**

Minor Q_{Known} (cfs)	2.9	0.1	4.7
Major Q_{Known} (cfs)	6.2	4.9	10.2

Bypass (Carry-Over) Flow from Upstream

Receive Bypass Flow from:	No Bypass Flow Received	No Bypass Flow Received	No Bypass Flow Received
Minor Bypass Flow Received, Q_b (cfs)	0.0	0.0	0.0
Major Bypass Flow Received, Q_b (cfs)	0.0	0.0	0.0

Watershed Characteristics

Subcatchment Area (acres)			
Percent Impervious			
NRCS Soil Type			

Watershed Profile

Overland Slope (ft/ft)			
Overland Length (ft)			
Channel Slope (ft/ft)			
Channel Length (ft)			

Minor Storm Rainfall Input

Design Storm Return Period, T_r (years)			
One-Hour Precipitation, P_1 (inches)			

Major Storm Rainfall Input

Design Storm Return Period, T_r (years)			
One-Hour Precipitation, P_1 (inches)			

CALCULATED OUTPUT

Minor Total Design Peak Flow, Q (cfs)	2.9	0.1	4.7
Major Total Design Peak Flow, Q (cfs)	6.2	4.9	10.2
Minor Flow Bypassed Downstream, Q_b (cfs)	N/A	0.0	N/A
Major Flow Bypassed Downstream, Q_b (cfs)	N/A	0.0	N/A

INLET MANAGEMENT

Worksheet Protected

INLET NAME	Inlet A4-1 Swale	Inlet A4-2
Site Type (Urban or Rural)	URBAN	URBAN
Inlet Application (Street or Area)	AREA	STREET
Hydraulic Condition	Swale	On Grade
Inlet Type	CDOT Type C	CDOT Type R Curb Opening

USER-DEFINED INPUT**User-Defined Design Flows**

Minor Q_{Known} (cfs)	3.6	4.5
Major Q_{Known} (cfs)	21.4	16.5

Bypass (Carry-Over) Flow from Upstream

Receive Bypass Flow from:	No Bypass Flow Received	No Bypass Flow Received
Minor Bypass Flow Received, Q_b (cfs)	0.0	0.0
Major Bypass Flow Received, Q_b (cfs)	0.0	0.0

Watershed Characteristics

Subcatchment Area (acres)		
Percent Impervious		
NRCS Soil Type		

Watershed Profile

Overland Slope (ft/ft)		
Overland Length (ft)		
Channel Slope (ft/ft)		
Channel Length (ft)		

Minor Storm Rainfall Input

Design Storm Return Period, T_r (years)		
One-Hour Precipitation, P_1 (inches)		

Major Storm Rainfall Input

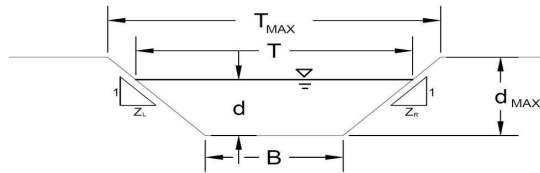
Design Storm Return Period, T_r (years)		
One-Hour Precipitation, P_1 (inches)		

CALCULATED OUTPUT

Minor Total Design Peak Flow, Q (cfs)	3.6	4.5
Major Total Design Peak Flow, Q (cfs)	21.4	16.5
Minor Flow Bypassed Downstream, Q_b (cfs)	1.2	0.0
Major Flow Bypassed Downstream, Q_b (cfs)	15.0	0.1

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A1-1 Swale



This worksheet uses the NRCS vegetal retardance method to determine Manning's n.

For more information see Section 7.2.3 of the USDCM.

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

NRCS Vegetal Retardance (A, B, C, D, or E)

Manning's n (Leave cell D16 blank to manually enter an n value)

Channel Invert Slope

Bottom Width

Left Side Slope

Right Side Slope

Check one of the following soil types:

Soil Type:	Max. Velocity (V _{MAX})	Max Froude No. (F _{MAX})
Non-Cohesive	5.0 fps	0.60
Cohesive	7.0 fps	0.80
Paved	N/A	N/A

A, B, C, D, or E =

n =	0.020
S ₀ =	0.0005 ft/ft
B =	0.00 ft
Z ₁ =	23.10 ft/ft
Z ₂ =	50.00 ft/ft

Choose One:

- ☐ Non-Cohesive
☐ Cohesive
☐ Paved

Maximum Allowable Top Width of Channel for Minor & Major Storm
Maximum Allowable Water Depth in Channel for Minor & Major Storm

	Minor Storm	Major Storm	
T _{MAX} =	38.20	38.20	ft
d _{MAX} =	0.74	0.74	ft

Maximum Channel Capacity Based On Allowable Top Width

Maximum Allowable Top Width

Water Depth

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
T _{MAX} =	38.20	38.20	ft
d =	0.52	0.52	ft
A =	9.98	9.98	sq ft
P =	38.22	38.22	ft
R =	0.26	0.26	ft
n =	0.020	0.020	
V =	0.68	0.68	fps
VR =	0.18	0.18	ft ² /s
D =	0.26	0.26	ft
Fr =	0.23	0.23	
Q _T =	6.8	6.8	cfs

Maximum Channel Capacity Based On Allowable Water Depth

Maximum Allowable Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
d _{MAX} =	0.74	0.74	ft
T =	54.09	54.09	ft
A =	20.01	20.01	sq ft
P =	54.11	54.11	ft
R =	0.37	0.37	ft
n =	0.020	0.020	
V =	0.86	0.86	fps
VR =	0.32	0.32	ft ² /s
D =	0.37	0.37	ft
Fr =	0.25	0.25	
Q _d =	17.2	17.2	cfs

Allowable Channel Capacity Based On Channel Geometry

MINOR STORM Allowable Capacity is based on Top Width Criterion

MAJOR STORM Allowable Capacity is based on Top Width Criterion

Water Depth in Channel Based On Design Peak Flow

Design Peak Flow

Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

	Minor Storm	Major Storm	
Q _o =	0.2	6.2	cfs
d =	0.14	0.50	ft
T =	10.19	36.91	ft
A =	0.71	9.32	sq ft
P =	10.19	36.93	ft
R =	0.07	0.25	ft
n =	0.020	0.020	
V =	0.28	0.67	fps
VR =	0.02	0.17	ft ² /s
D =	0.07	0.25	ft
Fr =	0.19	0.23	

Minor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Major storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A1-1 Swale

Inlet Design Information (Input)

Type of Inlet

CDOT Type C

Inlet Type =

CDOT Type C

Angle of Inclined Grate (must be ≤ 30 degrees)

Width of Grate

Length of Grate

Open Area Ratio

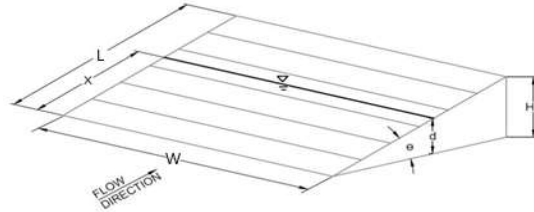
Height of Inclined Grate

Clogging Factor

Grate Discharge Coefficient

Orifice Coefficient

Weir Coefficient

 $\theta =$ 0.00 degrees

W = 3.00 ft

L = 3.00 ft

 $A_{\text{RATIO}} =$ 0.70 $H_B =$ 0.00 ft $C_f =$ 0.50 $C_d =$ 0.96 $C_o =$ 0.64 $C_w =$ 2.05

Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)

	MINOR	MAJOR
d =	0.14	0.50

Grate Capacity as a Weir

Submerged Side Weir Length

Inclined Side Weir Flow

Base Weir Flow

Interception Without Clogging

Interception With Clogging

	MINOR	MAJOR	
X =	3.00	3.00	ft
$Q_{ws} =$	0.6	3.9	cfs
$Q_{wb} =$	0.8	5.5	cfs
$Q_{wi} =$	1.9	13.3	cfs
$Q_{wa} =$	1.0	6.6	cfs

Grate Capacity as an Orifice

Interception Without Clogging

Interception With Clogging

	MINOR	MAJOR	
$Q_{oi} =$	12.1	23.0	cfs
$Q_{oa} =$	6.0	11.5	cfs

Total Inlet Interception Capacity (assumes clogged condition)

Bypassed Flow

Capture Percentage = Q_a/Q_o

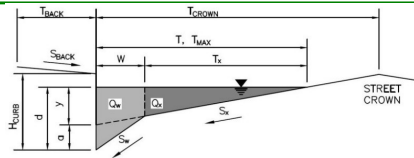
	MINOR	MAJOR	
$Q_a =$	1.0	6.6	cfs
$Q_b =$	0.0	0.0	cfs
$C\% =$	100	100	%

ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

Project: Dawson Trails Boulevard North

Inlet ID: Inlet A1-2

**Gutter Geometry:**

Maximum Allowable Width for Spread Behind Curb
 Side Slope Behind Curb (leave blank for no conveyance credit behind curb)
 Manning's Roughness Behind Curb (typically between 0.012 and 0.020)

$T_{BACK} = 21.0$ ft
 $S_{BACK} = 0.020$ ft/ft
 $n_{BACK} = 0.020$

Height of Curb at Gutter Flow Line
 Distance from Curb Face to Street Crown
 Gutter Width
 Street Transverse Slope
 Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)
 Street Longitudinal Slope - Enter 0 for sump condition
 Manning's Roughness for Street Section (typically between 0.012 and 0.020)

$H_{CURB} = 6.00$ inches
 $T_{CROWN} = 34.5$ ft
 $W = 2.00$ ft
 $S_x = 0.020$ ft/ft
 $S_w = 0.083$ ft/ft
 $S_o = 0.000$ ft/ft
 $n_{STREET} = 0.016$

Max. Allowable Spread for Minor & Major Storm
 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm
 Check boxes are not applicable in SUMP conditions

	Minor Storm	Major Storm	
$T_{MAX} =$	24.5	34.5	ft
$d_{MAX} =$	6.0	11.0	inches

☐ ☐

Maximum Capacity for 1/2 Street based On Allowable Spread

Water Depth without Gutter Depression ($T * S_x * 12$)
 Vertical Depth between Gutter Lip and Gutter Flowline ($W * S_w * 12$)
 Gutter Depression ($d_c - (W * S_x * 12)$)
 Water Depth at Gutter Flowline ($y + a$)
 Allowable Spread for Discharge outside the Gutter Section ($T - W$)
 Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)
 Discharge outside the Gutter Section, carried in Section T_x
 Discharge within the Gutter Section ($Q_T - Q_X - Q_{BACK}$)
 Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)
 Maximum Flow Based On Allowable Spread
 Flow Velocity within the Gutter Section
 $V*d$ Product: Flow Velocity times Gutter Flowline Depth

	Minor Storm	Major Storm	
$y =$	5.88	8.28	inches
$d_c =$	2.0	2.0	inches
$a =$	1.51	1.51	inches
$d =$	7.39	9.79	inches
$T_x =$	22.5	32.5	ft
$E_o =$	0.240	0.167	
$Q_X =$	0.0	0.0	cfs
$Q_W =$	0.0	0.0	cfs
$Q_{BACK} =$	0.0	0.0	cfs
$Q_T =$	SUMP	SUMP	cfs
$V =$	0.0	0.0	fps
$V*d =$	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth

Theoretical Water Spread
 Theoretical Spread for Discharge outside the Gutter Section ($T - W$)
 Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)
 Theoretical Discharge outside the Gutter Section, carried in Section T_{XTH}
 Actual Discharge outside the Gutter Section, (limited by distance T_{CROWN})
 Discharge within the Gutter Section ($Q_d - Q_X$)
 Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)
 Total Discharge for Major & Minor Storm (Pre-Safety Factor)
 Average Flow Velocity Within the Gutter Section
 $V*d$ Product: Flow Velocity Times Gutter Flowline Depth
 Slope-Based Safety Factor for Minor/Major Storm depth reduction, $d \geq 6"$
 Max Flow based on Allowable Depth (Safety Factor Applied)
 Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)
 Resultant Flow Depth at Street Crown (Safety Factor Applied)

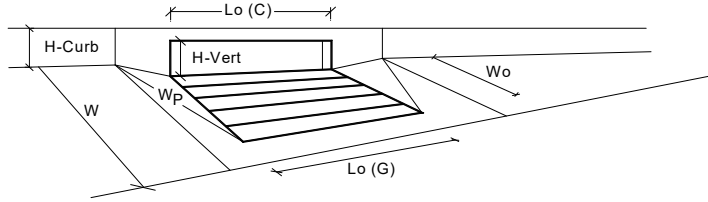
	Minor Storm	Major Storm	
$T_{TH} =$	18.7	39.5	ft
$T_{XTH} =$	16.7	37.5	ft
$E_o =$	0.318	0.145	
$Q_{XTH} =$	0.0	0.0	cfs
$Q_X =$	0.0	0.0	cfs
$Q_W =$	0.0	0.0	cfs
$Q_{BACK} =$	0.0	0.0	cfs
$Q =$	SUMP	SUMP	cfs
$V =$	0.0	0.0	fps
$V*d =$	0.0	0.0	
$R =$	SUMP	SUMP	
$Q_d =$	SUMP	SUMP	cfs
$d =$			inches
$d_{CROWN} =$			inches

MINOR STORM Allowable Capacity is not applicable to Sump Condition
 MAJOR STORM Allowable Capacity is not applicable to Sump Condition

	Minor Storm	Major Storm	
$Q_{allow} =$	SUMP	SUMP	cfs

INLET IN A SUMP OR SAG LOCATION

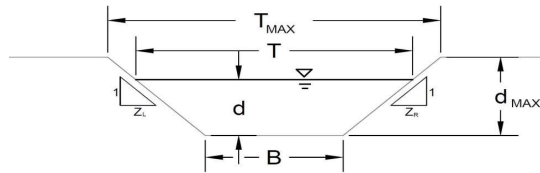
MHFD-Inlet, Version 5.02 (August 2022)



Design Information (Input)		MINOR		MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)		a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)		N_o =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	6.0	9.8	inches
Grate Information		MINOR		MAJOR	
Length of a Unit Grate		$L_o(G)$ =	N/A	N/A	feet
Width of a Unit Grate		W_o =	N/A	N/A	feet
Open Area Ratio for a Grate (typical values 0.15-0.90)		A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)		$C_f(G)$ =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)		$C_w(G)$ =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)		$C_o(G)$ =	N/A	N/A	
Curb Opening Information		MINOR		MAJOR	
Length of a Unit Curb Opening		$L_o(C)$ =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches		H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches		H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)		Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)		W_o =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)		$C_f(C)$ =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)		$C_w(C)$ =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)		$C_o(C)$ =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR		MAJOR	
Clogging Coefficient for Multiple Units		Coef =	N/A	N/A	
Clogging Factor for Multiple Units		Clog =	N/A	N/A	
Grate Capacity as a Weir (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{wi} =	N/A	N/A	cfs
Interception with Clogging		Q_{wa} =	N/A	N/A	cfs
Grate Capacity as an Orifice (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{oi} =	N/A	N/A	cfs
Interception with Clogging		Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR		MAJOR	
Interception without Clogging		Q_{mi} =	N/A	N/A	cfs
Interception with Clogging		Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)		Q_{Grate} =	N/A	N/A	cfs
Curb Opening Flow Analysis (Calculated)		MINOR		MAJOR	
Clogging Coefficient for Multiple Units		Coef =	1.25	1.25	
Clogging Factor for Multiple Units		Clog =	0.06	0.06	
Curb Capacity as a Weir (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{wi} =	8.8	25.7	cfs
Interception with Clogging		Q_{wa} =	8.3	24.1	cfs
Curb Capacity as an Orifice (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{oi} =	19.5	24.7	cfs
Interception with Clogging		Q_{oa} =	18.3	23.1	cfs
Curb Opening Capacity as Mixed Flow		MINOR		MAJOR	
Interception without Clogging		Q_{mi} =	12.2	23.4	cfs
Interception with Clogging		Q_{ma} =	11.4	21.9	cfs
Resulting Curb Opening Capacity (assumes clogged condition)		Q_{Curb} =	8.3	21.9	cfs
Resultant Street Conditions		MINOR		MAJOR	
Total Inlet Length		L =	10.00	10.00	feet
Resultant Street Flow Spread (based on street geometry from above)		T =	18.7	34.5	ft
Resultant Flow Depth at Street Crown		d_{CROWN} =	0.0	0.0	inches
Low Head Performance Reduction (Calculated)		MINOR		MAJOR	
Depth for Grate Midwidth		d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation		d_{Curb} =	0.33	0.65	ft
Grated Inlet Performance Reduction Factor for Long Inlets		RF_{Grate} =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets		RF_{Curb} =	0.93	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets		$RF_{Combination}$ =	N/A	N/A	
Total Inlet Interception Capacity (assumes clogged condition)		MINOR		MAJOR	
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)		Q_s =	8.3	21.9	cfs
$Q_{PEAK REQUIRED}$ =			6.5	13.9	cfs

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A2-1 Swale



This worksheet uses the NRCS vegetal retardance method to determine Manning's n.

For more information see Section 7.2.3 of the USDCM.

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

NRCS Vegetal Retardance (A, B, C, D, or E)

Manning's n (Leave cell D16 blank to manually enter an n value)

Channel Invert Slope

Bottom Width

Left Side Slope

Right Side Slope

Check one of the following soil types:

Soil Type:	Max. Velocity (V_{MAX})	Max Froude No. (F_{MAX})
Non-Cohesive	5.0 fps	0.60
Cohesive	7.0 fps	0.80
Paved	N/A	N/A

A, B, C, D, or E =

n =	0.020
S_o =	0.0005 ft/ft
B =	0.00 ft
Z1 =	23.10 ft/ft
Z2 =	50.00 ft/ft

Choose One:

- ☐ Non-Cohesive
☐ Cohesive
☐ Paved

Maximum Allowable Top Width of Channel for Minor & Major Storm
Maximum Allowable Water Depth in Channel for Minor & Major Storm

	Minor Storm	Major Storm	
T_{MAX} =	34.50	34.50	ft
d_{MAX} =	0.90	0.90	ft

Maximum Channel Capacity Based On Allowable Top Width

Maximum Allowable Top Width

Water Depth

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
T_{MAX} =	34.50	34.50	ft
d =	0.47	0.47	ft
A =	8.14	8.14	sq ft
P =	34.51	34.51	ft
R =	0.24	0.24	ft
n =	0.020	0.020	
V =	0.64	0.64	fps
VR =	0.15	0.15	ft ² /s
D =	0.24	0.24	ft
Fr =	0.23	0.23	
Q_T =	5.2	5.2	cfs

Maximum Channel Capacity Based On Allowable Water Depth

Maximum Allowable Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
d_{MAX} =	0.90	0.90	ft
T =	65.79	65.79	ft
A =	29.60	29.60	sq ft
P =	65.81	65.81	ft
R =	0.45	0.45	ft
n =	0.020	0.020	
V =	0.98	0.98	fps
VR =	0.44	0.44	ft ² /s
D =	0.45	0.45	ft
Fr =	0.26	0.26	
Q_d =	29.0	29.0	cfs

Allowable Channel Capacity Based On Channel Geometry

MINOR STORM Allowable Capacity is based on Top Width Criterion

MAJOR STORM Allowable Capacity is based on Top Width Criterion

Water Depth in Channel Based On Design Peak Flow

Design Peak Flow

Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

	Minor Storm	Major Storm	
Q_o =	0.0	2.9	cfs
d =	0.08	0.38	ft
T =	5.59	27.76	ft
A =	0.21	5.27	sq ft
P =	5.59	27.77	ft
R =	0.04	0.19	ft
n =	0.020	0.020	
V =	0.19	0.55	fps
VR =	0.01	0.10	ft ² /s
D =	0.04	0.19	ft
Fr =	0.17	0.22	

Minor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Major storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A2-1 Swale

Inlet Design Information (Input)

Type of Inlet

CDOT Type C

Inlet Type =

CDOT Type C

Angle of Inclined Grate (must be ≤ 30 degrees) $\theta = 0.00$ degrees

Width of Grate

 $W = 3.00$ ft

Length of Grate

 $L = 3.00$ ft

Open Area Ratio

 $A_{\text{RATIO}} = 0.70$

Height of Inclined Grate

 $H_B = 0.00$ ft

Clogging Factor

 $C_f = 0.50$

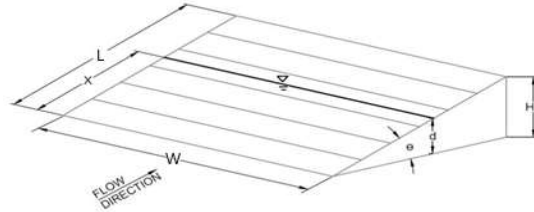
Grate Discharge Coefficient

 $C_d = 0.96$

Orifice Coefficient

 $C_o = 0.64$

Weir Coefficient

 $C_w = 2.05$ 

Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)

	MINOR	MAJOR
$d =$	0.08	0.38

Grate Capacity as a Weir

Submerged Side Weir Length

 $X = 3.00$ ft

Inclined Side Weir Flow

 $Q_{ws} = 0.2$ cfs

Base Weir Flow

 $Q_{wb} = 0.3$ cfs

Interception Without Clogging

 $Q_{wi} = 0.8$ cfs

Interception With Clogging

 $Q_{wa} = 0.4$ cfs

Grate Capacity as an Orifice

Interception Without Clogging

 $Q_{oi} = 8.9$ cfs

Interception With Clogging

 $Q_{oa} = 4.5$ cfs

Total Inlet Interception Capacity (assumes clogged condition)

 $Q_a = 0.4$ cfs

Bypassed Flow

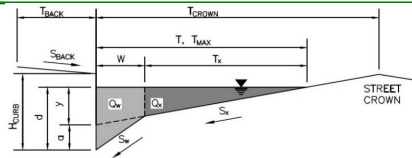
 $Q_b = 0.0$ cfsCapture Percentage = Q_a/Q_o $C\% = 100$ %

ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

Project: Dawson Trails Boulevard North

Inlet ID: Inlet A2-2

**Gutter Geometry:**

Maximum Allowable Width for Spread Behind Curb

Side Slope Behind Curb (leave blank for no conveyance credit behind curb)

Manning's Roughness Behind Curb (typically between 0.012 and 0.020)

Height of Curb at Gutter Flow Line

Distance from Curb Face to Street Crown

Gutter Width

Street Transverse Slope

Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)

Street Longitudinal Slope - Enter 0 for sump condition

Manning's Roughness for Street Section (typically between 0.012 and 0.020)

$T_{BACK} =$	21.0	ft
$S_{BACK} =$	0.020	ft/ft
$n_{BACK} =$	0.020	

$H_{CURB} =$	6.00	inches
$T_{CROWN} =$	34.5	ft
$W =$	2.00	ft
$S_x =$	0.020	ft/ft
$S_w =$	0.083	ft/ft
$S_o =$	0.000	ft/ft
$n_{STREET} =$	0.016	

Max. Allowable Spread for Minor & Major Storm

Max. Allowable Depth at Gutter Flowline for Minor & Major Storm

Check boxes are not applicable in SUMP conditions

	Minor Storm	Major Storm	
$T_{MAX} =$	24.5	34.5	ft
$d_{MAX} =$	6.0	11.0	inches
	☐	☐	

Maximum Capacity for 1/2 Street based On Allowable SpreadWater Depth without Gutter Depression ($T * S_x * 12$)Vertical Depth between Gutter Lip and Gutter Flowline ($W * S_w * 12$)Gutter Depression ($d_c - (W * S_x * 12)$)Water Depth at Gutter Flowline ($y + a$)Allowable Spread for Discharge outside the Gutter Section ($T - W$)

Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)

Discharge outside the Gutter Section, carried in Section T_x Discharge within the Gutter Section ($Q_T - Q_x - Q_{BACK}$)

Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)

Maximum Flow Based On Allowable Spread

Flow Velocity within the Gutter Section

 $V*d$ Product: Flow Velocity times Gutter Flowline Depth

	Minor Storm	Major Storm	
$y =$	5.88	8.28	inches
$d_c =$	2.0	2.0	inches
$a =$	1.51	1.51	inches
$d =$	7.39	9.79	inches
$T_x =$	22.5	32.5	ft
$E_o =$	0.240	0.167	
$Q_x =$	0.0	0.0	cfs
$Q_w =$	0.0	0.0	cfs
$Q_{BACK} =$	0.0	0.0	cfs
$Q_r =$	SUMP	SUMP	cfs
$V =$	0.0	0.0	fps
$V*d =$	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth

Theoretical Water Spread

Theoretical Spread for Discharge outside the Gutter Section ($T - W$)

Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)

Theoretical Discharge outside the Gutter Section, carried in Section T_{xTH} Actual Discharge outside the Gutter Section, (limited by distance T_{CROWN})Discharge within the Gutter Section ($Q_d - Q_x$)

Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)

Total Discharge for Major & Minor Storm (Pre-Safety Factor)

Average Flow Velocity Within the Gutter Section

 $V*d$ Product: Flow Velocity Times Gutter Flowline DepthSlope-Based Safety Factor for Minor/Major Storm depth reduction, $d \geq 6"$

Max Flow based on Allowable Depth (Safety Factor Applied)

Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)

Resultant Flow Depth at Street Crown (Safety Factor Applied)

	Minor Storm	Major Storm	
$T_{TH} =$	18.7	39.5	ft
$T_{xTH} =$	16.7	37.5	ft
$E_o =$	0.318	0.145	
$Q_{xTH} =$	0.0	0.0	cfs
$Q_x =$	0.0	0.0	cfs
$Q_w =$	0.0	0.0	cfs
$Q_{BACK} =$	0.0	0.0	cfs
$Q =$	SUMP	SUMP	cfs
$V =$	0.0	0.0	fps
$V*d =$	0.0	0.0	
$R =$	SUMP	SUMP	
$Q_d =$	SUMP	SUMP	cfs
$d =$			inches
$d_{CROWN} =$			inches

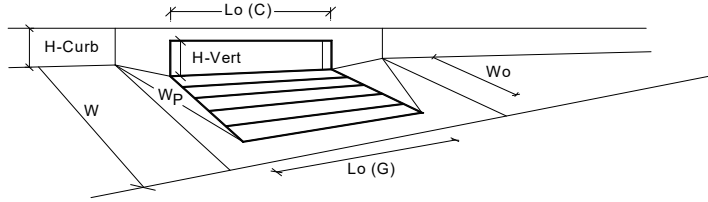
MINOR STORM Allowable Capacity is not applicable to Sump Condition

MAJOR STORM Allowable Capacity is not applicable to Sump Condition

	Minor Storm	Major Storm	
$Q_{allow} =$	SUMP	SUMP	cfs

INLET IN A SUMP OR SAG LOCATION

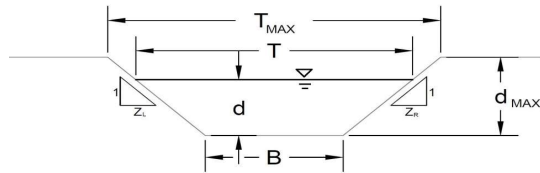
MHFD-Inlet, Version 5.02 (August 2022)



Design Information (Input)		MINOR		MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)		a_{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)		N_o =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	6.0	9.8	inches
Grate Information		MINOR		MAJOR	
Length of a Unit Grate		$L_o(G)$ =	N/A	N/A	feet
Width of a Unit Grate		W_o =	N/A	N/A	feet
Open Area Ratio for a Grate (typical values 0.15-0.90)		A_{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)		$C_f(G)$ =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)		$C_w(G)$ =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)		$C_o(G)$ =	N/A	N/A	
Curb Opening Information		MINOR		MAJOR	
Length of a Unit Curb Opening		$L_o(C)$ =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches		H_{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches		H_{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)		Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)		W_o =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)		$C_f(C)$ =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)		$C_w(C)$ =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)		$C_o(C)$ =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR		MAJOR	
Clogging Coefficient for Multiple Units		Coef =	N/A	N/A	
Clogging Factor for Multiple Units		Clog =	N/A	N/A	
Grate Capacity as a Weir (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{wi} =	N/A	N/A	cfs
Interception with Clogging		Q_{wa} =	N/A	N/A	cfs
Grate Capacity as an Orifice (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{oi} =	N/A	N/A	cfs
Interception with Clogging		Q_{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR		MAJOR	
Interception without Clogging		Q_{mi} =	N/A	N/A	cfs
Interception with Clogging		Q_{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)		Q_{Grate} =	N/A	N/A	cfs
Curb Opening Flow Analysis (Calculated)		MINOR		MAJOR	
Clogging Coefficient for Multiple Units		Coef =	1.25	1.25	
Clogging Factor for Multiple Units		Clog =	0.06	0.06	
Curb Capacity as a Weir (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{wi} =	8.8	25.7	cfs
Interception with Clogging		Q_{wa} =	8.3	24.1	cfs
Curb Capacity as an Orifice (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q_{oi} =	19.5	24.7	cfs
Interception with Clogging		Q_{oa} =	18.3	23.1	cfs
Curb Opening Capacity as Mixed Flow		MINOR		MAJOR	
Interception without Clogging		Q_{mi} =	12.2	23.4	cfs
Interception with Clogging		Q_{ma} =	11.4	21.9	cfs
Resulting Curb Opening Capacity (assumes clogged condition)		Q_{Curb} =	8.3	21.9	cfs
Resultant Street Conditions		MINOR		MAJOR	
Total Inlet Length		L =	10.00	10.00	feet
Resultant Street Flow Spread (based on street geometry from above)		T =	18.7	34.5	ft
Resultant Flow Depth at Street Crown		d_{CROWN} =	0.0	0.0	inches
Low Head Performance Reduction (Calculated)		MINOR		MAJOR	
Depth for Grate Midwidth		d_{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation		d_{Curb} =	0.33	0.65	ft
Grated Inlet Performance Reduction Factor for Long Inlets		RF_{Grate} =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets		RF_{Curb} =	0.93	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets		$RF_{Combination}$ =	N/A	N/A	
Total Inlet Interception Capacity (assumes clogged condition)		MINOR		MAJOR	
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)		Q_s =	8.3	21.9	cfs
$Q_{PEAK REQUIRED}$ =			2.9	6.2	cfs

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A2-3 Swale



This worksheet uses the NRCS vegetal retardance method to determine Manning's n.

For more information see Section 7.2.3 of the USDCM.

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

NRCS Vegetal Retardance (A, B, C, D, or E)

Manning's n (Leave cell D16 blank to manually enter an n value)

Channel Invert Slope

Bottom Width

Left Side Slope

Right Side Slope

Check one of the following soil types:

Soil Type:	Max. Velocity (V_{MAX})	Max Froude No. (F_{MAX})
Non-Cohesive	5.0 fps	0.60
Cohesive	7.0 fps	0.80
Paved	N/A	N/A

A, B, C, D, or E =

n =	0.020
S_o =	0.0005 ft/ft
B =	0.00 ft
Z1 =	23.10 ft/ft
Z2 =	50.00 ft/ft

Choose One:

- ☐ Non-Cohesive
☒ Cohesive
☐ Paved

Maximum Allowable Top Width of Channel for Minor & Major Storm

Maximum Allowable Water Depth in Channel for Minor & Major Storm

	Minor Storm	Major Storm	
T_{MAX} =	34.50	34.50	ft
d_{MAX} =	0.90	0.90	ft

Maximum Channel Capacity Based On Allowable Top Width

Maximum Allowable Top Width

Water Depth

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
T_{MAX} =	34.50	34.50	ft
d =	0.47	0.47	ft
A =	8.14	8.14	sq ft
P =	34.51	34.51	ft
R =	0.24	0.24	ft
n =	0.020	0.020	
V =	0.64	0.64	fps
VR =	0.15	0.15	ft ² /s
D =	0.24	0.24	ft
Fr =	0.23	0.23	
Q_T =	5.2	5.2	cfs

Maximum Channel Capacity Based On Allowable Water Depth

Maximum Allowable Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
d_{MAX} =	0.90	0.90	ft
T =	65.79	65.79	ft
A =	29.60	29.60	sq ft
P =	65.81	65.81	ft
R =	0.45	0.45	ft
n =	0.020	0.020	
V =	0.98	0.98	fps
VR =	0.44	0.44	ft ² /s
D =	0.45	0.45	ft
Fr =	0.26	0.26	
Q_d =	29.0	29.0	cfs

Allowable Channel Capacity Based On Channel Geometry

MINOR STORM Allowable Capacity is based on Top Width Criterion

MAJOR STORM Allowable Capacity is based on Top Width Criterion

Water Depth in Channel Based On Design Peak Flow

Design Peak Flow

Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

	Minor Storm	Major Storm	
Q_o =	0.1	4.9	cfs
d =	0.11	0.46	ft
T =	7.88	33.79	ft
A =	0.42	7.81	sq ft
P =	7.88	33.81	ft
R =	0.05	0.23	ft
n =	0.020	0.020	
V =	0.24	0.63	fps
VR =	0.01	0.14	ft ² /s
D =	0.05	0.23	ft
Fr =	0.18	0.23	

Minor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Major storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A2-3 Swale

Inlet Design Information (Input)

Type of Inlet

CDOT Type C

Inlet Type =

CDOT Type C

Angle of Inclined Grate (must be ≤ 30 degrees) $\theta = 0.00$ degrees

Width of Grate

 $W = 3.00$ ft

Length of Grate

 $L = 3.00$ ft

Open Area Ratio

 $A_{\text{RATIO}} = 0.70$

Height of Inclined Grate

 $H_B = 0.00$ ft

Clogging Factor

 $C_f = 0.50$

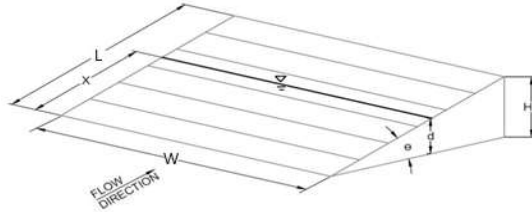
Grate Discharge Coefficient

 $C_d = 0.96$

Orifice Coefficient

 $C_o = 0.64$

Weir Coefficient

 $C_w = 2.05$ 

Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)

	MINOR	MAJOR
$d =$	0.11	0.46

Grate Capacity as a Weir

Submerged Side Weir Length

 $X = 3.00$ ft

Inclined Side Weir Flow

 $Q_{ws} = 0.4$ cfs

Base Weir Flow

 $Q_{wb} = 0.5$ cfs

Interception Without Clogging

 $Q_{wi} = 1.3$ cfs

Interception With Clogging

 $Q_{wa} = 0.7$ cfs

Grate Capacity as an Orifice

Interception Without Clogging

 $Q_{oi} = 10.6$ cfs

Interception With Clogging

 $Q_{oa} = 5.3$ cfs

Total Inlet Interception Capacity (assumes clogged condition)

 $Q_a = 0.7$ cfs

Bypassed Flow

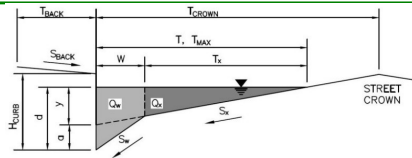
 $Q_b = 0.0$ cfsCapture Percentage = Q_a/Q_o $C\% = 100$ %

ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

Project: Dawson Trails Boulevard North

Inlet ID: Inlet A2-4

**Gutter Geometry:**

Maximum Allowable Width for Spread Behind Curb

Side Slope Behind Curb (leave blank for no conveyance credit behind curb)

Manning's Roughness Behind Curb (typically between 0.012 and 0.020)

Height of Curb at Gutter Flow Line

Distance from Curb Face to Street Crown

Gutter Width

Street Transverse Slope

Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)

Street Longitudinal Slope - Enter 0 for sump condition

Manning's Roughness for Street Section (typically between 0.012 and 0.020)

$T_{BACK} =$	21.0	ft
$S_{BACK} =$	0.020	ft/ft
$n_{BACK} =$	0.020	

$H_{CURB} =$	6.00	inches
$T_{CROWN} =$	34.5	ft
$W =$	2.00	ft
$S_x =$	0.020	ft/ft
$S_w =$	0.083	ft/ft
$S_o =$	0.000	ft/ft
$n_{STREET} =$	0.016	

Max. Allowable Spread for Minor & Major Storm

Max. Allowable Depth at Gutter Flowline for Minor & Major Storm

Check boxes are not applicable in SUMP conditions

	Minor Storm	Major Storm	
$T_{MAX} =$	24.5	34.5	ft
$d_{MAX} =$	6.0	11.0	inches
	☐	☐	

Maximum Capacity for 1/2 Street based On Allowable SpreadWater Depth without Gutter Depression ($T * S_x * 12$)Vertical Depth between Gutter Lip and Gutter Flowline ($W * S_w * 12$)Gutter Depression ($d_c - (W * S_x * 12)$)Water Depth at Gutter Flowline ($y + a$)Allowable Spread for Discharge outside the Gutter Section ($T - W$)

Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)

Discharge outside the Gutter Section, carried in Section T_x Discharge within the Gutter Section ($Q_T - Q_X - Q_{BACK}$)

Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)

Maximum Flow Based On Allowable Spread

Flow Velocity within the Gutter Section

 $V*d$ Product: Flow Velocity times Gutter Flowline Depth

	Minor Storm	Major Storm	
$y =$	5.88	8.28	inches
$d_c =$	2.0	2.0	inches
$a =$	1.51	1.51	inches
$d =$	7.39	9.79	inches
$T_x =$	22.5	32.5	ft
$E_o =$	0.240	0.167	
$Q_X =$	0.0	0.0	cfs
$Q_W =$	0.0	0.0	cfs
$Q_{BACK} =$	0.0	0.0	cfs
$Q_T =$	SUMP	SUMP	cfs
$V =$	0.0	0.0	fps
$V*d =$	0.0	0.0	

Maximum Capacity for 1/2 Street based on Allowable Depth

Theoretical Water Spread

Theoretical Spread for Discharge outside the Gutter Section ($T - W$)

Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)

Theoretical Discharge outside the Gutter Section, carried in Section T_{XTH} Actual Discharge outside the Gutter Section, (limited by distance T_{CROWN})Discharge within the Gutter Section ($Q_d - Q_X$)

Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)

Total Discharge for Major & Minor Storm (Pre-Safety Factor)

Average Flow Velocity Within the Gutter Section

 $V*d$ Product: Flow Velocity Times Gutter Flowline DepthSlope-Based Safety Factor for Minor/Major Storm depth reduction, $d \geq 6"$

Max Flow based on Allowable Depth (Safety Factor Applied)

Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)

Resultant Flow Depth at Street Crown (Safety Factor Applied)

	Minor Storm	Major Storm	
$T_{TH} =$	18.7	39.5	ft
$T_{XTH} =$	16.7	37.5	ft
$E_o =$	0.318	0.145	
$Q_{XTH} =$	0.0	0.0	cfs
$Q_X =$	0.0	0.0	cfs
$Q_W =$	0.0	0.0	cfs
$Q_{BACK} =$	0.0	0.0	cfs
$Q =$	SUMP	SUMP	cfs
$V =$	0.0	0.0	fps
$V*d =$	0.0	0.0	
$R =$	SUMP	SUMP	
$Q_d =$	SUMP	SUMP	cfs
$d =$			inches
$d_{CROWN} =$			inches

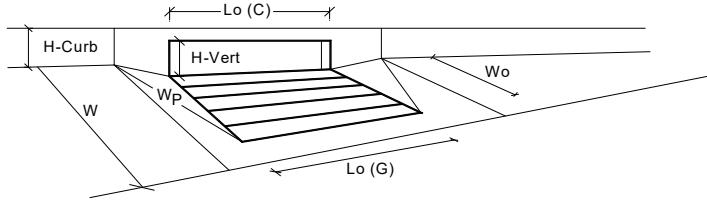
MINOR STORM Allowable Capacity is not applicable to Sump Condition

MAJOR STORM Allowable Capacity is not applicable to Sump Condition

	Minor Storm	Major Storm	
$Q_{allow} =$	SUMP	SUMP	cfs

INLET IN A SUMP OR SAG LOCATION

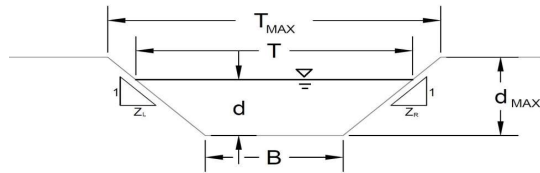
MHFD-Inlet, Version 5.02 (August 2022)



Design Information (Input)		MINOR		MAJOR	
Type of Inlet	CDOT Type R Curb Opening	Type =	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a' from above)		a _{local} =	3.00	3.00	inches
Number of Unit Inlets (Grate or Curb Opening)		No =	1	1	
Water Depth at Flowline (outside of local depression)		Ponding Depth =	6.0	9.8	inches
Grate Information		MINOR		MAJOR	
Length of a Unit Grate		L _o (G) =	N/A	N/A	feet
Width of a Unit Grate		W _o =	N/A	N/A	feet
Open Area Ratio for a Grate (typical values 0.15-0.90)		A _{ratio} =	N/A	N/A	
Clogging Factor for a Single Grate (typical value 0.50 - 0.70)		C _f (G) =	N/A	N/A	
Grate Weir Coefficient (typical value 2.15 - 3.60)		C _w (G) =	N/A	N/A	
Grate Orifice Coefficient (typical value 0.60 - 0.80)		C _o (G) =	N/A	N/A	
Curb Opening Information		MINOR		MAJOR	
Length of a Unit Curb Opening		L _o (C) =	10.00	10.00	feet
Height of Vertical Curb Opening in Inches		H _{vert} =	6.00	6.00	inches
Height of Curb Orifice Throat in Inches		H _{throat} =	6.00	6.00	inches
Angle of Throat (see USDCM Figure ST-5)		Theta =	63.40	63.40	degrees
Side Width for Depression Pan (typically the gutter width of 2 feet)		W _p =	2.00	2.00	feet
Clogging Factor for a Single Curb Opening (typical value 0.10)		C _f (C) =	0.10	0.10	
Curb Opening Weir Coefficient (typical value 2.3-3.7)		C _w (C) =	3.60	3.60	
Curb Opening Orifice Coefficient (typical value 0.60 - 0.70)		C _o (C) =	0.67	0.67	
Grate Flow Analysis (Calculated)		MINOR		MAJOR	
Clogging Coefficient for Multiple Units		Coef =	N/A	N/A	
Clogging Factor for Multiple Units		Clog =	N/A	N/A	
Grate Capacity as a Weir (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q _{wi} =	N/A	N/A	cfs
Interception with Clogging		Q _{wa} =	N/A	N/A	cfs
Grate Capacity as an Orifice (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q _{oi} =	N/A	N/A	cfs
Interception with Clogging		Q _{oa} =	N/A	N/A	cfs
Grate Capacity as Mixed Flow		MINOR		MAJOR	
Interception without Clogging		Q _{mi} =	N/A	N/A	cfs
Interception with Clogging		Q _{ma} =	N/A	N/A	cfs
Resulting Grate Capacity (assumes clogged condition)		Q _{Grate} =	N/A	N/A	cfs
Curb Opening Flow Analysis (Calculated)		MINOR		MAJOR	
Clogging Coefficient for Multiple Units		Coef =	1.25	1.25	
Clogging Factor for Multiple Units		Clog =	0.06	0.06	
Curb Capacity as a Weir (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q _{wi} =	8.8	25.7	cfs
Interception with Clogging		Q _{wa} =	8.3	24.1	cfs
Curb Capacity as an Orifice (based on MHFD - CSU 2010 Study)		MINOR		MAJOR	
Interception without Clogging		Q _{oi} =	19.5	24.7	cfs
Interception with Clogging		Q _{oa} =	18.3	23.1	cfs
Curb Opening Capacity as Mixed Flow		MINOR		MAJOR	
Interception without Clogging		Q _{mi} =	12.2	23.4	cfs
Interception with Clogging		Q _{ma} =	11.4	21.9	cfs
Resulting Curb Opening Capacity (assumes clogged condition)		Q _{Curb} =	8.3	21.9	cfs
Resultant Street Conditions		MINOR		MAJOR	
Total Inlet Length		L =	10.00	10.00	feet
Resultant Street Flow Spread (based on street geometry from above)		T =	18.7	34.5	ft
Resultant Flow Depth at Street Crown		d _{CROWN} =	0.0	0.0	inches
Low Head Performance Reduction (Calculated)		MINOR		MAJOR	
Depth for Grate Midwidth		d _{Grate} =	N/A	N/A	ft
Depth for Curb Opening Weir Equation		d _{Curb} =	0.33	0.65	ft
Grated Inlet Performance Reduction Factor for Long Inlets		RF _{Grate} =	N/A	N/A	
Curb Opening Performance Reduction Factor for Long Inlets		RF _{Curb} =	0.93	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets		RF _{Combination} =	N/A	N/A	
Total Inlet Interception Capacity (assumes clogged condition)		MINOR		MAJOR	
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)		Q _s =	8.3	21.9	cfs
Q _{PEAK REQUIRED} =			4.7	10.2	cfs

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A4-1 Swale



This worksheet uses the NRCS vegetal retardance method to determine Manning's n.

For more information see Section 7.2.3 of the USDCM.

Analysis of Trapezoidal Grass-Lined Channel Using SCS Method

NRCS Vegetal Retardance (A, B, C, D, or E)

Manning's n (Leave cell D16 blank to manually enter an n value)

Channel Invert Slope

Bottom Width

Left Side Slope

Right Side Slope

Check one of the following soil types:

Soil Type:	Max. Velocity (V_{MAX})	Max Froude No. (F_{MAX})
Non-Cohesive	5.0 fps	0.60
Cohesive	7.0 fps	0.80
Paved	N/A	N/A

A, B, C, D, or E =

n =	0.020	
S_o =	0.0050	ft/ft
B =	0.00	ft
Z1 =	23.10	ft/ft
Z2 =	61.35	ft/ft

Choose One:

- ☐ Non-Cohesive
☐ Cohesive
☐ Paved

Maximum Allowable Top Width of Channel for Minor & Major Storm

Maximum Allowable Water Depth in Channel for Minor & Major Storm

	Minor Storm	Major Storm	
T_{MAX} =	54.00	54.00	ft
d_{MAX} =	0.90	0.90	ft

Maximum Channel Capacity Based On Allowable Top Width

Maximum Allowable Top Width

Water Depth

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
T_{MAX} =	54.00	54.00	ft
d =	0.64	0.64	ft
A =	17.27	17.27	sq ft
P =	54.02	54.02	ft
R =	0.32	0.32	ft
n =	0.020	0.020	
V =	2.46	2.46	fps
VR =	0.79	0.79	ft ² /s
D =	0.32	0.32	ft
Fr =	0.77	0.77	
Q_T =	42.5	42.5	cfs

Maximum Channel Capacity Based On Allowable Water Depth

Maximum Allowable Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

Maximum Flow Based on Allowable Water Depth

	Minor Storm	Major Storm	
d_{MAX} =	0.90	0.90	ft
T =	76.00	76.00	ft
A =	34.20	34.20	sq ft
P =	76.03	76.03	ft
R =	0.45	0.45	ft
n =	0.020	0.020	
V =	3.09	3.09	fps
VR =	1.39	1.39	ft ² /s
D =	0.45	0.45	ft
Fr =	0.81	0.81	
Q_d =	105.8	105.8	cfs

Allowable Channel Capacity Based On Channel Geometry

MINOR STORM Allowable Capacity is based on Top Width Criterion

MAJOR STORM Allowable Capacity is based on Top Width Criterion

Water Depth in Channel Based On Design Peak Flow

Design Peak Flow

Water Depth

Top Width

Flow Area

Wetted Perimeter

Hydraulic Radius

Manning's n

Flow Velocity

Velocity-Depth Product

Hydraulic Depth

Froude Number

	Minor Storm	Major Storm	
Q_o =	3.6	21.4	cfs
d =	0.25	0.49	ft
T =	21.39	41.74	ft
A =	2.71	10.32	sq ft
P =	21.40	41.76	ft
R =	0.13	0.25	ft
n =	0.020	0.020	
V =	1.33	2.07	fps
VR =	0.17	0.51	ft ² /s
D =	0.13	0.25	ft
Fr =	0.66	0.74	

Minor storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

Major storm max. allowable capacity GOOD - greater than the design flow given on sheet 'Inlet Management'

AREA INLET IN A SWALE

Dawson Trails Boulevard North
Inlet A4-1 Swale

Inlet Design Information (Input)

Type of Inlet

CDOT Type C

Inlet Type =

CDOT Type C

Angle of Inclined Grate (must be ≤ 30 degrees)

Width of Grate

Length of Grate

Open Area Ratio

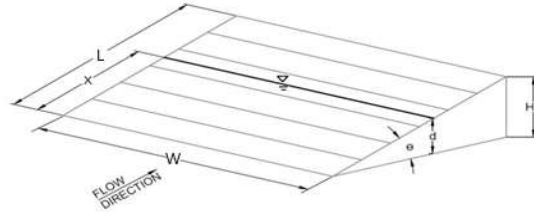
Height of Inclined Grate

Clogging Factor

Grate Discharge Coefficient

Orifice Coefficient

Weir Coefficient

 $\theta =$ 0.00 degrees

W = 3.00 ft

L = 3.00 ft

 $A_{RATIO} =$ 0.70 $H_B =$ 0.00 ft $C_f =$ 0.50 $C_d =$ 0.96 $C_o =$ 0.64 $C_w =$ 2.05

Water Depth at Inlet (for depressed inlets, 1 foot is added for depression)

	MINOR	MAJOR
d =	0.25	0.49

Grate Capacity as a Weir

Submerged Side Weir Length

Inclined Side Weir Flow

Base Weir Flow

Interception Without Clogging

Interception With Clogging

	MINOR	MAJOR	
X =	3.00	3.00	ft
$Q_{ws} =$	1.4	3.7	cfs
$Q_{wb} =$	2.0	5.4	cfs
$Q_{wi} =$	4.7	12.8	cfs
$Q_{wa} =$	2.4	6.4	cfs

Grate Capacity as an Orifice

Interception Without Clogging

Interception With Clogging

$Q_{oi} =$	16.3	22.7	cfs
$Q_{oa} =$	8.1	11.4	cfs

Total Inlet Interception Capacity (assumes clogged condition)

Bypassed Flow

Capture Percentage = Q_a/Q_o

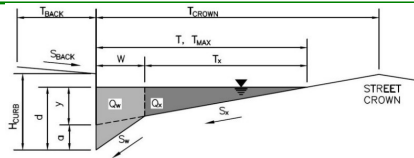
$Q_a =$	2.4	6.4	cfs
$Q_b =$	1.2	15.0	cfs
C% =	65	30	%

ALLOWABLE CAPACITY FOR ONE-HALF OF STREET (Minor & Major Storm)

(Based on Regulated Criteria for Maximum Allowable Flow Depth and Spread)

Project: Dawson Trails Boulevard North

Inlet ID: Inlet A4-2

**Gutter Geometry:**

Maximum Allowable Width for Spread Behind Curb
 Side Slope Behind Curb (leave blank for no conveyance credit behind curb)
 Manning's Roughness Behind Curb (typically between 0.012 and 0.020)

$T_{BACK} =$	21.0	ft
$S_{BACK} =$	0.020	ft/ft
$n_{BACK} =$	0.020	

Height of Curb at Gutter Flow Line
 Distance from Curb Face to Street Crown
 Gutter Width
 Street Transverse Slope
 Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)
 Street Longitudinal Slope - Enter 0 for sump condition
 Manning's Roughness for Street Section (typically between 0.012 and 0.020)

$H_{CURB} =$	6.00	inches
$T_{CROWN} =$	52.5	ft
$W =$	2.00	ft
$S_x =$	0.020	ft/ft
$S_w =$	0.083	ft/ft
$S_o =$	0.005	ft/ft
$n_{STREET} =$	0.016	

Max. Allowable Spread for Minor & Major Storm
 Max. Allowable Depth at Gutter Flowline for Minor & Major Storm
 Allow Flow Depth at Street Crown (check box for yes, leave blank for no)

	Minor Storm	Major Storm	
$T_{MAX} =$	42.5	52.5	ft
$d_{MAX} =$	6.0	11.0	inches
	☐	☐	

Maximum Capacity for 1/2 Street based On Allowable Spread

Water Depth without Gutter Depression ($T * S_x * 12$)
 Vertical Depth between Gutter Lip and Gutter Flowline ($W * S_w * 12$)
 Gutter Depression ($d_c - (W * S_x * 12)$)
 Water Depth at Gutter Flowline ($y + a$)
 Allowable Spread for Discharge outside the Gutter Section ($T - W$)
 Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)
 Discharge outside the Gutter Section, carried in Section T_x
 Discharge within the Gutter Section ($Q_T - Q_x - Q_{BACK}$)
 Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)
 Maximum Flow Based On Allowable Spread
 Flow Velocity within the Gutter Section
 $V*d$ Product: Flow Velocity times Gutter Flowline Depth

	Minor Storm	Major Storm	
$y =$	10.20	12.60	inches
$d_c =$	2.0	2.0	inches
$a =$	1.51	1.51	inches
$d =$	11.71	14.11	inches
$T_x =$	40.5	50.5	ft
$E_o =$	0.134	0.108	
$Q_x =$	70.5	127.1	cfs
$Q_w =$	10.9	15.3	cfs
$Q_{BACK} =$	13.6	32.2	cfs
$Q_T =$	95.1	174.6	cfs
$V =$	6.1	7.0	fps
$V*d =$	6.0	8.2	

Maximum Capacity for 1/2 Street based on Allowable Depth

Theoretical Water Spread
 Theoretical Spread for Discharge outside the Gutter Section ($T - W$)
 Gutter Flow to Design Flow Ratio by FHWA HEC-22 method (Eq. 7-7)
 Theoretical Discharge outside the Gutter Section, carried in Section T_{XTH}
 Actual Discharge outside the Gutter Section, (limited by distance T_{CROWN})
 Discharge within the Gutter Section ($Q_d - Q_x$)
 Discharge Behind the Curb (e.g., sidewalk, driveways, & lawns)
 Total Discharge for Major & Minor Storm (Pre-Safety Factor)
 Average Flow Velocity Within the Gutter Section
 $V*d$ Product: Flow Velocity Times Gutter Flowline Depth
 Slope-Based Safety Factor for Minor/Major Storm depth reduction, $d \geq 6"$
 Max Flow based on Allowable Depth (Safety Factor Applied)
 Resultant Flow Depth at Gutter Flowline (Safety Factor Applied)
 Resultant Flow Depth at Street Crown (Safety Factor Applied)

	Minor Storm	Major Storm	
$T_{TH} =$	18.7	39.5	ft
$T_{XTH} =$	16.7	37.5	ft
$E_o =$	0.318	0.145	
$Q_{XTH} =$	6.6	57.6	cfs
$Q_x =$	6.6	57.6	cfs
$Q_w =$	3.1	9.8	cfs
$Q_{BACK} =$	0.0	9.6	cfs
$Q =$	9.7	76.9	cfs
$V =$	3.7	5.9	fps
$V*d =$	1.9	5.4	
$R =$	1.00	1.00	
$Q_d =$	9.7	76.9	cfs
$d =$	6.00	11.00	inches
$d_{CROWN} =$	0.00	0.00	inches

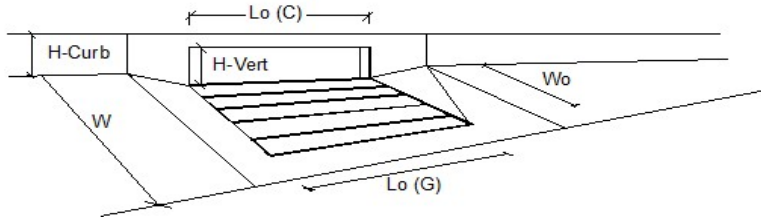
MINOR STORM Allowable Capacity is based on Depth Criterion
 MAJOR STORM Allowable Capacity is based on Depth Criterion

	Minor Storm	Major Storm	
$Q_{allow} =$	9.7	76.9	cfs

Minor storm max. allowable capacity GOOD - greater than the design peak flow of 4.50 cfs on sheet 'Inlet Management'
Major storm max. allowable capacity GOOD - greater than the design peak flow of 16.50 cfs on sheet 'Inlet Management'

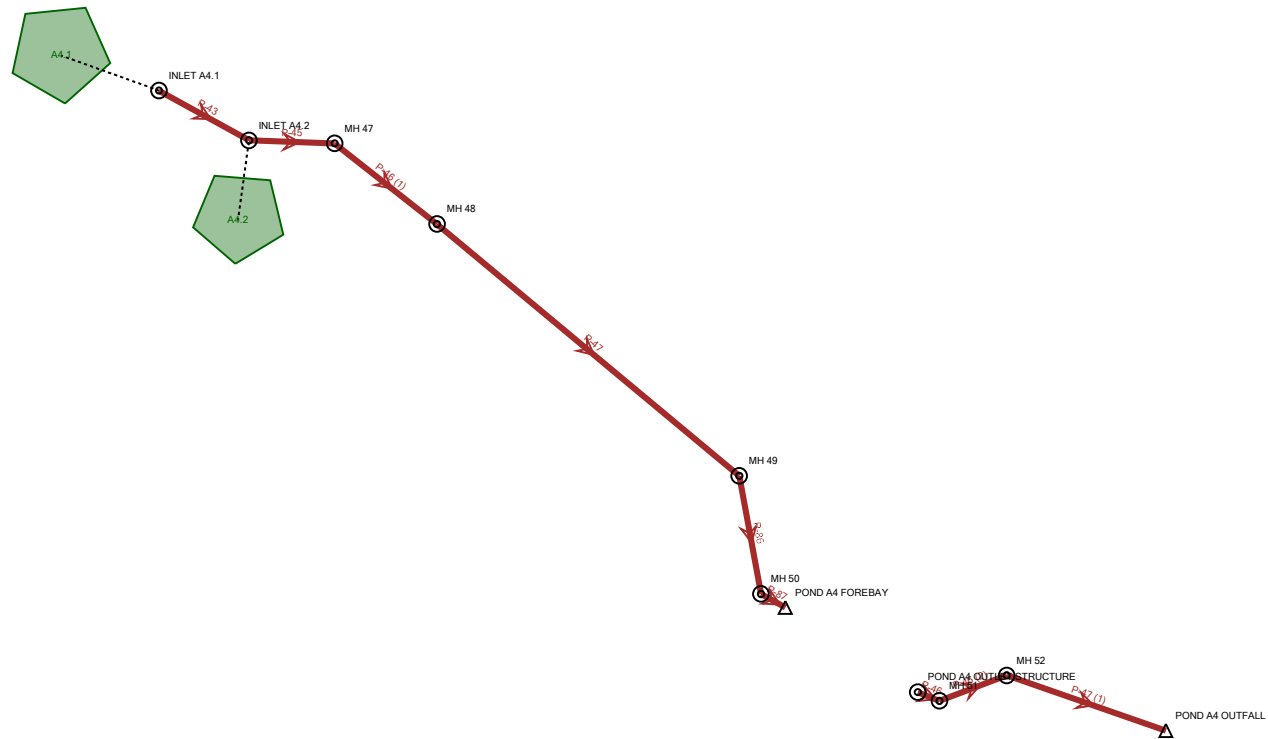
INLET ON A CONTINUOUS GRADE

MHFD-Inlet, Version 5.02 (August 2022)

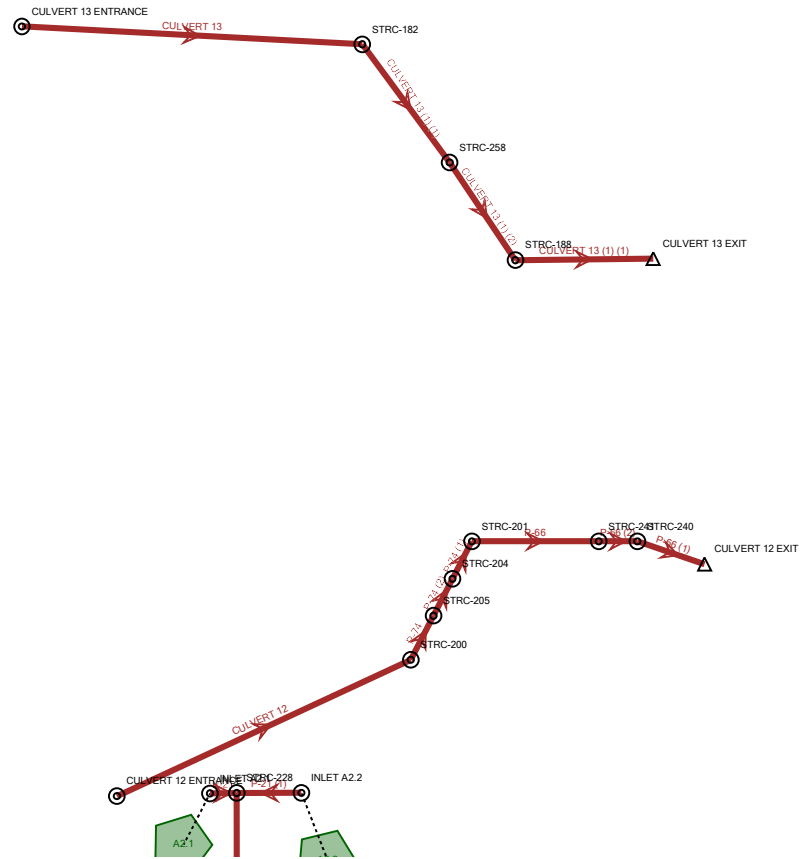


Design Information (Input)	MINOR	MAJOR	
Type of Inlet	CDOT Type R Curb Opening		
Local Depression (additional to continuous gutter depression 'a')	a _{LOCAL} = 3.0	3.0	inches
Total Number of Units in the Inlet (Grate or Curb Opening)	No = 2	2	
Length of a Single Unit Inlet (Grate or Curb Opening)	L _o = 15.00	15.00	ft
Width of a Unit Grate (cannot be greater than W, Gutter Width)	W _o = N/A	N/A	ft
Clogging Factor for a Single Unit Grate (typical min. value = 0.5)	C _f (G) = N/A	N/A	
Clogging Factor for a Single Unit Curb Opening (typical min. value = 0.1)	C _f (C) = 0.10	0.10	
Street Hydraulics: OK - Q < Allowable Street Capacity			
Design Discharge for Half of Street (from Inlet Management)	Q _o = 4.5	16.5	cfs
Water Spread Width	T = 13.6	23.0	ft
Water Depth at Flowline (outside of local depression)	d = 4.8	7.0	inches
Water Depth at Street Crown (or at T _{MAX})	d _{CROWN} = 0.0	0.0	inches
Ratio of Gutter Flow to Design Flow	E _o = 0.435	0.257	
Discharge outside the Gutter Section W, carried in Section T _x	Q _x = 2.5	12.2	cfs
Discharge within the Gutter Section W	Q _w = 2.0	4.2	cfs
Discharge Behind the Curb Face	Q _{BACK} = 0.0	0.1	cfs
Flow Area within the Gutter Section W	A _w = 0.63	1.00	sq ft
Velocity within the Gutter Section W	V _w = 3.1	4.2	fps
Water Depth for Design Condition	d _{LOCAL} = 7.8	10.0	inches
Grate Analysis (Calculated)			
Total Length of Inlet Grate Opening	L = N/A	N/A	ft
Ratio of Grate Flow to Design Flow	E _{o-GRATE} = N/A	N/A	
Under No-Clogging Condition			
Minimum Velocity Where Grate Splash-Over Begins	V _o = N/A	N/A	fps
Interception Rate of Frontal Flow	R _f = N/A	N/A	
Interception Rate of Side Flow	R _s = N/A	N/A	
Interception Capacity	Q _i = N/A	N/A	cfs
Under Clogging Condition			
Clogging Coefficient for Multiple-unit Grate Inlet	GrateCoeff = N/A	N/A	
Clogging Factor for Multiple-unit Grate Inlet	GrateClog = N/A	N/A	
Effective (unclogged) Length of Multiple-unit Grate Inlet	L _e = N/A	N/A	ft
Minimum Velocity Where Grate Splash-Over Begins	V _o = N/A	N/A	fps
Interception Rate of Frontal Flow	R _f = N/A	N/A	
Interception Rate of Side Flow	R _s = N/A	N/A	
Actual Interception Capacity	Q _a = N/A	N/A	cfs
Carry-Over Flow = Q _o - Q _a (to be applied to curb opening or next d/s inlet)	Q _b = N/A	N/A	cfs
Curb Opening or Slotted Inlet Analysis (Calculated)			
Equivalent Slope S _e	S _e = 0.102	0.068	ft/ft
Required Length L _T to Have 100% Interception	L _T = 11.54	26.78	ft
Under No-Clogging Condition			
Effective Length of Curb Opening or Slotted Inlet (minimum of L, L _T)	L = 11.54	26.78	ft
Interception Capacity	Q _i = 4.5	16.4	cfs
Under Clogging Condition			
Clogging Coefficient	CurbCoeff = 1.33	1.33	
Clogging Factor for Multiple-unit Curb Opening or Slotted Inlet	CurbClog = 0.02	0.02	
Effective (Unclogged) Length	L _e = 11.54	26.78	ft
Actual Interception Capacity	Q _a = 4.5	16.4	cfs
Carry-Over Flow = Q _o - Q _a	Q _b = 0.0	0.0	cfs
Summary			
Total Inlet Interception Capacity	Q = 4.5	16.4	cfs
Total Inlet Carry-Over Flow (flow bypassing inlet)	Q _b = 0.0	0.1	cfs
Capture Percentage = Q _a /Q _o	C% = 100	99	%

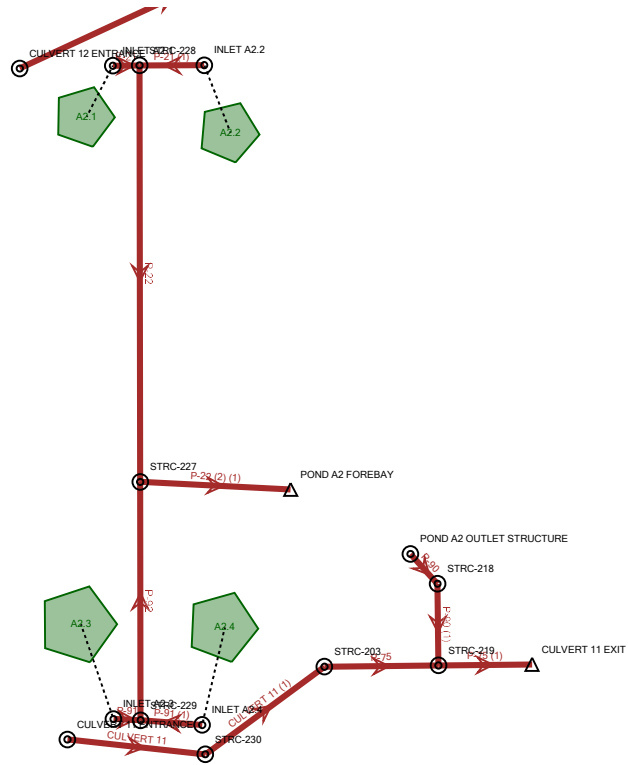
Scenario: Major



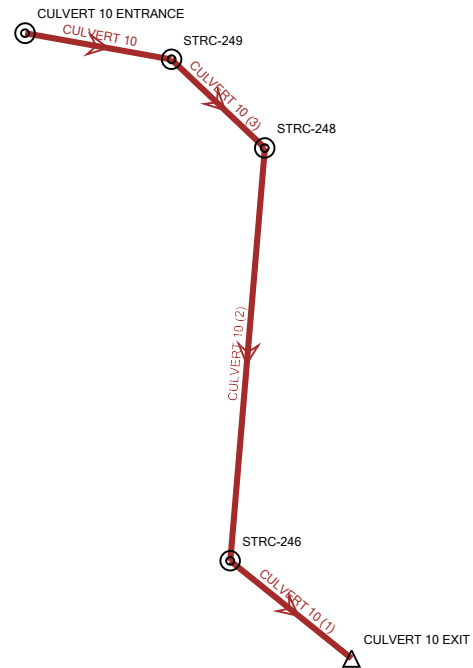
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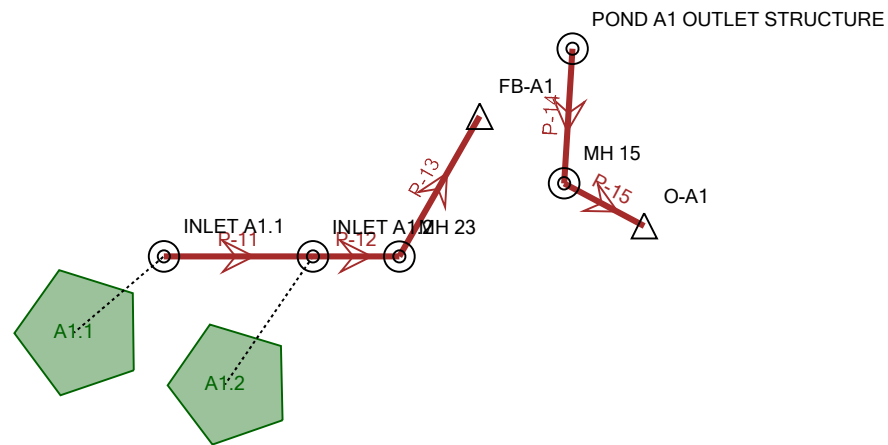
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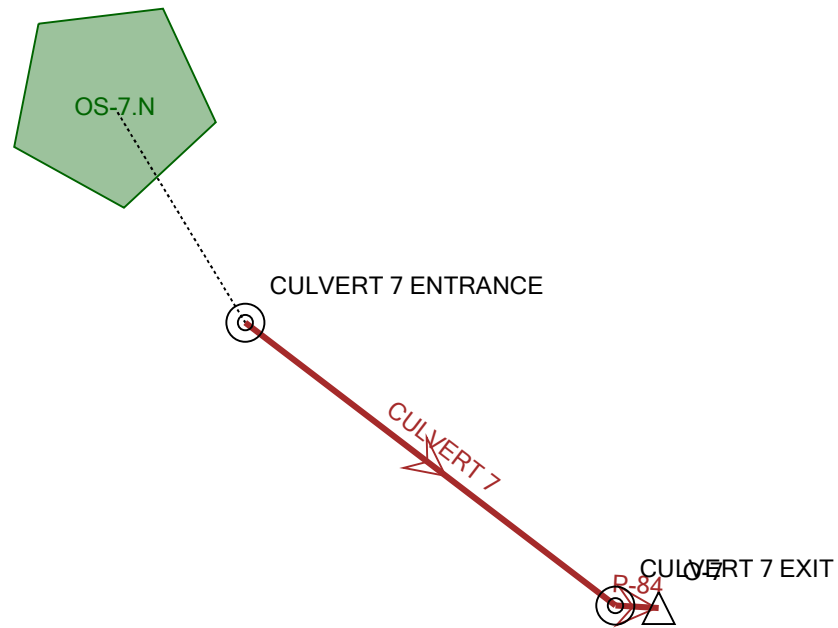
Scenario: Major



Scenario: Major



Scenario: Major



Major Storm: 100 Year Analysis

FlexTable: Manhole Table

Label	Elevation (Ground) (ft)	Elevation (Rim) (ft)	Flow (Known) (cfs)	Flow (Local In) (cfs)	Flow (Total Out) (cfs)	Hydraulic Grade Line (Out) (ft)	Hydraulic Grade Line (In) (ft)	Notes	Headloss Method	Headloss Coefficient (Standard)
CULVERT 7 ENTRANCE	6,339.77	6,339.77	0.00	0.00	24.43	6,339.02	6,339.52	30" FLARED END SECTION	Standard	0.050
CULVERT 7 EXIT	6,340.13	6,340.13	0.00	0.00	24.27	6,334.30	6,334.80	4' DIA MH	Standard	0.400
CULVERT 10 ENTRANCE	6,325.12	6,325.12	19.80	19.80	19.80	6,324.34	6,324.84	24" FLARED END SECTION	Standard	0.050
CULVERT 11 ENTRANCE	6,315.79	6,315.79	45.40	45.40	45.40	6,314.08	6,314.58	Null Structure	Standard	0.050
CULVERT 12 ENTRANCE	6,302.37	6,302.37	78.80	78.80	78.80	6,300.94	6,301.44	42" FLARED END SECTION	Standard	0.050
CULVERT 13 ENTRANCE	6,318.84	6,318.84	40.10	40.10	40.10	6,317.18	6,317.68	36" FLARED END SECTION	Standard	0.050
INLET A1.1	6,323.85	6,323.85	0.00	0.00	12.96	6,322.77	6,323.27	TYPE C INLET	Standard	0.050
INLET A1.2	6,323.86	6,323.86	0.00	0.00	26.51	6,322.03	6,322.53	10' TYPE R INLET	Standard	0.050
INLET A2.1	6,322.09	6,322.09	0.00	0.00	6.31	6,317.19	6,317.69	TYPE C INLET	Standard	0.050
INLET A2.2	6,322.59	6,322.59	0.00	0.00	6.31	6,317.39	6,317.89	10' TYPE R INLET	Standard	0.050
INLET A2.3	6,317.73	6,317.73	0.00	0.00	10.24	6,315.11	6,315.61	TYPE C INLET	Standard	0.050
INLET A2.4	6,317.73	6,317.73	0.00	0.00	10.30	6,315.59	6,316.09	10' TYPE R INLET	Standard	0.050
INLET A4.1	6,286.75	6,286.75	0.00	0.00	21.72	6,281.80	6,282.30	TYPE C INLET	Standard	0.050
INLET A4.2	6,285.30	6,285.30	0.00	0.00	36.70	6,280.20	6,280.70	20' TYPE R INLET	Standard	0.100
MH 15	6,321.51	6,321.51	0.00	0.00	4.00	6,313.07	6,313.57	5' DIA MH	Standard	0.640
MH 23	6,323.00	6,323.00	0.00	0.00	26.44	6,320.95	6,321.45	5' DIA MH	Standard	0.400
MH 47	6,286.93	6,286.93	0.00	0.00	36.54	6,277.72	6,278.22	5' DIA MH	Standard	0.100
MH 48	6,287.20	6,287.20	0.00	0.00	36.32	6,274.47	6,274.97	5' DIA MH	Standard	0.050
MH 49	6,284.16	6,284.16	0.00	0.00	35.55	6,269.65	6,270.15	5' DIA MH	Standard	0.100
MH 50	6,273.55	6,273.55	0.00	0.00	35.33	6,268.44	6,268.94	5' DIA MH	Standard	0.100
MH 51	6,269.29	6,269.29	0.00	0.00	15.70	6,256.26	6,256.76	5' DIA MH	Standard	0.400
MH 52	6,259.90	6,259.90	0.00	0.00	15.70	6,245.06	6,245.56	5' DIA MH	Standard	0.400
POND A1 OUTLET STRUCTURE	6,319.18	6,319.18	4.00	4.00	4.00	6,314.73	6,315.23		Standard	0.050
POND A2 OUTLET STRUCTURE	6,304.73	6,304.73	6.90	6.90	6.90	6,304.54	6,305.04	Null Structure	Standard	0.050
POND A4 OUTLET STRUCTURE	6,267.08	6,267.08	15.70	15.70	15.70	6,262.60	6,263.10	TYPE C INLET	Standard	0.050
STRC-182	6,322.54	6,322.54	0.00	0.00	40.10	6,301.86	6,302.36	5' DIA MH	Standard	0.400
STRC-188	6,298.74	6,298.74	0.00	0.00	40.10	6,289.06	6,289.56	5' DIA MH	Standard	0.400
STRC-200	6,300.20	6,300.20	0.00	0.00	78.80	6,287.35	6,287.85	6' DIA MH	Standard	0.100
STRC-201	6,283.48	6,283.48	0.00	0.00	78.80	6,272.79	6,273.37	6' DIA MH	Standard	0.400
STRC-203	6,307.48	6,307.48	0.00	0.00	45.40	6,300.58	6,301.08	5' DIA MH	Standard	0.400
STRC-204	6,288.77	6,288.77	0.00	0.00	78.80	6,277.60	6,278.10	6' DIA MH	Standard	0.050
STRC-205	6,293.92	6,293.92	0.00	0.00	78.80	6,282.41	6,282.91	6' DIA MH	Standard	0.050
STRC-218	6,308.79	6,308.79	0.00	0.00	6.90	6,303.72	6,304.22	4' DIA MH	Standard	0.400
STRC-219	6,302.44	6,302.44	0.00	0.00	52.30	6,297.49	6,299.62	5' DIA MH	Standard	1.770
STRC-227	6,321.35	6,321.35	0.00	0.00	30.66	6,312.32	6,313.95	5' DIA MH	Standard	1.770
STRC-228	6,322.86	6,322.86	0.00	0.00	12.43	6,316.20	6,317.16	5' DIA MH	Standard	1.770
STRC-229	6,318.05	6,318.05	0.00	0.00	20.38	6,314.45	6,315.02	5' DIA MH	Standard	1.770
STRC-230	6,317.88	6,317.88	0.00	0.00	45.40	6,307.23	6,307.73	5' DIA MH	Standard	0.400
STRC-240	6,271.92	6,271.92	0.00	0.00	78.80	6,259.63	6,260.13	6' DIA MH	Standard	0.100
STRC-241	6,274.94	6,274.94	0.00	0.00	78.80	6,265.38	6,265.88	6' DIA MH	Standard	0.050
STRC-246	6,325.94	6,325.94	0.00	0.00	19.80	6,314.60	6,315.14		Standard	0.640
STRC-248	6,328.45	6,328.45	0.00	0.00	19.80	6,321.10	6,321.60		Standard	0.400

Major Storm: 100 Year Analysis

FlexTable: Manhole Table

Label	Elevation (Ground) (ft)	Elevation (Rim) (ft)	Flow (Known) (cfs)	Flow (Local In) (cfs)	Flow (Total Out) (cfs)	Hydraulic Grade Line (Out) (ft)	Hydraulic Grade Line (In) (ft)	Notes	Headloss Method	Headloss Coefficient (Standard)
STRC-249	6,327.35	6,327.35	0.00	0.00	19.80	6,322.85	6,323.35	5' DIA MH	Standard	0.400
STRC-258	6,306.00	6,306.00	0.00	0.00	40.10	6,294.06	6,294.56		Standard	0.050

Major Storm: 100 Year Analysis

FlexTable: Conduit Table

Label	Start Node	Invert (Start) (ft)	Stop Node	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Capacity (Full Flow) (cfs)
P-21	INLET A2.1	6,315.31	STRC-228	6,315.13	7.0	0.026	18.0	0.013	6.31	3.57	6,317.19	6,317.16	16.88
P-21 (1)	INLET A2.2	6,315.64	STRC-228	6,315.13	62.0	0.008	18.0	0.013	6.31	3.57	6,317.39	6,317.16	9.52
P-11	INLET A1.1	6,319.22	INLET A1.2	6,318.49	73.2	0.010	24.0	0.013	12.96	4.13	6,322.77	6,322.53	22.59
P-15	MH 15	6,312.30	O-A1	6,311.80	63.0	0.008	18.0	0.013	4.00	5.09	6,313.07	6,312.49	9.36
P-91	INLET A2.3	6,312.68	STRC-229	6,312.50	9.8	0.018	18.0	0.013	10.24	5.79	6,315.11	6,315.02	14.24
P-91 (1)	INLET A2.4	6,313.01	STRC-229	6,312.50	59.2	0.009	18.0	0.013	10.30	5.83	6,315.59	6,315.02	9.75
P-43	INLET A4.1	6,280.30	INLET A4.2	6,279.73	98.8	0.006	36.0	0.013	21.72	6.89	6,281.80	6,281.10	50.67
P-14	POND A1 OUTLET STRUCTURE	6,313.96	MH 15	6,312.50	79.8	0.018	18.0	0.013	4.00	6.91	6,314.73	6,313.57	14.21
P-22	STRC-228	6,314.93	STRC-227	6,311.13	399.1	0.010	24.0	0.013	12.43	7.24	6,316.20	6,313.95	22.07
P-92	STRC-229	6,312.30	STRC-227	6,310.63	228.4	0.007	30.0	0.013	20.38	7.41	6,314.45	6,313.95	35.07
P-13	MH 23	6,317.10	FB-A1	6,315.54	65.7	0.024	24.0	0.013	26.44	8.42	6,320.95	6,320.05	34.85
P-12	INLET A1.2	6,318.29	MH 23	6,317.83	42.4	0.011	24.0	0.013	26.51	8.44	6,322.03	6,321.45	23.55
P-90	POND A2 OUTLET STRUCTURE	6,303.52	STRC-218	6,302.70	38.6	0.021	18.0	0.013	6.90	8.44	6,304.54	6,304.22	15.32
CULVERT 10	CULVERT 10 ENTRANCE	6,322.74	STRC-249	6,321.75	69.5	0.014	24.0	0.013	19.80	9.39	6,324.34	6,323.05	27.01
CULVERT 10 (3)	STRC-249	6,321.25	STRC-248	6,320.00	84.9	0.015	24.0	0.013	19.80	9.51	6,322.85	6,321.28	27.45
CULVERT 10 (2)	STRC-248	6,319.50	STRC-246	6,315.50	267.1	0.015	24.0	0.013	19.80	9.58	6,321.10	6,316.75	27.68
P-86	MH 49	6,267.80	MH 50	6,266.60	107.4	0.011	42.0	0.013	35.55	9.95	6,269.65	6,268.94	106.34
P-47	MH 48	6,272.60	MH 49	6,268.07	377.2	0.012	42.0	0.013	36.32	10.28	6,274.47	6,270.15	110.26
CULVERT 7	CULVERT 7 ENTRANCE	6,337.34	CULVERT 7 EXIT	6,334.41	177.1	0.017	30.0	0.013	24.43	10.54	6,339.02	6,335.61	52.76
CULVERT 10 (1)	STRC-246	6,313.00	CULVERT 10 EXIT	6,311.00	99.4	0.020	24.0	0.013	19.80	10.75	6,314.60	6,312.16	32.09
CULVERT 13 (1) (1)	STRC-188	6,287.00	CULVERT 13 EXIT	6,286.64	28.2	0.013	36.0	0.013	40.10	10.83	6,289.06	6,288.37	75.36
P-87	MH 50	6,266.30	POND A4 FOREBAY	6,265.91	27.2	0.014	42.0	0.013	35.33	10.88	6,268.44	6,268.59	120.47
P-45	INLET A4.2	6,278.23	MH 47	6,276.98	82.2	0.015	36.0	0.013	36.70	11.31	6,280.20	6,278.45	82.26
P-75 (1)	STRC-219	6,295.14	CULVERT 11 EXIT	6,294.24	70.1	0.013	36.0	0.013	52.30	11.54	6,297.49	6,296.17	75.55
P-66 (1)	STRC-240	6,256.85	CULVERT 12 EXIT	6,256.17	68.0	0.010	42.0	0.013	78.80	11.57	6,259.63	6,258.59	100.59
CULVERT 11	CULVERT 11 ENTRANCE	6,311.88	STRC-230	6,310.00	125.7	0.015	36.0	0.013	45.40	11.85	6,314.08	6,311.64	81.58
P-47 (1)	MH 52	6,243.63	POND A4 OUTFALL	6,238.16	161.6	0.034	24.0	0.013	15.70	12.32	6,245.06	6,239.02	41.61
P-46 (1)	MH 47	6,275.84	MH 48	6,273.33	124.9	0.020	42.0	0.013	36.54	12.41	6,277.72	6,274.57	142.61
P-90 (1)	STRC-218	6,302.70	STRC-219	6,296.63	78.5	0.077	18.0	0.013	6.90	13.53	6,303.72	6,299.62	29.21
P-75	STRC-203	6,298.38	STRC-219	6,295.34	128.9	0.024	36.0	0.013	45.40	14.06	6,300.58	6,299.62	102.42
P-22 (2) (1)	STRC-227	6,310.43	POND A2 FOREBAY	6,305.46	161.3	0.031	30.0	0.013	30.66	14.08	6,312.32	6,310.35	71.99
P-84	CULVERT 7 EXIT	6,332.62	O-7	6,331.96	17.0	0.039	30.0	0.013	24.27	14.38	6,334.30	6,333.16	80.71
P-66	STRC-201	6,270.02	STRC-241	6,267.59	121.6	0.020	42.0	0.013	78.80	15.16	6,272.79	6,269.56	142.22
P-74	STRC-200	6,284.58	STRC-205	6,283.63	47.5	0.020	42.0	0.013	78.80	15.16	6,287.35	6,285.77	142.24
P-74 (1)	STRC-204	6,274.83	STRC-201	6,274.02	40.4	0.020	42.0	0.013	78.80	15.18	6,277.60	6,276.19	142.39
P-74 (2)	STRC-205	6,279.63	STRC-204	6,278.83	39.9	0.020	42.0	0.013	78.80	15.19	6,282.41	6,281.01	142.52
P-66 (2)	STRC-241	6,262.60	STRC-240	6,261.85	37.0	0.020	42.0	0.013	78.80	15.24	6,265.38	6,264.04	143.24
CULVERT 13 (1) (2)	STRC-258	6,292.00	STRC-188	6,289.15	73.8	0.039	36.0	0.013	40.10	16.29	6,294.06	6,290.40	131.06
P-46	POND A4 OUTLET STRUCTURE	6,261.17	MH 51	6,259.33	22.4	0.082	24.0	0.013	15.70	16.99	6,262.60	6,260.15	64.77
CULVERT 13 (1) (1)	STRC-182	6,299.80	STRC-258	6,296.15	73.8	0.049	36.0	0.013	40.10	17.83	6,301.86	6,297.33	148.32
CULVERT 11 (1)	STRC-230	6,305.03	STRC-203	6,298.58	141.9	0.045	36.0	0.013	45.40	17.89	6,307.23	6,301.08	142.20
CULVERT 12	CULVERT 12 ENTRANCE	6,298.17	STRC-200	6,288.58	308.2	0.031	42.0	0.013	78.80	17.90	6,300.94	6,290.21	177.46
CULVERT 13	CULVERT 13 ENTRANCE	6,315.12	STRC-182	6,305.00	200.9	0.050	36.0	0.013	40.10	17.94	6,317.18	6,306.06	149.68
P-46 (2)	MH 51	6,254.83	MH 52	6,248.20	69.3	0.096	24.0	0.013	15.70	17.96	6,256.26	6,248.87	69.94

Major Storm: 100 Year Analysis

FlexTable: Catchment Table

ID	Label	Outflow Element	Area (User Defined) (acres)	Runoff Coefficient (Rational)	Time of Concentration (min)	Flow (Total Out) (cfs)	Notes
95	A1.1	INLET A1.1	2.850	0.750	14.400	12.96	
96	A1.2	INLET A1.2	2.880	0.790	13.800	14.07	
97	A2.3	INLET A2.3	1.690	0.800	8.400	10.24	
98	A2.4	INLET A2.4	1.700	0.800	8.400	10.30	
99	A2.1	INLET A2.1	0.910	0.800	5.500	6.31	
100	A2.2	INLET A2.2	0.910	0.800	5.500	6.31	
101	A4.1	INLET A4.1	6.140	0.530	11.600	21.72	
102	A4.2	INLET A4.2	3.820	0.600	9.600	16.51	
281	OS-7.N	CULVERT 7 ENTRANCE	10.670	0.490	24.000	24.43	

Major Storm: 100 Year Analysis

FlexTable: Outfall Table

ID	Label	Elevation (Ground) (ft)	Set Rim to Ground Elevation?	Elevation (Invert) (ft)	Boundary Condition Type	Boundary Element	Elevation (User Defined Tailwater) (ft)	Hydraulic Grade (ft)	Flow (Total Out) (cfs)	Notes
159	O-A1	6,315.36	True	6,311.95	Free Outfall	<None>		6,312.49	4.00	18" FLARED END SECTION
233	CULVERT 13 EXIT	6,290.15	True	6,286.64	Free Outfall	<None>		6,288.37	40.10	36" FES
234	O-7	6,334.93	True	6,331.96	Free Outfall	<None>		6,333.16	24.26	30" FES
236	FB-A1	6,317.93	True	6,315.51	User Defined Tailwater	<None>	6,320.05	6,320.05	26.33	24" FLARED END SECTION
352	POND A4 OUTFALL	6,240.54	True	6,238.16	Free Outfall	<None>		6,239.02	15.70	24" FES
353	CULVERT 12 EXIT	6,260.77	True	6,256.17	Free Outfall	<None>		6,258.59	78.80	42" FES
354	CULVERT 11 EXIT	6,297.75	True	6,294.24	Free Outfall	<None>		6,296.17	52.30	36" FES
355	POND A2 FOREBAY	6,308.45	True	6,305.46	User Defined Tailwater	<None>	6,310.35	6,310.35	30.42	POND A2 FOREBAY
356	POND A4 FOREBAY	6,269.62	True	6,265.91	User Defined Tailwater	<None>	6,268.59	6,268.59	35.28	42" FOREBAY HEADWALL
406	CULVERT 10 EXIT	6,311.00	True	6,311.00	Free Outfall	<None>		6,312.16	19.80	24" FLARED END SECTION

Minor Storm: 5 Year Analysis

FlexTable: Manhole Table

Label	Elevation (Ground) (ft)	Elevation (Rim) (ft)	Flow (Known) (cfs)	Flow (Local In) (cfs)	Flow (Total Out) (cfs)	Hydraulic Grade Line (Out) (ft)	Hydraulic Grade Line (In) (ft)	Notes	Headloss Method	Headloss Coefficient (Standard)
CULVERT 7 ENTRANCE	6,339.77	6,339.77	0.00	0.00	1.37	6,337.72	6,338.22	30" FLARED END SECTION	Standard	0.050
CULVERT 7 EXIT	6,340.13	6,340.13	0.00	0.00	1.35	6,333.00	6,333.50	4' DIA MH	Standard	0.400
CULVERT 10 ENTRANCE	6,325.12	6,325.12	1.90	1.90	1.90	6,323.22	6,323.72	24" FLARED END SECTION	Standard	0.050
CULVERT 11 ENTRANCE	6,315.79	6,315.79	1.90	1.90	1.90	6,312.31	6,312.81	Null Structure	Standard	0.050
CULVERT 12 ENTRANCE	6,302.37	6,302.37	1.20	1.20	1.20	6,298.50	6,299.00	42" FLARED END SECTION	Standard	0.050
CULVERT 13 ENTRANCE	6,318.84	6,318.84	0.60	0.60	0.60	6,315.36	6,315.86	36" FLARED END SECTION	Standard	0.050
INLET A1.1	6,323.85	6,323.85	0.00	0.00	5.61	6,320.06	6,320.56	TYPE C INLET	Standard	0.050
INLET A1.2	6,323.86	6,323.86	0.00	0.00	11.97	6,319.53	6,320.03	10' TYPE R INLET	Standard	0.050
INLET A2.1	6,322.09	6,322.09	0.00	0.00	2.17	6,316.21	6,316.71	TYPE C INLET	Standard	0.050
INLET A2.2	6,322.59	6,322.59	0.00	0.00	2.82	6,316.28	6,316.78	10' TYPE R INLET	Standard	0.050
INLET A2.3	6,317.73	6,317.73	0.00	0.00	4.29	6,313.96	6,314.46	TYPE C INLET	Standard	0.050
INLET A2.4	6,317.73	6,317.73	0.00	0.00	4.81	6,313.97	6,314.47	10' TYPE R INLET	Standard	0.050
INLET A4.1	6,286.75	6,286.75	0.00	0.00	3.61	6,280.89	6,281.39	TYPE C INLET	Standard	0.050
INLET A4.2	6,285.30	6,285.30	0.00	0.00	8.53	6,279.15	6,279.65	20' TYPE R INLET	Standard	0.100
MH 15	6,321.51	6,321.51	0.00	0.00	0.40	6,312.53	6,313.03	5' DIA MH	Standard	0.640
MH 23	6,323.00	6,323.00	0.00	0.00	11.93	6,318.68	6,319.18	5' DIA MH	Standard	0.400
MH 47	6,286.93	6,286.93	0.00	0.00	8.48	6,276.72	6,277.22	5' DIA MH	Standard	0.100
MH 48	6,287.20	6,287.20	0.00	0.00	8.40	6,273.48	6,273.98	5' DIA MH	Standard	0.050
MH 49	6,284.16	6,284.16	0.00	0.00	8.14	6,268.66	6,269.16	5' DIA MH	Standard	0.100
MH 50	6,273.55	6,273.55	0.00	0.00	8.06	6,267.16	6,267.66	5' DIA MH	Standard	0.100
MH 51	6,269.29	6,269.29	0.00	0.00	2.30	6,255.36	6,255.86	5' DIA MH	Standard	0.400
MH 52	6,259.90	6,259.90	0.00	0.00	2.30	6,244.16	6,244.66	5' DIA MH	Standard	0.400
POND A1 OUTLET STRUCTURE	6,319.18	6,319.18	0.40	0.40	0.40	6,314.19	6,314.69		Standard	0.050
POND A2 OUTLET STRUCTURE	6,304.73	6,304.73	0.60	0.60	0.60	6,303.81	6,304.31	Null Structure	Standard	0.050
POND A4 OUTLET STRUCTURE	6,267.08	6,267.08	2.30	2.30	2.30	6,261.70	6,262.20	TYPE C INLET	Standard	0.050
STRC-182	6,322.54	6,322.54	0.00	0.00	0.60	6,300.04	6,300.54	5' DIA MH	Standard	0.400
STRC-188	6,298.74	6,298.74	0.00	0.00	0.60	6,287.24	6,287.74	5' DIA MH	Standard	0.400
STRC-200	6,300.20	6,300.20	0.00	0.00	1.20	6,284.91	6,285.41	6' DIA MH	Standard	0.100
STRC-201	6,283.48	6,283.48	0.00	0.00	1.20	6,270.35	6,270.85	6' DIA MH	Standard	0.400
STRC-203	6,307.48	6,307.48	0.00	0.00	1.90	6,298.81	6,299.31	5' DIA MH	Standard	0.400
STRC-204	6,288.77	6,288.77	0.00	0.00	1.20	6,275.16	6,275.66	6' DIA MH	Standard	0.050
STRC-205	6,293.92	6,293.92	0.00	0.00	1.20	6,279.96	6,280.46	6' DIA MH	Standard	0.050
STRC-218	6,308.79	6,308.79	0.00	0.00	0.60	6,302.99	6,303.49	4' DIA MH	Standard	0.400
STRC-219	6,302.44	6,302.44	0.00	0.00	2.50	6,295.63	6,296.13	5' DIA MH	Standard	1.770
STRC-227	6,321.35	6,321.35	0.00	0.00	13.01	6,311.64	6,312.48	5' DIA MH	Standard	1.770
STRC-228	6,322.86	6,322.86	0.00	0.00	4.93	6,315.71	6,316.23	5' DIA MH	Standard	1.770
STRC-229	6,318.05	6,318.05	0.00	0.00	9.04	6,313.30	6,313.97	5' DIA MH	Standard	1.770
STRC-230	6,317.88	6,317.88	0.00	0.00	1.90	6,305.46	6,305.96	5' DIA MH	Standard	0.400
STRC-240	6,271.92	6,271.92	0.00	0.00	1.20	6,257.18	6,257.68	6' DIA MH	Standard	0.100
STRC-241	6,274.94	6,274.94	0.00	0.00	1.20	6,262.93	6,263.43	6' DIA MH	Standard	0.050
STRC-246	6,325.94	6,325.94	0.00	0.00	1.90	6,313.48	6,313.98		Standard	0.640
STRC-248	6,328.45	6,328.45	0.00	0.00	1.90	6,319.98	6,320.48		Standard	0.400

Minor Storm: 5 Year Analysis

FlexTable: Manhole Table

Label	Elevation (Ground) (ft)	Elevation (Rim) (ft)	Flow (Known) (cfs)	Flow (Local In) (cfs)	Flow (Total Out) (cfs)	Hydraulic Grade Line (Out) (ft)	Hydraulic Grade Line (In) (ft)	Notes	Headloss Method	Headloss Coefficient (Standard)
STRC-249	6,327.35	6,327.35	0.00	0.00	1.90	6,321.73	6,322.23	5' DIA MH	Standard	0.400
STRC-258	6,306.00	6,306.00	0.00	0.00	0.60	6,292.24	6,292.74		Standard	0.050

Minor Storm: 5 Year Analysis

FlexTable: Conduit Table

Label	Start Node	Invert (Start) (ft)	Stop Node	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Capacity (Full Flow) (cfs)
P-15	MH 15	6,312.30	O-A1	6,311.80	63.0	0.008	18.0	0.013	0.40	2.63	6,312.53	6,312.01	9.36
CULVERT 13 (1) (1)	STRC-188	6,287.00	CULVERT 13 EXIT	6,286.64	28.2	0.013	36.0	0.013	0.60	3.19	6,287.24	6,286.83	75.36
P-14	POND A1 OUTLET STRUCTURE	6,313.96	MH 15	6,312.50	79.8	0.018	18.0	0.013	0.40	3.53	6,314.19	6,313.03	14.21
P-66 (1)	STRC-240	6,256.85	CULVERT 12 EXIT	6,256.17	68.0	0.010	42.0	0.013	1.20	3.54	6,257.18	6,256.44	100.59
P-43	INLET A4.1	6,280.30	INLET A4.2	6,279.73	98.8	0.006	36.0	0.013	3.61	4.15	6,280.89	6,280.27	50.67
P-90	POND A2 OUTLET STRUCTURE	6,303.52	STRC-218	6,302.70	38.6	0.021	18.0	0.013	0.60	4.20	6,303.81	6,303.49	15.32
P-66	STRC-201	6,270.02	STRC-241	6,267.59	121.6	0.020	42.0	0.013	1.20	4.51	6,270.35	6,267.82	142.22
P-74	STRC-200	6,284.58	STRC-205	6,283.63	47.5	0.020	42.0	0.013	1.20	4.51	6,284.91	6,283.86	142.24
P-74 (1)	STRC-204	6,274.83	STRC-201	6,274.02	40.4	0.020	42.0	0.013	1.20	4.51	6,275.16	6,274.25	142.39
P-74 (2)	STRC-205	6,279.63	STRC-204	6,278.83	39.9	0.020	42.0	0.013	1.20	4.52	6,279.96	6,279.06	142.52
P-66 (2)	STRC-241	6,262.60	STRC-240	6,261.85	37.0	0.020	42.0	0.013	1.20	4.53	6,262.93	6,262.08	143.24
CULVERT 7	CULVERT 7 ENTRANCE	6,337.34	CULVERT 7 EXIT	6,334.41	177.1	0.017	30.0	0.013	1.37	4.60	6,337.72	6,334.69	52.76
CULVERT 13 (1) (2)	STRC-258	6,292.00	STRC-188	6,289.15	73.8	0.039	36.0	0.013	0.60	4.69	6,292.24	6,289.30	131.06
P-21 (1)	INLET A2.2	6,315.64	STRC-228	6,315.13	62.0	0.008	18.0	0.013	2.82	4.70	6,316.28	6,316.23	9.52
CULVERT 11	CULVERT 11 ENTRANCE	6,311.88	STRC-230	6,310.00	125.7	0.015	36.0	0.013	1.90	4.78	6,312.31	6,310.32	81.58
P-75 (1)	STRC-219	6,295.14	CULVERT 11 EXIT	6,294.24	70.1	0.013	36.0	0.013	2.50	4.92	6,295.63	6,294.61	75.55
CULVERT 10	CULVERT 10 ENTRANCE	6,322.74	STRC-249	6,321.75	69.5	0.014	24.0	0.013	1.90	4.96	6,323.22	6,322.11	27.01
CULVERT 10 (3)	STRC-249	6,321.25	STRC-248	6,320.00	84.9	0.015	24.0	0.013	1.90	5.02	6,321.73	6,320.36	27.45
CULVERT 10 (2)	STRC-248	6,319.50	STRC-246	6,315.50	267.1	0.015	24.0	0.013	1.90	5.05	6,319.98	6,315.85	27.68
CULVERT 13 (1) (1)	STRC-182	6,299.80	STRC-258	6,296.15	73.8	0.049	36.0	0.013	0.60	5.12	6,300.04	6,296.29	148.32
CULVERT 13	CULVERT 13 ENTRANCE	6,315.12	STRC-182	6,305.00	200.9	0.050	36.0	0.013	0.60	5.15	6,315.36	6,305.14	149.68
CULVERT 12	CULVERT 12 ENTRANCE	6,298.17	STRC-200	6,288.58	308.2	0.031	42.0	0.013	1.20	5.26	6,298.50	6,288.79	177.46
P-91 (1)	INLET A2.4	6,313.01	STRC-229	6,312.50	59.2	0.009	18.0	0.013	4.81	5.50	6,313.97	6,313.97	9.75
CULVERT 10 (1)	STRC-246	6,313.00	CULVERT 10 EXIT	6,311.00	99.4	0.020	24.0	0.013	1.90	5.60	6,313.48	6,311.33	32.09
P-75	STRC-203	6,298.38	STRC-219	6,295.34	128.9	0.024	36.0	0.013	1.90	5.61	6,298.81	6,296.13	102.42
P-22	STRC-228	6,314.93	STRC-227	6,311.13	399.1	0.010	24.0	0.013	4.93	5.66	6,315.71	6,312.48	22.07
P-11	INLET A1.1	6,319.22	INLET A1.2	6,318.49	73.2	0.010	24.0	0.013	5.61	5.97	6,320.06	6,320.03	22.59
P-92	STRC-229	6,312.30	STRC-227	6,310.63	228.4	0.007	30.0	0.013	9.04	5.99	6,313.30	6,312.48	35.07
P-84	CULVERT 7 EXIT	6,332.62	O-7	6,331.96	17.0	0.039	30.0	0.013	1.35	6.17	6,333.00	6,332.19	80.71
P-86	MH 49	6,267.80	MH 50	6,266.60	107.4	0.011	42.0	0.013	8.14	6.54	6,268.66	6,267.66	106.34
P-21	INLET A2.1	6,315.31	STRC-228	6,315.13	7.0	0.026	18.0	0.013	2.17	6.57	6,316.21	6,316.23	16.88
P-90 (1)	STRC-218	6,302.70	STRC-219	6,296.63	78.5	0.077	18.0	0.013	0.60	6.60	6,302.99	6,296.78	29.21
P-47	MH 48	6,272.60	MH 49	6,268.07	377.2	0.012	42.0	0.013	8.40	6.76	6,273.48	6,269.16	110.26
CULVERT 11 (1)	STRC-230	6,305.03	STRC-203	6,298.58	141.9	0.045	36.0	0.013	1.90	7.05	6,305.46	6,299.31	142.20
P-91	INLET A2.3	6,312.68	STRC-229	6,312.50	9.8	0.018	18.0	0.013	4.29	7.05	6,313.96	6,313.97	14.24
P-47 (1)	MH 52	6,243.63	POND A4 OUTFALL	6,238.16	161.6	0.034	24.0	0.013	2.30	7.11	6,244.16	6,238.48	41.61
P-87	MH 50	6,266.30	POND A4 FOREBAY	6,265.91	27.2	0.014	42.0	0.013	8.06	7.12	6,267.16	6,267.28	120.47
P-45	INLET A4.2	6,278.23	MH 47	6,276.98	82.2	0.015	36.0	0.013	8.53	7.52	6,279.15	6,277.63	82.26
P-12	INLET A1.2	6,318.29	MH 23	6,317.83	42.4	0.011	24.0	0.013	11.97	7.53	6,319.53	6,319.18	23.55
P-46 (1)	MH 47	6,275.84	MH 48	6,273.33	124.9	0.020	42.0	0.013	8.48	8.13	6,276.72	6,273.91	142.61
P-46	POND A4 OUTLET STRUCTURE	6,261.17	MH 51	6,259.33	22.4	0.082	24.0	0.013	2.30	9.71	6,261.70	6,259.60	64.77
P-13	MH 23	6,317.10	FB-A1	6,315.54	65.7	0.024	24.0	0.013	11.93	10.05	6,318.68	6,318.59	34.85
P-46 (2)	MH 51	6,254.83	MH 52	6,248.20	69.3	0.096	24.0	0.013	2.30	10.22	6,255.36	6,248.45	69.94
P-22 (2) (1)	STRC-227	6,310.43	POND A2 FOREBAY	6,305.46	161.3	0.031	30.0	0.013	13.01	11.13	6,311.64	6,309.01	71.99

Minor Storm: 5 Year Analysis

FlexTable: Catchment Table

ID	Label	Outflow Element	Area (User Defined) (acres)	Runoff Coefficient (Rational)	Time of Concentration (min)	Flow (Total Out) (cfs)	Notes
95	A1.1	INLET A1.1	2.850	0.590	14.400	5.61	
96	A1.2	INLET A1.2	2.880	0.670	13.800	6.56	
97	A2.3	INLET A2.3	1.690	0.610	8.400	4.29	
98	A2.4	INLET A2.4	1.700	0.680	8.400	4.81	
99	A2.1	INLET A2.1	0.910	0.500	5.500	2.17	
100	A2.2	INLET A2.2	0.910	0.650	5.500	2.82	
101	A4.1	INLET A4.1	6.140	0.160	11.600	3.61	
102	A4.2	INLET A4.2	3.820	0.360	9.600	5.45	
281	OS-7.N	CULVERT 7 ENTRANCE	10.670	0.050	24.000	1.37	

Minor Storm: 5 Year Analysis

FlexTable: Outfall Table

ID	Label	Elevation (Ground) (ft)	Set Rim to Ground Elevation?	Elevation (Invert) (ft)	Boundary Condition Type	Boundary Element	Elevation (User Defined Tailwater) (ft)	Hydraulic Grade (ft)	Flow (Total Out) (cfs)	Notes
159	O-A1	6,315.36	True	6,311.95	Free Outfall	<None>		6,312.01	0.40	18" FLARED END SECTION
233	CULVERT 13 EXIT	6,290.15	True	6,286.64	Free Outfall	<None>		6,286.83	0.60	36" FES
234	O-7	6,334.93	True	6,331.96	Free Outfall	<None>		6,332.19	1.35	30" FES
236	FB-A1	6,317.93	True	6,315.51	User Defined Tailwater	<None>	6,318.59	6,318.59	11.89	24" FLARED END SECTION
352	POND A4 OUTFALL	6,240.54	True	6,238.16	Free Outfall	<None>		6,238.48	2.30	24" FES
353	CULVERT 12 EXIT	6,260.77	True	6,256.17	Free Outfall	<None>		6,256.44	1.20	42" FES
354	CULVERT 11 EXIT	6,297.75	True	6,294.24	Free Outfall	<None>		6,294.61	2.50	36" FES
355	POND A2 FOREBAY	6,308.45	True	6,305.46	User Defined Tailwater	<None>	6,309.01	6,309.01	12.89	POND A2 FOREBAY
356	POND A4 FOREBAY	6,269.62	True	6,265.91	User Defined Tailwater	<None>	6,267.28	6,267.28	8.05	42" FOREBAY HEADWALL
406	CULVERT 10 EXIT	6,311.00	True	6,311.00	Free Outfall	<None>		6,311.33	1.90	24" FLARED END SECTION

Minimum Velocity Analysis

References:

- 1. ToCR SDDTCM, Section 9.8.1: "a flow depth equal to 25 percent of the pipe diameter and the corresponding flow rate shall be used to check the minimum velocity"
- 2. MHFD Volume 1 Section 4.4.1: Equation 7-45 (Manning's equation)

Pipe Label	Length	Slope (ft/ft)	Diameter (in)	25% Diameter (ft)	Area (ft2)	Area at 25% of D (ft^2)	P	Rh	Q at 25% pipe D	Start Node (Manhole)
CULVERT 7	177.1	0.017	30	0.625	4.90625	0.959375	2.6125	0.367225	7.4	CULVERT 7 ENTRANCE
CULVERT 10	69.5	0.014	24	0.5	3.14	0.614	2.09	0.29378	3.7	CULVERT 10 ENTRANCE
CULVERT 10 (1)	99.4	0.02	24	0.5	3.14	0.614	2.09	0.29378	4.4	STRC-246
CULVERT 10 (2)	267.1	0.015	24	0.5	3.14	0.614	2.09	0.29378	3.8	STRC-248
CULVERT 10 (3)	84.9	0.015	24	0.5	3.14	0.614	2.09	0.29378	3.8	STRC-249
CULVERT 11	125.7	0.015	36	0.75	7.065	1.3815	3.135	0.44067	11.2	CULVERT 11 ENTRANCE
CULVERT 11 (1)	141.9	0.045	36	0.75	7.065	1.3815	3.135	0.44067	19.5	STRC-230
CULVERT 12	308.2	0.031	42	0.875	9.61625	1.880375	3.6575	0.514115	24.4	CULVERT 12 ENTRANCE
CULVERT 13	200.9	0.05	36	0.75	7.065	1.3815	3.135	0.44067	20.5	CULVERT 13 ENTRANCE
CULVERT 13 (1) (1)	73.8	0.049	36	0.75	7.065	1.3815	3.135	0.44067	20.3	STRC-182
CULVERT 13 (1) (1)	28.2	0.013	36	0.75	7.065	1.3815	3.135	0.44067	10.5	STRC-188
CULVERT 13 (1) (2)	73.8	0.039	36	0.75	7.065	1.3815	3.135	0.44067	18.1	STRC-258
P-11	73.2	0.01	24	0.5	3.14	0.614	2.09	0.29378	3.1	INLET A1.1
P-12	42.4	0.011	24	0.5	3.14	0.614	2.09	0.29378	3.3	INLET A1.2
P-13	65.7	0.024	24	0.5	3.14	0.614	2.09	0.29378	4.8	MH 13
P-14	79.8	0.018	18	0.375	1.76625	0.345375	1.5675	0.220335	1.9	POND A1 OUTLET STRUCTURE
P-15	63	0.008	18	0.375	1.76625	0.345375	1.5675	0.220335	1.3	MH 15
P-21	7	0.026	18	0.375	1.76625	0.345375	1.5675	0.220335	2.3	INLET A2.1
P-21 (1)	62	0.008	18	0.375	1.76625	0.345375	1.5675	0.220335	1.3	INLET A2.2
P-22	399.1	0.01	24	0.5	3.14	0.614	2.09	0.29378	3.1	STRC-228
P-22 (2) (1)	161.3	0.031	30	0.625	4.90625	0.959375	2.6125	0.367225	9.9	STRC-227
P-43	98.8	0.006	36	0.75	7.065	1.3815	3.135	0.44067	7.1	INLET A4.1
P-45	82.2	0.015	36	0.75	7.065	1.3815	3.135	0.44067	11.2	INLET A4.2
P-46	22.4	0.082	24	0.5	3.14	0.614	2.09	0.29378	8.9	POND A4 OUTLET STRUCTURE
P-46 (1)	124.9	0.02	42	0.875	9.61625	1.880375	3.6575	0.514115	19.6	MH 47
P-46 (2)	69.3	0.096	24	0.5	3.14	0.614	2.09	0.29378	9.6	MH 51
P-47	377.2	0.012	42	0.875	9.61625	1.880375	3.6575	0.514115	15.2	MH 48
P-47 (1)	161.6	0.034	24	0.5	3.14	0.614	2.09	0.29378	5.7	MH 52
P-66	121.6	0.02	42	0.875	9.61625	1.880375	3.6575	0.514115	19.6	STRC-201
P-66 (1)	68	0.01	42	0.875	9.61625	1.880375	3.6575	0.514115	13.8	STRC-240
P-66 (2)	37	0.02	42	0.875	9.61625	1.880375	3.6575	0.514115	19.6	STRC-241
P-74	47.5	0.02	42	0.875	9.61625	1.880375	3.6575	0.514115	19.6	STRC-200
P-74 (1)	40.4	0.02	42	0.875	9.61625	1.880375	3.6575	0.514115	19.6	STRC-204
P-74 (2)	39.9	0.02	42	0.875	9.61625	1.880375	3.6575	0.514115	19.6	STRC-205
P-75	128.9	0.024	36	0.75	7.065	1.3815	3.135	0.44067	14.2	STRC-203
P-75 (1)	70.1	0.013	36	0.75	7.065	1.3815	3.135	0.44067	10.5	STRC-219
P-84	17	0.039	30	0.625	4.90625	0.959375	2.6125	0.367225	11.1	CULVERT 7 EXIT

Minimum Velocity Analysis

References:

- 1. ToCR SDDTCM, Section 9.8.1: "a flow depth equal to 25 percent of the pipe diameter and the corresponding flow rate shall be used to check the minimum velocity"
- 2. MHFD Volume 1 Section 4.4.1: Equation 7-45 (Manning's equation)

Pipe Label	Length	Slope (ft/ft)	Diameter (in)	25% Diameter (ft)	Area (ft2)	Area at 25% of D (ft^2)	P	Rh	Q at 25% pipe D	Start Node (Manhole)
P-86	107.4	0.011	42	0.875	9.61625	1.880375	3.6575	0.514115	14.5	MH 49
P-87	27.2	0.014	42	0.875	9.61625	1.880375	3.6575	0.514115	16.4	MH 50
P-90	38.6	0.021	18	0.375	1.76625	0.345375	1.5675	0.220335	2.1	POND A2 OUTLET STRUCTURE
P-90 (1)	78.5	0.077	18	0.375	1.76625	0.345375	1.5675	0.220335	4.0	STRC-218
P-91	9.8	0.018	18	0.375	1.76625	0.345375	1.5675	0.220335	1.9	INLET A2.3
P-91 (1)	59.2	0.009	18	0.375	1.76625	0.345375	1.5675	0.220335	1.4	INLET A2.4
P-92	228.4	0.007	30	0.625	4.90625	0.959375	2.6125	0.367225	4.7	STRC-229

Minimum Velocity Analysis: 25% of Diameter Flow Depth

FlexTable: Manhole Table

Label	Elevation (Ground) (ft)	Elevation (Rim) (ft)	Flow (Known) (cfs)	Flow (Local In) (cfs)	Flow (Total Out) (cfs)	Hydraulic Grade Line (Out) (ft)	Hydraulic Grade Line (In) (ft)	Notes	Headloss Method	Headloss Coefficient (Standard)
CULVERT 7 ENTRANCE	6,339.77	6,339.77	7.35	7.35	7.35	6,338.24	6,338.74	30" FLARED END SECTION	Standard	0.050
CULVERT 7 EXIT	6,340.13	6,340.13	11.14	11.14	11.14	6,333.74	6,334.24	4' DIA MH	Standard	0.400
CULVERT 10 ENTRANCE	6,325.12	6,325.12	3.68	3.68	3.68	6,323.41	6,323.91	24" FLARED END SECTION	Standard	0.050
CULVERT 11 ENTRANCE	6,315.79	6,315.79	11.23	11.23	11.23	6,312.94	6,313.44	Null Structure	Standard	0.050
CULVERT 12 ENTRANCE	6,302.37	6,302.37	24.35	24.35	24.35	6,299.69	6,300.19	42" FLARED END SECTION	Standard	0.050
CULVERT 13 ENTRANCE	6,318.84	6,318.84	20.50	20.50	20.50	6,316.57	6,317.07	36" FLARED END SECTION	Standard	0.050
INLET A1.1	6,323.85	6,323.85	3.11	3.11	3.11	6,319.84	6,320.34	TYPE C INLET	Standard	0.050
INLET A1.2	6,323.86	6,323.86	3.26	3.26	3.26	6,318.92	6,319.42	10' TYPE R INLET	Standard	0.050
INLET A2.1	6,322.09	6,322.09	2.33	2.33	2.33	6,315.98	6,316.48	TYPE C INLET	Standard	0.050
INLET A2.2	6,322.59	6,322.59	1.29	1.29	1.29	6,316.07	6,316.57	10' TYPE R INLET	Standard	0.050
INLET A2.3	6,317.73	6,317.73	1.94	1.94	1.94	6,313.50	6,314.00	TYPE C INLET	Standard	0.050
INLET A2.4	6,317.73	6,317.73	1.37	1.37	1.37	6,313.45	6,313.95	10' TYPE R INLET	Standard	0.050
INLET A4.1	6,286.75	6,286.75	7.10	7.10	7.10	6,281.14	6,281.64	TYPE C INLET	Standard	0.050
INLET A4.2	6,285.30	6,285.30	11.23	11.23	11.23	6,279.29	6,279.79	20' TYPE R INLET	Standard	0.100
MH 13	6,322.96	6,322.96	4.82	4.82	4.82	6,317.87	6,318.37	5' DIA MH	Standard	0.400
MH 15	6,321.51	6,321.51	1.29	1.29	1.29	6,312.73	6,313.23	5' DIA MH	Standard	0.640
MH 47	6,286.93	6,286.93	19.56	19.56	19.56	6,277.19	6,277.69	5' DIA MH	Standard	0.100
MH 48	6,287.20	6,287.20	15.15	15.15	15.15	6,273.79	6,274.29	5' DIA MH	Standard	0.050
MH 49	6,284.16	6,284.16	14.51	14.51	14.51	6,268.96	6,269.46	5' DIA MH	Standard	0.100
MH 50	6,273.55	6,273.55	16.37	16.37	16.37	6,268.57	6,269.07	5' DIA MH	Standard	0.100
MH 51	6,269.29	6,269.29	9.64	9.64	9.64	6,255.94	6,256.44	5' DIA MH	Standard	0.400
MH 52	6,259.90	6,259.90	5.73	5.73	5.73	6,244.48	6,244.98	5' DIA MH	Standard	0.400
POND A1 OUTLET STRUCTURE	6,319.18	6,319.18	1.94	1.94	1.94	6,314.48	6,314.98		Standard	0.050
POND A2 OUTLET STRUCTURE	6,304.73	6,304.73	2.09	2.09	2.09	6,304.07	6,304.57	Null Structure	Standard	0.050
POND A4 OUTLET STRUCTURE	6,267.08	6,267.08	8.91	8.91	8.91	6,262.23	6,262.73	TYPE C INLET	Standard	0.050
STRC-182	6,322.54	6,322.54	20.30	20.30	20.30	6,301.25	6,301.75	5' DIA MH	Standard	0.400
STRC-188	6,298.74	6,298.74	10.45	10.45	10.45	6,288.02	6,288.52	5' DIA MH	Standard	0.400
STRC-200	6,300.20	6,300.20	19.56	19.56	19.56	6,285.93	6,286.43	6' DIA MH	Standard	0.100
STRC-201	6,283.48	6,283.48	19.56	19.56	19.56	6,271.37	6,271.87	6' DIA MH	Standard	0.400
STRC-203	6,307.48	6,307.48	14.21	14.21	14.21	6,299.58	6,300.08	5' DIA MH	Standard	0.400
STRC-204	6,288.77	6,288.77	19.56	19.56	19.56	6,276.18	6,276.68	6' DIA MH	Standard	0.050
STRC-205	6,293.92	6,293.92	19.56	19.56	19.56	6,280.98	6,281.48	6' DIA MH	Standard	0.050
STRC-218	6,308.79	6,308.79	4.01	4.01	4.01	6,303.47	6,303.97	4' DIA MH	Standard	0.400
STRC-219	6,302.44	6,302.44	10.45	10.45	10.45	6,296.16	6,296.83	5' DIA MH	Standard	1.770
STRC-227	6,321.35	6,321.35	9.93	9.93	9.93	6,311.48	6,312.19	5' DIA MH	Standard	1.770
STRC-228	6,322.86	6,322.86	3.11	3.11	3.11	6,315.55	6,316.05	5' DIA MH	Standard	1.770
STRC-229	6,318.05	6,318.05	4.72	4.72	4.72	6,313.02	6,313.52	5' DIA MH	Standard	1.770
STRC-230	6,317.88	6,317.88	19.45	19.45	19.45	6,306.44	6,306.94	5' DIA MH	Standard	0.400
STRC-240	6,271.92	6,271.92	13.83	13.83	13.83	6,257.98	6,258.48	6' DIA MH	Standard	0.100
STRC-241	6,274.94	6,274.94	19.56	19.56	19.56	6,263.95	6,264.45	6' DIA MH	Standard	0.050
STRC-246	6,325.94	6,325.94	4.40	4.40	4.40	6,313.74	6,314.24		Standard	0.640
STRC-248	6,328.45	6,328.45	3.81	3.81	3.81	6,320.18	6,320.68		Standard	0.400

Minimum Velocity Analysis: 25% of Diameter Flow Depth

FlexTable: Manhole Table

Label	Elevation (Ground) (ft)	Elevation (Rim) (ft)	Flow (Known) (cfs)	Flow (Local In) (cfs)	Flow (Total Out) (cfs)	Hydraulic Grade Line (Out) (ft)	Hydraulic Grade Line (In) (ft)	Notes	Headloss Method	Headloss Coefficient (Standard)
STRC-249	6,327.35	6,327.35	3.81	3.81	3.81	6,321.93	6,322.43	5' DIA MH	Standard	0.400
STRC-258	6,306.00	6,306.00	18.11	18.11	18.11	6,293.36	6,293.86		Standard	0.050

Minimum Velocity Analysis: 25% of Diameter Flow Depth

FlexTable: Conduit Table

Label	Start Node	Invert (Start) (ft)	Stop Node	Invert (Stop) (ft)	Length (User Defined) (ft)	Slope (Calculated) (ft/ft)	Diameter (in)	Manning's n	Flow (cfs)	Velocity (ft/s)	Hydraulic Grade Line (In) (ft)	Hydraulic Grade Line (Out) (ft)	Capacity (Full Flow) (cfs)
P-15	MH 15	6,312.30	O-A1	6,311.80	63.0	0.008	18.0	0.013	1.29	3.72	6,312.73	6,312.18	9.36
P-21 (1)	INLET A2.2	6,315.64	STRC-228	6,315.13	62.0	0.008	18.0	0.013	1.29	3.77	6,316.07	6,316.05	9.52
P-91 (1)	INLET A2.4	6,313.01	STRC-229	6,312.50	59.2	0.009	18.0	0.013	1.37	3.89	6,313.45	6,313.52	9.75
P-22	STRC-228	6,314.93	STRC-227	6,311.13	399.1	0.010	24.0	0.013	3.11	4.96	6,315.55	6,312.19	22.07
P-92	STRC-229	6,312.30	STRC-227	6,310.63	228.4	0.007	30.0	0.013	4.72	4.98	6,313.02	6,312.19	35.07
P-11	INLET A1.1	6,319.22	INLET A1.2	6,318.49	73.2	0.010	24.0	0.013	3.11	5.05	6,319.84	6,319.42	22.59
P-43	INLET A4.1	6,280.30	INLET A4.2	6,279.73	98.8	0.006	36.0	0.013	7.10	5.06	6,281.14	6,280.49	50.67
P-12	INLET A1.2	6,318.29	MH 13	6,317.83	42.4	0.011	24.0	0.013	3.26	5.27	6,318.92	6,318.33	23.55
P-14	POND A1 OUTLET STRUCTURE	6,313.96	MH 15	6,312.50	79.8	0.018	18.0	0.013	1.94	5.63	6,314.48	6,313.23	14.21
P-91	INLET A2.3	6,312.68	STRC-229	6,312.50	9.8	0.018	18.0	0.013	1.94	5.64	6,313.50	6,313.52	14.24
CULVERT 10	CULVERT 10 ENTRANCE	6,322.74	STRC-249	6,321.75	69.5	0.014	24.0	0.013	3.68	6.01	6,323.41	6,322.43	27.01
P-90	POND A2 OUTLET STRUCTURE	6,303.52	STRC-218	6,302.70	38.6	0.021	18.0	0.013	2.09	6.07	6,304.07	6,303.97	15.32
CULVERT 10 (3)	STRC-249	6,321.25	STRC-248	6,320.00	84.9	0.015	24.0	0.013	3.81	6.14	6,321.93	6,320.50	27.45
CULVERT 10 (2)	STRC-248	6,319.50	STRC-246	6,315.50	267.1	0.015	24.0	0.013	3.81	6.18	6,320.18	6,316.00	27.68
P-21	INLET A2.1	6,315.31	STRC-228	6,315.13	7.0	0.026	18.0	0.013	2.33	6.71	6,315.98	6,316.05	16.88
CULVERT 10 (1)	STRC-246	6,313.00	CULVERT 10 EXIT	6,311.00	99.4	0.020	24.0	0.013	4.40	7.16	6,313.74	6,311.50	32.09
P-66 (1)	STRC-240	6,256.85	CULVERT 12 EXIT	6,256.17	68.0	0.010	42.0	0.013	13.83	7.33	6,257.98	6,257.06	100.59
CULVERT 13 (1) (1)	STRC-188	6,287.00	CULVERT 13 EXIT	6,286.64	28.2	0.013	36.0	0.013	10.45	7.50	6,288.02	6,287.44	75.36
P-75 (1)	STRC-219	6,295.14	CULVERT 11 EXIT	6,294.24	70.1	0.013	36.0	0.013	10.45	7.51	6,296.16	6,295.00	75.55
CULVERT 7	CULVERT 7 ENTRANCE	6,337.34	CULVERT 7 EXIT	6,334.41	177.1	0.017	30.0	0.013	7.35	7.57	6,338.24	6,335.04	52.76
P-86	MH 49	6,267.80	MH 50	6,266.60	107.4	0.011	42.0	0.013	14.51	7.74	6,268.96	6,269.07	106.34
P-13	MH 13	6,317.10	FB-A1	6,315.54	65.7	0.024	24.0	0.013	4.82	7.79	6,317.87	6,316.04	34.85
P-47	MH 48	6,272.60	MH 49	6,268.07	377.2	0.012	42.0	0.013	15.15	8.04	6,273.79	6,269.46	110.26
CULVERT 11	CULVERT 11 ENTRANCE	6,311.88	STRC-230	6,310.00	125.7	0.015	36.0	0.013	11.23	8.10	6,312.94	6,310.75	81.58
P-45	INLET A4.2	6,278.23	MH 47	6,276.98	82.2	0.015	36.0	0.013	11.23	8.15	6,279.29	6,277.73	82.26
P-87	MH 50	6,266.30	POND A4 FOREBAY	6,265.91	27.2	0.014	42.0	0.013	16.37	8.75	6,268.57	6,268.59	120.47
P-47 (1)	MH 52	6,243.63	POND A4 OUTFALL	6,238.16	161.6	0.034	24.0	0.013	5.73	9.30	6,244.48	6,238.67	41.61
P-75	STRC-203	6,298.38	STRC-219	6,295.34	128.9	0.024	36.0	0.013	14.21	10.19	6,299.58	6,296.83	102.42
P-22 (2) (1)	STRC-227	6,310.43	POND A2 FOREBAY	6,305.46	161.3	0.031	30.0	0.013	9.93	10.30	6,311.48	6,310.35	71.99
P-66	STRC-201	6,270.02	STRC-241	6,267.59	121.6	0.020	42.0	0.013	19.56	10.37	6,271.37	6,268.47	142.22
P-74	STRC-200	6,284.58	STRC-205	6,283.63	47.5	0.020	42.0	0.013	19.56	10.37	6,285.93	6,284.58	142.24
P-74 (1)	STRC-204	6,274.83	STRC-201	6,274.02	40.4	0.020	42.0	0.013	19.56	10.38	6,276.18	6,274.98	142.39
P-74 (2)	STRC-205	6,279.63	STRC-204	6,278.83	39.9	0.020	42.0	0.013	19.56	10.38	6,280.98	6,279.79	142.52
P-46 (1)	MH 47	6,275.84	MH 48	6,273.33	124.9	0.020	42.0	0.013	19.56	10.39	6,277.19	6,274.21	142.61
P-66 (2)	STRC-241	6,262.60	STRC-240	6,261.85	37.0	0.020	42.0	0.013	19.56	10.42	6,263.95	6,262.82	143.24
P-84	CULVERT 7 EXIT	6,332.62	O-7	6,331.96	17.0	0.039	30.0	0.013	11.14	11.54	6,333.74	6,332.72	80.71
P-90 (1)	STRC-218	6,302.70	STRC-219	6,296.63	78.5	0.077	18.0	0.013	4.01	11.59	6,303.47	6,297.01	29.21
CULVERT 12	CULVERT 12 ENTRANCE	6,298.17	STRC-200	6,288.58	308.2	0.031	42.0	0.013	24.35	12.93	6,299.69	6,289.46	177.46
CULVERT 13 (1) (2)	STRC-258	6,292.00	STRC-188	6,289.15	73.8	0.039	36.0	0.013	18.11	13.02	6,293.36	6,289.94	131.06
CULVERT 11 (1)	STRC-230	6,305.03	STRC-203	6,298.58	141.9	0.045	36.0	0.013	19.45	14.09	6,306.44	6,300.08	142.20
P-46	POND A4 OUTLET STRUCTURE	6,261.17	MH 51	6,259.33	22.4	0.082	24.0	0.013	8.91	14.46	6,262.23	6,259.91	64.77
CULVERT 13 (1) (1)	STRC-182	6,299.80	STRC-258	6,296.15	73.8	0.049	36.0	0.013	20.30	14.70	6,301.25	6,296.94	148.32
CULVERT 13	CULVERT 13 ENTRANCE	6,315.12	STRC-182	6,305.00	200.9	0.050	36.0	0.013	20.50	14.84	6,316.57	6,305.75	149.68
P-46 (2)	MH 51	6,254.83	MH 52	6,248.20	69.3	0.096	24.0	0.013	9.64	15.62	6,255.94	6,248.71	69.94

HY-8 Culvert Analysis Report

Table 1 - Project Headwater Table

Crossing Name	Culvert Name	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Outlet Velocity (ft/s)
OS-8 Box Culvert	2x8x6 Box	877.00	877.00	6327.22	7.41	7.254	1.24	4.56	4.54	4.54	12.08

Crossing Input: OS-8 Box Culvert

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	876.000	cfs
Design Flow	877.000	cfs
Maximum Flow	878.000	cfs
TAILWATER DATA		
Channel Type	Trapezoidal Channel	
Bottom Width	22.000	ft
Side Slope (H:V)	4.000	_:1
Channel Slope	0.0040	ft/ft
Manning's n (channel)	0.030	
Channel Invert Elevation	6319.230	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	6329.700	ft
Roadway Surface	Paved	
Top Width	111.000	ft

Culvert Input: OS-8 Box Culvert

Parameter	Value	Units
CULVERT DATA		
Name	2x8x6 Box	
Shape	Concrete Box	
Material	Concrete	
Span	8.000	ft
Rise	6.000	ft
Embedment Depth	0.000	in
Manning's n	0.013	
Culvert Type	Straight	
Inlet Configuration	1:1 Bevel (45° flare) Wingwall (Ke=0.2)	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	6319.810	ft
Outlet Station	144.600	ft
Outlet Elevation	6319.230	ft
Number of Barrels	2	
Computed Culvert Slope	0.004011	ft/ft

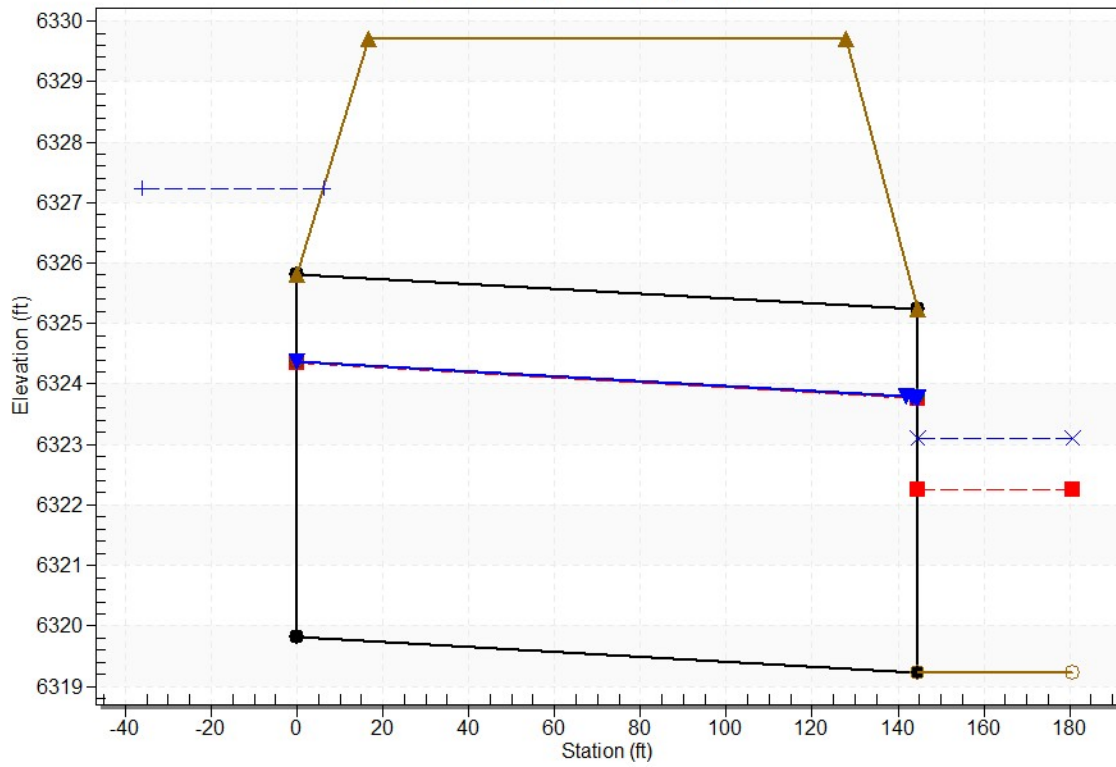
Table 2 - Culvert Summary Table: 2x8x6 Box

Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
876.00	876.00	6327.22	7.41	7.249	1.23	7-M2c	4.56	4.53	4.53	3.86	12.08	6.05
876.20	876.20	6327.22	7.41	7.250	1.23	7-M2c	4.56	4.53	4.53	3.86	12.08	6.05
876.40	876.40	6327.22	7.41	7.251	1.23	7-M2c	4.56	4.53	4.53	3.86	12.08	6.06
876.60	876.60	6327.22	7.41	7.252	1.24	7-M2c	4.56	4.53	4.53	3.86	12.08	6.06
876.80	876.80	6327.22	7.41	7.253	1.24	7-M2c	4.56	4.53	4.53	3.87	12.08	6.06
877.00	877.00	6327.22	7.41	7.254	1.24	7-M2c	4.56	4.54	4.54	3.87	12.08	6.06
877.20	877.20	6327.22	7.41	7.255	1.24	7-M2c	4.56	4.54	4.54	3.87	12.09	6.06
877.40	877.40	6327.23	7.42	7.256	1.24	7-M2c	4.56	4.54	4.54	3.87	12.09	6.06
877.60	877.60	6327.23	7.42	7.257	1.24	7-M2c	4.56	4.54	4.54	3.87	12.09	6.06
877.80	877.80	6327.23	7.42	7.258	1.24	7-M2c	4.57	4.54	4.54	3.87	12.09	6.06
878.00	878.00	6327.23	7.42	7.259	1.24	7-M2c	4.57	4.54	4.54	3.87	12.09	6.06
1436.73	1273.39	6330.36	10.55	9.570	1.76	7-M2c	6.00	5.82	5.82	4.95	13.68	6.94

Water Surface Profile Plot for Culvert: 2x8x6 Box

Crossing - OS-8 Box Culvert , Design Discharge - 877.0 cfs

Culvert - 2x8x6 Box, Culvert Discharge - 877.0 cfs



HY-8 Culvert Analysis Report

Table 1 - Project Headwater Table

Crossing Name	Culvert Name	Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Outlet Velocity (ft/s)
OS-9	Culvert 9	78.00	78.00	6321.74	3.40	0.106	1.13	1.37	2.03	1.39	12.12

Crossing Input: OS-9

Parameter	Value	Units
DISCHARGE DATA		
Discharge Method	Minimum, Design, and Maximum	
Minimum Flow	0.000	cfs
Design Flow	78.000	cfs
Maximum Flow	78.000	cfs
TAILWATER DATA		
Channel Type	Rectangular Channel	
Bottom Width	10.000	ft
Channel Slope	0.0300	ft/ft
Manning's n (channel)	0.030	
Channel Invert Elevation	6315.000	ft
Rating Curve	View...	
ROADWAY DATA		
Roadway Profile Shape	Constant Roadway Elevation	
First Roadway Station	0.000	ft
Crest Length	100.000	ft
Crest Elevation	6324.250	ft
Roadway Surface	Paved	
Top Width	111.000	ft

Culvert Input: OS-9

Parameter	Value	Units
CULVERT DATA		
Name	Culvert 9	
Shape	Circular	
Material	Concrete	
Diameter	3.000	ft
Embedment Depth	0.000	in
Manning's n	0.013	
Culvert Type	Straight	
Inlet Configuration	Mitered to Conform to Slope (Ke=0.7)	
Inlet Depression?	No	
SITE DATA		
Site Data Input Option	Culvert Invert Data	
Inlet Station	0.000	ft
Inlet Elevation	6318.340	ft
Outlet Station	179.400	ft
Outlet Elevation	6315.000	ft
Number of Barrels	2	
Computed Culvert Slope	0.018618	ft/ft

Table 2 - Culvert Summary Table: Culvert 9

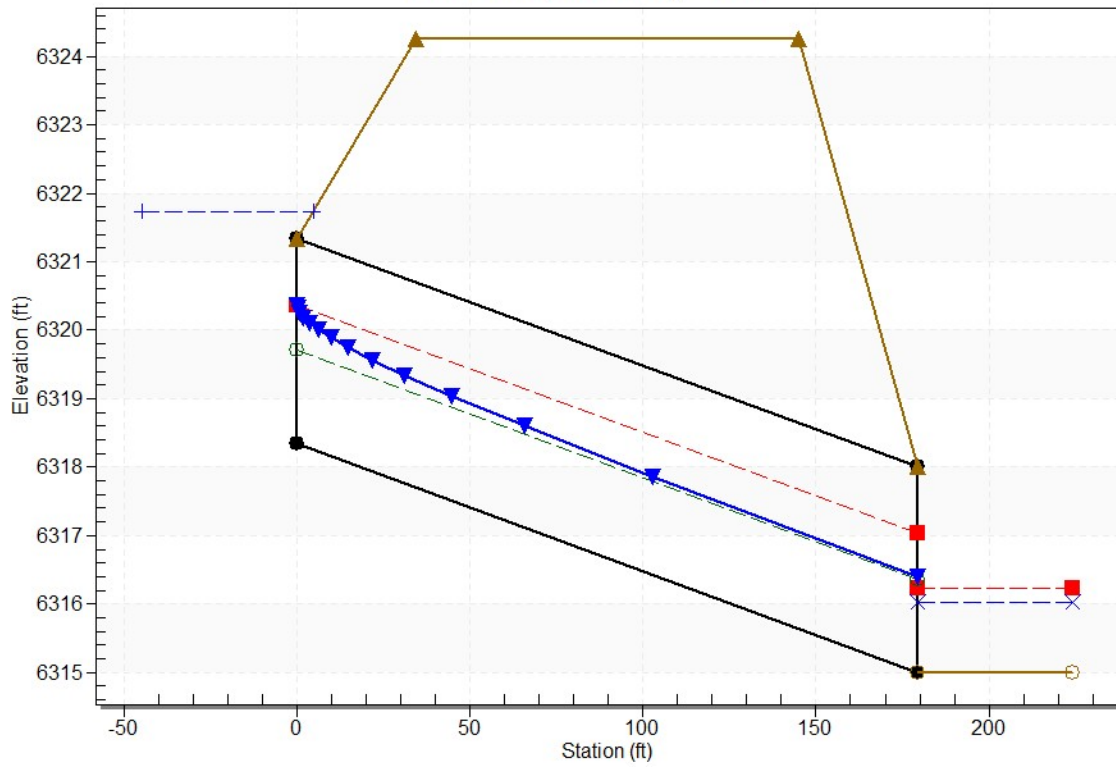
Total Discharge (cfs)	Culvert Discharge (cfs)	Headwater Elevation (ft)	Inlet Control Depth (ft)	Outlet Control Depth (ft)	HW / D (ft)	Flow Type	Normal Depth (ft)	Critical Depth (ft)	Outlet Depth (ft)	Tailwater Depth (ft)	Outlet Velocity (ft/s)	Tailwater Velocity (ft/s)
0.00	0.00	6318.34	0.00	0.000	0.00	0-NF	0.00	0.00	0.00	0.00	0.00	0.00
7.80	7.80	6319.23	0.89	0.0*	0.30	1-S2n	0.42	0.62	0.42	0.24	6.44	3.23
15.60	15.60	6319.62	1.28	0.0*	0.43	1-S2n	0.59	0.88	0.59	0.37	7.89	4.22
23.40	23.40	6319.94	1.60	0.0*	0.53	1-S2n	0.72	1.09	0.72	0.48	8.88	4.92
31.20	31.20	6320.21	1.87	0.0*	0.62	1-S2n	0.84	1.26	0.84	0.57	9.64	5.48
39.00	39.00	6320.46	2.12	0.0*	0.71	1-S2n	0.94	1.42	0.94	0.65	10.27	5.96
46.80	46.80	6320.69	2.35	0.0*	0.78	1-S2n	1.04	1.56	1.04	0.73	10.81	6.37
54.60	54.60	6320.93	2.59	0.0*	0.86	1-S2n	1.12	1.69	1.12	0.81	11.27	6.74
62.40	62.40	6321.18	2.84	0.0*	0.95	1-S2n	1.21	1.81	1.23	0.88	11.44	7.08
70.20	70.20	6321.44	3.10	0.0*	1.03	5-S2n	1.29	1.93	1.32	0.95	11.71	7.39
78.00	78.00	6321.74	3.40	0.106	1.13	5-S2n	1.37	2.03	1.39	1.02	12.12	7.67
134.73	127.51	6324.33	5.99	3.220	2.00	5-S2n	1.85	2.57	1.90	1.45	13.53	9.28

* Full Flow Headwater elevation is below inlet invert.

Water Surface Profile Plot for Culvert: Culvert 9

Crossing - OS-9, Design Discharge - 78.0 cfs

Culvert - Culvert 9, Culvert Discharge - 78.0 cfs



Dawson Trails Boulevard (North)

CORE Project #: 20-224
Prepared By: ACJ

ROADSIDE SWALE DESIGN SUMMARY

-REFERENCE UDFCD Vol.1 Section 8.6.3

SUB-BASIN DATA				ROADSIDE SWALE PARAMETERS					SWALE DESIGN		OPEN CHANNEL FLOW PROPERTIES			CULVERT APPROACH		
Offsite Sub-basin	Design Pt.	AREA ac.	Q100 cfs	Begin Road Sta.	End Road Sta.	Bottom Width (ft)	Side Slope	Swale Grade (%)	Depth per Fig. 8-22 (in.)	Lining per Fig. 8-22	Max velocity ft/s	Flow depth ft	Froude Number	Swale Grade (%)	Depth per Fig. 8-22 (in.)	Lining per Fig. 8-22
OS-7N	7N	10.7	24.4	62+50	67+48	4	5:1	1.10%	18	Type VL	3.48	0.85	0.67	10.0%	6	Type L
OS-8S	8S	5.7	11.8	67+48	79+00	4	5:1	1.10%	12	Grass	2.81	0.60	0.64	10.0%	6	Type M
OS-9S	9S	10.2	19.2	80+30	83+70	4	5:1	1.32%	12	Grass	3.44	0.73	0.71	5.0%	18	Type M
OS-9N	9N	21.0	34.6	83+70	89+97	4	5:1	1.34%	18	Type VL	4.10	0.96	0.74	5.0%	18	Type M
OS-10S	10S	0.5	1.6	89+97	92+20	4	5:1	1.38%	6	Grass	1.70	0.19	0.69	10.0%	6	Grass
OS-11S	11S	3.4	6.6	92+50	97+00	4	5:1	1.55%	12	Grass	2.66	0.41	0.73	10.0%	6	Type M
OS-11	OS11	18.8	23.8	97+00	99+75	4	5:1	1.30%	18	Type VL	3.65	0.81	0.71	5.0%	18	Type M
OS-11N	11N	5.0	10.6	99+75	103+25	4	5:1	2.78%	12	Grass	3.77	0.45	0.99	10.0%	12	Type M
OS-12S	12S	0.3	0.8	103+25	104+50	4	5:1	1.25%	6	Grass	1.22	0.14	0.57	10.0%	6	Grass
OS-12N	12N	0.6	1.5	106+75	110+20	4	5:1	1.90%	6	Grass	1.82	0.17	0.78	10.0%	6	Grass
OS-13S	13S	0.3	0.7	110+60	111+40	4	5:1	0.80%	6	Grass	1.06	0.14	0.50	10.0%	6	Grass

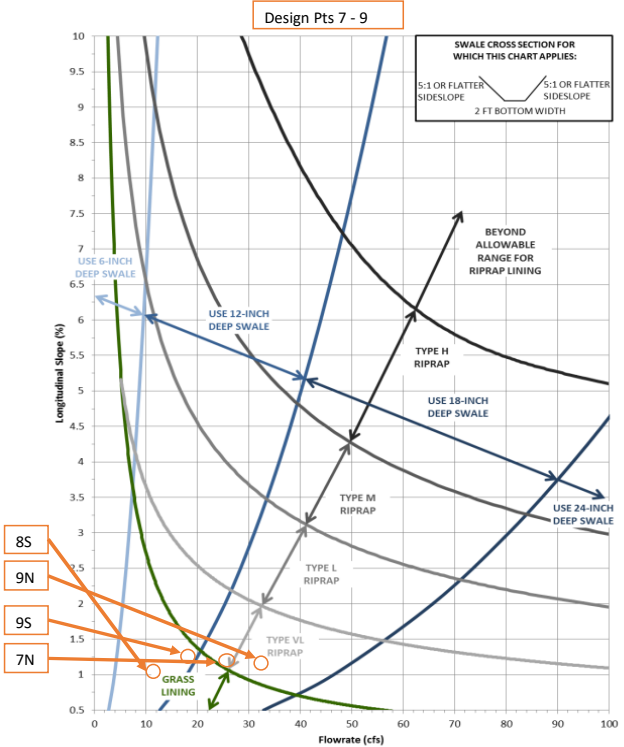


Figure 8-22. Swale stability chart; 2- to 4-foot bottom width and side slopes between 5:1 and 10:1 (Note: Riprap classifications refer to gradation for riprap used in soil riprap or void-filled riprap. See Figure 8-34 for gradations.) (Source: Muller Engineering Company)

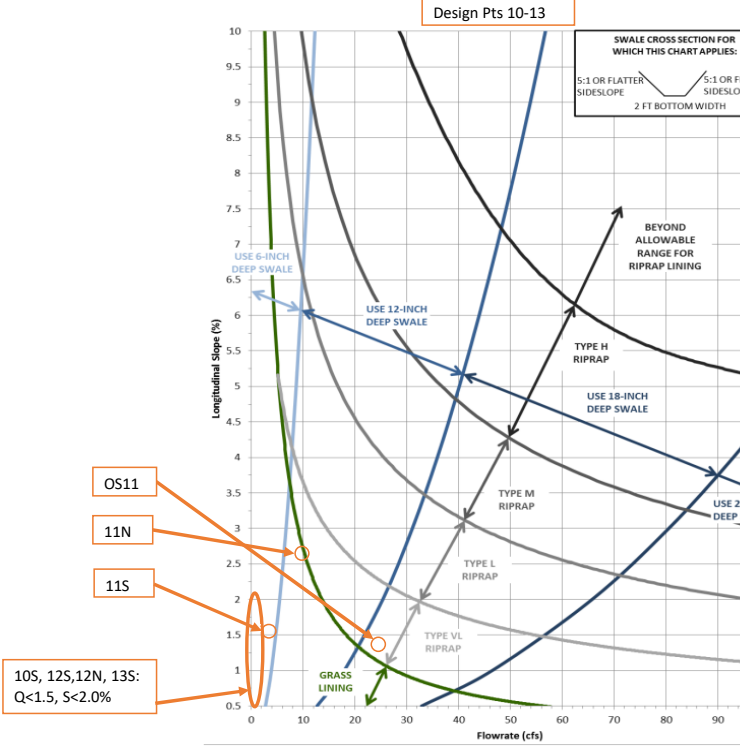


Figure 8-22. Swale stability chart; 2- to 4-foot bottom width and side slopes between 5:1 and 10:1 (Note: Riprap classifications refer to gradation for riprap used in soil riprap or void-filled riprap. See Figure 8-34 for gradations.) (Source: Muller Engineering Company)

Channel Report

Roadside Swale OS-7N

Trapezoidal

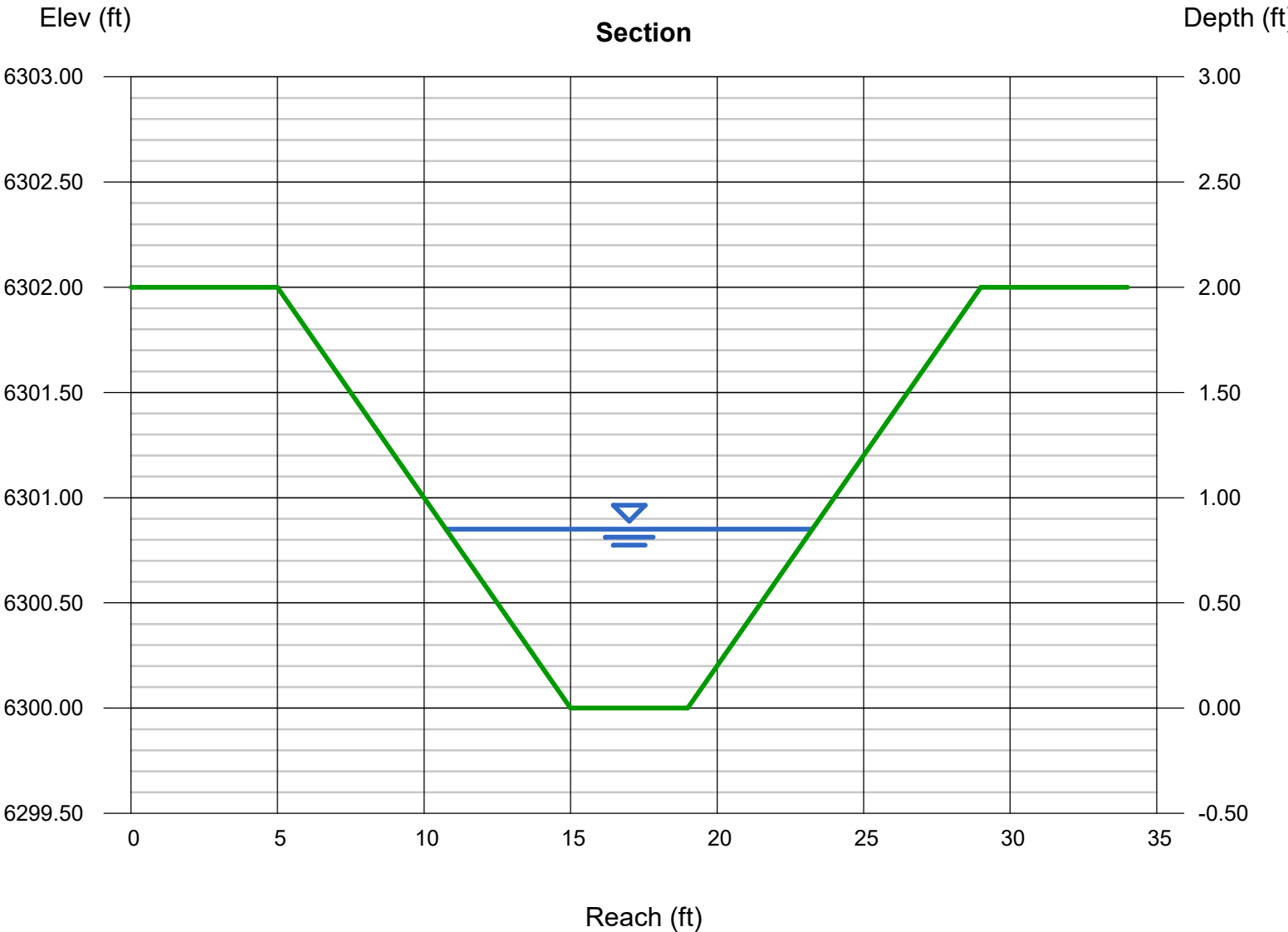
Bottom Width (ft)	= 4.00
Side Slopes (z:1)	= 5.00, 5.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 6300.00
Slope (%)	= 1.10
N-Value	= 0.030

Highlighted

Depth (ft)	= 0.85
Q (cfs)	= 24.40
Area (sqft)	= 7.01
Velocity (ft/s)	= 3.48
Wetted Perim (ft)	= 12.67
Crit Depth, Yc (ft)	= 0.77
Top Width (ft)	= 12.50
EGL (ft)	= 1.04

Calculations

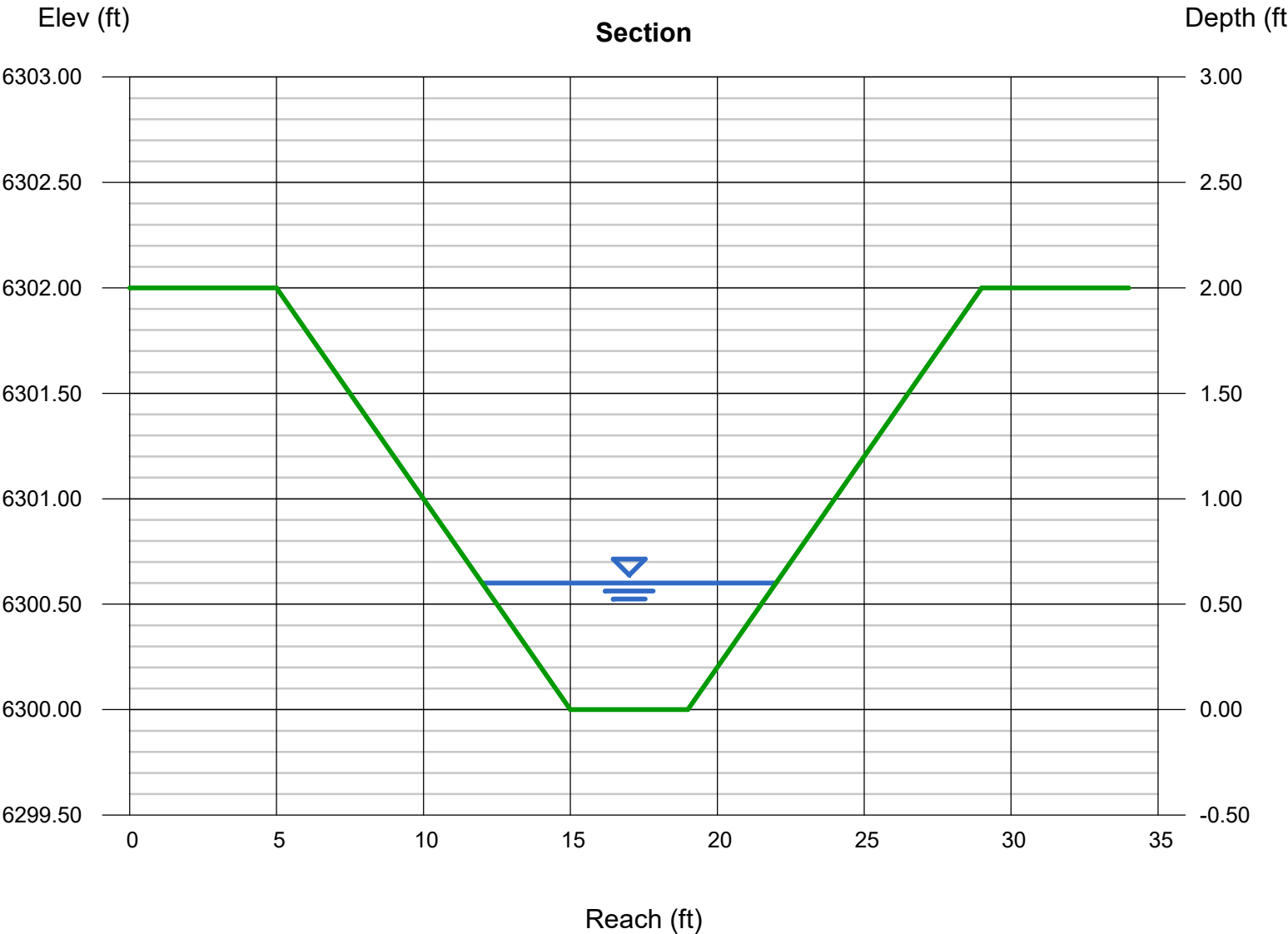
Compute by:	Known Q
Known Q (cfs)	= 24.40



Channel Report

Roadside Swale OS-8S

Trapezoidal		Highlighted	
Bottom Width (ft)	= 4.00	Depth (ft)	= 0.60
Side Slopes (z:1)	= 5.00, 5.00	Q (cfs)	= 11.80
Total Depth (ft)	= 2.00	Area (sqft)	= 4.20
Invert Elev (ft)	= 6300.00	Velocity (ft/s)	= 2.81
Slope (%)	= 1.10	Wetted Perim (ft)	= 10.12
N-Value	= 0.030	Crit Depth, Yc (ft)	= 0.52
Calculations		Top Width (ft)	= 10.00
Compute by:	Known Q	EGL (ft)	= 0.72
Known Q (cfs)	= 11.80		



Channel Report

Roadside Swale OS-9S

Trapezoidal

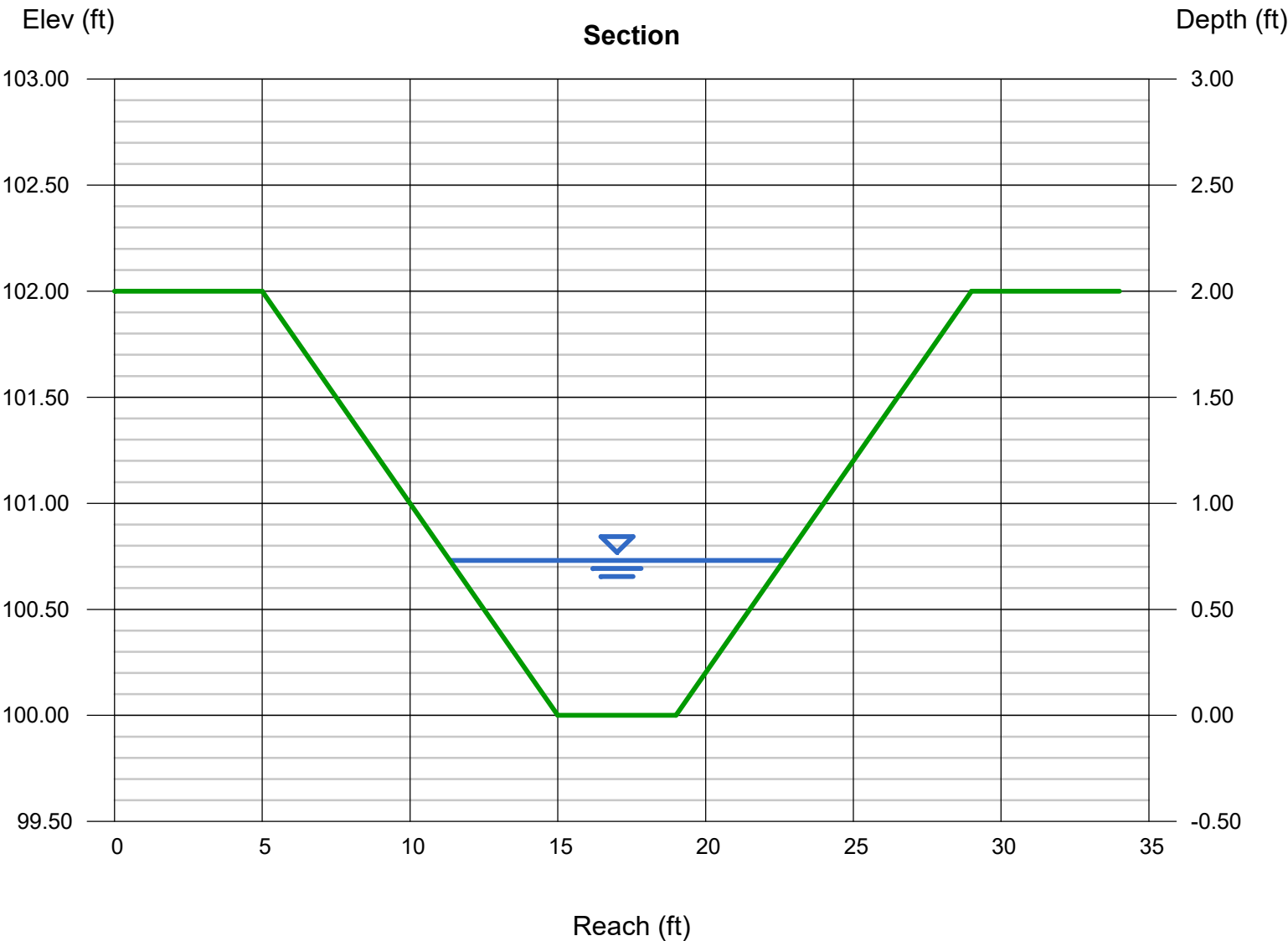
Bottom Width (ft)	= 4.00
Side Slopes (z:1)	= 5.00, 5.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 100.00
Slope (%)	= 1.32
N-Value	= 0.030

Highlighted

Depth (ft)	= 0.73
Q (cfs)	= 19.20
Area (sqft)	= 5.58
Velocity (ft/s)	= 3.44
Wetted Perim (ft)	= 11.44
Crit Depth, Yc (ft)	= 0.68
Top Width (ft)	= 11.30
EGL (ft)	= 0.91

Calculations

Compute by:	Known Q
Known Q (cfs)	= 19.20



Channel Report

Roadside Swale OS-9N

Trapezoidal

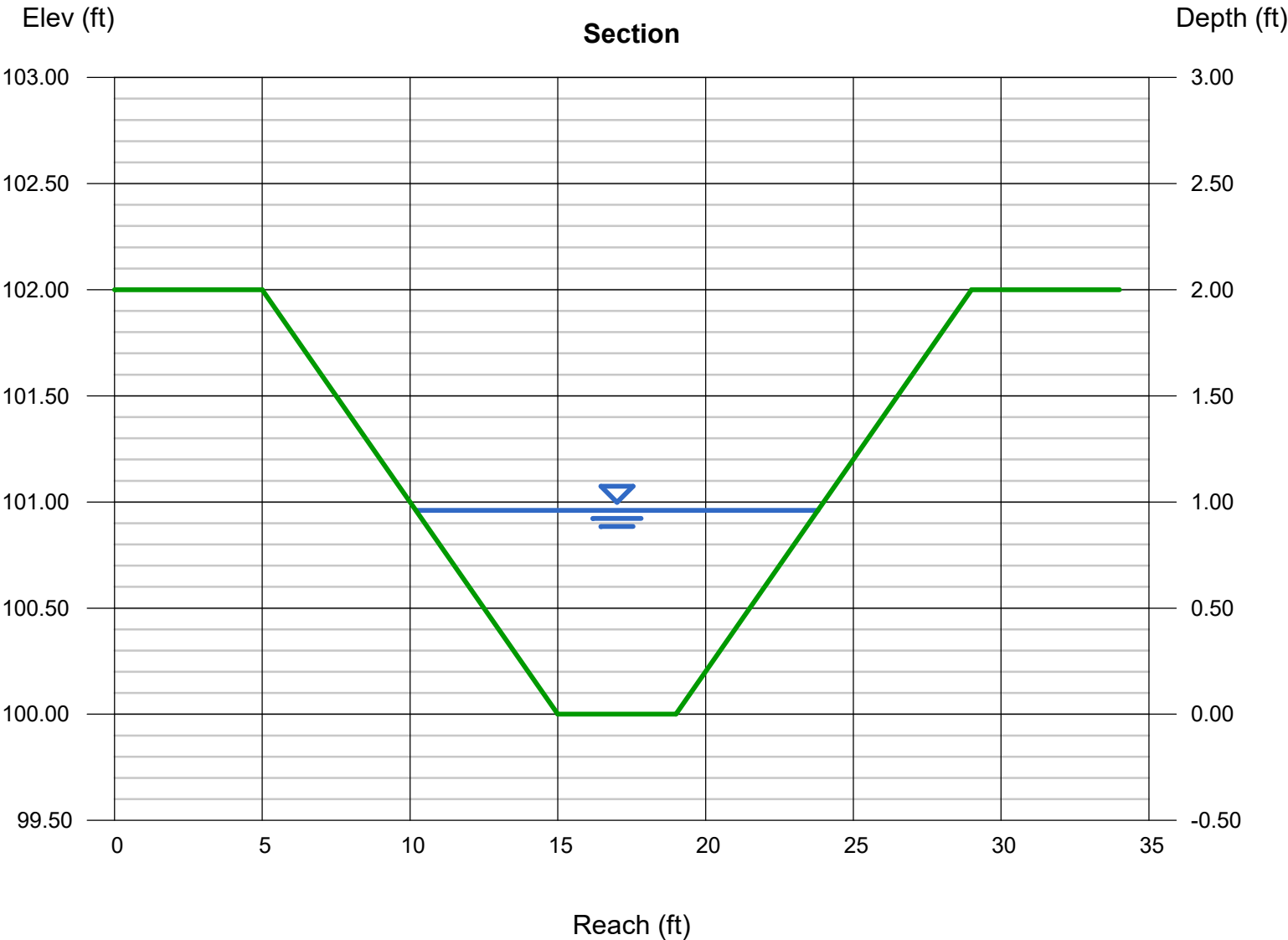
Bottom Width (ft)	= 4.00
Side Slopes (z:1)	= 5.00, 5.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 100.00
Slope (%)	= 1.34
N-Value	= 0.030

Calculations

Compute by:	Known Q
Known Q (cfs)	= 34.60

Highlighted

Depth (ft)	= 0.96
Q (cfs)	= 34.60
Area (sqft)	= 8.45
Velocity (ft/s)	= 4.10
Wetted Perim (ft)	= 13.79
Crit Depth, Yc (ft)	= 0.92
Top Width (ft)	= 13.60
EGL (ft)	= 1.22



Channel Report

Roadside Swale OS-10S

Trapezoidal

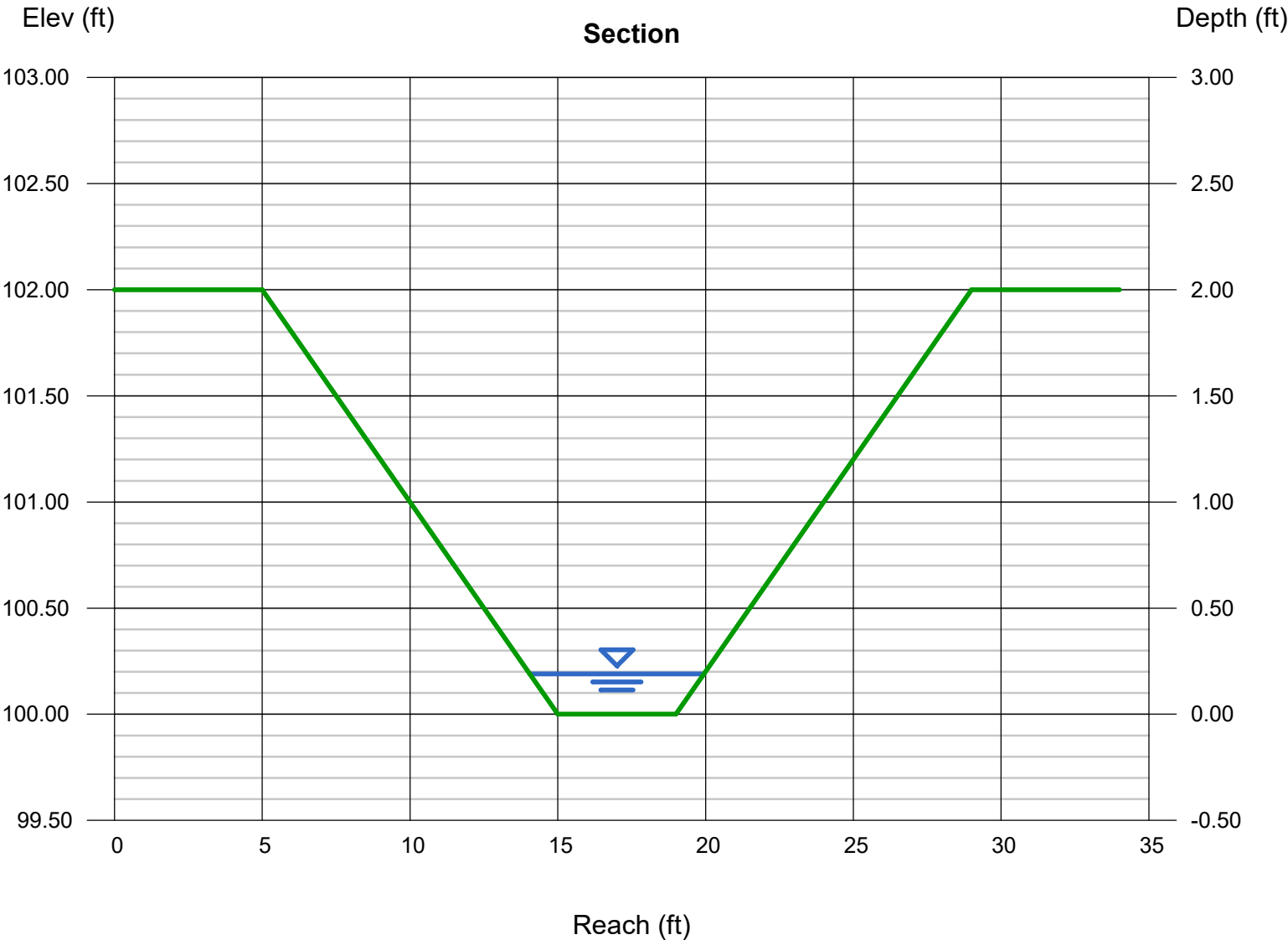
Bottom Width (ft)	= 4.00
Side Slopes (z:1)	= 5.00, 5.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 100.00
Slope (%)	= 1.38
N-Value	= 0.030

Highlighted

Depth (ft)	= 0.19
Q (cfs)	= 1.600
Area (sqft)	= 0.94
Velocity (ft/s)	= 1.70
Wetted Perim (ft)	= 5.94
Crit Depth, Yc (ft)	= 0.16
Top Width (ft)	= 5.90
EGL (ft)	= 0.23

Calculations

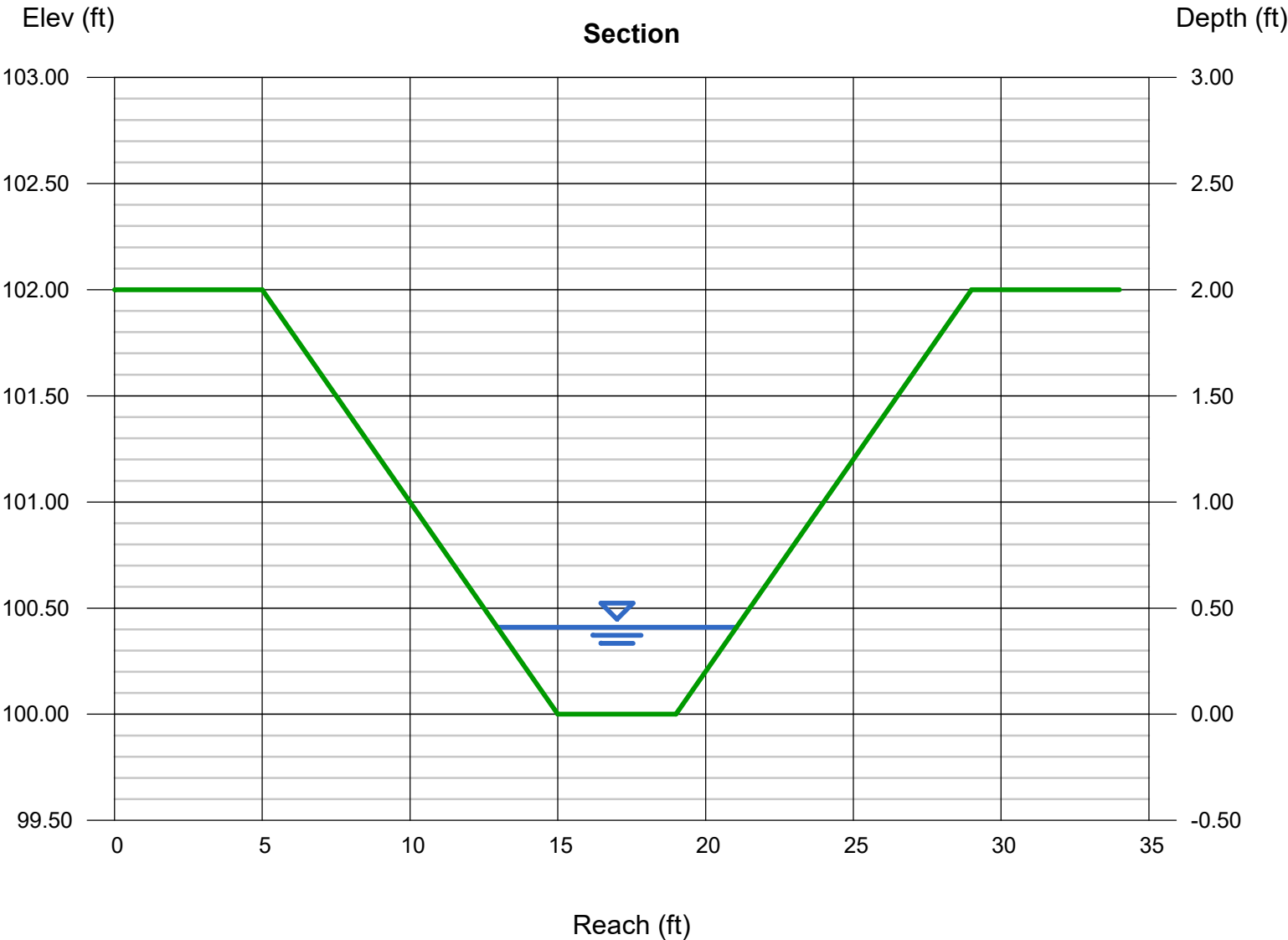
Compute by:	Known Q
Known Q (cfs)	= 1.60



Channel Report

Roadside Swale OS-11S

Trapezoidal		Highlighted	
Bottom Width (ft)	= 4.00	Depth (ft)	= 0.41
Side Slopes (z:1)	= 5.00, 5.00	Q (cfs)	= 6.600
Total Depth (ft)	= 2.00	Area (sqft)	= 2.48
Invert Elev (ft)	= 100.00	Velocity (ft/s)	= 2.66
Slope (%)	= 1.55	Wetted Perim (ft)	= 8.18
N-Value	= 0.030	Crit Depth, Yc (ft)	= 0.38
Calculations		Top Width (ft)	= 8.10
Compute by:	Known Q	EGL (ft)	= 0.52
Known Q (cfs)	= 6.60		



Channel Report

Roadside Swale OS-11

Trapezoidal

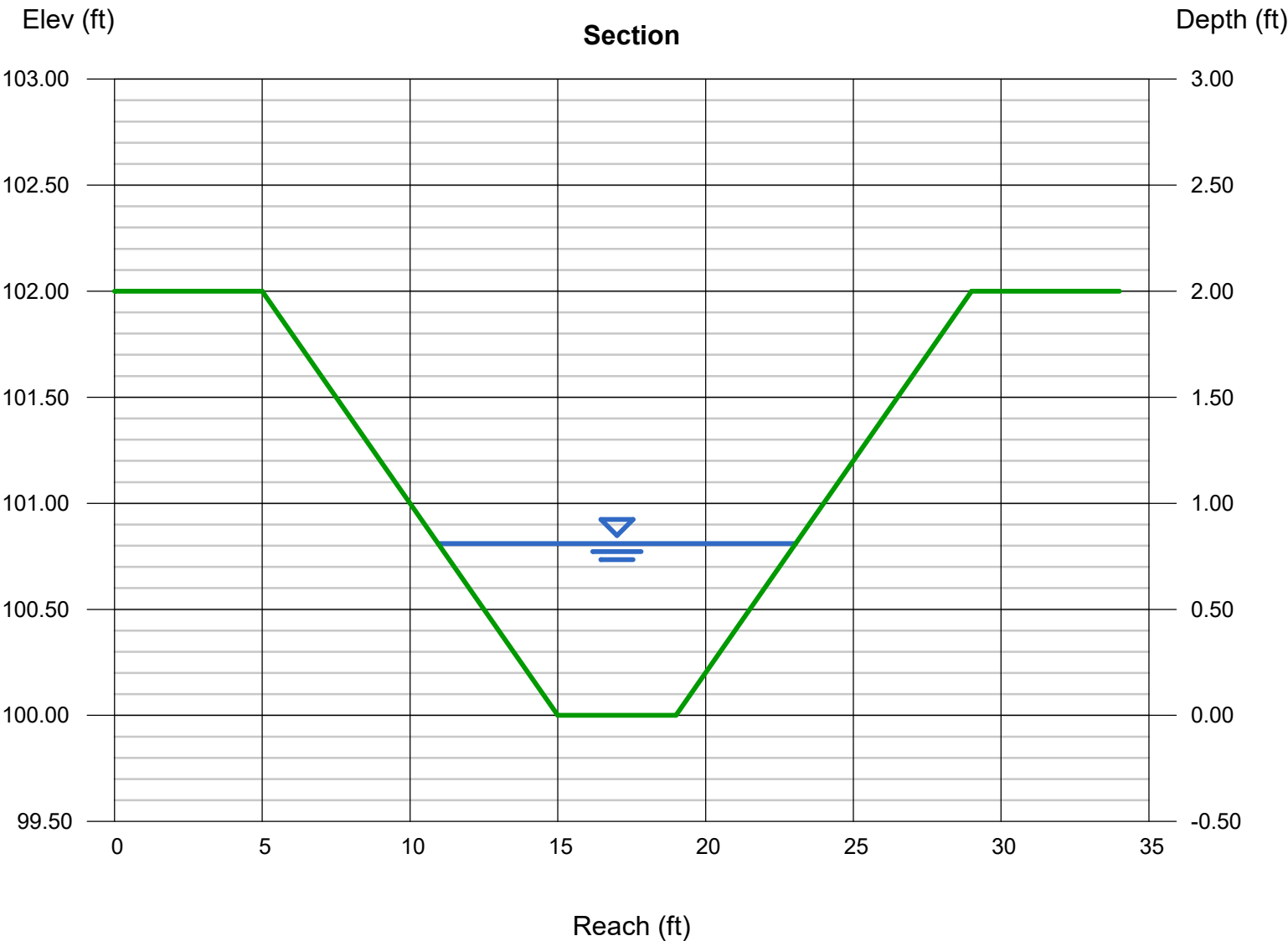
Bottom Width (ft) = 4.00
Side Slopes (z:1) = 5.00, 5.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 100.00
Slope (%) = 1.30
N-Value = 0.030

Calculations

Compute by: Known Q
Known Q (cfs) = 23.80

Highlighted

Depth (ft) = 0.81
Q (cfs) = 23.80
Area (sqft) = 6.52
Velocity (ft/s) = 3.65
Wetted Perim (ft) = 12.26
Crit Depth, Yc (ft) = 0.76
Top Width (ft) = 12.10
EGL (ft) = 1.02



Channel Report

Roadside Swale OS-11N

Trapezoidal

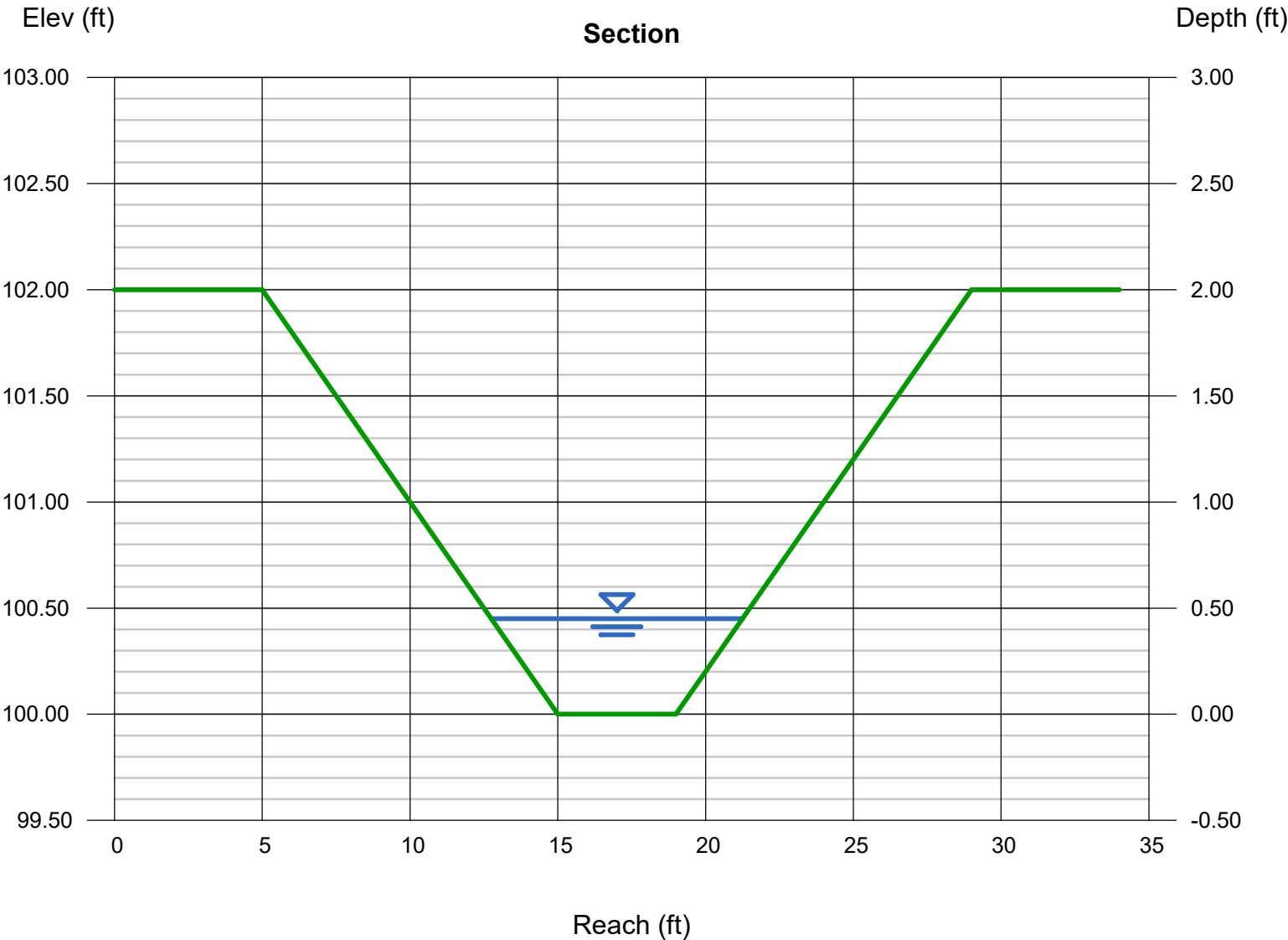
Bottom Width (ft) = 4.00
Side Slopes (z:1) = 5.00, 5.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 100.00
Slope (%) = 2.78
N-Value = 0.030

Calculations

Compute by: Known Q
Known Q (cfs) = 10.60

Highlighted

Depth (ft) = 0.45
Q (cfs) = 10.60
Area (sqft) = 2.81
Velocity (ft/s) = 3.77
Wetted Perim (ft) = 8.59
Crit Depth, Yc (ft) = 0.49
Top Width (ft) = 8.50
EGL (ft) = 0.67



Channel Report

Roadside Swale OS-12S

Trapezoidal

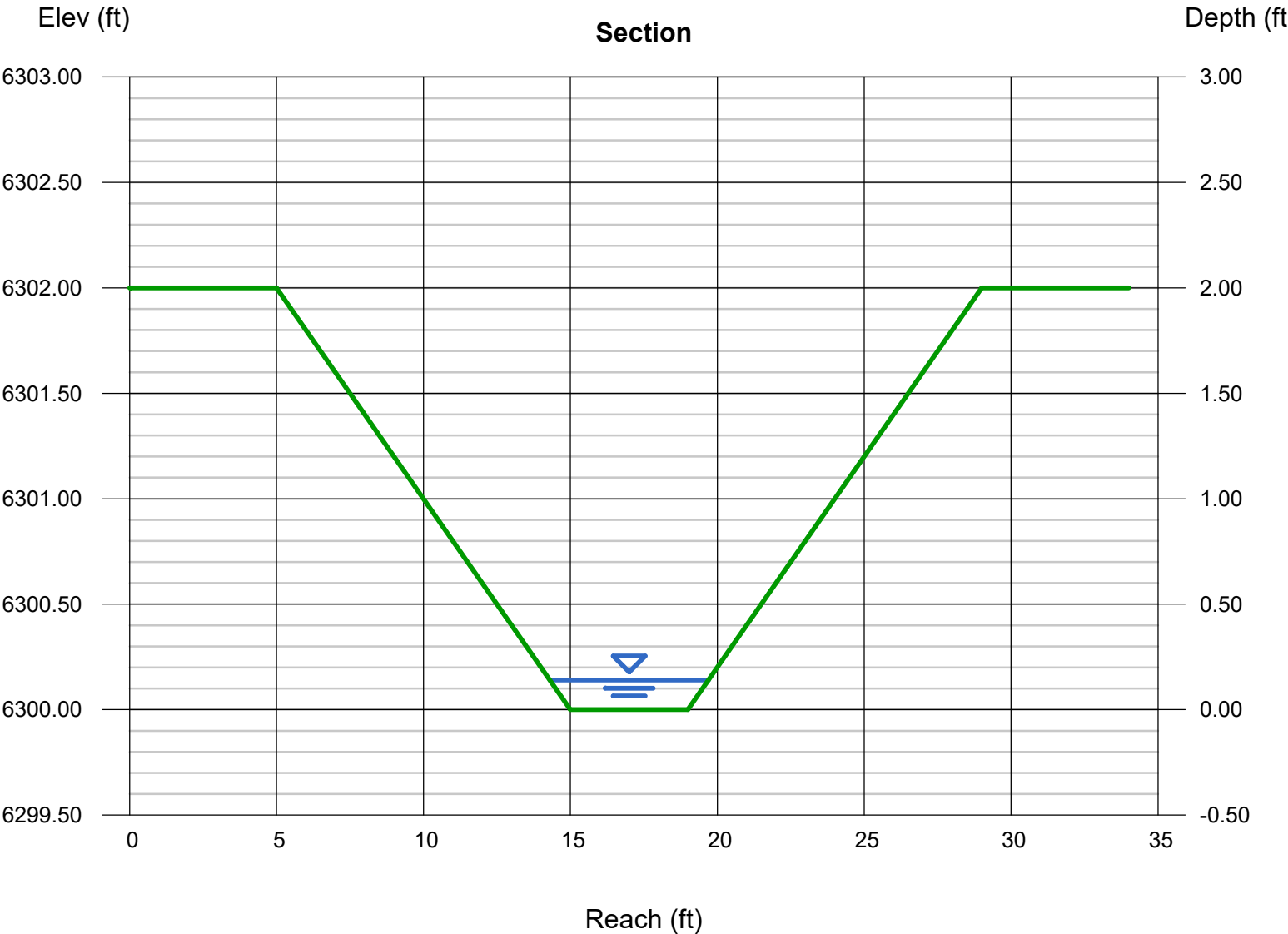
Bottom Width (ft) = 4.00
Side Slopes (z:1) = 5.00, 5.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 6300.00
Slope (%) = 1.25
N-Value = 0.030

Calculations

Compute by: Known Q
Known Q (cfs) = 0.80

Highlighted

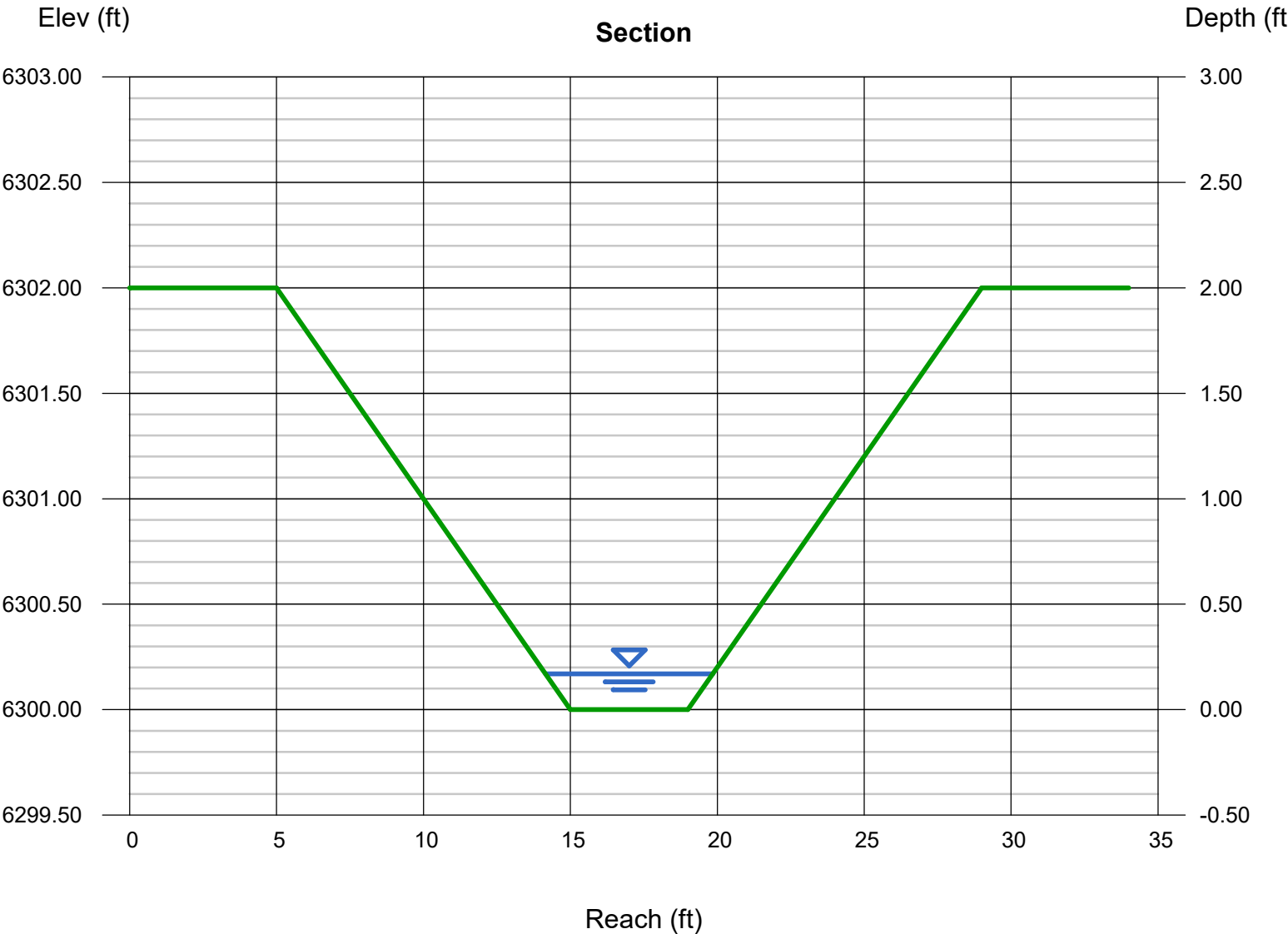
Depth (ft) = 0.14
Q (cfs) = 0.800
Area (sqft) = 0.66
Velocity (ft/s) = 1.22
Wetted Perim (ft) = 5.43
Crit Depth, Yc (ft) = 0.11
Top Width (ft) = 5.40
EGL (ft) = 0.16



Channel Report

Roadside Swale OS-12N

Trapezoidal		Highlighted	
Bottom Width (ft)	= 4.00	Depth (ft)	= 0.17
Side Slopes (z:1)	= 5.00, 5.00	Q (cfs)	= 1.500
Total Depth (ft)	= 2.00	Area (sqft)	= 0.82
Invert Elev (ft)	= 6300.00	Velocity (ft/s)	= 1.82
Slope (%)	= 1.90	Wetted Perim (ft)	= 5.73
N-Value	= 0.030	Crit Depth, Yc (ft)	= 0.16
Calculations		Top Width (ft)	= 5.70
Compute by:		EGL (ft)	= 0.22
Known Q (cfs)	= 1.50		



Channel Report

Roadside Swale OS-13S

Trapezoidal

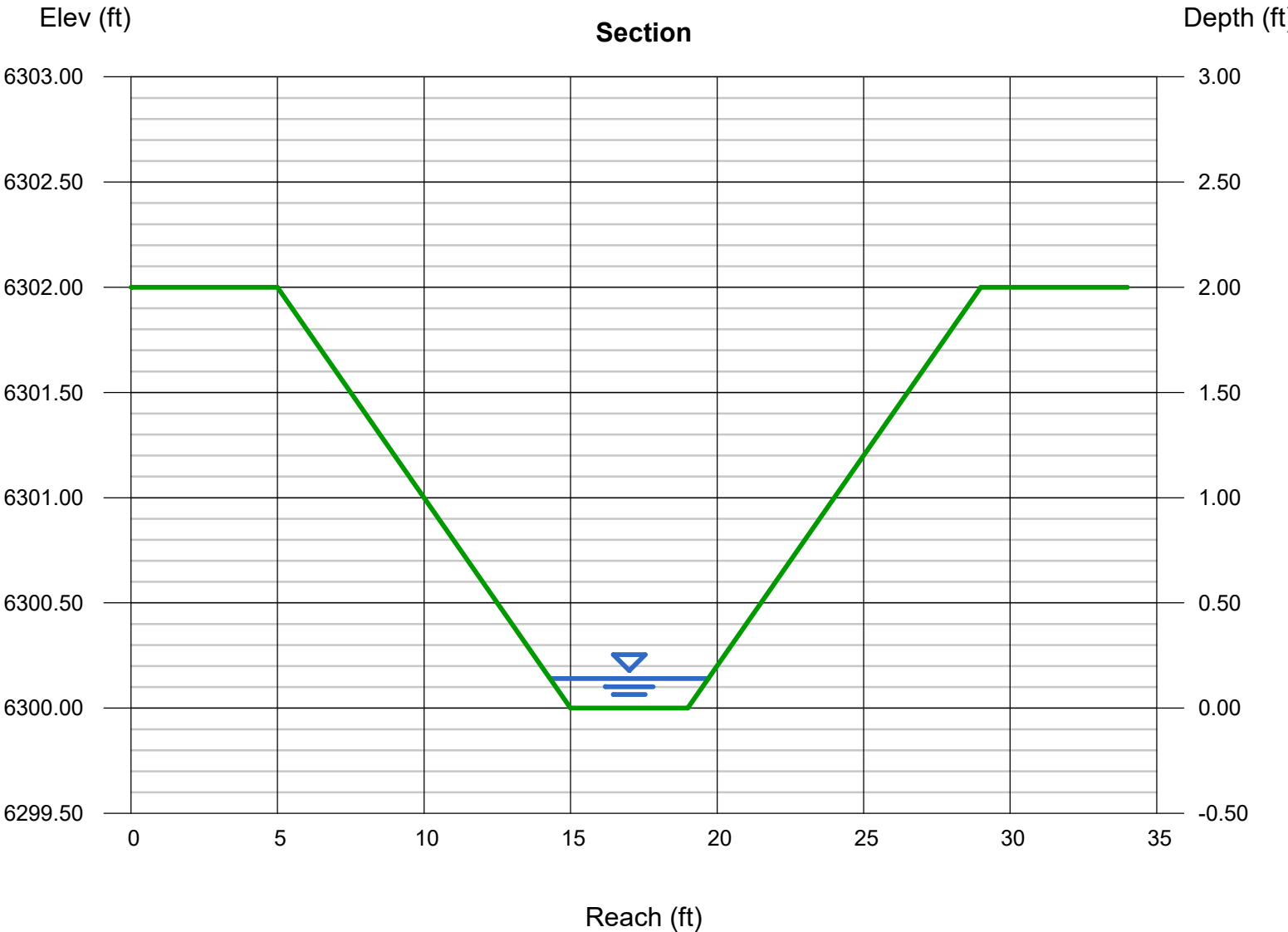
Bottom Width (ft)	= 4.00
Side Slopes (z:1)	= 5.00, 5.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 6300.00
Slope (%)	= 0.80
N-Value	= 0.030

Highlighted

Depth (ft)	= 0.14
Q (cfs)	= 0.700
Area (sqft)	= 0.66
Velocity (ft/s)	= 1.06
Wetted Perim (ft)	= 5.43
Crit Depth, Yc (ft)	= 0.10
Top Width (ft)	= 5.40
EGL (ft)	= 0.16

Calculations

Compute by:	Known Q
Known Q (cfs)	= 0.70



Channel Report

Culvert 9 Swale

Trapezoidal

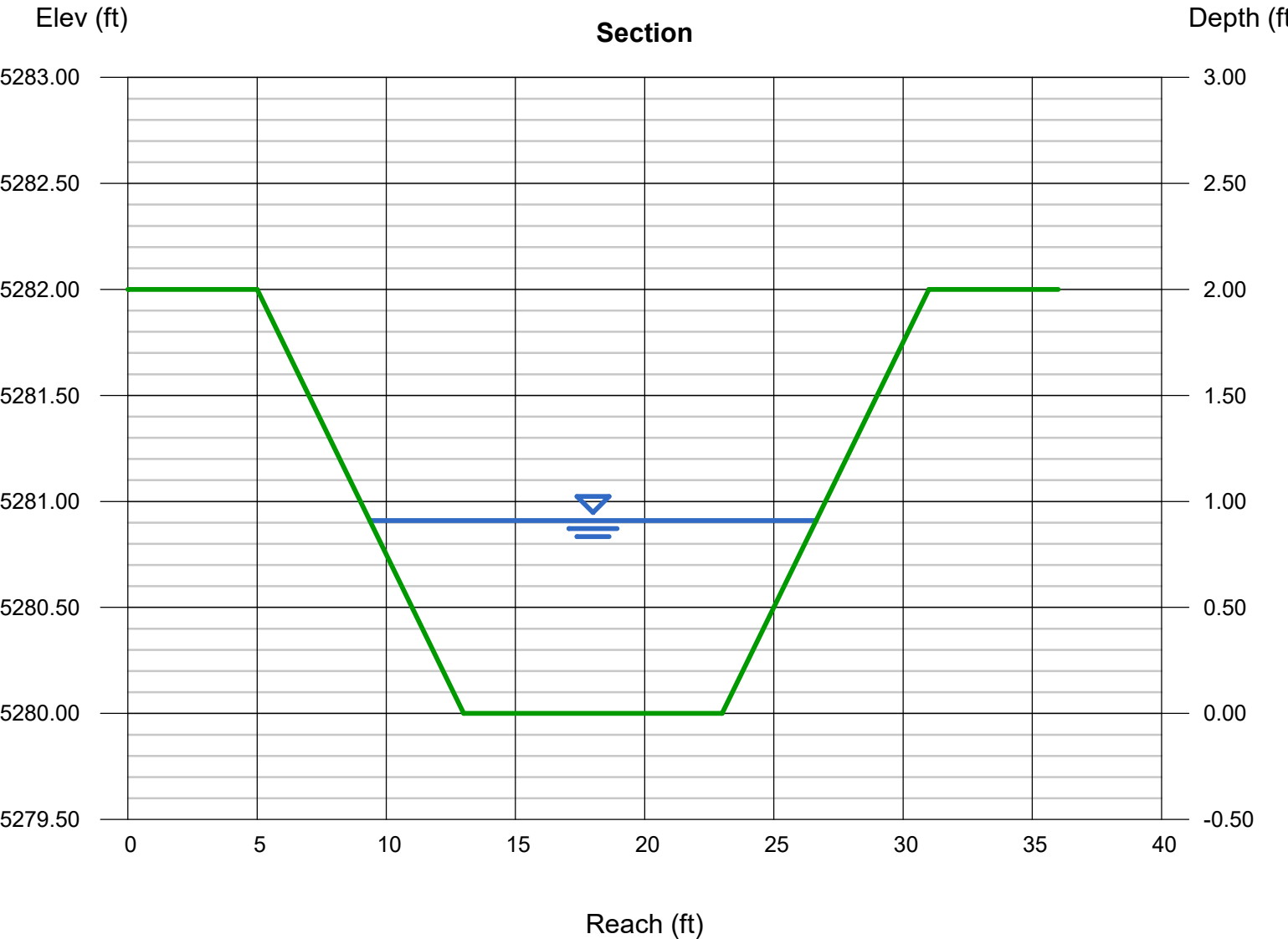
Bottom Width (ft)	= 10.00
Side Slopes (z:1)	= 4.00, 4.00
Total Depth (ft)	= 2.00
Invert Elev (ft)	= 5280.00
Slope (%)	= 3.50
N-Value	= 0.035

Calculations

Compute by:	Known Q
Known Q (cfs)	= 78.00

Highlighted

Depth (ft)	= 0.91
Q (cfs)	= 78.00
Area (sqft)	= 12.41
Velocity (ft/s)	= 6.28
Wetted Perim (ft)	= 17.50
Crit Depth, Yc (ft)	= 1.07
Top Width (ft)	= 17.28
EGL (ft)	= 1.52



Channel Report

Culvert 10 Swale

Trapezoidal

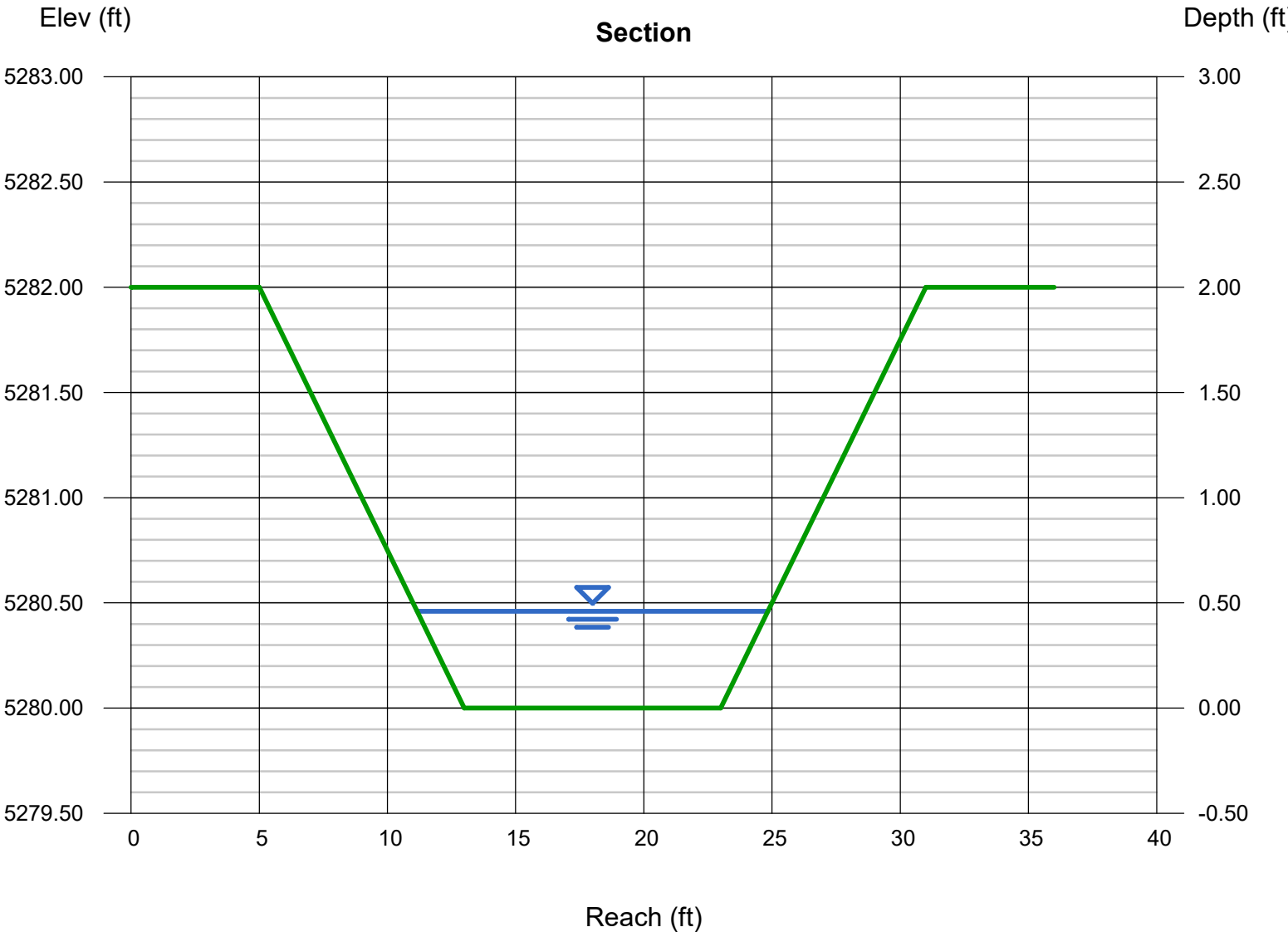
Bottom Width (ft) = 10.00
Side Slopes (z:1) = 4.00, 4.00
Total Depth (ft) = 2.00
Invert Elev (ft) = 5280.00
Slope (%) = 2.60
N-Value = 0.035

Calculations

Compute by: Known Q
Known Q (cfs) = 19.80

Highlighted

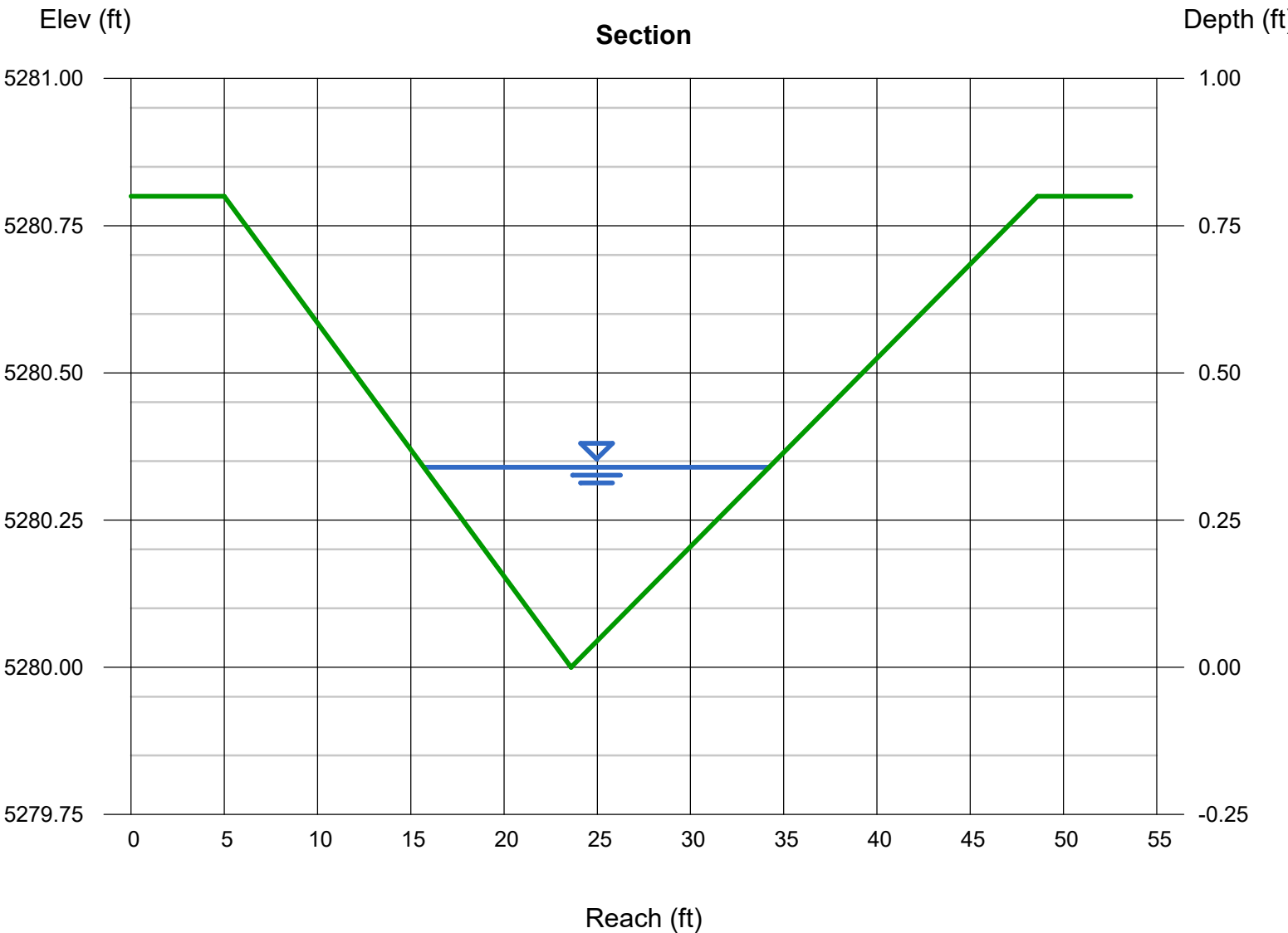
Depth (ft) = 0.46
Q (cfs) = 19.80
Area (sqft) = 5.45
Velocity (ft/s) = 3.64
Wetted Perim (ft) = 13.79
Crit Depth, Yc (ft) = 0.47
Top Width (ft) = 13.68
EGL (ft) = 0.67



Channel Report

TEMPORARY DRAINAGE CONVEYANCE (BASIN 7.1)

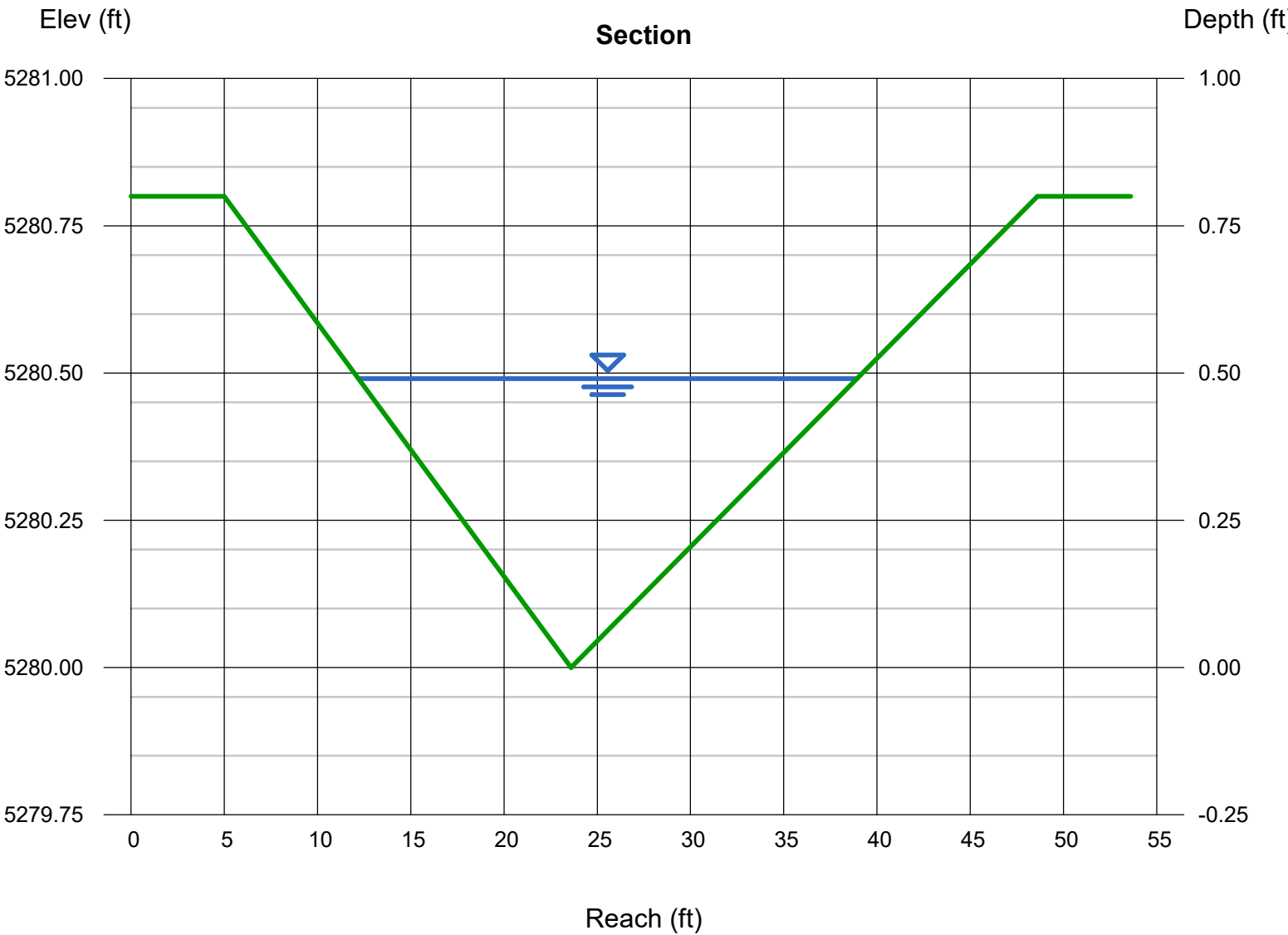
Triangular		Highlighted	
Side Slopes (z:1)	= 23.25, 31.25	Depth (ft)	= 0.34
Total Depth (ft)	= 0.80	Q (cfs)	= 4.100
		Area (sqft)	= 3.15
Invert Elev (ft)	= 5280.00	Velocity (ft/s)	= 1.30
Slope (%)	= 1.10	Wetted Perim (ft)	= 18.54
N-Value	= 0.035	Crit Depth, Yc (ft)	= 0.27
		Top Width (ft)	= 18.53
		EGL (ft)	= 0.37
Calculations			
Compute by:	Known Q		
Known Q (cfs)	= 4.10		



Channel Report

TEMPORARY DRAINAGE CONVEYANCE (BASIN A1.1)

Triangular		Highlighted	
Side Slopes (z:1)	= 23.25, 31.25	Depth (ft)	= 0.49
Total Depth (ft)	= 0.80	Q (cfs)	= 12.80
		Area (sqft)	= 6.54
Invert Elev (ft)	= 5280.00	Velocity (ft/s)	= 1.96
Slope (%)	= 1.50	Wetted Perim (ft)	= 26.72
N-Value	= 0.035	Crit Depth, Yc (ft)	= 0.43
		Top Width (ft)	= 26.70
		EGL (ft)	= 0.55
Calculations			
Compute by:	Known Q		
Known Q (cfs)	= 12.80		



Channel Report

TEMPORARY DRAINAGE CONVEYANCE (BASIN A2.1)

Triangular

Side Slopes (z:1) = 23.25, 31.25
Total Depth (ft) = 0.80

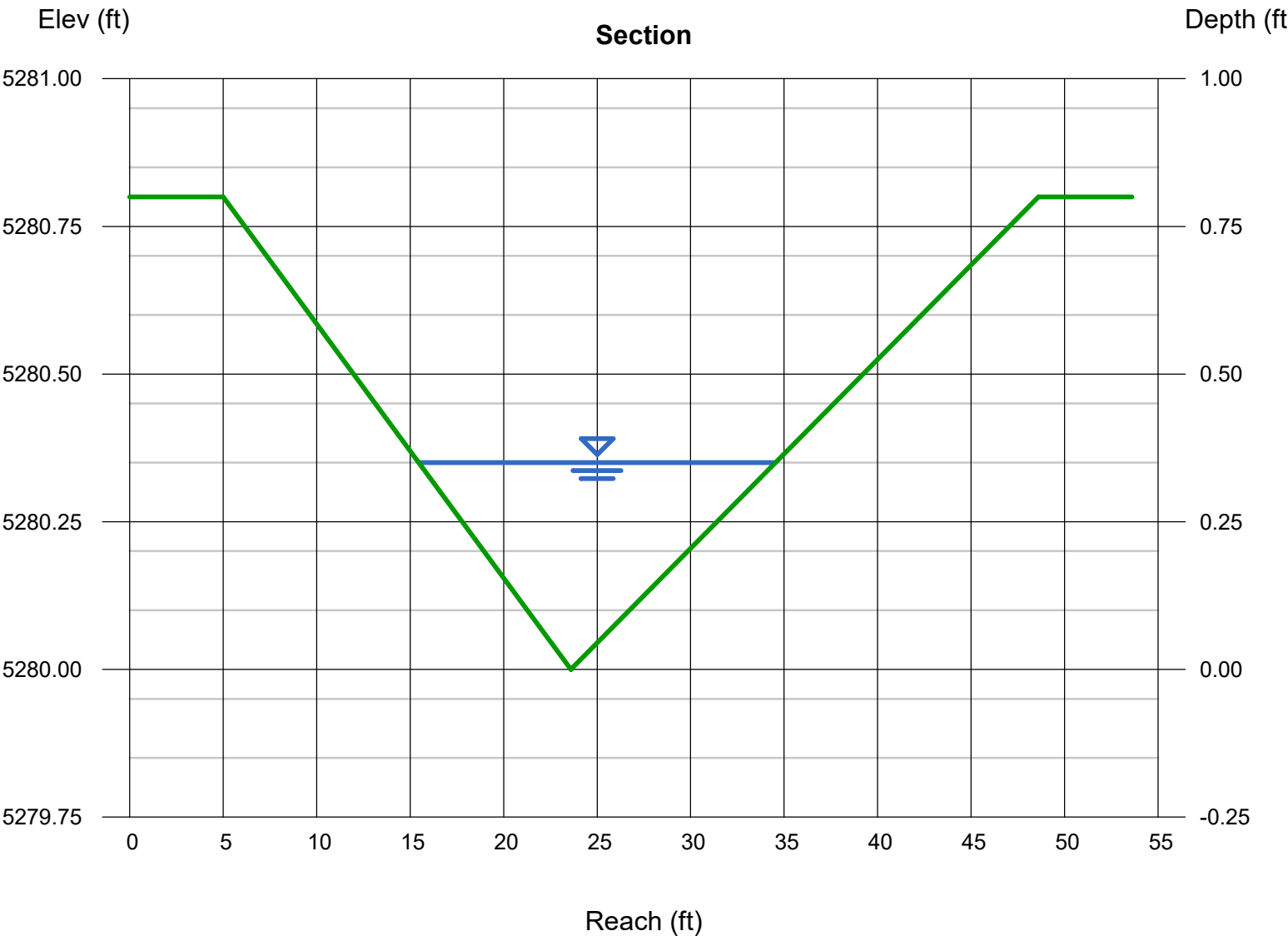
Invert Elev (ft) = 5280.00
Slope (%) = 2.00
N-Value = 0.035

Calculations

Compute by: Known Q
Known Q (cfs) = 6.20

Highlighted

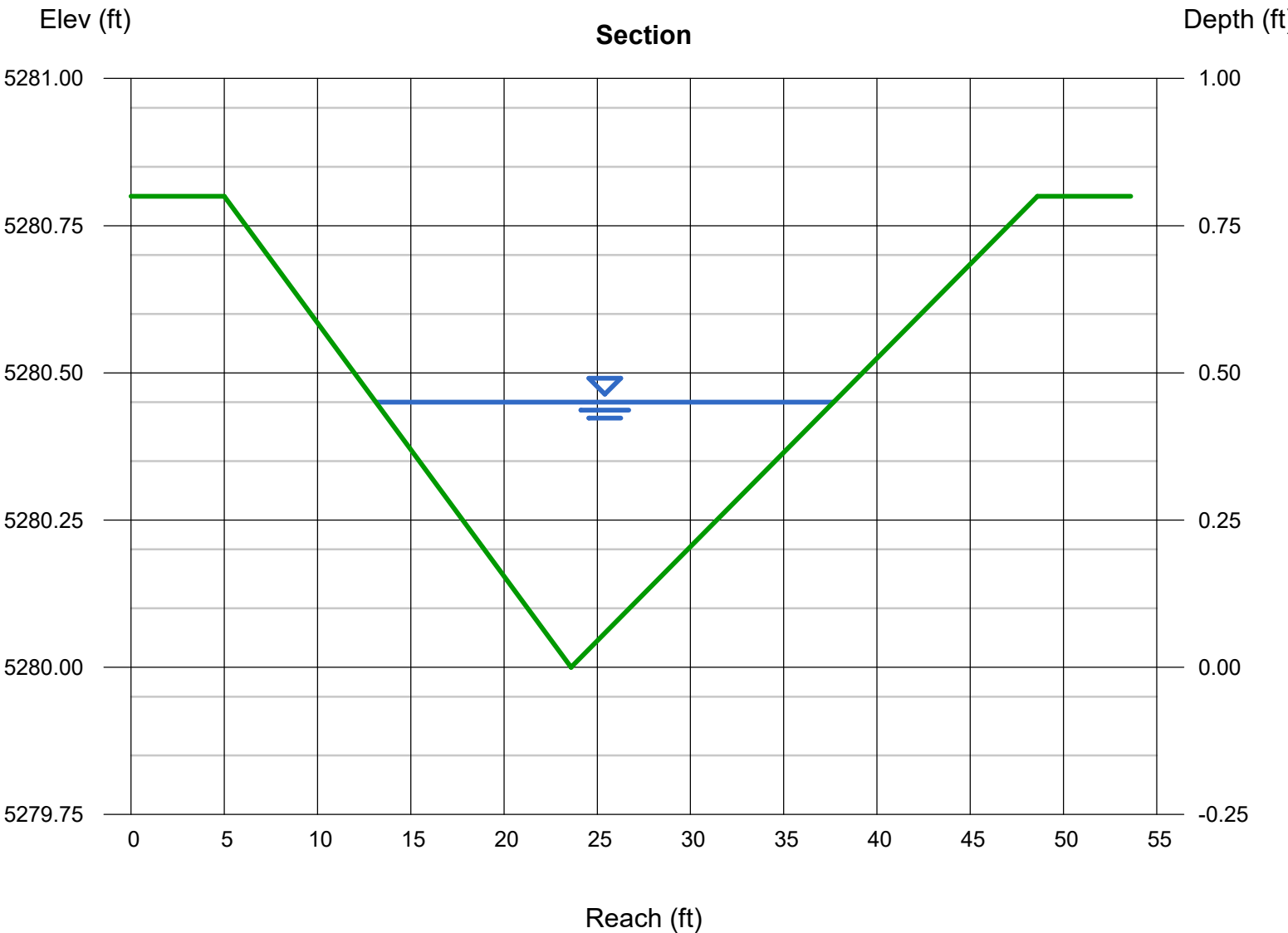
Depth (ft) = 0.35
Q (cfs) = 6.200
Area (sqft) = 3.34
Velocity (ft/s) = 1.86
Wetted Perim (ft) = 19.09
Crit Depth, Yc (ft) = 0.32
Top Width (ft) = 19.08
EGL (ft) = 0.40



Channel Report

TEMPORARY DRAINAGE CONVEYANCE (BASIN A2.3)

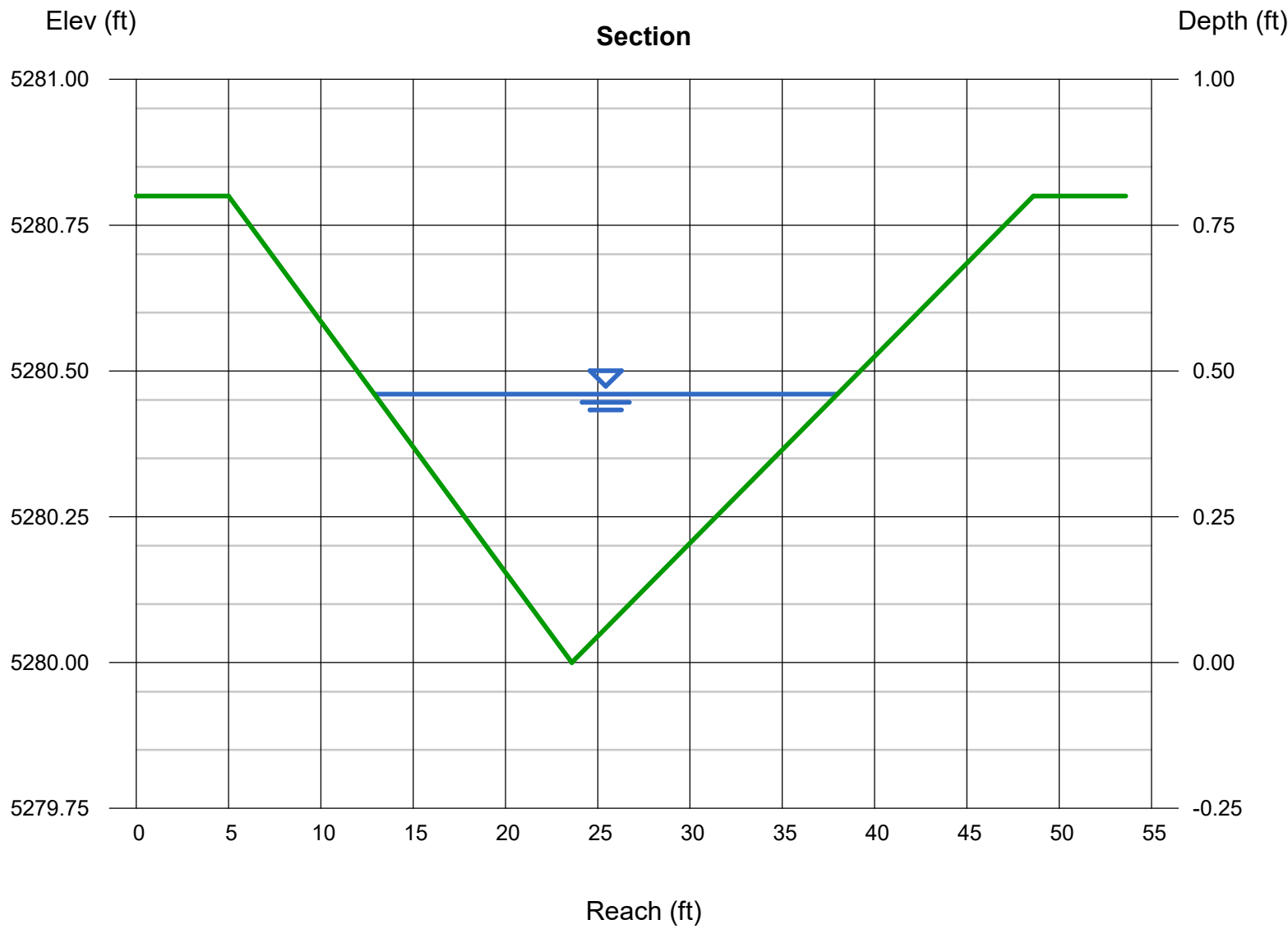
Triangular		Highlighted	
Side Slopes (z:1)	= 23.25, 31.25	Depth (ft)	= 0.45
Total Depth (ft)	= 0.80	Q (cfs)	= 10.10
		Area (sqft)	= 5.52
Invert Elev (ft)	= 5280.00	Velocity (ft/s)	= 1.83
Slope (%)	= 1.50	Wetted Perim (ft)	= 24.54
N-Value	= 0.035	Crit Depth, Yc (ft)	= 0.39
		Top Width (ft)	= 24.52
		EGL (ft)	= 0.50
Calculations			
Compute by:	Known Q		
Known Q (cfs)	= 10.10		



Channel Report

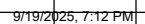
TEMPORARY DRAINAGE CONVEYANCE (BASIN A4.1)

Triangular		Highlighted	
Side Slopes (z:1)	= 23.25, 31.25	Depth (ft)	= 0.46
Total Depth (ft)	= 0.80	Q (cfs)	= 21.40
		Area (sqft)	= 5.77
Invert Elev (ft)	= 5280.00	Velocity (ft/s)	= 3.71
Slope (%)	= 5.50	Wetted Perim (ft)	= 25.09
N-Value	= 0.035	Crit Depth, Yc (ft)	= 0.53
		Top Width (ft)	= 25.07
		EGL (ft)	= 0.67
Calculations			
Compute by:	Known Q		
Known Q (cfs)	= 21.40		



MHFD-Detention, Version 4.05 (January 2022)

Basin ID: Pond A1 - Final Condition

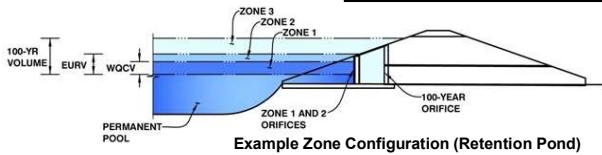


DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.05 (January 2022)

Project: **Dawson Trails Blvd (North)**

Basin ID: **Pond A1 - Final Condition**



Example Zone Configuration (Retention Pond)

	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.54	0.152	Orifice Plate
Zone 2 (EURV)	4.46	0.355	Orifice Plate
Zone 3 (100+1/2WQCV)	5.95	0.386	Weir&Pipe (Restrict)
Total (all zones)		0.894	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth = ft (distance below the filtration media surface)
Underdrain Orifice Diameter = inches

Calculated Parameters for Underdrain
Underdrain Orifice Area = ft²
Underdrain Orifice Centroid = feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation BMP)

Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate = ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing = inches
Orifice Plate: Orifice Area per Row = sq. inches

Calculated Parameters for Plate
WQ Orifice Area per Row = ft²
Elliptical Half-Width = feet
Elliptical Slot Centroid = feet
Elliptical Slot Area = ft²

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	3.00						
Orifice Area (sq. inches)	0.99	7.00						

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter = inches

Calculated Parameters for Vertical Orifice
Vertical Orifice Area = ft²
Vertical Orifice Centroid = feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

Overflow Weir Front Edge Height, H_o = ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length = feet
Overflow Weir Grate Slope = H:V
Horiz. Length of Weir Sides = feet
Overflow Grate Type =
Debris Clogging % = %

Calculated Parameters for Overflow Weir
Height of Grate Upper Edge, H_u = feet
Overflow Weir Slope Length = feet
Grate Open Area / 100-yr Orifice Area =
Overflow Grate Open Area w/o Debris = ft²
Overflow Grate Open Area w/ Debris = ft²

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

Depth to Invert of Outlet Pipe = ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter = inches
Restrictor Plate Height Above Pipe Invert = inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
Outlet Orifice Area = ft²
Outlet Orifice Centroid = feet
Half-Central Angle of Restrictor Plate on Pipe = radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage = ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length = feet
Spillway End Slopes = H:V
Freeboard above Max Water Surface = feet

Calculated Parameters for Spillway
Spillway Design Flow Depth = feet
Stage at Top of Freeboard = feet
Basin Area at Top of Freeboard = acres
Basin Volume at Top of Freeboard = acre-ft

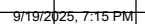
Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Design Storm Return Period =	N/A	N/A	1.06	1.43	1.66	1.72	2.26	2.60	3.07
One-Hour Rainfall Depth (in) =	0.152	0.507	0.395	0.574	0.690	0.741	1.033	1.233	1.494
CUHP Runoff Volume (acre-ft) =	N/A	N/A	0.395	0.574	0.690	0.741	1.033	1.233	1.494
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.0	0.6	1.0	1.7	3.2	4.5	5.8
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A							
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.01	0.10	0.17	0.27	0.51	0.71	0.93
Peak Inflow Q (cfs) =	N/A	N/A	4.0	5.8	6.9	8.0	11.2	13.3	16.1
Peak Outflow Q (cfs) =	0.1	0.4	0.3	0.4	0.4	0.4	2.9	4.0	7.4
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.6	0.4	0.2	0.9	0.9	1.3
Structure Controlling Flow =	Plate	Plate	Plate	Plate	Plate	Plate	Overflow Weir 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	0.4	0.6	0.6
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	40	67	65	69	72	73	73	71	69
Time to Drain 99% of Inflow Volume (hours) =	41	70	68	73	76	78	80	79	78
Maximum Ponding Depth (ft) =	2.53	4.45	3.73	4.49	4.94	5.14	5.75	5.95	6.17
Area at Maximum Ponding Depth (acres) =	0.14	0.22	0.19	0.23	0.25	0.25	0.28	0.29	0.30
Maximum Volume Stored (acre-ft) =	0.152	0.507	0.354	0.516	0.622	0.670	0.837	0.894	0.957

MHFD-Detention, Version 4.05 (January 2022)

Basin ID: Pond A1 - Interim Condition

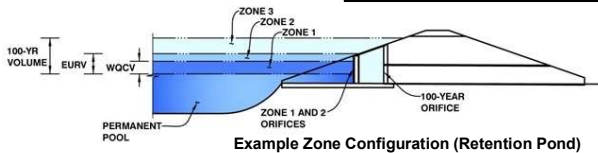


DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.05 (January 2022)

Project: **Dawson Trails Blvd (North)**

Basin ID: **Pond A1 - Interim Condition**



Example Zone Configuration (Retention Pond)

	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.12	0.097	Orifice Plate
Zone 2 (EURV)	3.27	0.174	Orifice Plate
Zone 3 (100+1/2WQCV)	4.74	0.302	Weir&Pipe (Restrict)
Total (all zones)		0.573	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth = ft (distance below the filtration media surface)
Underdrain Orifice Diameter = inches

Calculated Parameters for Underdrain
Underdrain Orifice Area = ft²
Underdrain Orifice Centroid = feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation BMP)

Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate = ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing = inches
Orifice Plate: Orifice Area per Row = sq. inches

Calculated Parameters for Plate
WQ Orifice Area per Row = ft²
Elliptical Half-Width = feet
Elliptical Slot Centroid = feet
Elliptical Slot Area = ft²

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	2.00						
Orifice Area (sq. inches)	0.60	4.00						

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter = inches

Calculated Parameters for Vertical Orifice
Vertical Orifice Area = ft²
Vertical Orifice Centroid = feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

Overflow Weir Front Edge Height, Ho = ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length = feet
Overflow Weir Grate Slope = H:V
Horiz. Length of Weir Sides = feet
Overflow Grate Type =
Debris Clogging % = %

Calculated Parameters for Overflow Weir
Height of Grate Upper Edge, H_u = feet
Overflow Weir Slope Length = feet
Grate Open Area / 100-yr Orifice Area =
Overflow Grate Open Area w/o Debris = ft²
Overflow Grate Open Area w/ Debris = ft²

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

Depth to Invert of Outlet Pipe = ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter = inches
Restrictor Plate Height Above Pipe Invert = inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
Outlet Orifice Area = ft²
Outlet Orifice Centroid = feet
Half-Central Angle of Restrictor Plate on Pipe = radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage = ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length = feet
Spillway End Slopes = H:V
Freeboard above Max Water Surface = feet

Calculated Parameters for Spillway
Spillway Design Flow Depth = feet
Stage at Top of Freeboard = feet
Basin Area at Top of Freeboard = acres
Basin Volume at Top of Freeboard = acre-ft

Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Design Storm Return Period =	N/A	N/A	1.06	1.43	1.66	1.72	2.26	2.60	3.07
One-Hour Rainfall Depth (in) =	0.097	0.270	0.214	0.353	0.450	0.521	0.795	1.003	1.254
CUHP Runoff Volume (acre-ft) =	N/A	N/A	0.214	0.353	0.450	0.521	0.795	1.003	1.254
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.0	0.6	1.0	1.7	3.2	4.5	5.8
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A							
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.01	0.10	0.17	0.27	0.51	0.71	0.93
Peak Inflow Q (cfs) =	N/A	N/A	1.7	2.8	3.6	4.5	6.9	8.7	10.8
Peak Outflow Q (cfs) =	0.1	0.2	0.1	0.2	0.2	0.2	0.3	2.1	4.0
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.3	0.2	0.1	0.1	0.5	0.7
Structure Controlling Flow =	Plate	Plate	Plate	Plate	Plate	Plate	Plate	Overflow Weir 1	Outlet Plate 1
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	0.3	0.5
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	42	54	53	59	62	65	72	71	67
Time to Drain 99% of Inflow Volume (hours) =	44	58	56	63	68	71	81	82	80
Maximum Ponding Depth (ft) =	2.12	3.27	2.75	3.49	3.95	4.28	5.36	5.70	5.87
Area at Maximum Ponding Depth (acres) =	0.13	0.18	0.15	0.18	0.20	0.22	0.27	0.28	0.29
Maximum Volume Stored (acre-ft) =	0.097	0.272	0.184	0.311	0.401	0.468	0.730	0.820	0.871

POND A1 - FOREBAY & TRICKLE CHANNEL CALCULATIONS

Project Name: Dawson Trails Blvd (North)

Project No.: 20-224-003

Per MHFD USDCM Vol. 3, Ch. 3, S. 3.0 & Table EDB-4; Vol. 2, Ch. 12, S. 5.14.2; Vol. 1, & Ch. 8, S. 7.1. Based on an octagonal forebay typical of local installations.

Water Quality Capture Volume (WQCV)

$$WQCV = (\alpha(0.91I^3 - 1.19I^2 + 0.78I)/12)A$$

Water Quality Capture Volume (ac-ft); Equations 3-1 & 3-2 combined

$$\alpha = 1.0$$

BMP type coefficient; typ. 1.0 for 40 hr drain time; Table 3-2

$$I = 0.73$$

Imperviousness (percent expressed as a decimal)

$$A = 6.30 \text{ ac}$$

watershed tributary area upstream

WQCV = 0.152 ac-ft

Forebay Sizing

$$V = p(WQCV)43560/100$$

minimum forebay volume (ft³)

$$p = 3 \%$$

percentage of WQCV; Table EDB-4

V = 198.79 ft ³

minimum forebay volume

$$A_R = V/D_F$$

minimum required forebay area (ft²)

D _M = 1.50 ft

maximum forebay depth (sidewall depth); Table EDB-4

D _F = 1.25 ft

forebay front edge depth (D_M - 3"); Figure 14-8

$$A_R = 159.0 \text{ ft}^2$$

A _p = 111.0 ft ²
--

forebay area provided (must be ≥ A_R);

determined by increasing the central longitudinal dimension shown on the typical forebay detail;

see detail in construction drawings for complete forebay configuration and dimensions

Forebay Notch Sizing

$$Q_{100} = 26.33 \text{ cfs}$$

undetained peak discharge (from StormCAD calcs)

$$Q_{Rm} = 0.02 * Q_{100}$$

minimum forebay release (cfs); 2% of Q₁₀₀; Table EDB-4

$$Q_{Rm} = 0.53 \text{ cfs}$$

H = 1.25 ft

notch depth = D_F

$$L_{Nm} = Q_{Rm}/C_{BCW}H^{1.5}$$

minimum notch width (ft); (broad crested weir calculation); Equation 12-8; CBCW = 3.3

$$L_{Nm} = 0.11 \text{ ft}$$

L _N = 0.36 ft

notch width used for design;

(minimum notch width [L_{Nm}] increased by 20% of notch depth [D_F] for clogged condition

per MHFD BMP Worksheet and rounded up to nearest whole inch)

= 5.0 in

Trickle Channel Sizing

$$Q_R = C_{BCW}L_NH^{1.5}$$

maximum forebay notch capacity (cfs); (broad crested weir calculation); Equation 12-8; CBCW = 3.3

$$Q_R = 1.68 \text{ cfs}$$

$$Q_T = (1.49/n)AR^{2/3}S^{1/2}$$

flow in trickle channel (cfs); (Manning's Equation for uniform open channel flow); Equation 8-7

$$n = 0.013$$

Manning's roughness coefficient for concrete

$$S = 0.005 \text{ ft/ft}$$

friction slope (approx. slope of trickle channel for normal flow); 0.4% - 1% recommended by MHFD

$$A = WH$$

cross-sectional area of flow in channel (ft²); assume channel flowing full

W = 2.0 ft

width of trickle channel bottom

H = 0.5 ft

height of trickle channel sides

$$A = 1.00 \text{ ft}^2$$

$$R = A/P$$

hydraulic radius (ft)

$$R = 0.33 \text{ ft}$$

$$P = W+2H$$

wetted perimeter (ft)

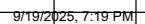
$$P = 3.00 \text{ ft}$$

Q _T = 3.90 cfs

maximum capacity of trickle channel (must be ≥ Q_R)

MHFD-Detention, Version 4.05 (January 2022)

Basin ID: Pond A2 - Final Condition

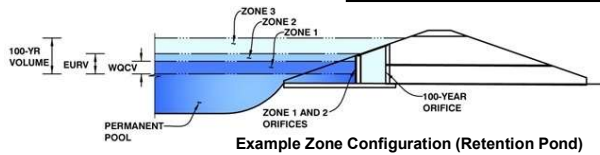


DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.05 (January 2022)

Project: **Dawson Trails Blvd (North)**

Basin ID: **Pond A2 - Final Condition**



Example Zone Configuration (Retention Pond)

	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.93	0.195	Orifice Plate
Zone 2 (EURV)	5.15	0.456	Orifice Plate
Zone 3 (100+1/2WQCV)	6.90	0.492	Weir&Pipe (Restrict)
Total (all zones)		1.142	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth = ft (distance below the filtration media surface)
Underdrain Orifice Diameter = inches

Calculated Parameters for Underdrain
Underdrain Orifice Area = ft²
Underdrain Orifice Centroid = feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation BMP)

Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate = ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing = inches
Orifice Plate: Orifice Area per Row = sq. inches

Calculated Parameters for Plate
WQ Orifice Area per Row = ft²
Elliptical Half-Width = feet
Elliptical Slot Centroid = feet
Elliptical Slot Area = ft²

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	2.50						
Orifice Area (sq. inches)	0.99	3.00						

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter = inches

Calculated Parameters for Vertical Orifice
Vertical Orifice Area = ft²
Vertical Orifice Centroid = feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

Overflow Weir Front Edge Height, H_o = ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length = feet
Overflow Weir Grate Slope = H:V
Horiz. Length of Weir Sides = feet
Overflow Grate Type =
Debris Clogging % = %

Calculated Parameters for Overflow Weir
Height of Grate Upper Edge, H_u = feet
Overflow Weir Slope Length = feet
Grate Open Area / 100-yr Orifice Area =
Overflow Grate Open Area w/o Debris = ft²
Overflow Grate Open Area w/ Debris = ft²

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

Depth to Invert of Outlet Pipe = ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter = inches
Restrictor Plate Height Above Pipe Invert = inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
Outlet Orifice Area = ft²
Outlet Orifice Centroid = feet
Half-Central Angle of Restrictor Plate on Pipe = radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage = ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length = feet
Spillway End Slopes = H:V
Freeboard above Max Water Surface = feet

Calculated Parameters for Spillway
Spillway Design Flow Depth = feet
Stage at Top of Freeboard = feet
Basin Area at Top of Freeboard = acres
Basin Volume at Top of Freeboard = acre-ft

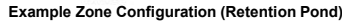
Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Design Storm Return Period =	N/A	N/A	1.06	1.43	1.66	1.72	2.26	2.60	3.07
One-Hour Rainfall Depth (in) =	0.195	0.650	0.493	0.724	0.872	0.935	1.303	1.552	1.881
CUHP Runoff Volume (acre-ft) =	N/A	N/A	0.493	0.724	0.872	0.935	1.303	1.552	1.881
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.1	1.8	2.8	4.3	7.6	10.1	13.0
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A							
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.01	0.22	0.35	0.53	0.93	1.25	1.60
Peak Inflow Q (cfs) =	N/A	N/A	8.1	12.0	14.0	15.4	21.5	26.0	31.3
Peak Outflow Q (cfs) =	0.1	0.3	0.2	0.6	1.9	2.6	6.6	6.9	13.2
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.3	0.7	0.6	0.9	0.7	1.0
Structure Controlling Flow =	Plate	Overflow Weir 1	Plate	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	0.00	N/A	0.0	0.1	0.1	0.3	0.3	0.3
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	42	67	61	69	68	68	65	63	60
Time to Drain 99% of Inflow Volume (hours) =	44	71	64	74	74	74	73	72	71
Maximum Ponding Depth (ft) =	2.93	5.15	4.33	5.24	5.47	5.55	6.00	6.58	7.01
Area at Maximum Ponding Depth (acres) =	0.16	0.25	0.21	0.25	0.26	0.26	0.28	0.30	0.32
Maximum Volume Stored (acre-ft) =	0.196	0.651	0.460	0.673	0.729	0.753	0.874	1.043	1.174

MHFD-Detention, Version 4.05 (January 2022)

Basin ID: Pond A2 - Interim Condition



6303.77

Water Quality Capture Volume (WQCV) =	0.136	acre-feet
Excess Urban Runoff Volume (EURV) =	0.414	acre-feet
2-yr Runoff Volume (P1 = 1.06 in.) =	0.318	acre-feet
5-yr Runoff Volume (P1 = 1.43 in.) =	0.519	acre-feet
10-yr Runoff Volume (P1 = 1.66 in.) =	0.652	acre-feet
25-yr Runoff Volume (P1 = 1.72 in.) =	0.737	acre-feet
50-yr Runoff Volume (P1 = 2.26 in.) =	1.092	acre-feet
100-yr Runoff Volume (P1 = 2.6 in.) =	1.352	acre-feet
500-yr Runoff Volume (P1 = 3.07 in.) =	1.676	acre-feet
Approximate 2-yr Detention Volume =	0.277	acre-feet
Approximate 5-yr Detention Volume =	0.408	acre-feet
Approximate 10-yr Detention Volume =	0.541	acre-feet
Approximate 25-yr Detention Volume =	0.539	acre-feet
Approximate 50-yr Detention Volume =	0.658	acre-feet
Approximate 100-yr Detention Volume =	0.759	acre-feet

Initial Surge Area (A_{ISV}) =	user	ft ²
Surcharge Volume Length (L_{ISV}) =	user	ft
Surcharge Volume Width (W_{ISV}) =	user	ft
Depth of Basin Floor (H_{FLOOR}) =	user	ft
Length of Basin Floor (L_{FLOOR}) =	user	ft
Width of Basin Floor (W_{FLOOR}) =	user	ft
Area of Basin Floor (A_{FLOOR}) =	user	ft ²
Volume of Basin Floor (V_{FLOOR}) =	user	ft ³
Depth of Main Basin (H_{MAIN}) =	user	ft
Length of Main Basin (L_{MAIN}) =	user	ft
Width of Main Basin (W_{MAIN}) =	user	ft
Area of Main Basin (A_{MAIN}) =	user	ft ²
Volume of Main Basin (V_{MAIN}) =	user	ft ³
Calculated Total Basin Volume (V_{TOTAL}) =	user	acre-feet

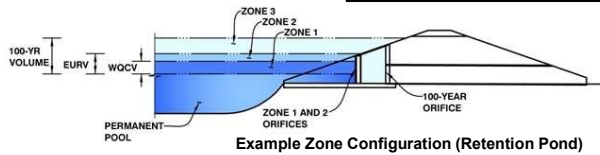
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DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.05 (January 2022)

Project: **Dawson Trails Blvd (North)**

Basin ID: **Pond A2 - Interim Condition**



	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.54	0.136	Orifice Plate
Zone 2 (EURV)	4.11	0.278	Orifice Plate
Zone 3 (100+1/2WQCV)	5.83	0.413	Weir&Pipe (Restrict)
Total (all zones)		0.827	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth = ft (distance below the filtration media surface)
Underdrain Orifice Diameter = inches

Underdrain Orifice Area = ft²
Underdrain Orifice Centroid = feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation BMP)

Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate = ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing = inches
Orifice Plate: Orifice Area per Row = sq. inches

WQ Orifice Area per Row = ft²
Elliptical Half-Width = feet
Elliptical Slot Centroid = feet
Elliptical Slot Area = ft²

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	2.50						
Orifice Area (sq. inches)	0.79	3.50						

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter = inches

Calculated Parameters for Vertical Orifice
Not Selected Not Selected
Vertical Orifice Area = ft²
Vertical Orifice Centroid = feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

Overflow Weir Front Edge Height, Ho = ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length = feet
Overflow Weir Grate Slope = H:V
Horiz. Length of Weir Sides = feet
Overflow Grate Type =
Debris Clogging % = %

Calculated Parameters for Overflow Weir
Zone 3 Weir Not Selected
Height of Grate Upper Edge, H_u = feet
Overflow Weir Slope Length = feet
Grate Open Area / 100-yr Orifice Area =
Overflow Grate Open Area w/o Debris = ft²
Overflow Grate Open Area w/ Debris = ft²

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

Depth to Invert of Outlet Pipe = ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter = inches
Restrictor Plate Height Above Pipe Invert = inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
Zone 3 Restrictor Not Selected
Outlet Orifice Area = ft²
Outlet Orifice Centroid = feet
Half-Central Angle of Restrictor Plate on Pipe = radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage = ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length = feet
Spillway End Slopes = H:V
Freeboard above Max Water Surface = feet

Calculated Parameters for Spillway
Spillway Design Flow Depth = feet
Stage at Top of Freeboard = feet
Basin Area at Top of Freeboard = acres
Basin Volume at Top of Freeboard = acre-ft

Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Design Storm Return Period =	N/A	N/A	1.06	1.43	1.66	1.72	2.26	2.60	3.07
One-Hour Rainfall Depth (in) =	0.136	0.414	0.318	0.519	0.652	0.737	1.092	1.352	1.676
CUHP Runoff Volume (acre-ft) =	N/A	N/A	0.318	0.519	0.652	0.737	1.092	1.352	1.676
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.1	1.8	2.8	4.3	7.6	10.1	13.0
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A							
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.01	0.22	0.35	0.53	0.93	1.25	1.60
Peak Inflow Q (cfs) =	N/A	N/A	4.4	7.6	9.3	10.7	16.0	19.8	24.3
Peak Outflow Q (cfs) =	0.1	0.2	0.2	0.2	0.2	0.7	4.8	6.7	7.0
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.1	0.1	0.2	0.6	0.7	0.5
Structure Controlling Flow =	Plate	Plate	Plate	Plate	Plate	Overflow Weir 1	Overflow Weir 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	N/A	N/A	0.0	0.2	0.3	0.3
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	44	64	60	69	74	76	72	69	66
Time to Drain 99% of Inflow Volume (hours) =	46	68	63	74	80	82	81	80	79
Maximum Ponding Depth (ft) =	2.54	4.11	3.48	4.43	4.99	5.26	5.76	6.13	6.80
Area at Maximum Ponding Depth (acres) =	0.14	0.21	0.19	0.22	0.24	0.25	0.27	0.28	0.31
Maximum Volume Stored (acre-ft) =	0.136	0.416	0.292	0.482	0.610	0.678	0.808	0.911	1.111

POND A2 - FOREBAY & TRICKLE CHANNEL CALCULATIONS

Project Name: Dawson Trails Blvd (North)

Project No.: 20-224-003

Per MHFD USDCM Vol. 3, Ch. 3, S. 3.0 & Table EDB-4; Vol. 2, Ch. 12, S. 5.14.2; Vol. 1, & Ch. 8, S. 7.1. Based on an octagonal forebay typical of local installations.

Water Quality Capture Volume (WQCV)

$$WQCV = (\alpha(0.91I^3 - 1.19I^2 + 0.78I)/12)A$$

Water Quality Capture Volume (ac-ft); Equations 3-1 & 3-2 combined

$$\alpha = 1.0$$

BMP type coefficient; typ. 1.0 for 40 hr drain time; Table 3-2

$$I = 0.7680$$

Imperviousness (percent expressed as a decimal)

$$A = 5.74 \text{ ac}$$

watershed tributary area upstream

WQCV = 0.148 ac-ft

Forebay Sizing

$$V = p(WQCV)43560/100$$

minimum forebay volume (ft³)

$$p = 3 \%$$

percentage of WQCV; Table EDB-4

V = 193.38 ft ³

minimum forebay volume

$$A_R = V/D_F$$

minimum required forebay area (ft²)

D _M = 1.50 ft

maximum forebay depth (sidewall depth); Table EDB-4

D _F = 1.25 ft

forebay front edge depth (D_M - 3"); Figure 14-8

$$A_R = 154.7 \text{ ft}^2$$

A _p = 198.0 ft ²
--

forebay area provided (must be ≥ A_R);

determined by increasing the central longitudinal dimension shown on the typical forebay detail;

see detail in construction drawings for complete forebay configuration and dimensions

Forebay Notch Sizing

$$Q_{100} = 30.42 \text{ cfs}$$

undetained peak discharge (from StormCAD calcs)

$$Q_{Rm} = 0.02 * Q_{100}$$

minimum forebay release (cfs); 2% of Q₁₀₀; Table EDB-4

$$Q_{Rm} = 0.61 \text{ cfs}$$

H = 1.25 ft

notch depth = D_F

$$L_{Nm} = Q_{Rm}/C_{BCW}H^{1.5}$$

minimum notch width (ft); (broad crested weir calculation); Equation 12-8; CBCW = 3.3

$$L_{Nm} = 0.13 \text{ ft}$$

L _N = 0.38 ft

notch width used for design;

(minimum notch width [L_{Nm}] increased by 20% of notch depth [D_F] for clogged condition

per MHFD BMP Worksheet and rounded up to nearest whole inch)

= 5.0 in

Trickle Channel Sizing

$$Q_R = C_{BCW}L_NH^{1.5}$$

maximum forebay notch capacity (cfs); (broad crested weir calculation); Equation 12-8; CBCW = 3.3

$$Q_R = 1.76 \text{ cfs}$$

$$Q_T = (1.49/n)AR^{2/3}S^{1/2}$$

flow in trickle channel (cfs); (Manning's Equation for uniform open channel flow); Equation 8-7

$$n = 0.013$$

Manning's roughness coefficient for concrete

$$S = 0.005 \text{ ft/ft}$$

friction slope (approx. slope of trickle channel for normal flow); 0.4% - 1% recommended by MHFD

$$A = WH$$

cross-sectional area of flow in channel (ft²); assume channel flowing full

W = 2.0 ft

width of trickle channel bottom

H = 0.5 ft

height of trickle channel sides

$$A = 1.00 \text{ ft}^2$$

$$R = A/P$$

hydraulic radius (ft)

$$R = 0.33 \text{ ft}$$

$$P = W+2H$$

wetted perimeter (ft)

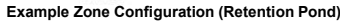
$$P = 3.00 \text{ ft}$$

Q _T = 3.90 cfs

maximum capacity of trickle channel (must be ≥ Q_R)

MHFD-Detention, Version 4.05 (January 2022)

Basin ID: Pond A4 - Final Condition



6263.67

Water Quality Capture Volume (WQCV) =	0.143	acre-feet
Excess Urban Runoff Volume (EURV) =	0.286	acre-feet
2-yr Runoff Volume (P1 = 1.06 in.) =	0.206	acre-feet
5-yr Runoff Volume (P1 = 1.43 in.) =	0.525	acre-feet
10-yr Runoff Volume (P1 = 1.66 in.) =	0.747	acre-feet
25-yr Runoff Volume (P1 = 1.72 in.) =	0.954	acre-feet
50-yr Runoff Volume (P1 = 2.26 in.) =	1.615	acre-feet
100-yr Runoff Volume (P1 = 2.6 in.) =	2.136	acre-feet
500-yr Runoff Volume (P1 = 3.07 in.) =	2.744	acre-feet
Approximate 2-yr Detention Volume =	0.174	acre-feet
Approximate 5-yr Detention Volume =	0.277	acre-feet
Approximate 10-yr Detention Volume =	0.455	acre-feet
Approximate 25-yr Detention Volume =	0.517	acre-feet
Approximate 50-yr Detention Volume =	0.638	acre-feet
Approximate 100-yr Detention Volume =	0.810	acre-feet

Initial Surcharge Area (A_{ISV}) =	user	ft ²
Surcharge Volume Length (L_{ISV}) =	user	ft
Surcharge Volume Width (W_{ISV}) =	user	ft
Depth of Basin Floor (H_{FLOOR}) =	user	ft
Length of Basin Floor (L_{FLOOR}) =	user	ft
Width of Basin Floor (W_{FLOOR}) =	user	ft
Area of Basin Floor (A_{FLOOR}) =	user	ft ²
Volume of Basin Floor (V_{FLOOR}) =	user	ft ³
Depth of Main Basin (H_{MAIN}) =	user	ft
Length of Main Basin (L_{MAIN}) =	user	ft
Width of Main Basin (W_{MAIN}) =	user	ft
Area of Main Basin (A_{MAIN}) =	user	ft ²
Volume of Main Basin (V_{MAIN}) =	user	ft ³
Calculated Total Basin Volume (V_{TOTAL}) =	user	acre-feet

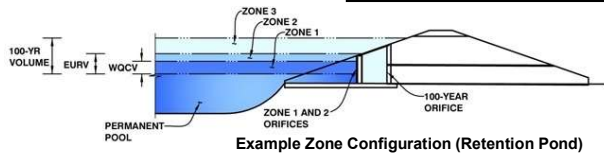
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DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.05 (January 2022)

Project: **Dawson Trails Blvd (North)**

Basin ID: **Pond A4 - Final Condition**



Example Zone Configuration (Retention Pond)

	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.54	0.143	Orifice Plate
Zone 2 (EURV)	3.28	0.144	Orifice Plate
3 (100+1/2WQCV)	5.50	0.595	Weir&Pipe (Restrict)
Total (all zones)		0.881	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth = ft (distance below the filtration media surface)
Underdrain Orifice Diameter = inches

Calculated Parameters for Underdrain
Underdrain Orifice Area = ft²
Underdrain Orifice Centroid = feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation BMP)

Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate = ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing = inches
Orifice Plate: Orifice Area per Row = sq. inches

Calculated Parameters for Plate
WQ Orifice Area per Row = ft²
Elliptical Half-Width = feet
Elliptical Slot Centroid = feet
Elliptical Slot Area = ft²

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	2.50						
Orifice Area (sq. inches)	0.79	1.50						

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter = inches

Calculated Parameters for Vertical Orifice
Vertical Orifice Area = ft²
Vertical Orifice Centroid = feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

Overflow Weir Front Edge Height, H_o = ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length = feet
Overflow Weir Grate Slope = H:V
Horiz. Length of Weir Sides = feet
Overflow Grate Type =
Debris Clogging % = %

Calculated Parameters for Overflow Weir
Height of Grate Upper Edge, H_u = feet
Overflow Weir Slope Length = feet
Grate Open Area / 100-yr Orifice Area =
Overflow Grate Open Area w/o Debris = ft²
Overflow Grate Open Area w/ Debris = ft²

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

Depth to Invert of Outlet Pipe = ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter = inches
Restrictor Plate Height Above Pipe Invert = inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
Outlet Orifice Area = ft²
Outlet Orifice Centroid = feet
Half-Central Angle of Restrictor Plate on Pipe = radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage = ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length = feet
Spillway End Slopes = H:V
Freeboard above Max Water Surface = feet

Calculated Parameters for Spillway
Spillway Design Flow Depth = feet
Stage at Top of Freeboard = feet
Basin Area at Top of Freeboard = acres
Basin Volume at Top of Freeboard = acre-ft

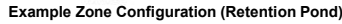
Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Design Storm Return Period =	N/A	N/A	1.06	1.43	1.66	1.72	2.26	2.60	3.07
One-Hour Rainfall Depth (in) =	0.143	0.286	0.206	0.525	0.747	0.954	1.615	2.136	2.744
CUHP Runoff Volume (acre-ft) =	N/A	N/A	0.206	0.525	0.747	0.954	1.615	2.136	2.744
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.2	4.0	6.1	9.1	16.2	21.6	27.7
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A							
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.02	0.26	0.39	0.59	1.04	1.38	1.78
Peak Inflow Q (cfs) =	N/A	N/A	2.4	6.5	8.8	11.8	19.3	24.5	30.9
Peak Outflow Q (cfs) =	0.1	0.1	0.1	2.3	4.8	7.8	14.9	15.7	19.0
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.6	0.8	0.9	0.9	0.7	0.7
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Outlet Plate 1	Outlet Plate 1	Spillway
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.2	0.5	0.8	1.6	1.7	1.8
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	46	67	58	71	69	67	60	55	49
Time to Drain 99% of Inflow Volume (hours) =	48	70	60	76	75	74	72	69	67
Maximum Ponding Depth (ft) =	2.54	3.28	2.81	3.61	3.75	3.88	4.19	4.92	5.84
Area at Maximum Ponding Depth (acres) =	0.17	0.22	0.19	0.23	0.24	0.25	0.26	0.29	0.34
Maximum Volume Stored (acre-ft) =	0.144	0.287	0.192	0.360	0.393	0.427	0.503	0.707	0.996

MHFD-Detention, Version 4.05 (January 2022)

Basin ID: Pond A4 - Interim Condition



6263.67

Water Quality Capture Volume (WQCV) =	0.097	acre-feet
Excess Urban Runoff Volume (EURV) =	0.169	acre-feet
2-yr Runoff Volume (P1 = 1.06 in.) =	0.116	acre-feet
5-yr Runoff Volume (P1 = 1.43 in.) =	0.413	acre-feet
10-yr Runoff Volume (P1 = 1.66 in.) =	0.622	acre-feet
25-yr Runoff Volume (P1 = 1.72 in.) =	0.847	acre-feet
50-yr Runoff Volume (P1 = 2.26 in.) =	1.495	acre-feet
100-yr Runoff Volume (P1 = 2.6 in.) =	2.021	acre-feet
500-yr Runoff Volume (P1 = 3.07 in.) =	2.626	acre-feet
Approximate 2-yr Detention Volume =	0.097	acre-feet
Approximate 5-yr Detention Volume =	0.161	acre-feet
Approximate 10-yr Detention Volume =	0.316	acre-feet
Approximate 25-yr Detention Volume =	0.391	acre-feet
Approximate 50-yr Detention Volume =	0.479	acre-feet
Approximate 100-yr Detention Volume =	0.631	acre-feet

Initial Surcharge Area (A_{ISV})	=	user	ft ²
Surcharge Volume Length (L_{ISV})	=	user	ft
Surcharge Volume Width (W_{ISV})	=	user	ft
Depth of Basin Floor (H_{FLOOR})	=	user	ft
Length of Basin Floor (L_{FLOOR})	=	user	ft
Width of Basin Floor (W_{FLOOR})	=	user	ft
Area of Basin Floor (A_{FLOOR})	=	user	ft ²
Volume of Basin Floor (V_{FLOOR})	=	user	ft ³
Depth of Main Basin (H_{MAIN})	=	user	ft
Length of Main Basin (L_{MAIN})	=	user	ft
Width of Main Basin (W_{MAIN})	=	user	ft
Area of Main Basin (A_{MAIN})	=	user	ft ²
Volume of Main Basin (V_{MAIN})	=	user	ft ³
Calculated Total Basin Volume (V_{TOTAL})	=	user	acre-feet

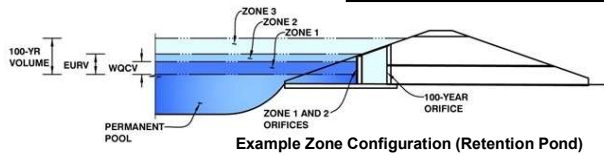
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DETENTION BASIN OUTLET STRUCTURE DESIGN

MHFD-Detention, Version 4.05 (January 2022)

Project: **Dawson Trails Blvd (North)**

Basin ID: **Pond A4 - Interim Condition**



	Estimated Stage (ft)	Estimated Volume (ac-ft)	Outlet Type
Zone 1 (WQCV)	2.25	0.097	Orifice Plate
Zone 2 (EURV)	2.69	0.072	Orifice Plate
3 (100+1/2WQCV)	4.83	0.511	Weir&Pipe (Restrict)
Total (all zones)		0.680	

User Input: Orifice at Underdrain Outlet (typically used to drain WQCV in a Filtration BMP)

Underdrain Orifice Invert Depth = ft (distance below the filtration media surface)
Underdrain Orifice Diameter = inches

Calculated Parameters for Underdrain
Underdrain Orifice Area = ft²
Underdrain Orifice Centroid = feet

User Input: Orifice Plate with one or more orifices or Elliptical Slot Weir (typically used to drain WQCV and/or EURV in a sedimentation BMP)

Centroid of Lowest Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Orifice Plate = ft (relative to basin bottom at Stage = 0 ft)
Orifice Plate: Orifice Vertical Spacing = inches
Orifice Plate: Orifice Area per Row = sq. inches

Calculated Parameters for Plate
WQ Orifice Area per Row = ft²
Elliptical Half-Width = feet
Elliptical Slot Centroid = feet
Elliptical Slot Area = ft²

User Input: Stage and Total Area of Each Orifice Row (numbered from lowest to highest)

	Row 1 (required)	Row 2 (optional)	Row 3 (optional)	Row 4 (optional)	Row 5 (optional)	Row 6 (optional)	Row 7 (optional)	Row 8 (optional)
Stage of Orifice Centroid (ft)	0.00	2.50						
Orifice Area (sq. inches)	0.60	6.00						

	Row 9 (optional)	Row 10 (optional)	Row 11 (optional)	Row 12 (optional)	Row 13 (optional)	Row 14 (optional)	Row 15 (optional)	Row 16 (optional)
Stage of Orifice Centroid (ft)								
Orifice Area (sq. inches)								

User Input: Vertical Orifice (Circular or Rectangular)

Invert of Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Depth at top of Zone using Vertical Orifice = ft (relative to basin bottom at Stage = 0 ft)
Vertical Orifice Diameter = inches

Calculated Parameters for Vertical Orifice
Vertical Orifice Area = ft²
Vertical Orifice Centroid = feet

User Input: Overflow Weir (Dropbox with Flat or Sloped Grate and Outlet Pipe OR Rectangular/Trapezoidal Weir and No Outlet Pipe)

Overflow Weir Front Edge Height, H_o = ft (relative to basin bottom at Stage = 0 ft)
Overflow Weir Front Edge Length = feet
Overflow Weir Grate Slope = H:V
Horiz. Length of Weir Sides = feet
Overflow Grate Type =
Debris Clogging % = %

Calculated Parameters for Overflow Weir
Height of Grate Upper Edge, H_u = feet
Overflow Weir Slope Length = feet
Grate Open Area / 100-yr Orifice Area =
Overflow Grate Open Area w/o Debris = ft²
Overflow Grate Open Area w/ Debris = ft²

User Input: Outlet Pipe w/ Flow Restriction Plate (Circular Orifice, Restrictor Plate, or Rectangular Orifice)

Depth to Invert of Outlet Pipe = ft (distance below basin bottom at Stage = 0 ft)
Outlet Pipe Diameter = inches
Restrictor Plate Height Above Pipe Invert = inches

Calculated Parameters for Outlet Pipe w/ Flow Restriction Plate
Outlet Orifice Area = ft²
Outlet Orifice Centroid = feet
Half-Central Angle of Restrictor Plate on Pipe = radians

User Input: Emergency Spillway (Rectangular or Trapezoidal)

Spillway Invert Stage = ft (relative to basin bottom at Stage = 0 ft)
Spillway Crest Length = feet
Spillway End Slopes = H:V
Freeboard above Max Water Surface = feet

Calculated Parameters for Spillway
Spillway Design Flow Depth = feet
Stage at Top of Freeboard = feet
Basin Area at Top of Freeboard = acres
Basin Volume at Top of Freeboard = acre-ft

Routed Hydrograph Results

The user can override the default CUHP hydrographs and runoff volumes by entering new values in the Inflow Hydrographs table (Columns W through AF).

	WQCV	EURV	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year	500 Year
Design Storm Return Period =	N/A	N/A	1.06	1.43	1.66	1.72	2.26	2.60	3.07
One-Hour Rainfall Depth (in) =	0.097	0.169	0.116	0.413	0.622	0.847	1.495	2.021	2.626
CUHP Runoff Volume (acre-ft) =	N/A	N/A	0.116	0.413	0.622	0.847	1.495	2.021	2.626
Inflow Hydrograph Volume (acre-ft) =	N/A	N/A	0.2	4.0	6.1	9.1	16.2	21.6	27.7
CUHP Predevelopment Peak Q (cfs) =	N/A	N/A							
OPTIONAL Override Predevelopment Peak Q (cfs) =	N/A	N/A							
Predevelopment Unit Peak Flow, q (cfs/acre) =	N/A	N/A	0.02	0.26	0.39	0.59	1.04	1.38	1.78
Peak Inflow Q (cfs) =	N/A	N/A	1.4	5.3	7.5	10.5	17.8	22.9	29.1
Peak Outflow Q (cfs) =	0.0	0.1	0.0	1.0	3.5	6.5	14.2	15.5	16.5
Ratio Peak Outflow to Predevelopment Q =	N/A	N/A	N/A	0.3	0.6	0.7	0.9	0.7	0.6
Structure Controlling Flow =	Plate	Plate	Plate	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Overflow Weir 1	Outlet Plate 1	Outlet Plate 1
Max Velocity through Grate 1 (fps) =	N/A	N/A	N/A	0.1	0.3	0.7	1.5	1.6	1.7
Max Velocity through Grate 2 (fps) =	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Time to Drain 97% of Inflow Volume (hours) =	44	63	52	71	68	65	56	49	41
Time to Drain 99% of Inflow Volume (hours) =	46	66	54	77	75	74	70	68	65
Maximum Ponding Depth (ft) =	2.25	2.69	2.33	3.51	3.67	3.82	4.11	4.73	5.68
Area at Maximum Ponding Depth (acres) =	0.15	0.18	0.16	0.23	0.24	0.24	0.26	0.28	0.33
Maximum Volume Stored (acre-ft) =	0.097	0.170	0.109	0.337	0.377	0.410	0.485	0.649	0.939

POND A4 - FOREBAY & TRICKLE CHANNEL CALCULATIONS

Project Name: Dawson Trails Blvd (North)

Project No.: 20-224-003

Per MHFD USDCM Vol. 3, Ch. 3, S. 3.0 & Table EDB-4; Vol. 2, Ch. 12, S. 5.14.2; Vol. 1, & Ch. 8, S. 7.1. Based on an octagonal forebay typical of local installations.

Water Quality Capture Volume (WQCV)

$$WQCV = (\alpha(0.91I^3 - 1.19I^2 + 0.78I)/12)A$$

Water Quality Capture Volume (ac-ft); Equations 3-1 & 3-2 combined

$$\alpha = 1.0$$

BMP type coefficient; typ. 1.0 for 40 hr drain time; Table 3-2

$$I = 0.1860$$

Imperviousness (percent expressed as a decimal)

$$A = 15.58 \text{ ac}$$

watershed tributary area upstream

WQCV = 0.143 ac-ft

Forebay Sizing

$$V = p(WQCV)43560/100$$

minimum forebay volume (ft³)

$$p = 3 \%$$

percentage of WQCV; Table EDB-4

V = 186.24 ft ³

minimum forebay volume

$$A_R = V/D_F$$

minimum required forebay area (ft²)

D _M = 1.50 ft

maximum forebay depth (sidewall depth); Table EDB-4

D _F = 1.25 ft

forebay front edge depth (D_M - 3"); Figure 14-8

$$A_R = 149.0 \text{ ft}^2$$

A _p = 198.0 ft ²
--

forebay area provided (must be ≥ A_R);

determined by increasing the central longitudinal dimension shown on the typical forebay detail;

see detail in construction drawings for complete forebay configuration and dimensions

Forebay Notch Sizing

$$Q_{100} = 35.28 \text{ cfs}$$

undetained peak discharge (from StormCAD calcs)

$$Q_{Rm} = 0.02 * Q_{100}$$

minimum forebay release (cfs); 2% of Q₁₀₀; Table EDB-4

$$Q_{Rm} = 0.71 \text{ cfs}$$

H = 1.25 ft

notch depth = D_F

$$L_{Nm} = Q_{Rm}/C_{BCW}H^{1.5}$$

minimum notch width (ft); (broad crested weir calculation); Equation 12-8; CBCW = 3.3

$$L_{Nm} = 0.15 \text{ ft}$$

L _N = 0.40 ft

notch width used for design;

(minimum notch width [L_{Nm}] increased by 20% of notch depth [D_F] for clogged condition

per MHFD BMP Worksheet and rounded up to nearest whole inch)

= 5.0 in

Trickle Channel Sizing

$$Q_R = C_{BCW}L_NH^{1.5}$$

maximum forebay notch capacity (cfs); (broad crested weir calculation); Equation 12-8; CBCW = 3.3

$$Q_R = 1.86 \text{ cfs}$$

$$Q_T = (1.49/n)AR^{2/3}S^{1/2}$$

flow in trickle channel (cfs); (Manning's Equation for uniform open channel flow); Equation 8-7

$$n = 0.013$$

Manning's roughness coefficient for concrete

$$S = 0.008 \text{ ft/ft}$$

friction slope (approx. slope of trickle channel for normal flow); 0.4% - 1% recommended by MHFD

$$A = WH$$

cross-sectional area of flow in channel (ft²); assume channel flowing full

W = 2.0 ft

width of trickle channel bottom

H = 0.5 ft

height of trickle channel sides

$$A = 1.00 \text{ ft}^2$$

$$R = A/P$$

hydraulic radius (ft)

$$R = 0.33 \text{ ft}$$

$$P = W+2H$$

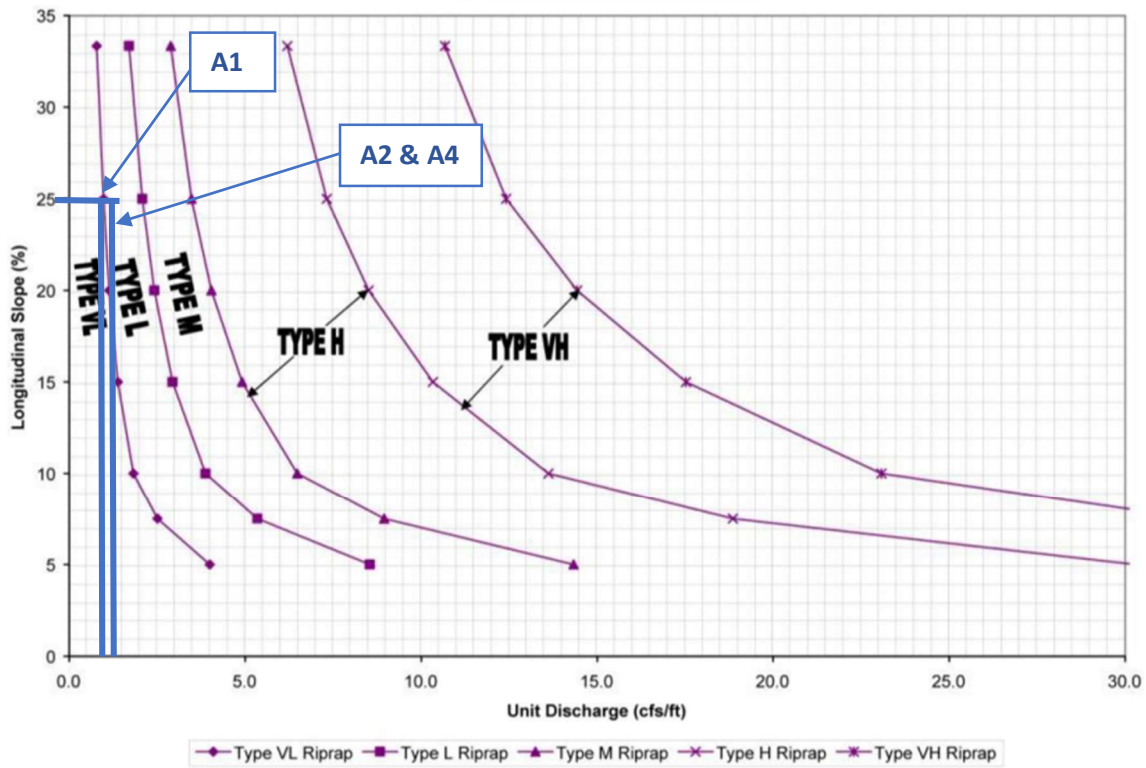
wetted perimeter (ft)

$$P = 3.00 \text{ ft}$$

Q _T = 4.77 cfs

maximum capacity of trickle channel (must be ≥ Q_R)

SPILLWAY CHANNEL AT CREST AND DOWNSTREAM SIDE OF EMBANKMENT



Pond A1:

Developed Q100 = 13.3 cfs
 Spillway Crest Length = 15 ft
 Unit Discharge = 0.9 cfs/ft
 Use Type VL Riprap

Pond A4:

Developed Q100 = 24.5 cfs
 Spillway Crest Length = 20 ft
 Unit Discharge = 1.2 cfs/ft
 Use Type L Riprap

Pond A2:

Developed Q100 = 26.0 cfs
 Spillway Crest Length = 20 ft
 Unit Discharge = 1.3 cfs/ft
 Use Type L Riprap

Outfall A4 - CIRCULAR PIPE END TREATMENT CALCULATIONS**1 OF 3**

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2 & S. 3.1

Riprap Sizing per Section 3.2.3

$$D_c = \begin{array}{l} 2.00 \text{ ft} \\ 24 \text{ in} \end{array}$$

Diameter of outlet pipe

$$D_a = (D_c + Y_n)/2 \text{ ft}$$

Adjusted value of H for supercritical flow; Equation 9-19

$$Y_n = 0.86 \text{ ft}$$

Culvert normal depth

$$D_a = 1.43 \text{ ft}$$

$$Y_t = 0.40 \text{ ft}$$

Tailwater depth

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 15.7 \text{ cfs}$$

100-year discharge

$$V = 5.00 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 3.14 \text{ ft}^2$$

Y

Supercritical flow Condition (Y OR N)?

$$Q/D^{1.5} = 9.18$$

Froude Parameter (for Rock Sizing figure 9-39)

$$Y_t/H = 0.28$$

Tailwater Depth/Conduit Height

$$\text{Use } Y_t/H = 0.40$$

(use 0.40 if Y_t unknown or hydraulic jump downstream)

$$D_{50} = 0.023Q/(Y_t^{1.2} D_c^{0.3})$$

Mean rock diameter by weight; Equation 9-17

$$D_{50} = \begin{array}{l} 0.97 \text{ ft} \\ 11.69 \text{ in} \end{array}$$

Use Type L Riprap per Figure 9-38 $D_{50} = 9 \text{ in}$ Thickness of Riprap Layer = $2 \times D_{50} = 1.5 \text{ ft}$ Low Tailwater Sizing per Section 3.2.2, Figure 9-37

$$D_c = 24 \text{ in}$$

Diameter of pipe

$$D = 1.0 \text{ ft}$$

$$W = 4.0 \text{ ft}$$

$$L = 15.0 \text{ ft}$$

Dimensions of Low Tailwater Basin per detail Figure 9-37

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.3

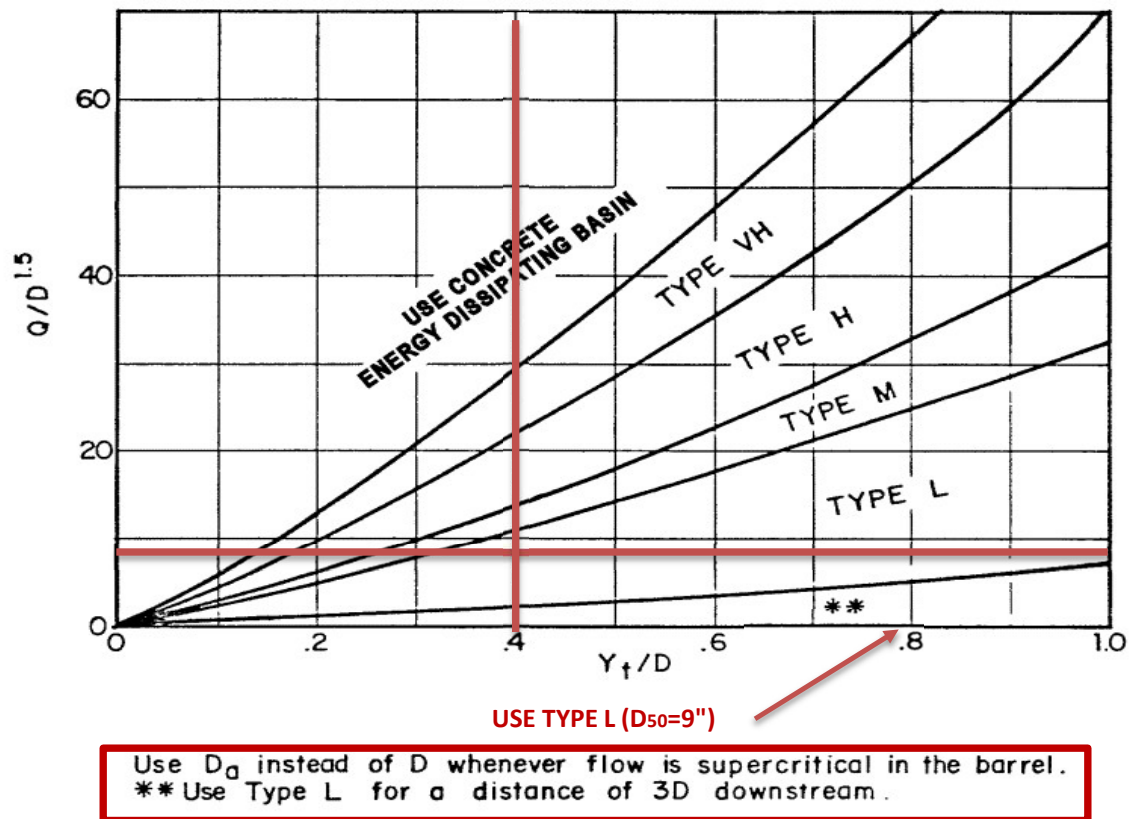


Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)

RIPRAP DESIGNATION	THICKNESS REQUIREMENTS FOR GRANULAR BEDDING		
	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

Outfall A4 - CIRCULAR PIPE END TREATMENT CALCULATIONS

3 OF 3

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.2

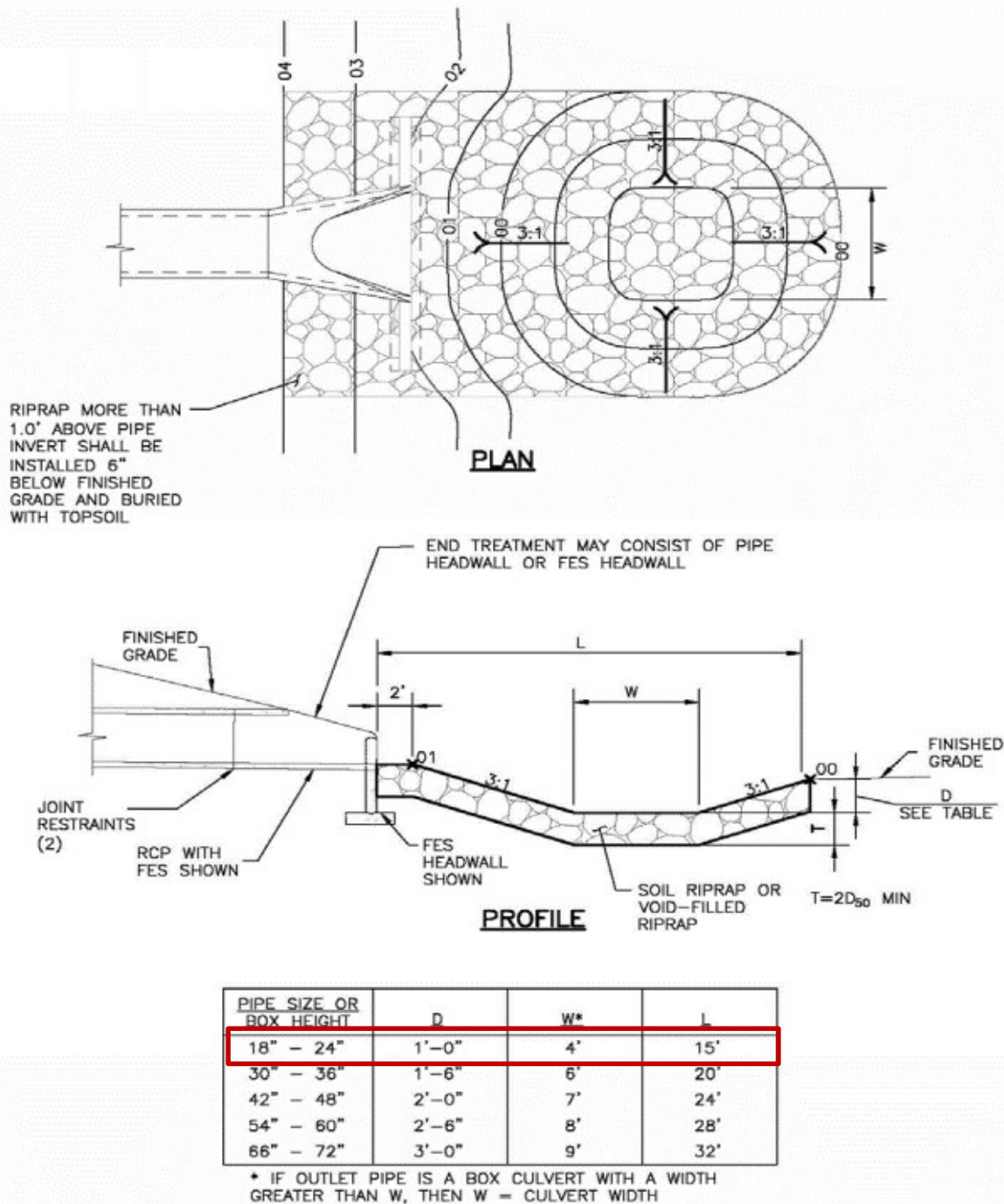


Figure 9-37. Low tailwater riprap basin

Outfall A1 - CIRCULAR PIPE END TREATMENT CALCULATIONS**1 OF 3**Outfall A1 Low Tailwater Basin Calcs

Per USDCM Vol. 2, Ch. 9, S. 3.2 & S. 3.1

Riprap Sizing per Section 3.2.3

$$D_c = \begin{array}{ll} 2.00 & \text{ft} \\ 24 & \text{in} \end{array}$$

Diameter of outlet pipe

$$D_a = (D_c + Y_n)/2 \text{ ft}$$

Adjusted value of H for supercritical flow; Equation 9-19

$$Y_n = 0.54 \text{ ft}$$

Culvert normal depth

$$V = 5.09 \text{ fps}$$

$$D_a = 1.27 \text{ ft}$$

$$Fr = 1.220655$$

$$Y_t = 0.20 \text{ ft}$$

Tailwater depth

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 4 \text{ cfs}$$

100-year discharge

$$V = 5.00 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 0.80 \text{ ft}^2$$

Y

Supercritical flow Condition (Y OR N)?

$$Q/D^{1.5} = 2.79$$

Froude Parameter (for Rock Sizing figure 9-39)

$$Y_t/H = 0.16$$

Tailwater Depth/Conduit Height

$$\text{Use } Y_t/H = 0.40$$

(use 0.40 if Y_t unknown or hydraulic jump downstream)

$$D_{50} = 0.023Q/(Y_t^{1.2} D_c^{0.3})$$

Mean rock diameter by weight; Equation 9-17

$$D_{50} = 0.59 \text{ ft}$$

$$7.09 \text{ in}$$

Use Type L Riprap per Figure 9-38	$D_{50} =$	9	in
Thickness of Riprap Layer = $2 \times D_{50} =$			
		1.5	ft

Low Tailwater Sizing per Section 3.2.2, Figure 9-37

$$D_c = 24 \text{ in}$$

Diameter of pipe

$$D = 1.0 \text{ ft}$$

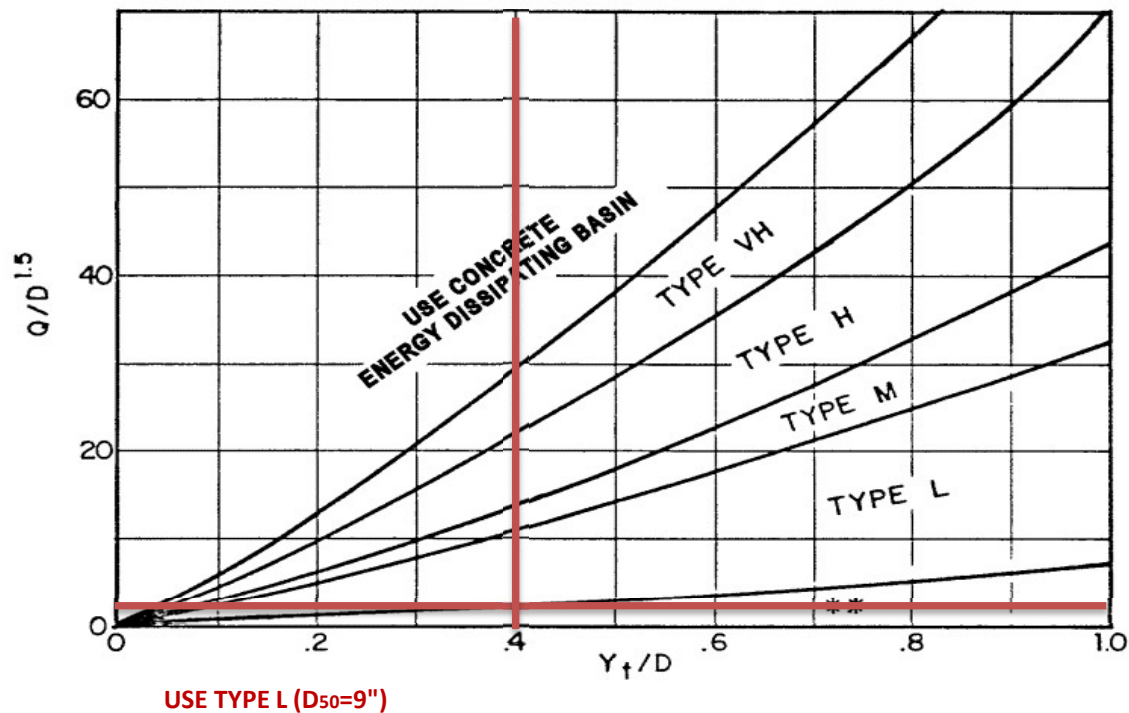
$$W = 4.0 \text{ ft}$$

$$L = 15.0 \text{ ft}$$

Dimensions of Low Tailwater Basin per detail Figure 9-37

Outfall A1 Low Tailwater Basin Calcs

Per USDCM Vol. 2, Ch. 9, S. 3.2.3



Use D_a instead of D whenever flow is supercritical in the barrel.

** Use Type L for a distance of $3D$ downstream.

Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)

THICKNESS REQUIREMENTS FOR GRANULAR BEDDING			
RIPRAP DESIGNATION	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

Outfall A1 Low Tailwater Basin Calcs

Per USDCM Vol. 2, Ch. 9, S. 3.2.2

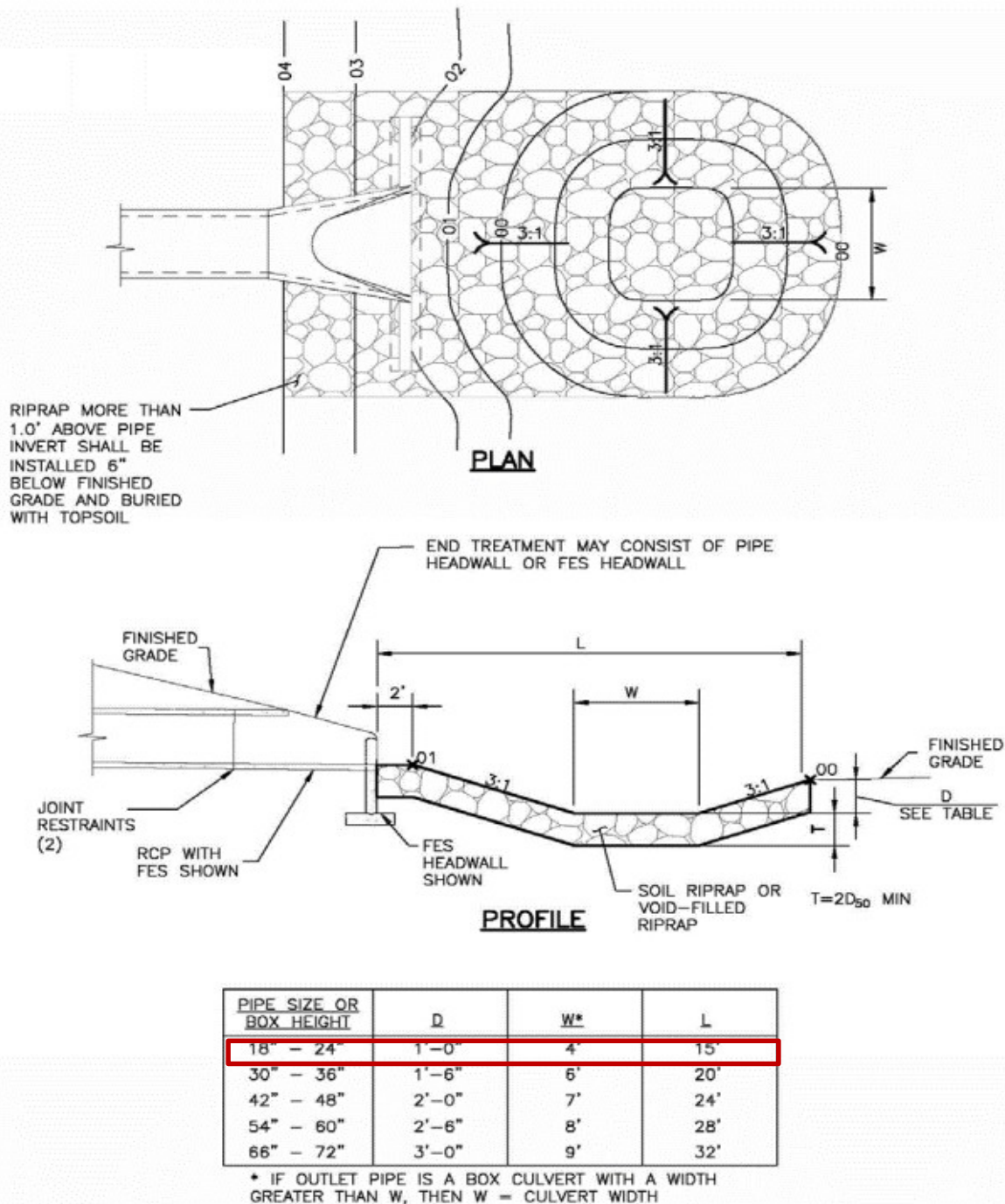


Figure 9-37. Low tailwater riprap basin

CULVERT 7 - OUTLET RIPRAP CALCULATIONS

SHEET 1 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1 & 3.2

Riprap Sizing per Section 3.2.3

$$L_p = (1/2 \tan \theta) [(A_t/Y_t) - W]$$

Length of protection; Equation 9-11

$$D = \frac{30.00}{2.50} \text{ in ft}$$

Pipe diameter

$$Y_n = 1.21 \text{ ft}$$

Normal depth of flow at culvert outlet

$$V = 14.4 \text{ ft/s}$$

Flow velocity at culvert outlet

$$FR = \frac{2.31}{Y}$$

Froude Number

Supercritical flow?

$$Y_t = 0.61 \text{ ft}$$

Tailwater depth

$$D_a = 1.86$$

D parameter for supercritical flow

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 24.26 \text{ cfs}$$

100-year discharge

$$V = 5 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 4.85 \text{ ft}^2$$

$$Q/D^{1.5} = 6.14$$

Froude Parameter (for Rock Sizing figure 9-38)

$$Y_t/D = 0.24$$

Tailwater Depth/Conduit Height

$$Y_t/D_a = 0.33$$

(use 0.40 if unknown or hydraulic jump is suspected downstream)

Use Type L Riprap per Figure 9-38	$D_{50} = 9 \text{ in}$
-----------------------------------	-------------------------

See Figure 9-38

Thickness of Riprap Layer = $2D_{50} = 1.5 \text{ ft}$
--

See Figure 9-34

$$Q/D^{2.5} = 2.45$$

Froude Parameter (for Extent of Protection figure 9-35)
(must be ≤ 6.0)

$$1/2 \tan \theta = 2.5$$

 (F_E) Expansion factor; Figure 9-35

$L_p = 14 \text{ ft}$

See Figure 9-34

$$T = 2(L_p \tan \theta) + W$$

Width of riprap at furthest downstream point; Equation 9-13

$$\theta = \tan^{-1} [1/(2 * F_E)]$$

Equation 9-13

$$\theta = 0.20^\circ$$

$T = 8 \text{ ft}$

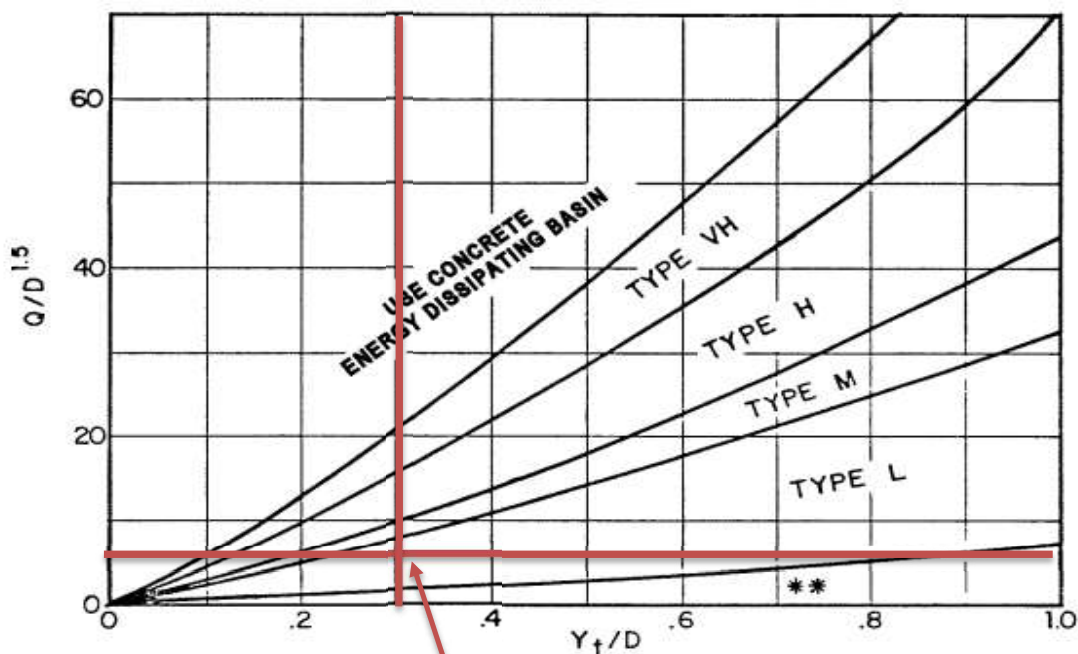
See Figure 9-34

CULVERT 7 - OUTLET RIPRAP CALCULATIONS

SHEET 2 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.3



Use D_d instead of D whenever flow is supercritical in the barrel.
** Use Type L for a distance of $3D$ downstream.

USE TYPE L ($D_{50}=9"$)

Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)

THICKNESS REQUIREMENTS FOR GRANULAR BEDDING			
RIPRAP DESIGNATION	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

- 1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING.** THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

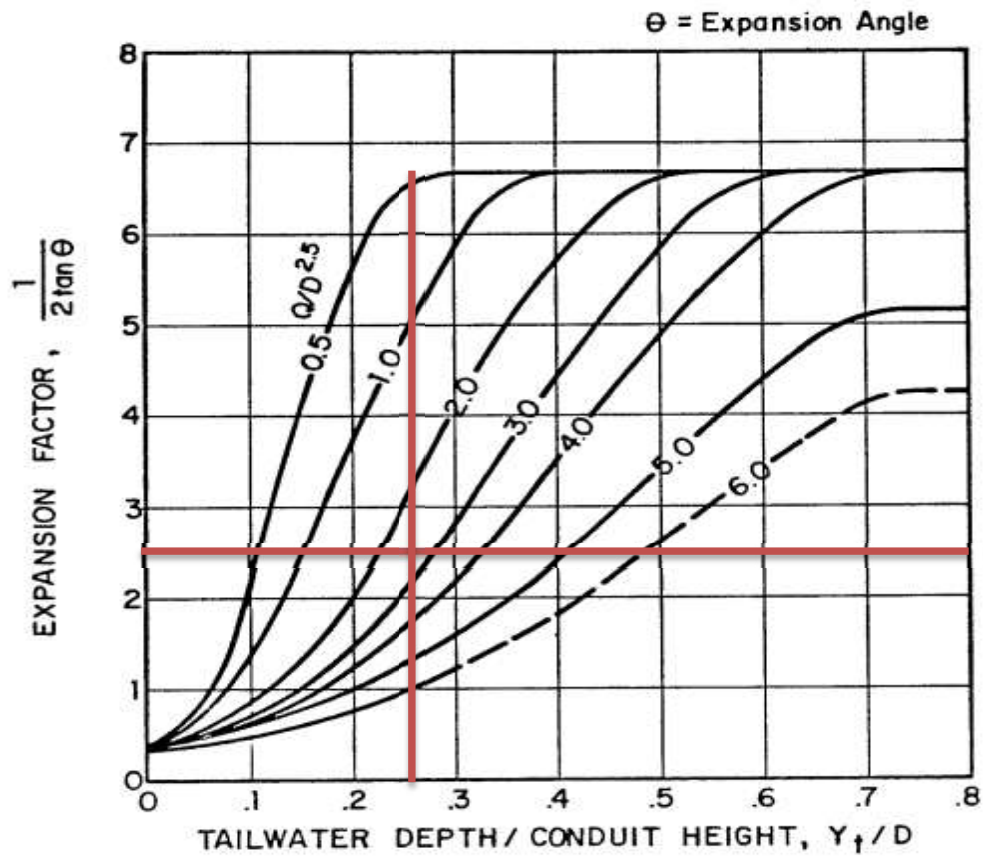


Figure 9-35. Expansion factor for circular conduits

CULVERT 7 - OUTLET RIPRAP CALCULATIONS

SHEET 4 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

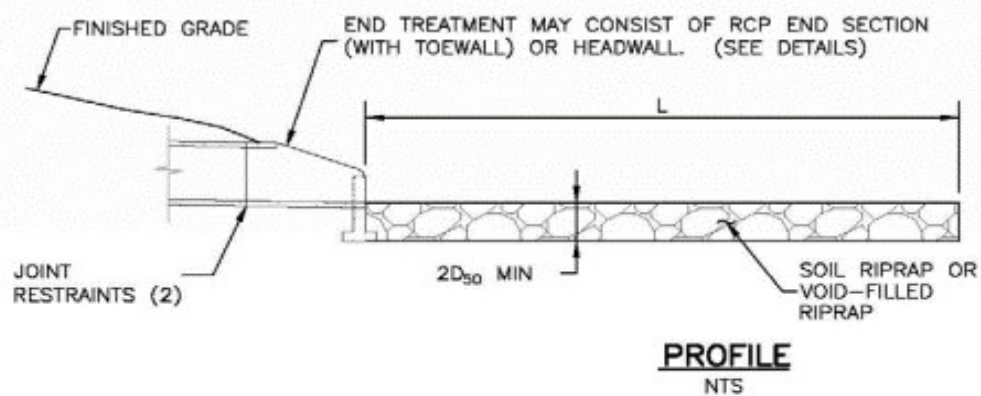
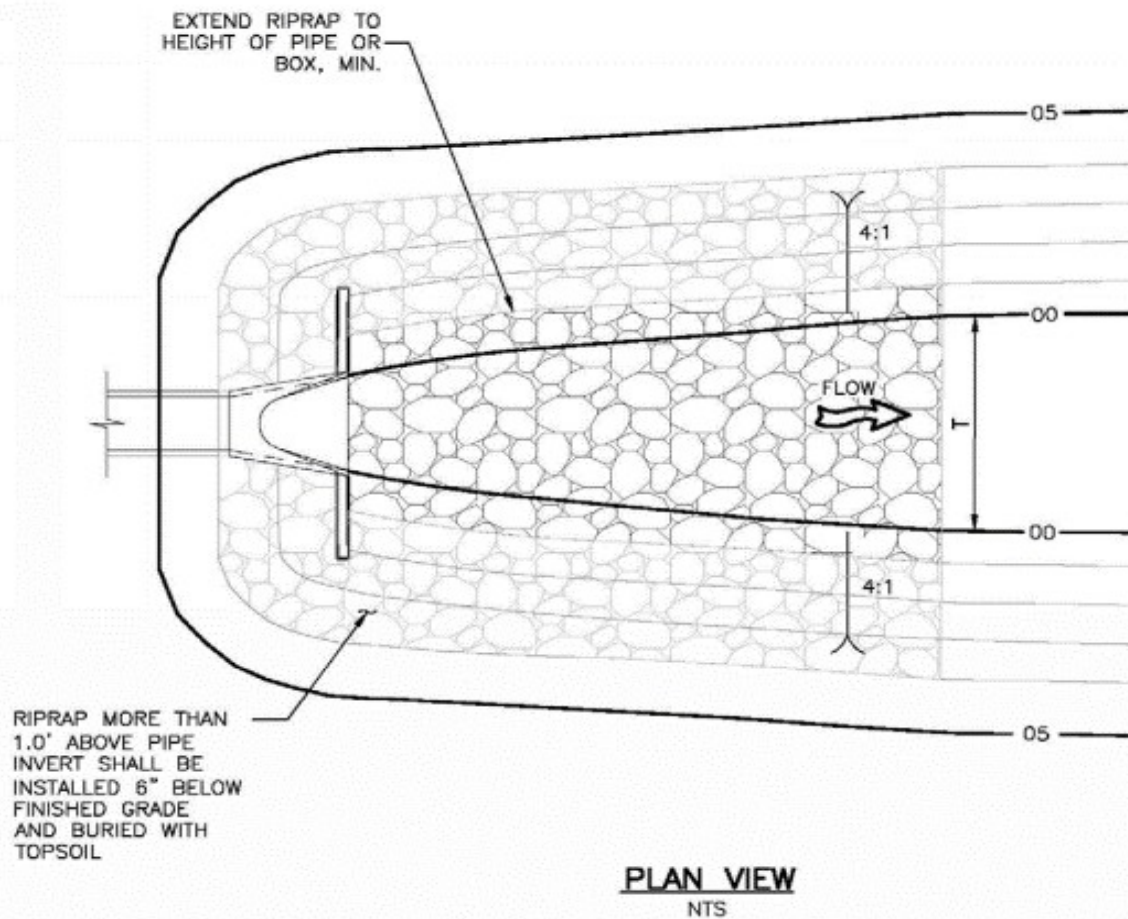


Figure 9-34. Riprap apron detail for culverts in-line with the channel

CULVERT 7 - OUTLET RIPRAP CALCULATIONS

SHEET 5 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1.2

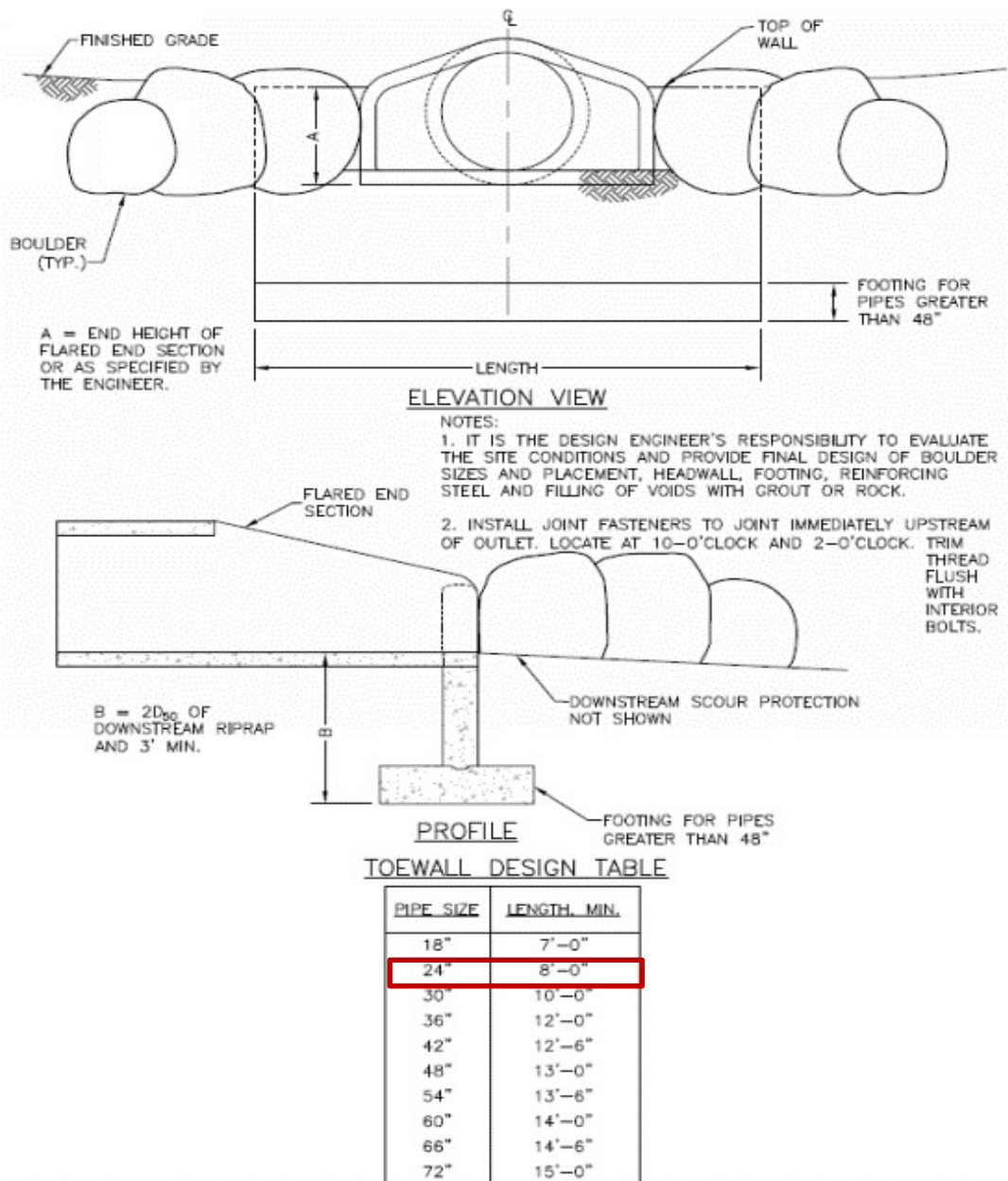


Figure 9-30. Flared end section (FES) headwall concept

Toe wall Dimensions		
Length =	8	ft
Depth (B) =	3.0	ft
Footing Required =	N	

BOX CULVERT OS-8 END TREATMENT CALCULATIONS 1 OF 3

Project: Prairie Hawk North 6x14 Box

Per USDCM Vol. 2, Ch. 9, S. 3.2 & S. 3.1

Riprap Sizing per Section 3.2.3

$$W = 16.00 \text{ ft}$$

Width of rectangular outlet pipe

$$H = 6.00 \text{ ft}$$

Height of rectangular outlet pipe

$$H_a = (H + Y_n)/2 \text{ ft}$$

Adjusted value of H for supercritical flow; Equation 9-19

$$Y_n = 4.56 \text{ ft}$$

Culvert normal depth

$$H_a = 5.28 \text{ ft}$$

$$Y_t = 4.54 \text{ ft}$$

Tailwater depth

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 877 \text{ cfs}$$

100-year discharge

$$V = 12.08 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 72.60 \text{ ft}^2$$

N

Supercritical flow Condition (Y OR N)?

$$Q/WH^{0.5} = 22.38$$

Froude Parameter (for Rock Sizing figure 9-39)

$$Y_t/H = 0.76$$

Tailwater Depth/Conduit Height

(use 0.40 if Y_t unknown or hydraulic jump downstream)

$$D_{50} = 0.014H^{0.5}Q/Y_tW$$

Mean rock diameter by weight; Equation 9-17

$$D_{50} = 0.41 \text{ ft}$$

$$4.97 \text{ in}$$

Use Type L Riprap per Figure 9-38 $D_{50} = 9 \text{ in}$ **Thickness of Riprap Layer = $2xD_{50} = 1.5 \text{ ft}$** Low Tailwater Sizing per Section 3.2.2, Figure 9-37

$$\text{Box Height} = 72 \text{ in}$$

$$D = 3.0 \text{ ft}$$

$$W = 9.0 \text{ ft}$$

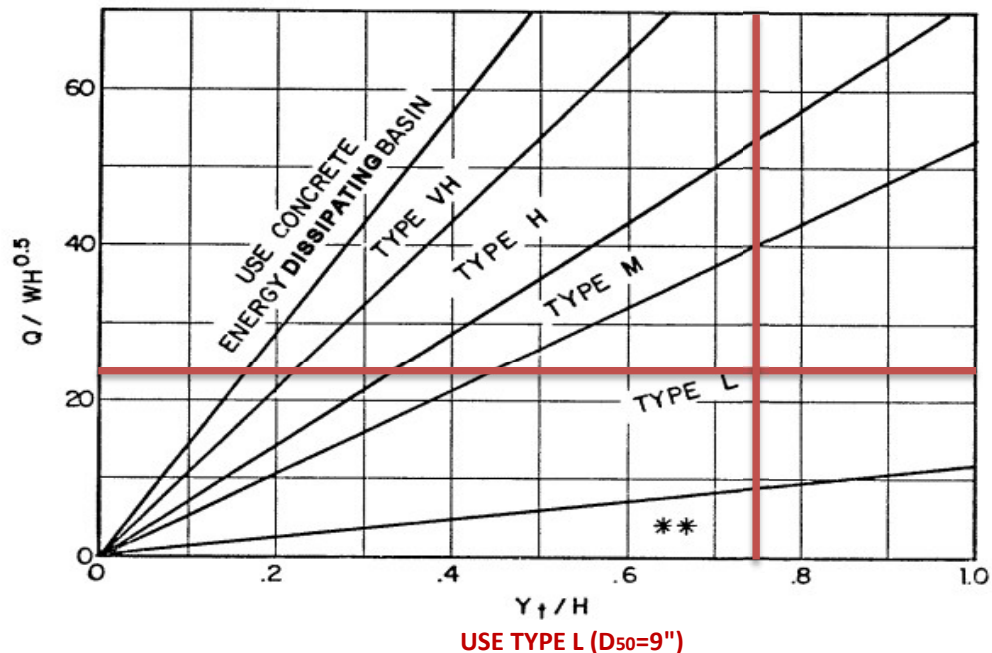
$$L = 32.0 \text{ ft}$$

Dimensions of Low Tailwater Basin per detail Figure 9-37

BOX CULVERT OS-8 END TREATMENT CALCULATIONS 2 OF 3

Project: Prairie Hawk North 6x14 Box

Per USDCM Vol. 2, Ch. 9, S. 3.2.3



Use H_d instead of H whenever culvert has supercritical flow in the barrel.

**Use Type L for a distance of $3H$ downstream.

Figure 9-39. Riprap erosion protection at rectangular conduit outlet (valid for $Q/WH^{1.5} \leq 8.0$)

THICKNESS REQUIREMENTS FOR GRANULAR BEDDING			
RIPRAP DESIGNATION	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.

2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

BOX CULVERT OS-8 END TREATMENT CALCULATIONS 3 OF 3

Project: Prairie Hawk North 6x14 Box

Per USDCM Vol. 2, Ch. 9, S. 3.2.2

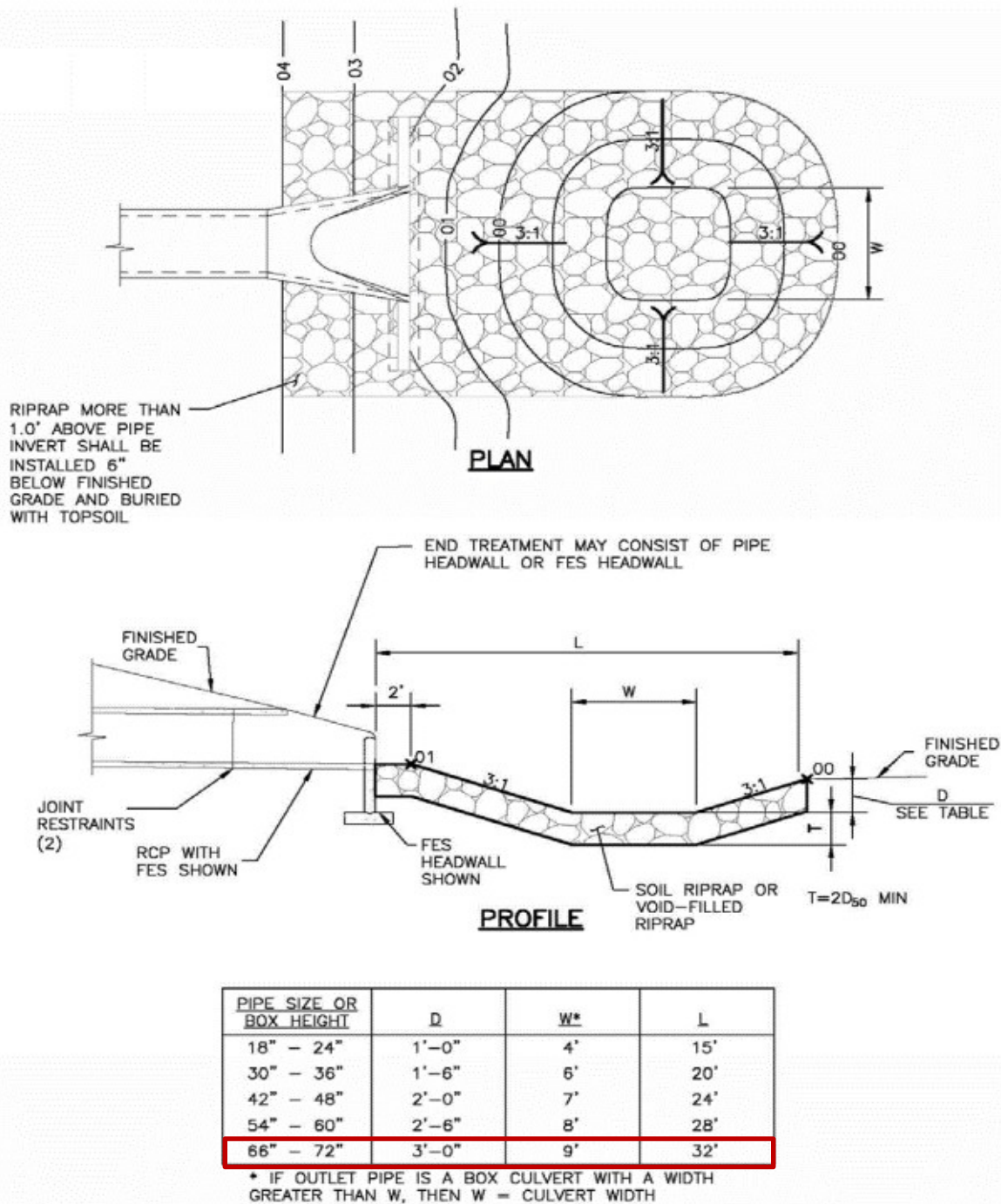


Figure 9-37. Low tailwater riprap basin

CULVERT 9 - OUTLET RIPRAP CALCULATIONS

SHEET 1 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1 & 3.2

Riprap Sizing per Section 3.2.3

$$L_p = (1/2 \tan \theta) [(A_t/Y_t) - W]$$

Length of protection; Equation 9-11

$$D = \begin{matrix} 72.00 & \text{in} \\ 6.00 & \text{ft} \end{matrix}$$

Pipe diameter (2 - 36" pipes)

$$Y_n = 1.39 \text{ ft}$$

Normal depth of flow at culvert outlet

$$V = 12.1 \text{ ft/s}$$

Flow velocity at culvert outlet

$$FR = 1.81$$

Froude Number

Y

Supercritical flow?

$$Y_t = 0.91 \text{ ft}$$

Tailwater depth

$$D_a = 3.70$$

D parameter for supercritical flow

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 78 \text{ cfs}$$

100-year discharge

$$V = 5 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 15.60 \text{ ft}^2$$

$$Q/D^{1.5} = 5.31$$

Froude Parameter (for Rock Sizing figure 9-38)

$$Y_t/D = 0.15$$

Tailwater Depth/Conduit Height

$$Y_t/D_a = 0.25$$

(use 0.40 if unknown or hydraulic jump is suspected downstream)

Use Type L Riprap per Figure 9-38

$$D_{50} = 9 \text{ in}$$

See Figure 9-38

$$\text{Thickness of Riprap Layer} = 2D_{50} = 1.5 \text{ ft}$$

See Figure 9-34

$$Q/D^{2.5} = 2.97$$

Froude Parameter (for Extent of Protection figure 9-35)
(must be ≤ 6.0)

$$1/2 \tan \theta = 1$$

 (F_E) Expansion factor; Figure 9-35

$$L_p = 11 \text{ ft}$$

See Figure 9-34

$$T = 2(L_p \tan \theta) + W$$

Width of riprap at furthest downstream point; Equation 9-13

$$\theta = \tan^{-1} [1/(2 * F_E)]$$

Equation 9-13

$$\theta = 0.46^\circ$$

$$T = 17 \text{ ft}$$

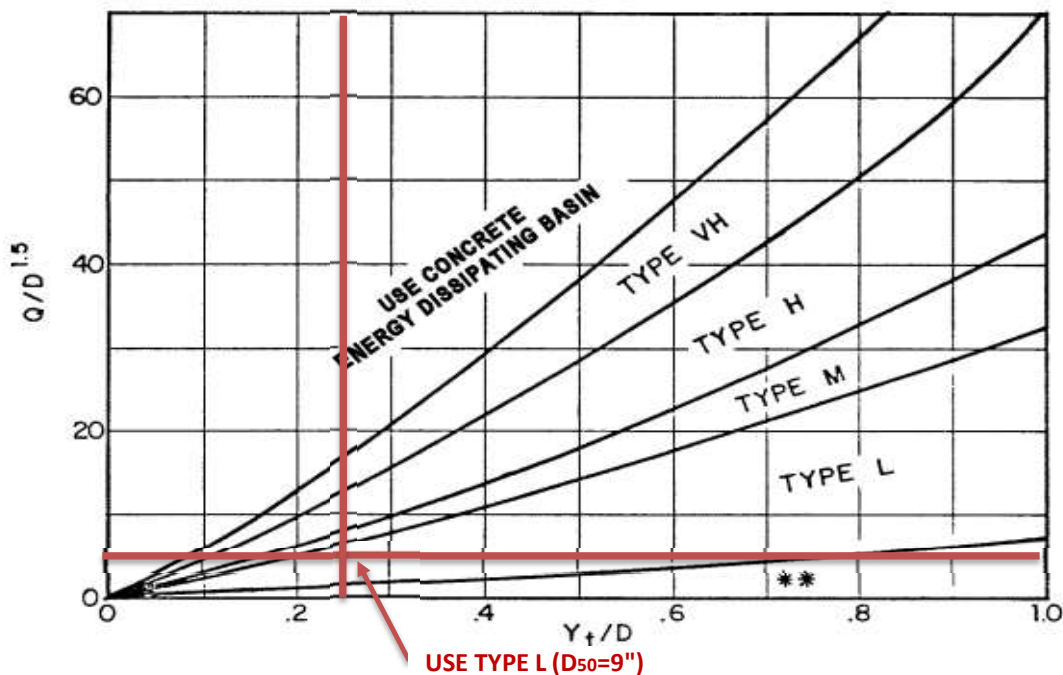
See Figure 9-34

CULVERT 9 - OUTLET RIPRAP CALCULATIONS

SHEET 2 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.3

Use D_d instead of D whenever flow is supercritical in the barrel.** Use Type L for a distance of $3D$ downstream.**Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)**

RIPRAP DESIGNATION	THICKNESS REQUIREMENTS FOR GRANULAR BEDDING		
	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

CULVERT 9 - OUTLET RIPRAP CALCULATIONS

SHEET 3 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

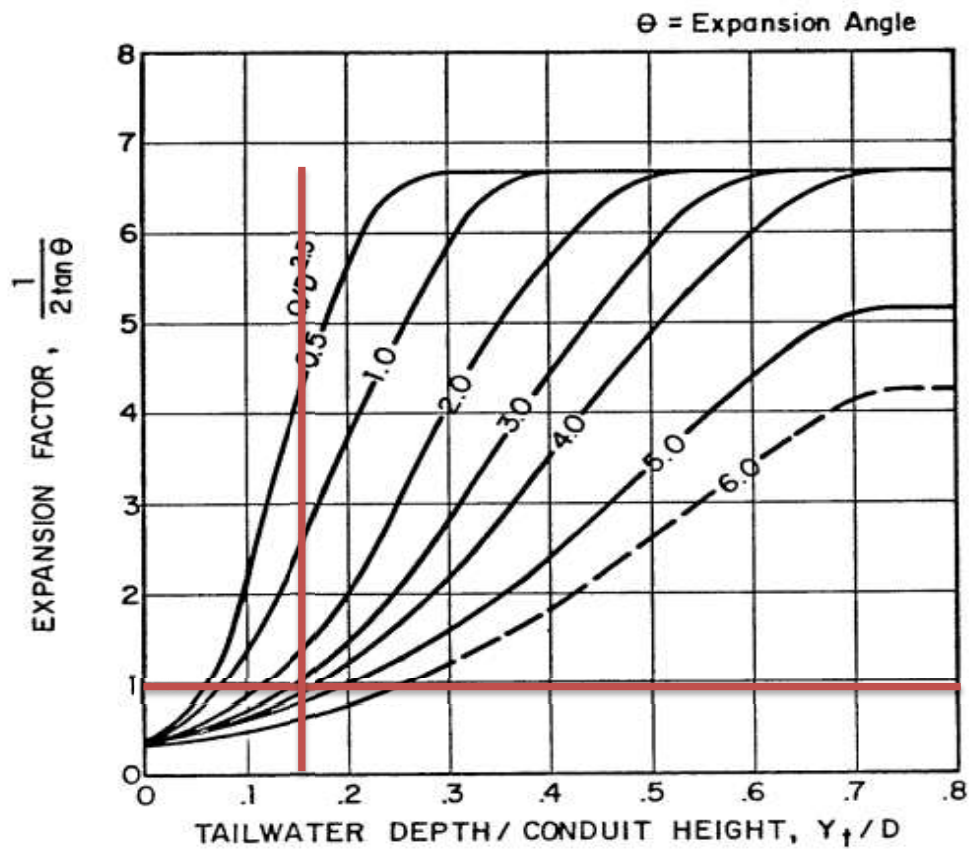


Figure 9-35. Expansion factor for circular conduits

CULVERT 9 - OUTLET RIPRAP CALCULATIONS

SHEET 4 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

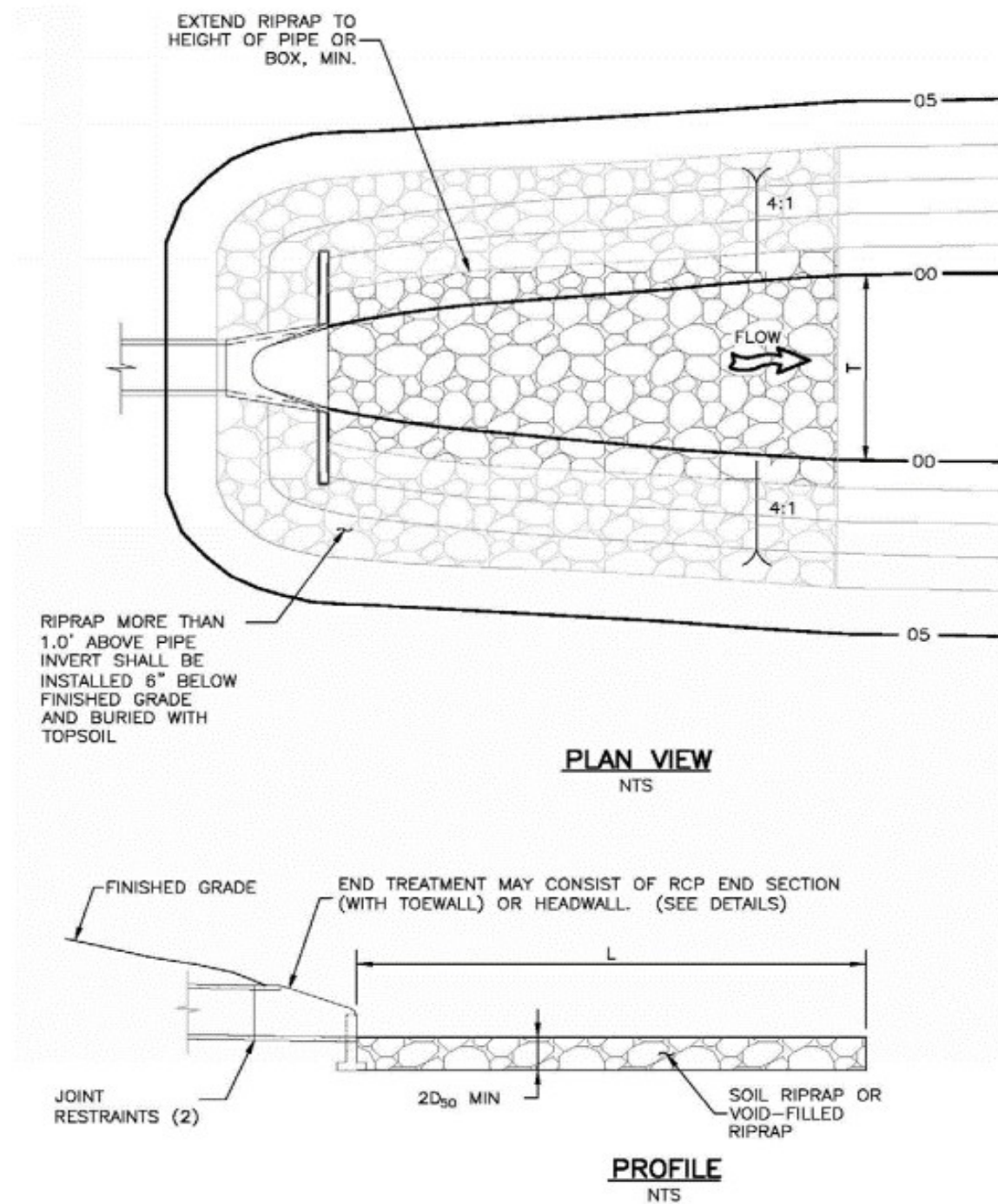


Figure 9-34. Riprap apron detail for culverts in-line with the channel

CULVERT 9 - OUTLET RIPRAP CALCULATIONS

SHEET 5 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1.2

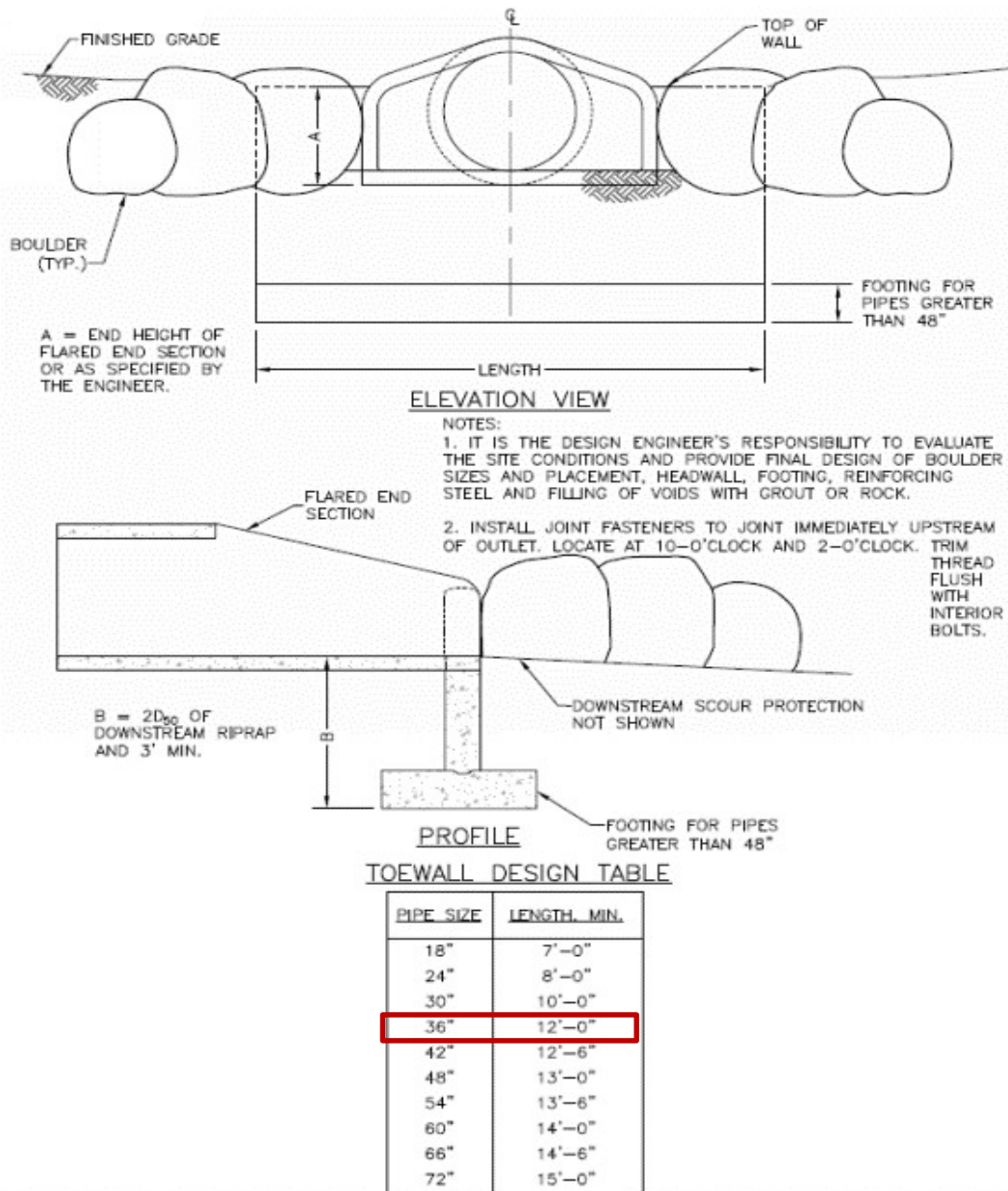


Figure 9-30. Flared end section (FES) headwall concept

Toe wall Dimensions		
Length =	12	ft
Depth (B) =	3.0	ft
Footing Required =	N	

CULVERT 10 - OUTLET RIPRAP CALCULATIONS

SHEET 1 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1 & 3.2

Riprap Sizing per Section 3.2.3

$$L_p = (1/2 \tan \theta) [(A_t/Y_t) - W]$$

Length of protection; Equation 9-11

$$D = \begin{matrix} 24.00 & \text{in} \\ 2.00 & \text{ft} \end{matrix}$$

Pipe diameter

$$Y_n = 1.16 \text{ ft}$$

Normal depth of flow at culvert outlet

$$V = 7.4 \text{ ft/s}$$

Flow velocity at culvert outlet

$$FR = 1.20$$

Froude Number

Y

Supercritical flow?

$$Y_t = 0.47 \text{ ft}$$

Tailwater depth

$$D_a = 1.58$$

D parameter for supercritical flow

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 19.8 \text{ cfs}$$

100-year discharge

$$V = 5 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 3.96 \text{ ft}^2$$

$$Q/D^{1.5} = 7.00$$

Froude Parameter (for Rock Sizing figure 9-38)

$$Y_t/D = 0.24$$

Tailwater Depth/Conduit Height

$$Y_t/D_a = 0.30$$

(use 0.40 if unknown or hydraulic jump is suspected downstream)

Use Type L Riprap per Figure 9-38

$$D_{50} = 9 \text{ in}$$

See Figure 9-38

$$\text{Thickness of Riprap Layer} = 2D_{50} = 1.5 \text{ ft}$$

See Figure 9-34

$$Q/D^{2.5} = 6.31$$

Froude Parameter (for Extent of Protection figure 9-35)
(must be ≤ 6.0)

$$1/2 \tan \theta = 1$$

 (F_E) Expansion factor; Figure 9-35

$$L_p = 6 \text{ ft}$$

See Figure 9-34

$$T = 2(L_p \tan \theta) + W$$

Width of riprap at furthest downstream point; Equation 9-13

$$\theta = \tan^{-1}[1/(2 \cdot F_E)]$$

Equation 9-13

$$\theta = 0.46^\circ$$

$$T = 8 \text{ ft}$$

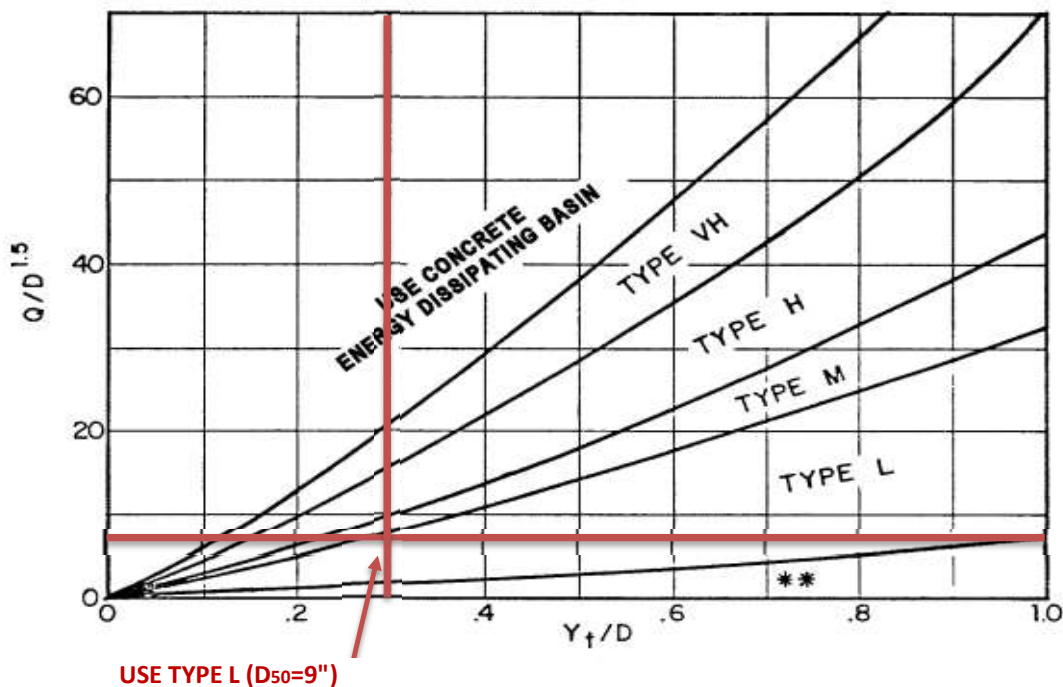
See Figure 9-34

CULVERT 10 - OUTLET RIPRAP CALCULATIONS

SHEET 2 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.3

Use D_d instead of D whenever flow is supercritical in the barrel.** Use Type L for a distance of $3D$ downstream.**Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)**

RIPRAP DESIGNATION	THICKNESS REQUIREMENTS FOR GRANULAR BEDDING		
	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

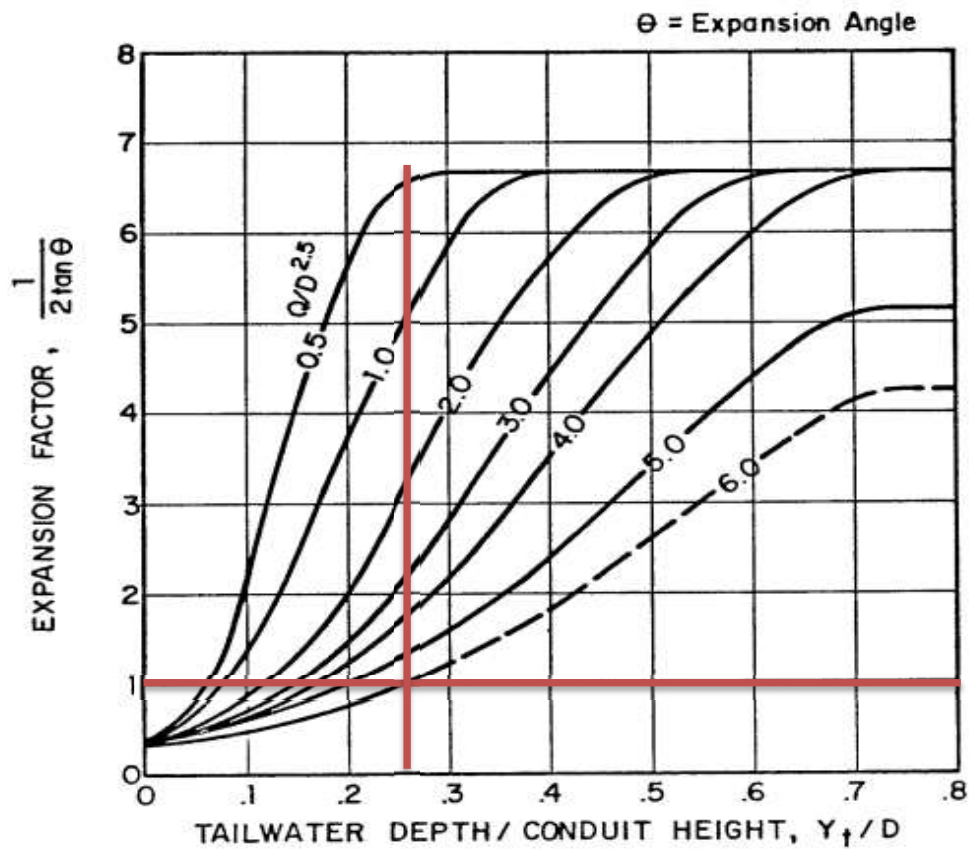
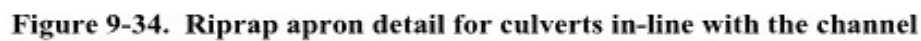


Figure 9-35. Expansion factor for circular conduits

SHEET 4 OF 5

Per USDCM Vol. 2, Ch. 9, S. 3.2.1



CULVERT 10 - OUTLET RIPRAP CALCULATIONS

SHEET 5 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1.2

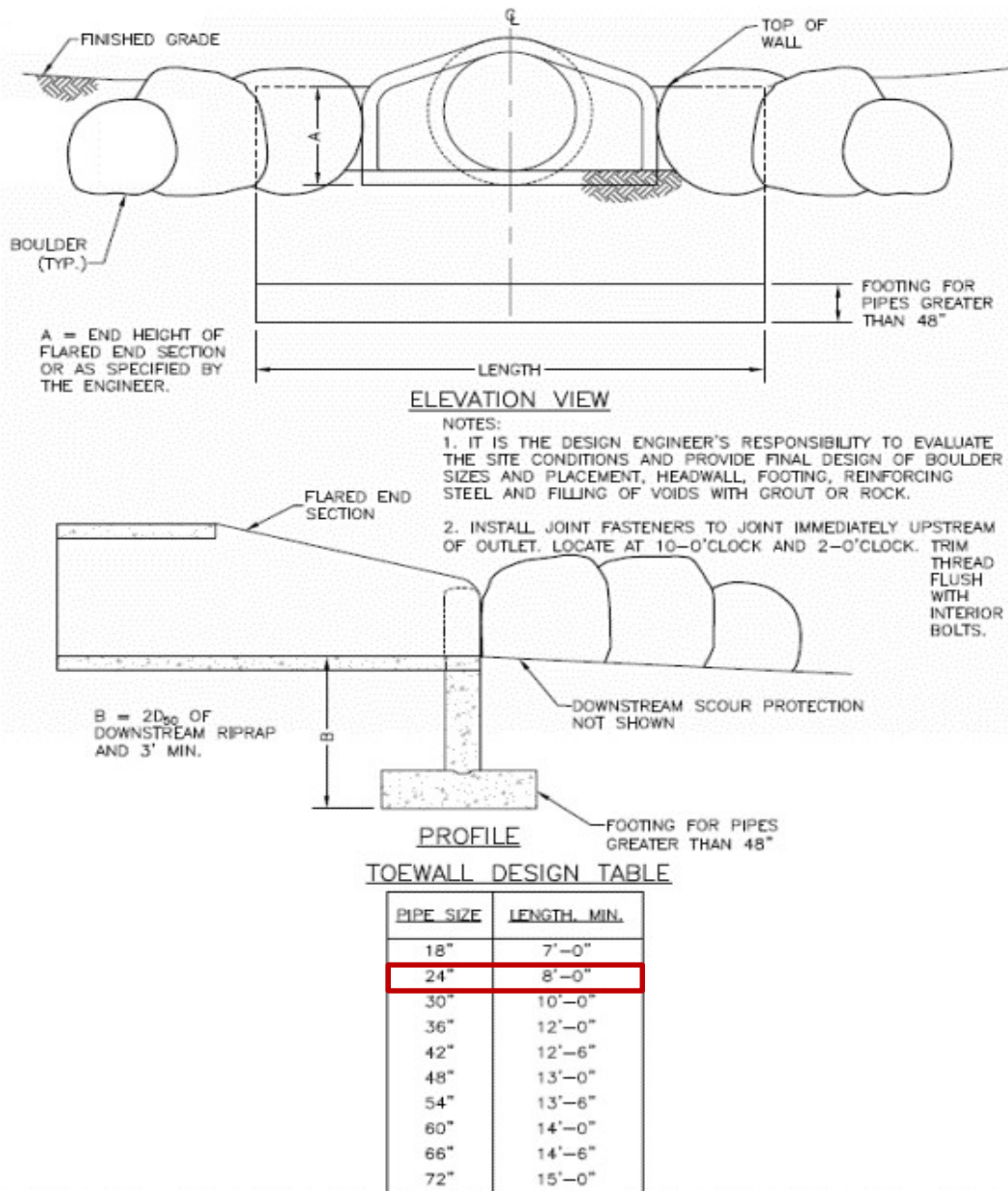


Figure 9-30. Flared end section (FES) headwall concept

Toe wall Dimensions		
Length =	8	ft
Depth (B) =	3.0	ft
Footing Required =	N	

CULVERT 11 - OUTLET RIPRAP CALCULATIONS

SHEET 1 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1 & 3.2

Riprap Sizing per Section 3.2.3

$$L_p = (1/2 \tan \theta) [(A_t/Y_t) - W]$$

Length of protection; Equation 9-11

$$D = \begin{matrix} 36.00 & \text{in} \\ 3.00 & \text{ft} \end{matrix}$$

Pipe diameter

$$Y_n = 1.93 \text{ ft}$$

Normal depth of flow at culvert outlet

$$V = 11.5 \text{ ft/s}$$

Flow velocity at culvert outlet

$$FR = 1.46$$

Froude Number

Y

Supercritical flow?

$$Y_t = 0.95 \text{ ft}$$

Tailwater depth

$$D_a = 2.47$$

D parameter for supercritical flow

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 52.3 \text{ cfs}$$

100-year discharge

$$V = 5 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 10.46 \text{ ft}^2$$

$$Q/D^{1.5} = 10.07$$

Froude Parameter (for Rock Sizing figure 9-38)

$$Y_t/D = 0.32$$

Tailwater Depth/Conduit Height

$$Y_t/D_a = 0.39$$

(use 0.40 if unknown or hydraulic jump is suspected downstream)

Use Type L Riprap per Figure 9-38

$$D_{50} = 9 \text{ in}$$

See Figure 9-38

$$\text{Thickness of Riprap Layer} = 2D_{50} = 1.5 \text{ ft}$$

See Figure 9-34

$$Q/D^{2.5} = 5.48$$

Froude Parameter (for Extent of Protection figure 9-35)
(must be ≤ 6.0)

$$1/2 \tan \theta = 2.2$$

(F_E) Expansion factor; Figure 9-35

$$L_p = 18 \text{ ft}$$

See Figure 9-34

$$T = 2(L_p \tan \theta) + W$$

Width of riprap at furthest downstream point; Equation 9-13

$$\theta = \tan^{-1}[1/(2 \cdot F_E)]$$

Equation 9-13

$$\theta = 0.22^\circ$$

$$T = 11 \text{ ft}$$

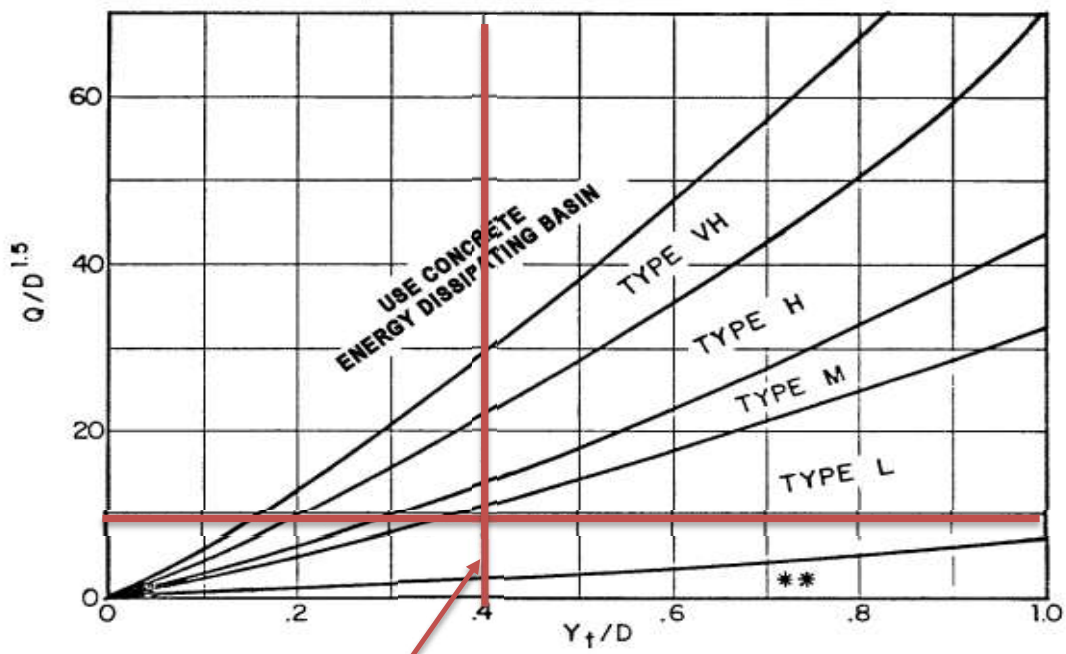
See Figure 9-34

CULVERT 11 - OUTLET RIPRAP CALCULATIONS

SHEET 2 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.3

**USE TYPE L ($D_{50}=9''$)**Use D_d instead of D whenever flow is supercritical in the barrel.** Use Type L for a distance of $3D$ downstream.**Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D2.5 \leq 6.0$)**

THICKNESS REQUIREMENTS FOR GRANULAR BEDDING			
RIPRAP DESIGNATION	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

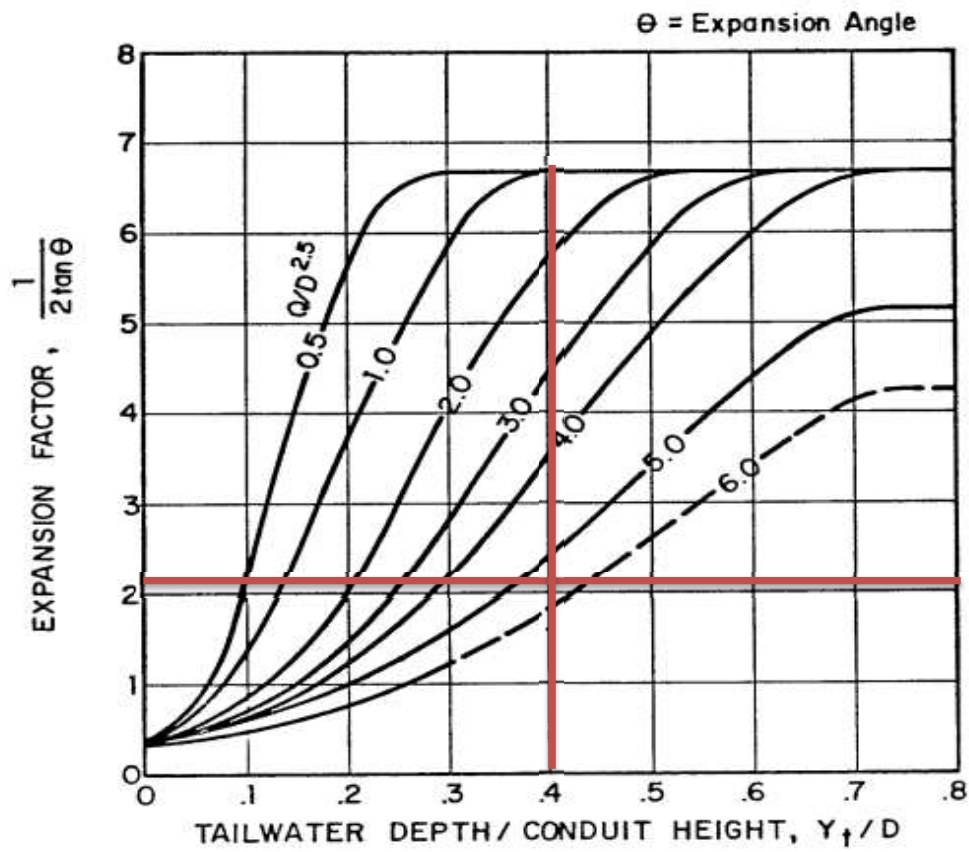
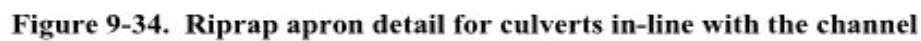


Figure 9-35. Expansion factor for circular conduits

SHEET 4 OF 5

Per USDCM Vol. 2, Ch. 9, S. 3.2.1



CULVERT 11 - OUTLET RIPRAP CALCULATIONS

SHEET 5 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1.2

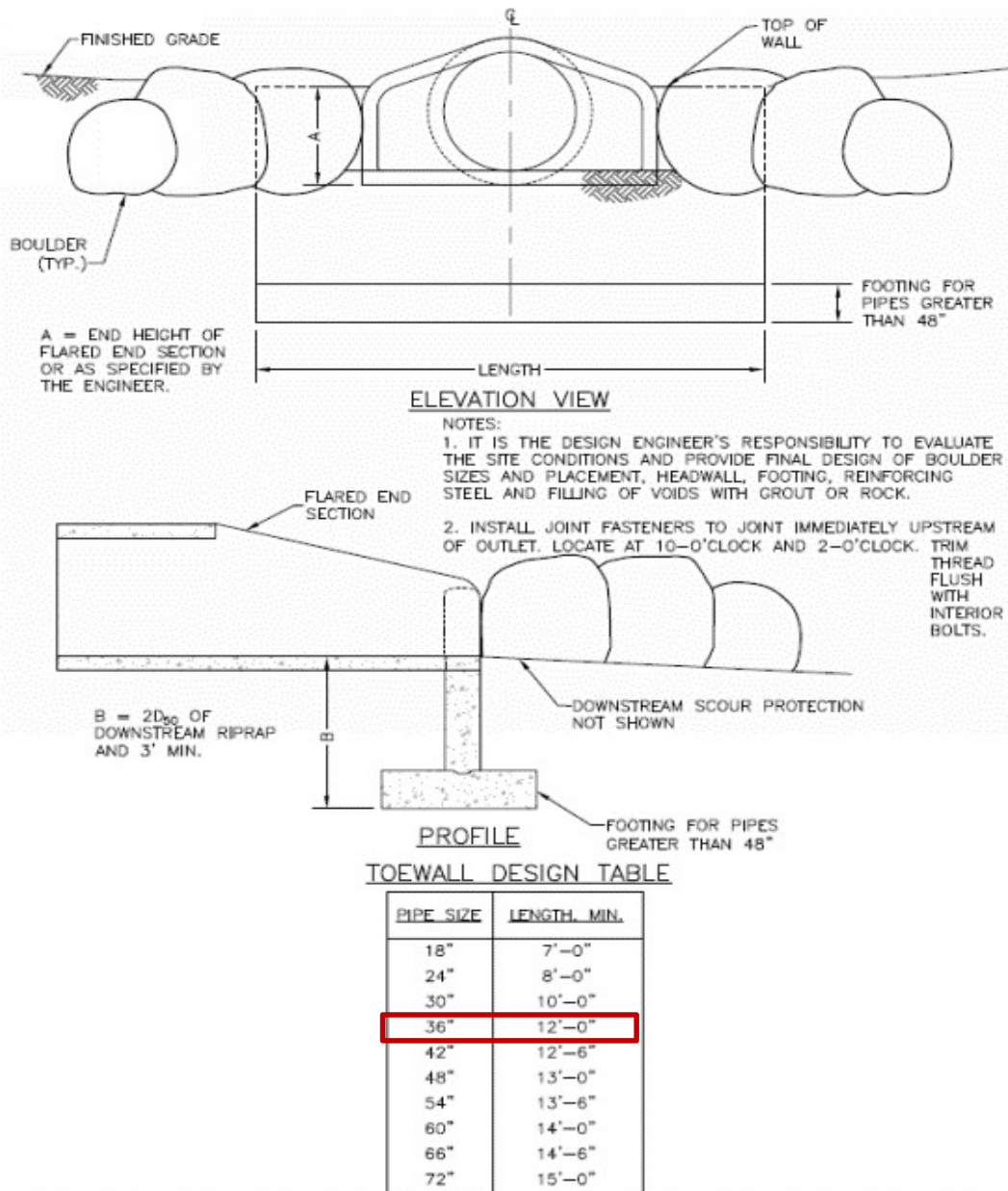


Figure 9-30. Flared end section (FES) headwall concept

Toe wall Dimensions		
Length =	12	ft
Depth (B) =	3.0	ft
Footing Required =	N	

CULVERT 12 - OUTLET RIPRAP CALCULATIONS

SHEET 1 OF 5

Project: Dawson Trails Boulevard North
 Per USDCM Vol. 2, Ch. 9, S. 3.1 & 3.2

Riprap Sizing per Section 3.2.3

$$L_p = (1/2 \tan \theta) [(A_t/Y_t) - W]$$

Length of protection; Equation 9-11

$$D = \begin{matrix} 42.00 & \text{in} \\ 3.50 & \text{ft} \end{matrix}$$

Pipe diameter

$$Y_n = 2.42 \text{ ft}$$

Normal depth of flow at culvert outlet

$$V = 11.6 \text{ ft/s}$$

Flow velocity at culvert outlet

$$FR = \begin{matrix} 1.31 \\ Y \end{matrix}$$

Froude Number

Supercritical flow?

$$Y_t = 0.73 \text{ ft}$$

Tailwater depth

$$D_a = 2.96$$

D parameter for supercritical flow

$$A_t = Q/V \text{ ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 78.8 \text{ cfs}$$

100-year discharge

$$V = 5 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 15.76 \text{ ft}^2$$

$$Q/D^{1.5} = 12.03$$

Froude Parameter (for Rock Sizing figure 9-38)

$$Y_t/D = 0.21$$

Tailwater Depth/Conduit Height

$$Y_t/D_a = 0.25$$

(use 0.40 if unknown or hydraulic jump
 is suspected downstream)

Use Type H Riprap per Figure 9-38	$D_{50} = 18 \text{ in}$
-----------------------------------	--------------------------

See Figure 9-38

Thickness of Riprap Layer = $2D_{50} = 3.0 \text{ ft}$
--

See Figure 9-34

$$Q/D^{2.5} = 5.23$$

Froude Parameter (for Extent of Protection figure 9-35)
 (must be ≤ 6.0)

$$1/2 \tan \theta = 1$$

(F_E) Expansion factor; Figure 9-35

$L_p = 18 \text{ ft}$

See Figure 9-34

$$T = 2(L_p \tan \theta) + W$$

Width of riprap at furthest downstream point; Equation 9-13

$$\theta = \tan^{-1} [1/(2 \cdot F_E)]$$

Equation 9-13

$$\theta = 0.46^\circ$$

$T = 22 \text{ ft}$

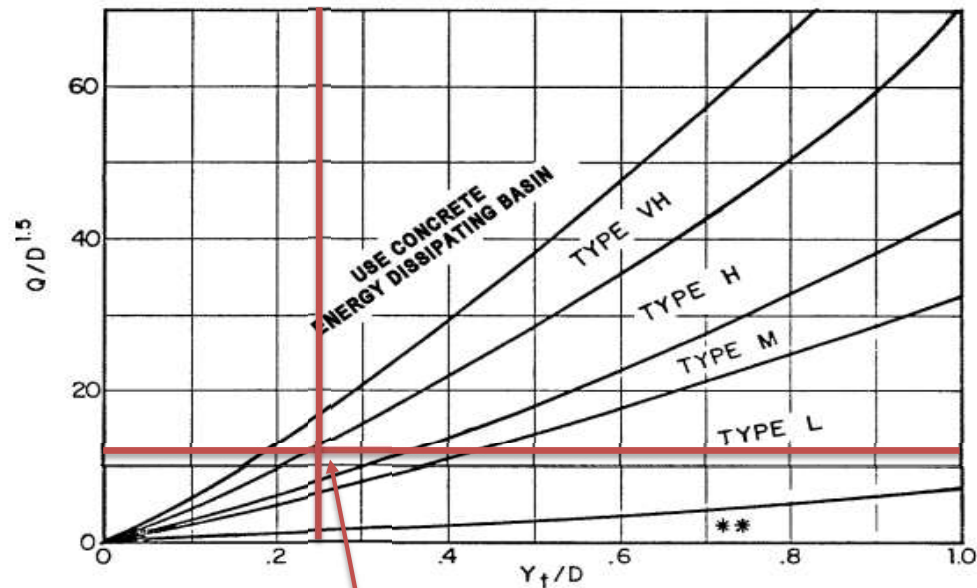
See Figure 9-34

CULVERT 12 - OUTLET RIPRAP CALCULATIONS

SHEET 2 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.3



Use D_d instead of D whenever flow is supercritical in the barrel.
 ** Use Type L for a distance of $3D$ downstream.

USE TYPE H ($D_{50}=18"$)

Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)

RIPRAP DESIGNATION	THICKNESS REQUIREMENTS FOR GRANULAR BEDDING		
	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

CULVERT 12 - OUTLET RIPRAP CALCULATIONS

SHEET 3 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

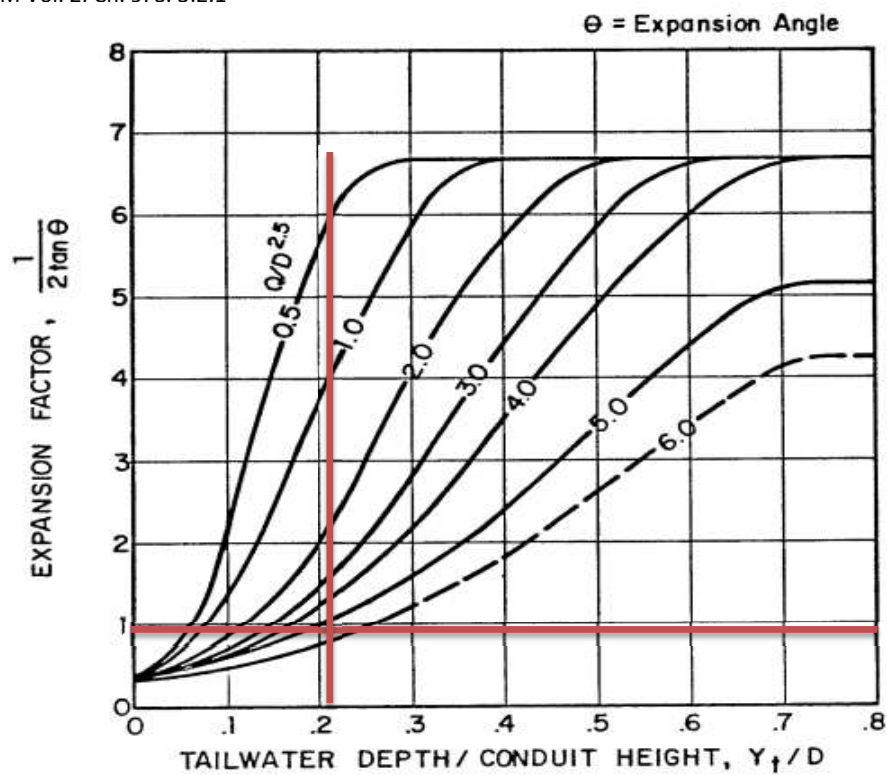


Figure 9-35. Expansion factor for circular conduits

CULVERT 12 - OUTLET RIPRAP CALCULATIONS

SHEET 4 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

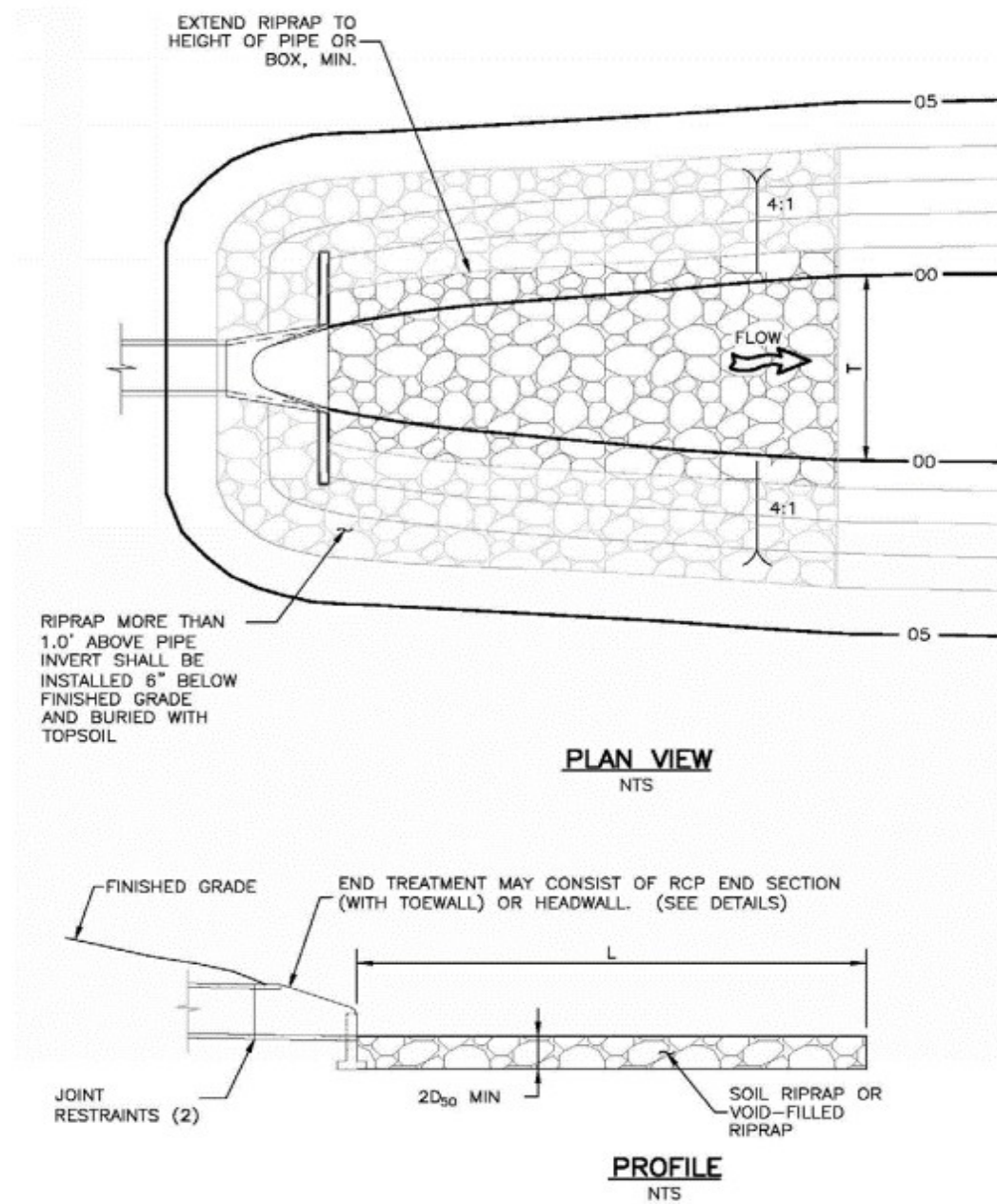


Figure 9-34. Riprap apron detail for culverts in-line with the channel

CULVERT 12 - OUTLET RIPRAP CALCULATIONS

SHEET 5 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1.2

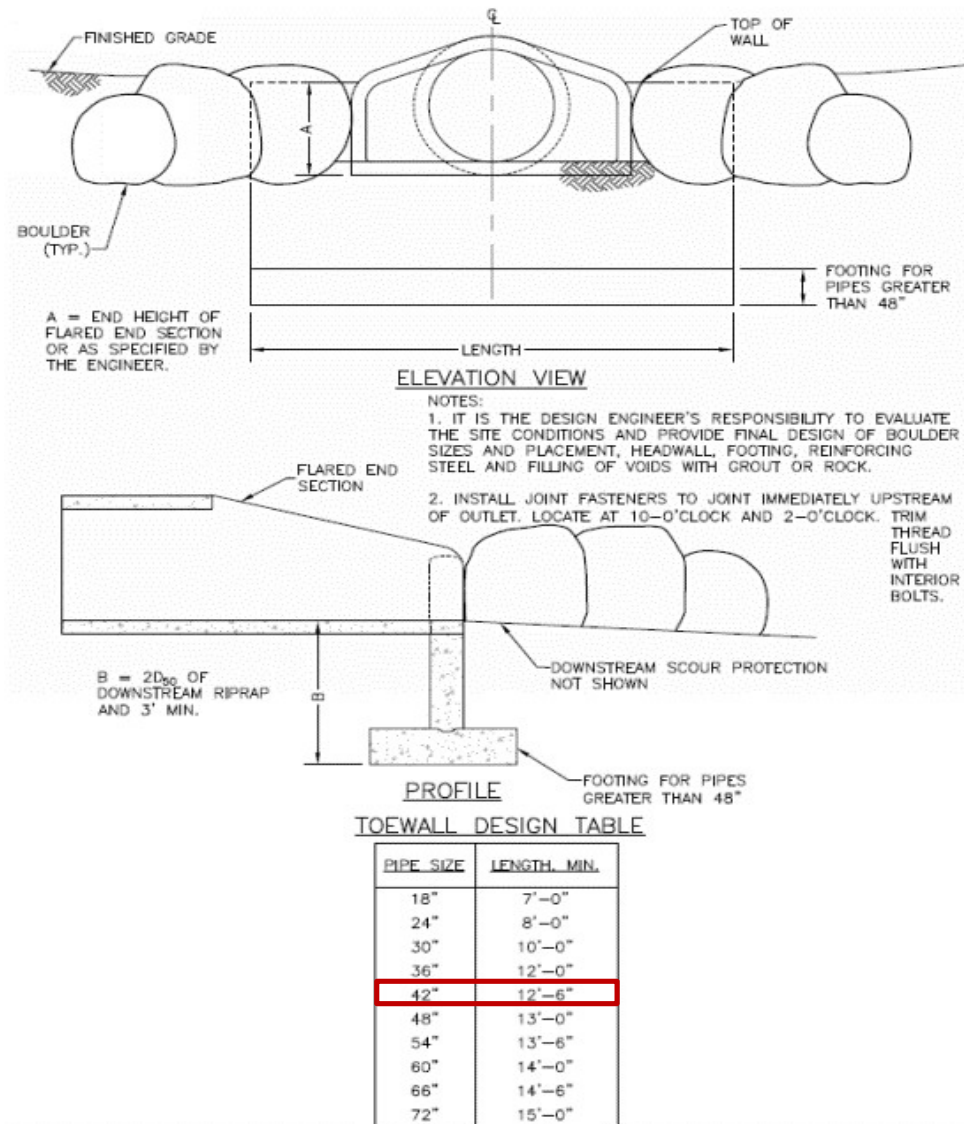


Figure 9-30. Flared end section (FES) headwall concept

Toe wall Dimensions	
Length =	12.5 ft
Depth (B) =	3.0 ft
Footing Required =	N

CULVERT 13 - OUTLET RIPRAP CALCULATIONS

SHEET 1 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1 & 3.2

Riprap Sizing per Section 3.2.3

$$L_p = (1/2 \tan \theta) [(A_t/Y_t) - W]$$

Length of protection; Equation 9-11

$$D = \begin{matrix} 36.00 & \text{in} \\ 3.00 & \text{ft} \end{matrix}$$

Pipe diameter

$$Y_n = 1.44 \text{ ft}$$

Normal depth of flow at culvert outlet

$$V = 15.9 \text{ ft/s}$$

Flow velocity at culvert outlet

$$FR = 2.33$$

Froude Number

$$Y$$

Supercritical flow?

$$Y_t = 0.73 \text{ ft}$$

Tailwater depth

$$D_a = 2.22$$

D parameter for supercritical flow

$$A_t = Q/V \quad \text{ft}^2$$

Required area of flow at allowable velocity; Equation 9-12

$$Q = 40.1 \text{ cfs}$$

100-year discharge

$$V = 5 \text{ ft/sec}$$

Max velocity allowed by UDFCD for non-cohesive soils

$$A_t = 8.02 \text{ ft}^2$$

$$Q/D^{1.5} = 7.72$$

Froude Parameter (for Rock Sizing figure 9-38)

$$Y_t/D = 0.24$$

Tailwater Depth/Conduit Height

$$Y_t/D_a = 0.33$$

(use 0.40 if unknown or hydraulic jump is suspected downstream)

Use Type L Riprap per Figure 9-38	$D_{50} = 9 \text{ in}$
-----------------------------------	-------------------------

See Figure 9-38

Thickness of Riprap Layer = $2D_{50} = 1.5 \text{ ft}$
--

See Figure 9-34

$$Q/D^{2.5} = 5.46$$

Froude Parameter (for Extent of Protection figure 9-35)
(must be ≤ 6.0)

$$1/2 \tan \theta = 1.1$$

 (F_E) Expansion factor; Figure 9-35

$L_p = 9 \text{ ft}$

See Figure 9-34

$$T = 2(L_p \tan \theta) + W$$

Width of riprap at furthest downstream point; Equation 9-13

$$\theta = \tan^{-1} [1/(2 * F_E)]$$

Equation 9-13

$$\theta = 0.43^\circ$$

$T = 11 \text{ ft}$

See Figure 9-34

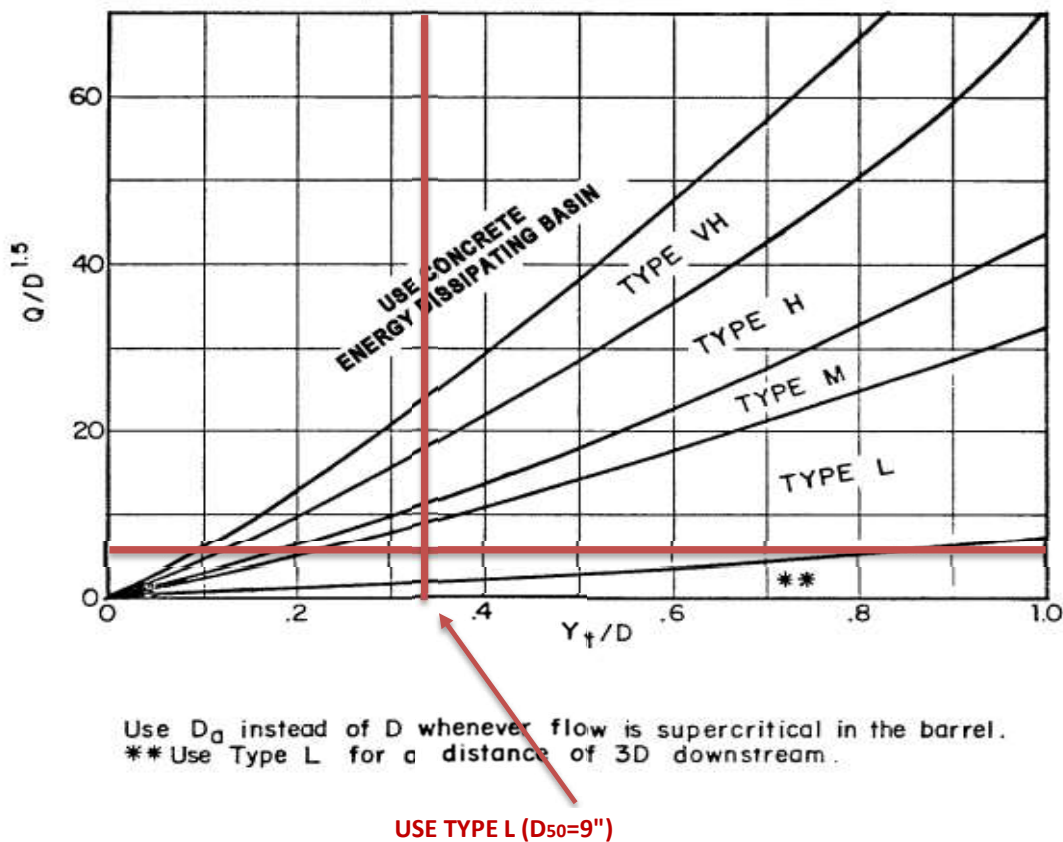


Figure 9-38. Riprap erosion protection at circular conduit outlet (valid for $Q/D^{2.5} \leq 6.0$)

THICKNESS REQUIREMENTS FOR GRANULAR BEDDING			
RIPRAP DESIGNATION	MINIMUM BEDDING THICKNESS (INCHES)		
	FINE-GRAINED SOILS ¹		COARSE-GRAINED SOILS ²
	TYPE I (LOWER LAYER)	TYPE II (UPPER LAYER)	TYPE II
VL ($D_{50} = 6$ IN)	4	4	6
L ($D_{50} = 9$ IN)	4	4	6
M ($D_{50} = 12$ IN)	4	4	6
H ($D_{50} = 18$ IN)	4	6	8
VH ($D_{50} = 24$ IN)	4	6	8

NOTES:

1. MAY SUBSTITUTE ONE 12-INCH LAYER OF TYPE II BEDDING. THE SUBSTITUTION OF ONE LAYER OF TYPE II BEDDING SHALL NOT BE PERMITTED AT DROP STRUCTURES. THE USE OF A COMBINATION OF FILTER FABRIC AND TYPE II BEDDING AT DROP STRUCTURES IS ACCEPTABLE.
2. FIFTY PERCENT OR MORE BY WEIGHT RETAINED ON THE #40 SIEVE.

Figure 8-34. Riprap and soil riprap placement and gradation (part 3 of 3)

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

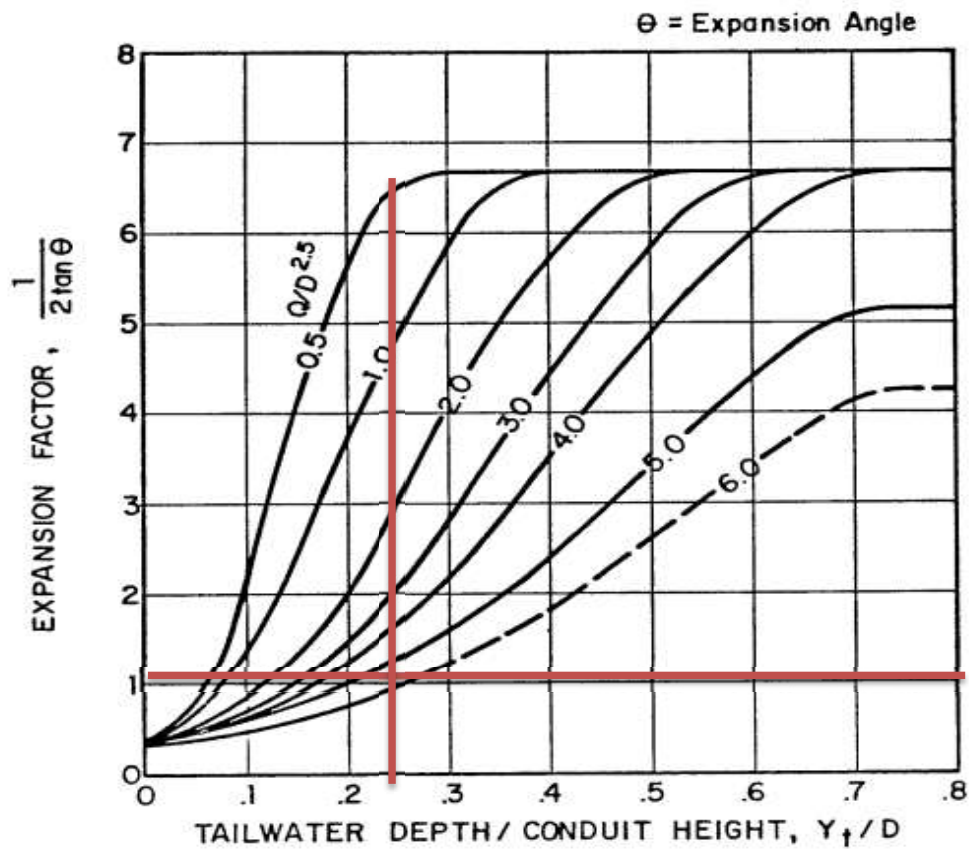


Figure 9-35. Expansion factor for circular conduits

CULVERT 13 - OUTLET RIPRAP CALCULATIONS

SHEET 4 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.2.1

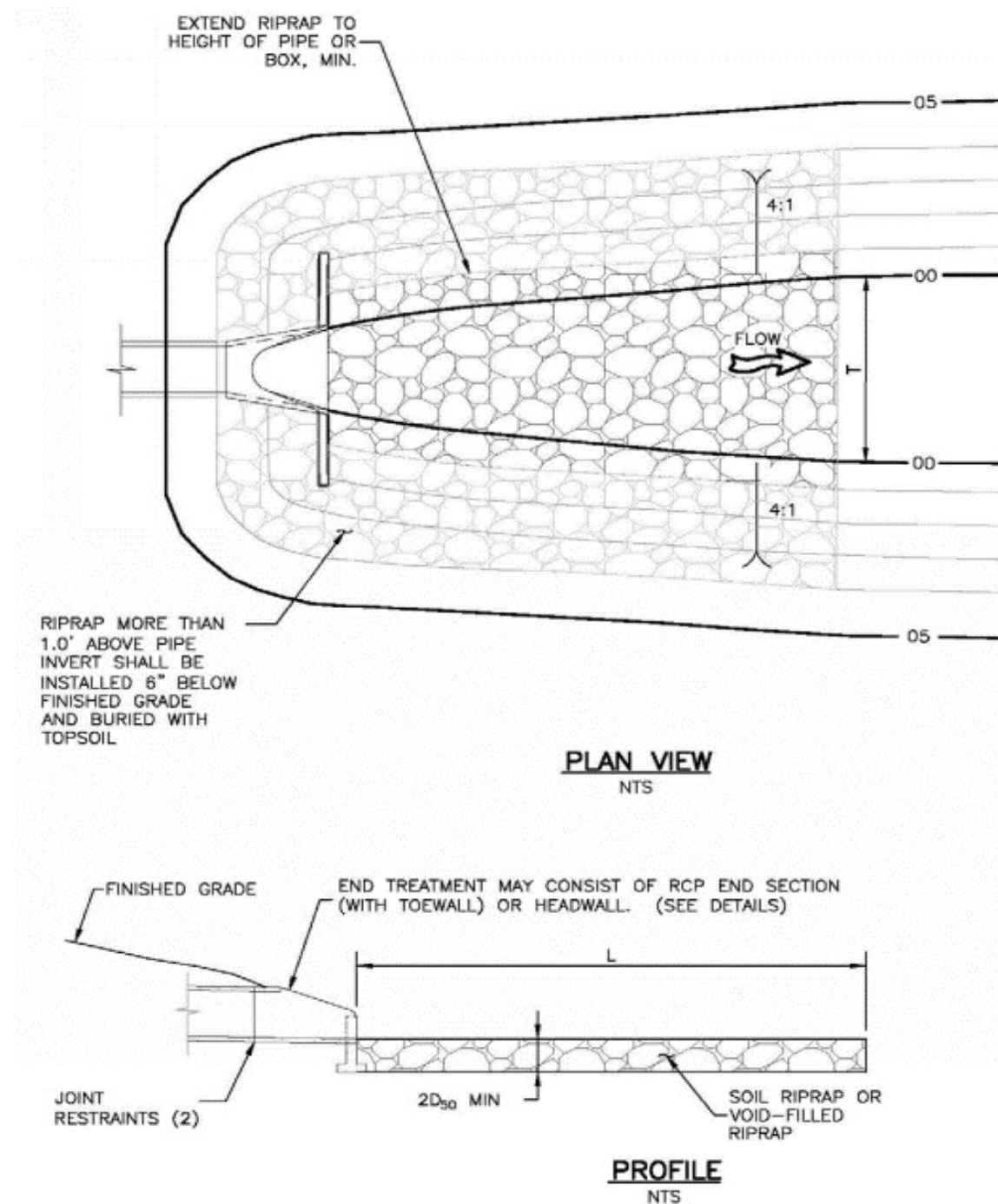


Figure 9-34. Riprap apron detail for culverts in-line with the channel

CULVERT 13 - OUTLET RIPRAP CALCULATIONS

SHEET 5 OF 5

Project: Dawson Trails Boulevard North

Per USDCM Vol. 2, Ch. 9, S. 3.1.2

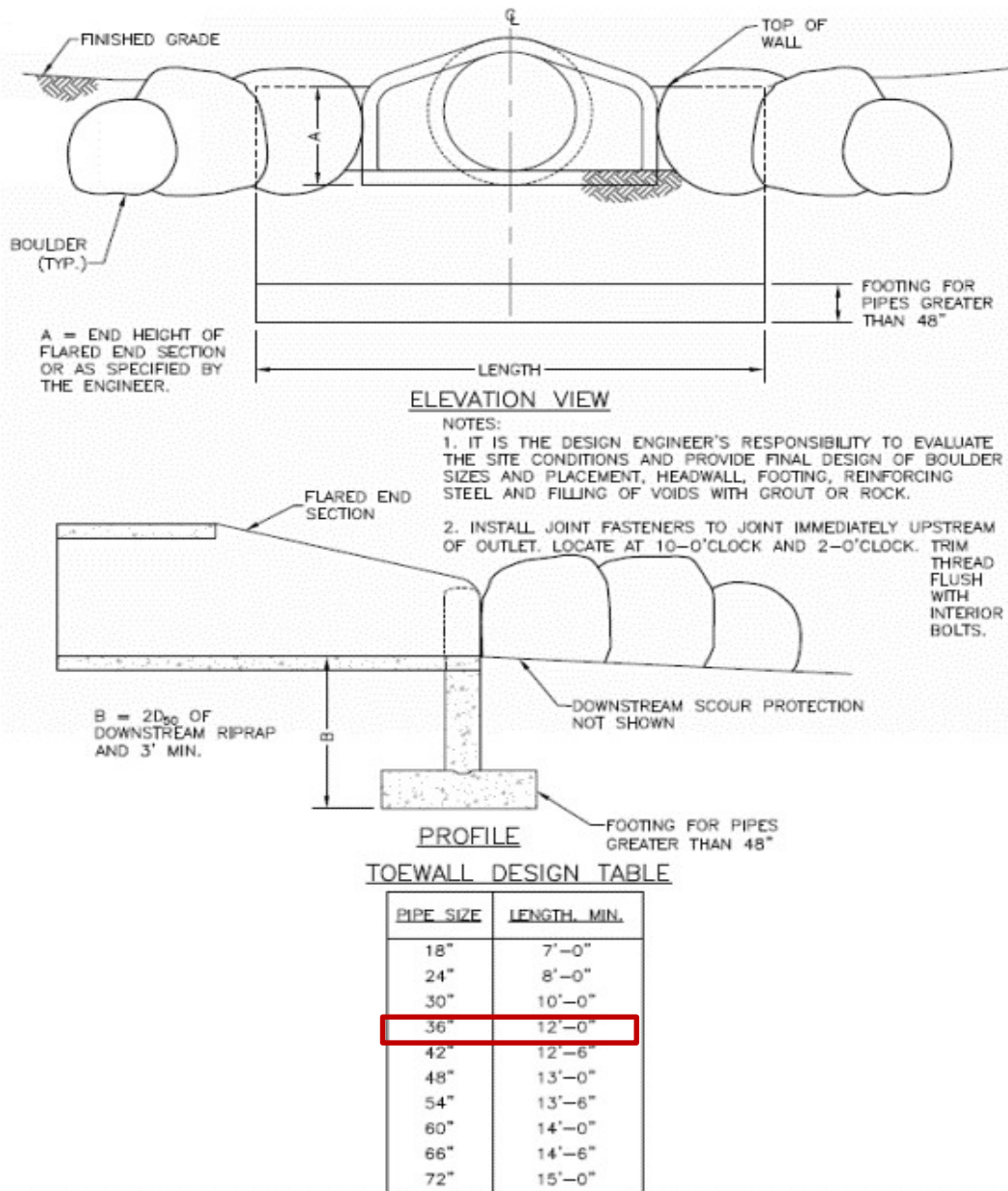


Figure 9-30. Flared end section (FES) headwall concept

Toe wall Dimensions		
Length =	12	ft
Depth (B) =	1.5	ft
Footing Required =	N	

Dawson Trails Boulevard North

Worksheet to Solve for Depth of Emergency Overflow

For Sump Inlets Using MHFD Formula for a Broad Crested Weir

MHFD USDCM Vol. 2, Sec. 5.14.2:

$$Q = C_{BCW} L H^{1.5}$$

Equation 12-8

Where:

Q = discharge (cfs)

C_{BCW} = broad-crested weir coefficient (This ranges from 2.6 to 3.0. A value of 3.0 is often used in practice.) See Hydraulic Engineering Circular No. 22 for additional information.

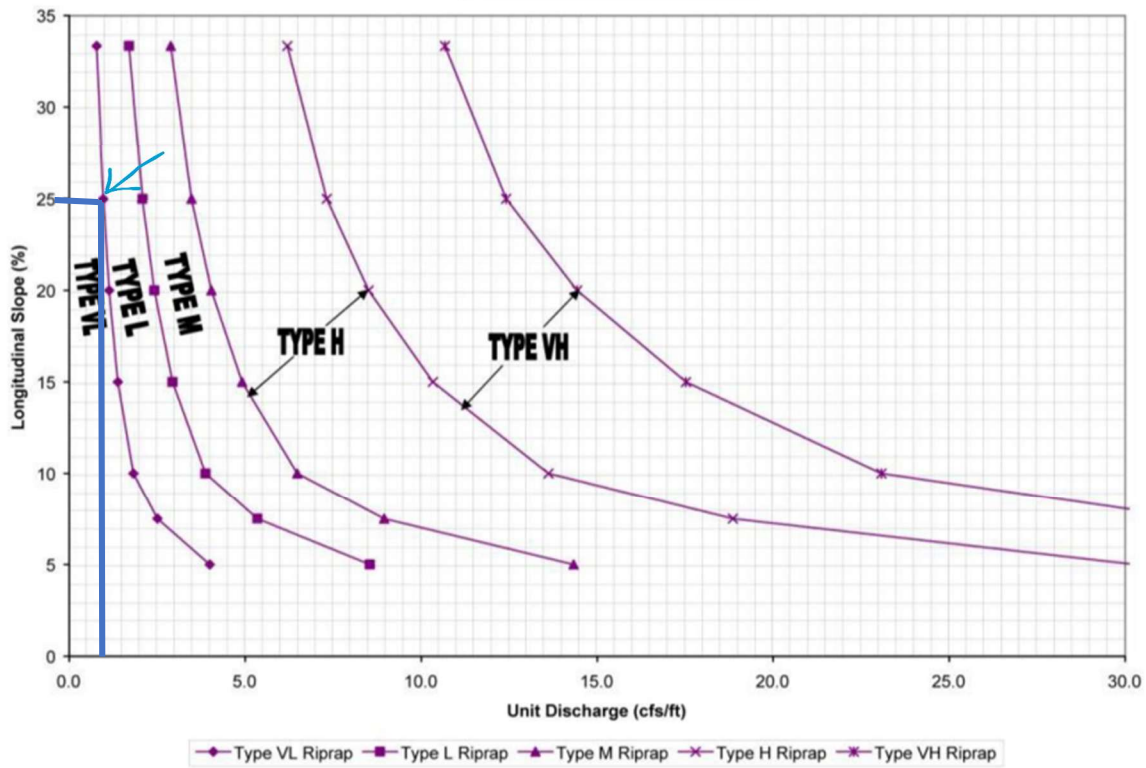
L = broad-crested weir length (ft)

H = head above weir crest (ft)

Sump Inlet	Q100 (cfs)	C	L (ft)	H (ft)
A1.1	12.8	3	100	0.12
A1.2	13.9	3	100	0.13
A2.1	6.2	3	100	0.08
A2.2	6.2	3	100	0.08
A2.3	10.1	3	100	0.10
A2.4	10.2	3	100	0.10
A4.1	21.4	3	100	0.17

EMERGENCY OVERFLOW RIPRAP PROTECTION

SPILLWAY CHANNEL AT CREST AND DOWNSTREAM SIDE OF EMBANKMENT



This table will be used to size the emergency overflow riprap protection. Emergency overflows will overtop the right of way, and will flow over a broad crested weir to an embankment, this situation is very similar hydraulically to a spillway with an embankment.

The largest Developed Q100 for an inlet observed on site = 21.4 cfs

Spillway Crest Length = 25 ft

Maximum Unit Discharge = 0.9 cfs/ft

Use **Type VL Riprap** protection behind all sump inlets on site

APPENDIX C

WATER QUALITY ENHANCEMENT CONTROL MEASURES

APPENDIX D

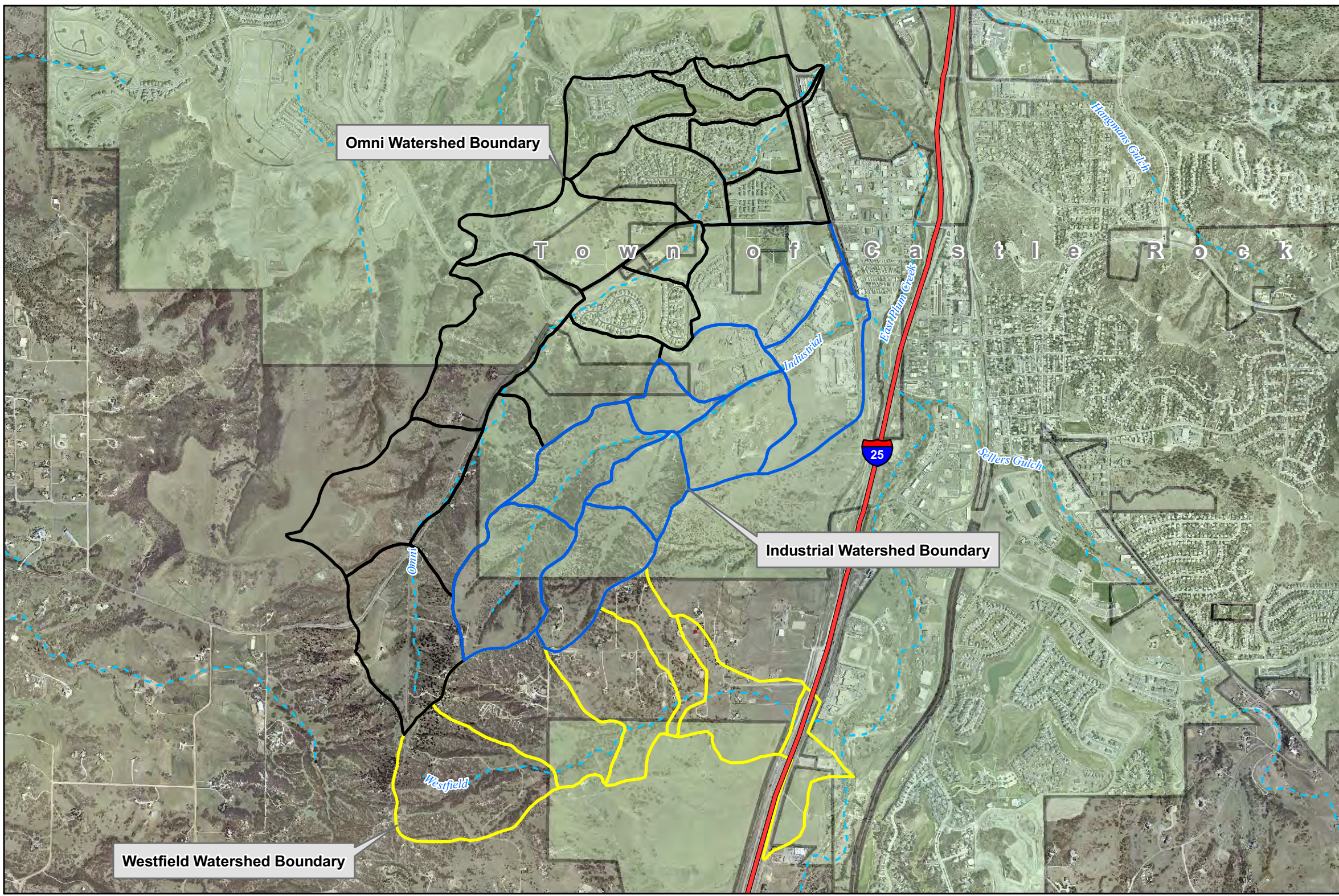
REFERENCE EXCERPTS

OMNI, INDUSTRIAL AND WESTFIELD WATERSHEDS FLOOD HAZARD AREA DELINEATION APRIL 2013

PREPARED FOR:



PREPARED BY:



Reach 5 - Station 77+35 to Station 104+97

In this reach, a private drive with no outlet structure causes a backwater effect and the floodplain encroaches onto private property near Station 81+80. The floodplain does not inundate any buildings on this property but does encroach closely to adjacent structures. A similar situation exists at Station 79+03 where a private driveway causes a backwater effect; however, no private structures are impacted. Further upstream, the floodplain is not entirely contained within the channel banks, but no private structures are impacted.

Reach 6 - Station 104+97 to Station 149+98

In the upper reach of the Omni Tributary, flows inundate Wolfensberger Road before running in parallel roadside ditches along either side of the road. There is no culvert under Wolfensberger Road so the flow overtops the road. Further downstream, the flow from the north roadside ditch overtops Wolfensberger Road to the south roadside ditch. Downstream, two private structures are impacted by the floodplain. At Station 113+28, the floodplain comes close to, but does not affect any buildings. Upstream of the intersection of Wolfensberger Road and Hier Lane, private driveways with undersized or no conveyance structures cause inadvertent detention in multiple locations.

Un-named Tributary to Omni

Reach 1 - Station 0+00 to Station 18+41

There are few flood hazards within Reach 1 of the un-named tributary. No private structures are impacted by the floodplain; however, there are two bike crossings with culverts that are inundated during flows greater than the 10-year event. The channel has steep banks that are eroding and degrading with exposed utility crossings within the reach.

Reach 2 - Station 18+41 to Station 27+74

Reach 2 of the un-named tributary is similar to the first reach. No structures are impacted by the floodplain and the channel is incised with undercut banks. In general, the channel is 10 – 20 feet below adjacent grade and there are little to no flooding issues.

Industrial Tributary

Reach 1 - Station 0+00 to Station 25+22

There are few flood hazards in Reach 1 of the Industrial Tributary. The stacked 7-foot CMP below a 4-foot CMP beneath Prairie Hawk Drive are undersized for the 100-year future flows but are adequate for the existing 100-year flows. The 9-foot arch under the BNSF Railroad passes both the existing and future flows. The floodplain exhibits backwater effects directly upstream of Prairie Hawk Drive but does not affect any private property. The flow throughout the remainder of this reach is contained within the channel banks that are 10 – 20 feet below surrounding property.

Reach 2 - Station 25+22 to Station 40+76

Within Reach 2, there are no culverts or bridges. Between Station 31+17 and Station 33+58, the floodplain extends outside of the channel banks to the north and enters private industrial property. This is likely caused by filling in the channel thalweg. The floodplain for the remainder of this reach is contained within the channel banks.

Reach 3 - Station 40+76 to Station 50+71

Within this reach, shallow overland flow occurs with multiple split flow conditions. The stock pond at the top of this reach releases flows through a spillway channel while flows overtopping the pond enter the historic channel. Flows in the spillway channel that do not drain to the historic channel enter private industrial property to the north near Station 44+59. The floodplain continues in private property on the spillway channel until Station 40+76. No structures are impacted by the shallow overland flow.

Reach 4 - Station 50+71 to Station 75+42

In this reach, there are no flooding or infrastructure issues. The flow is contained within the channel banks and no private properties are impacted by the floodplain.

Reach 5 - Station 75+42 to Station 103+65

In this reach, there are no flooding or infrastructure hazards. The flow is contained within the channel banks and no private properties are impacted by the floodplain.

Un-named Tributary to Industrial

Reach 1 - Station 0+00 to Station 29+47

In this reach, there are no flooding or infrastructure issues. The flow is contained within the channel banks and no private properties are impacted by the floodplain.

Westfield Tributary

Reach 1 - Station 0+00 to Station 35+50

In the downstream reach of the Westfield Tributary, a split flow condition exists with a portion of the flow running north along the railroad ditch to a 72-inch CMP under the BNSF Railroad. The other portion of flow runs south along the railroad ditch to a 36-inch CMP outfall under the BNSF Railroad. Neither one of these outfalls is sized for the full 100-year future nor the existing flows. In this reach, a 24-inch RCP under a private drive near Station 33+65 is undersized for all flows. The undersized culvert causes a backwater effect in the floodplain upstream. Within this reach, two properties are impacted by the 100-year floodplain. A house located near Station 33+00 on the north side of the Westfield Tributary is within the 100-year floodplain and another house, located on the south side of the Westfield Tributary near Station 33+60, is near the 100-year floodplain.

Reach 2 - Station 35+50 to Station 46+00

Within Reach 2, multiple culverts located at private driveways are undersized for the 10-year existing and future flows. The 24-inch CMP under a private drive at Station 35+50 is undersized for all flows while the 4 – 52-inch by 36-inch elliptical RCPs under a private drive near Station 40+50 are undersized for the 100-year existing and future flows. Upstream of both undersized culverts, backwater effects are seen in the floodplain. Although this reach is severely degraded and the floodplain is not entirely contained within the channel banks, no properties are at risk from flooding.

Reach 3 - Station 46+00 to Station 87+50

Within the lower parts of Reach 3, the floodplain is not contained within the channel banks, but there are no properties that are at risk from flooding. In the upper portions of this reach, the floodplain is contained within the channel banks and no properties are at risk. **Reach 4 - Station 87+50 to Station 95+75**

Similar to Reach 3, the floodplain within this reach is entirely contained within the poorly defined channel banks. No properties are at risk from flooding within this reach.

Previous Analyses

Previous studies within the Study Area have been limited to hydrologic and hydraulic analysis of small sections of the Omni, Industrial, and Westfield Watersheds. Federal Emergency Management Agency (FEMA) Letter of Map Revisions (LOMR) was accepted for the lower parts of the Omni and Industrial Tributaries. This will be the first full-length hydraulic analysis of the Omni, Industrial, and Westfield Tributaries.

Omni Tributary: Only the lower, developed portion of the Omni Tributary has been studied in detail. There are two detention ponds on the Omni Tributary as well as an engineered channel leading into each pond. Industrial Tributary: Only the lower, developed portion of the Industrial Tributary has been hydraulically studied in detail. The CDOT culvert under Park Street has a FEMA LOMR in the process of being finalized.

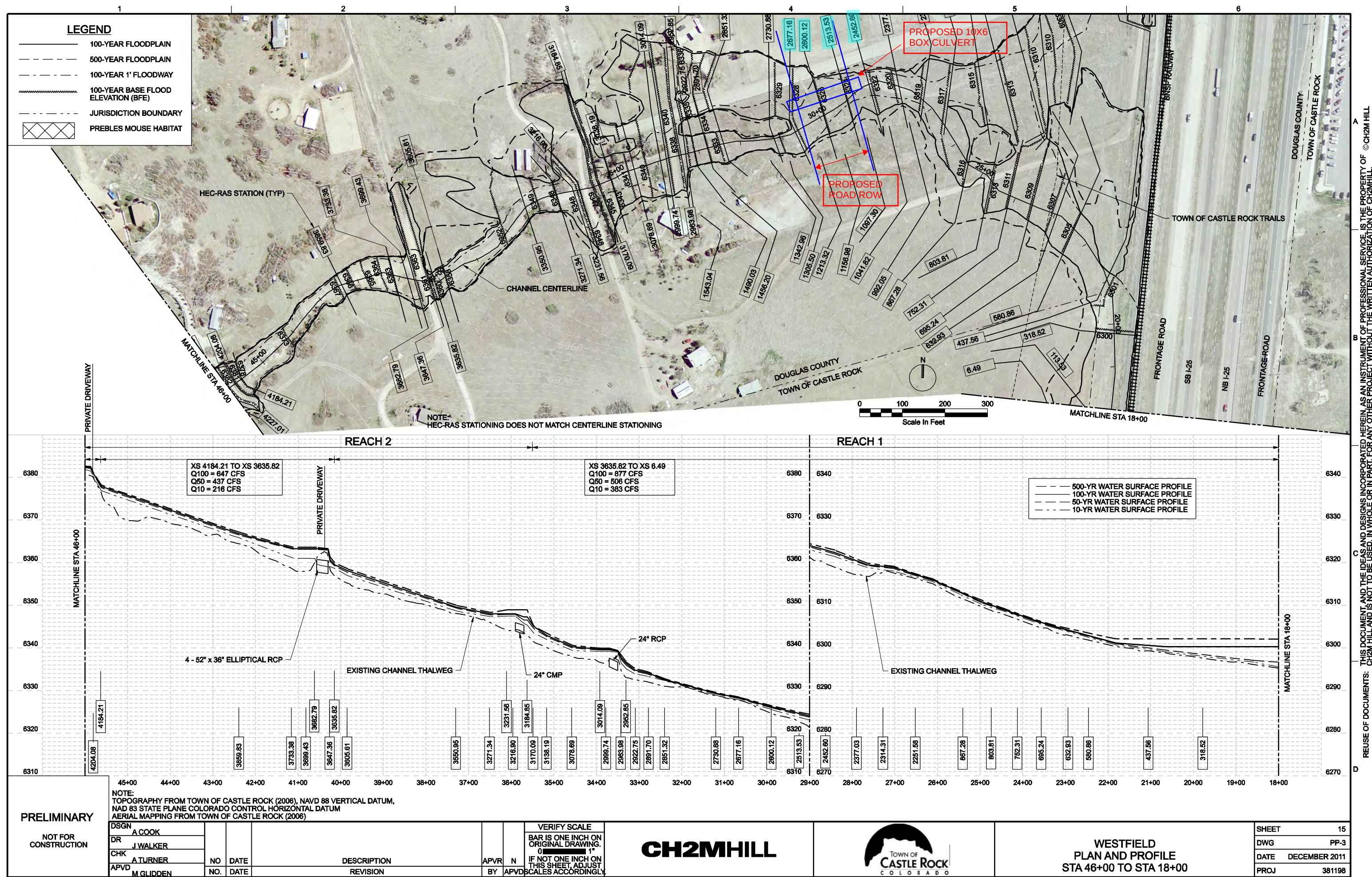
Westfield Tributary: The Westfield Tributary is currently being studied by Moser Engineering. The current plan for this study is to upsize the Westfield Outfall Tributary under I-25 to an 8’ x 6’ CBC.

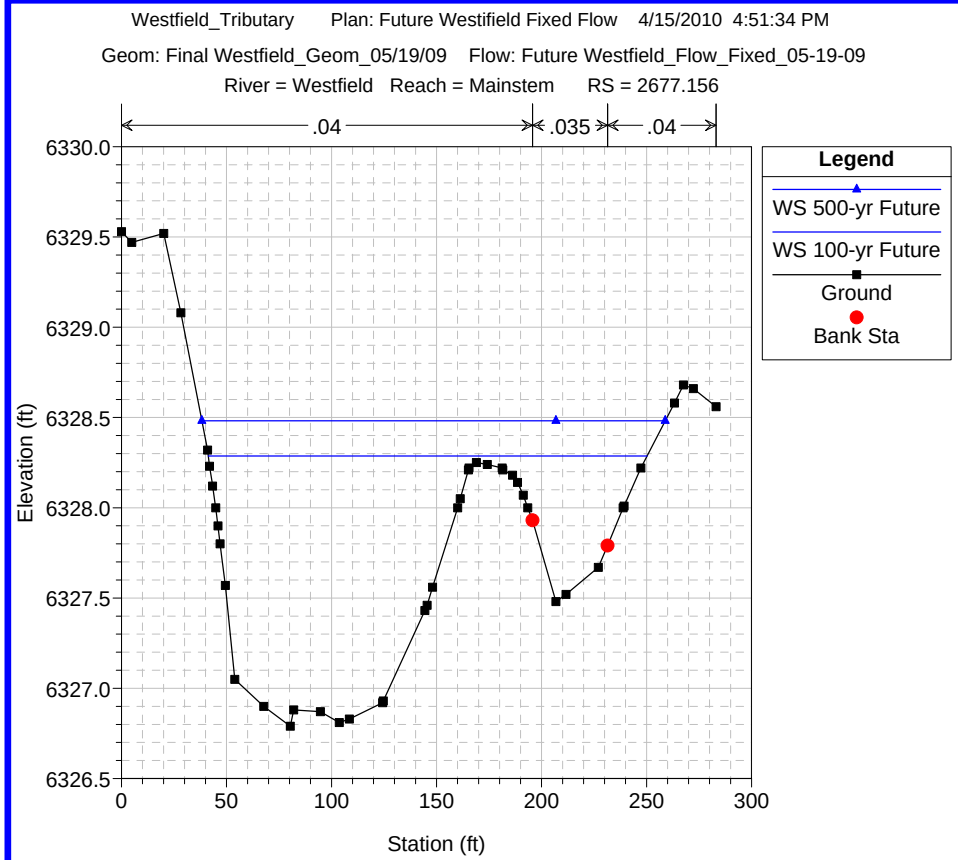
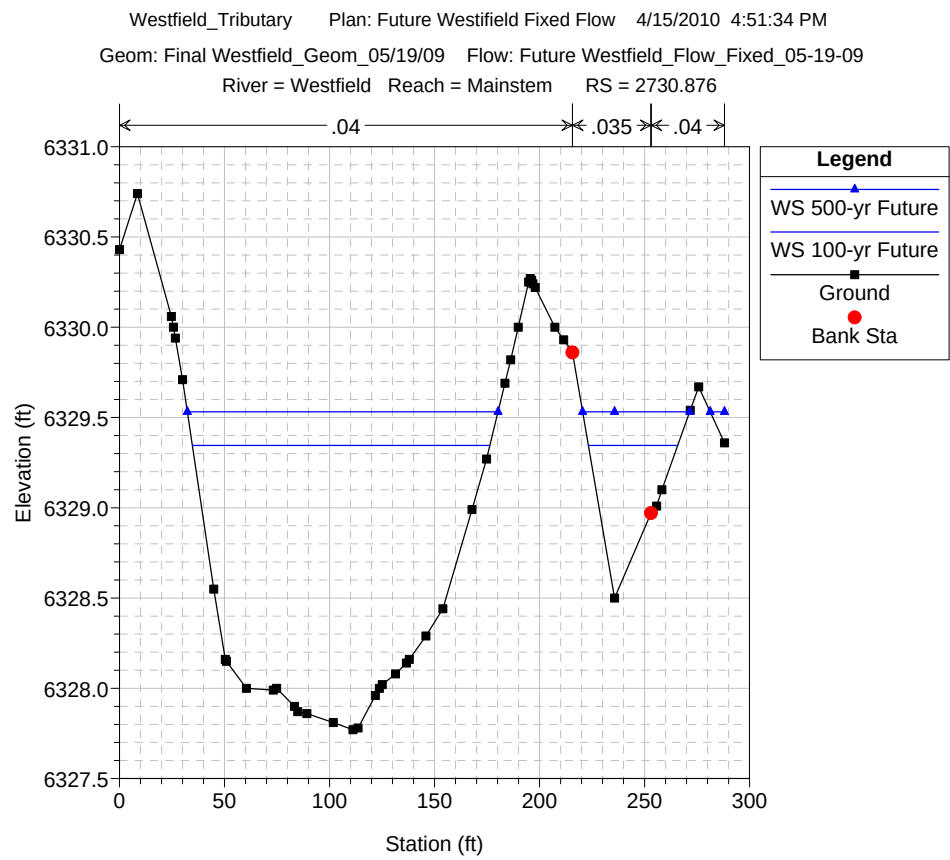
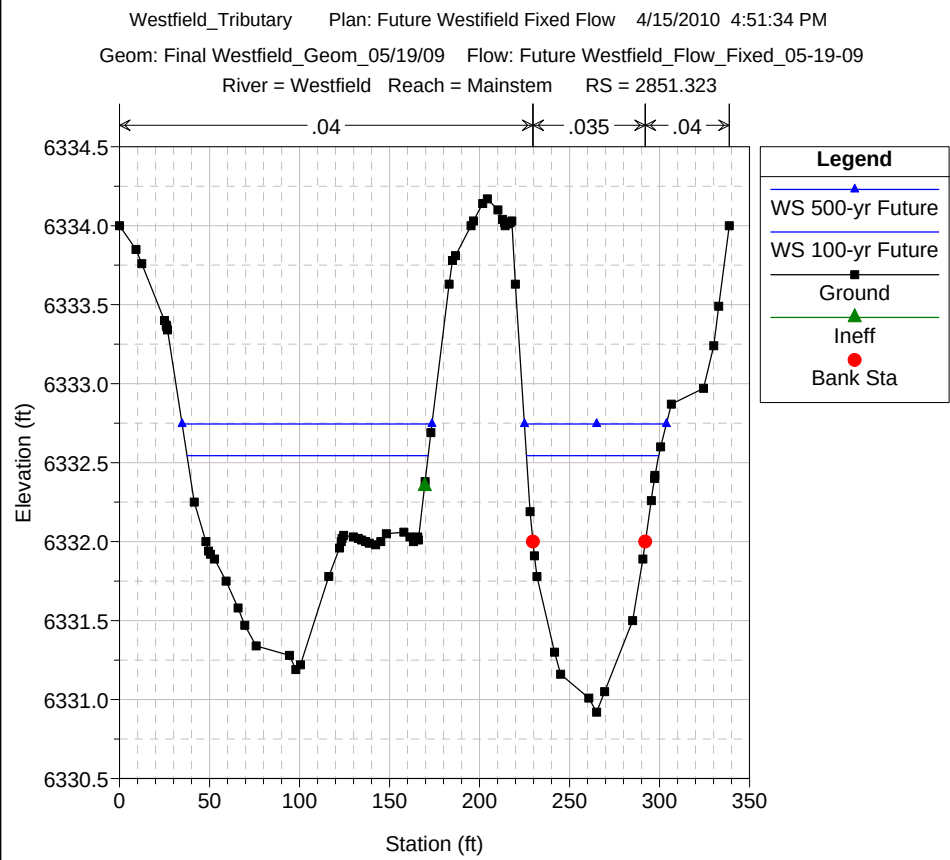
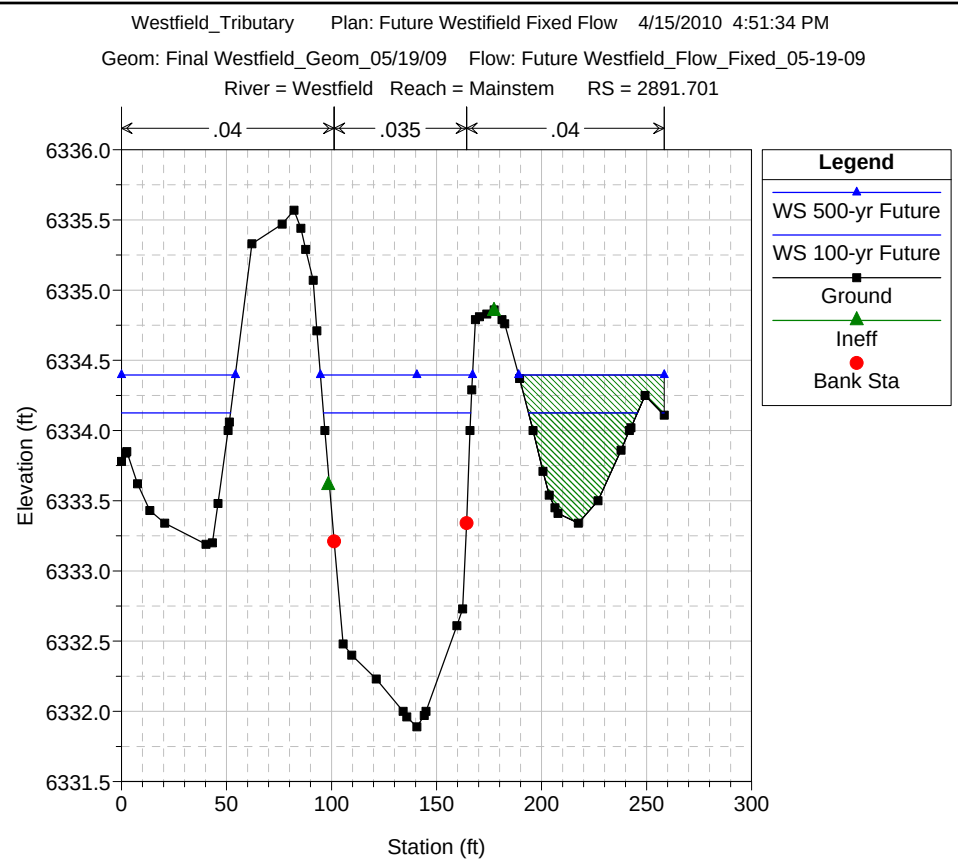
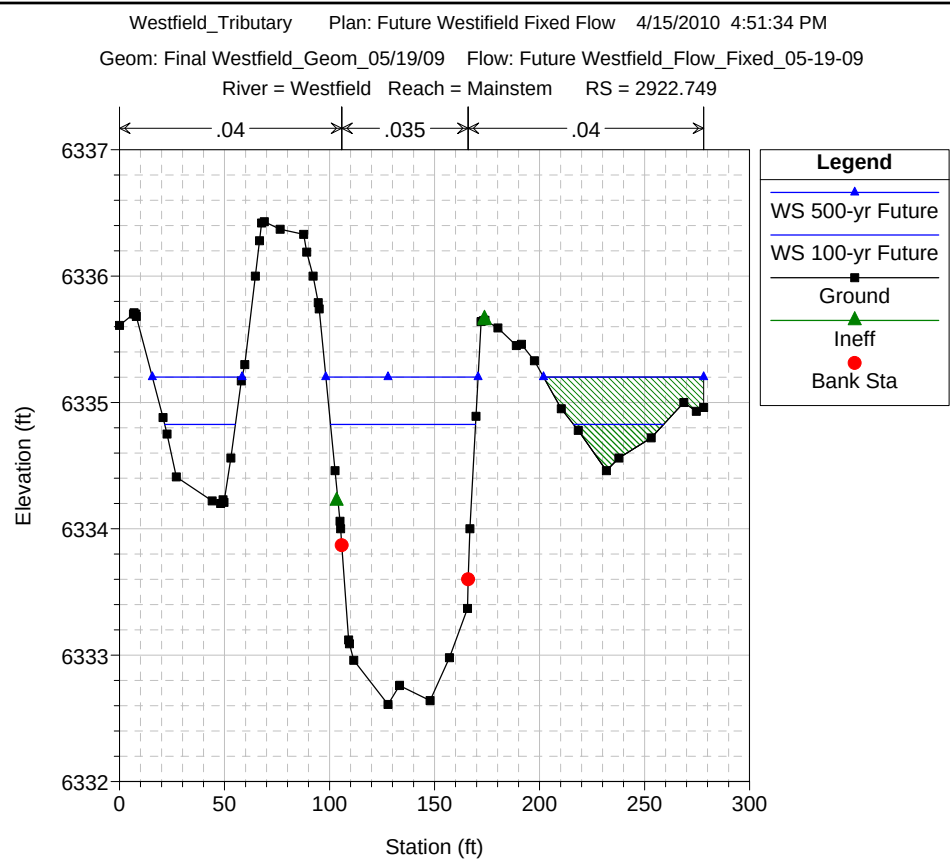
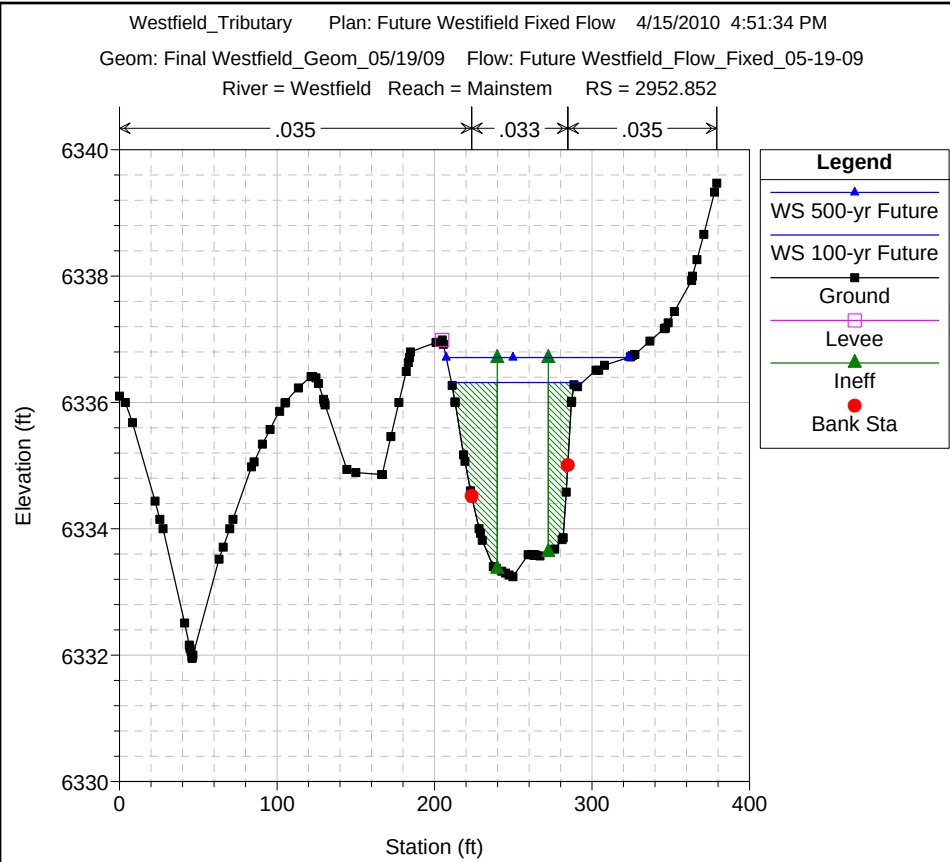
Floodplain Results

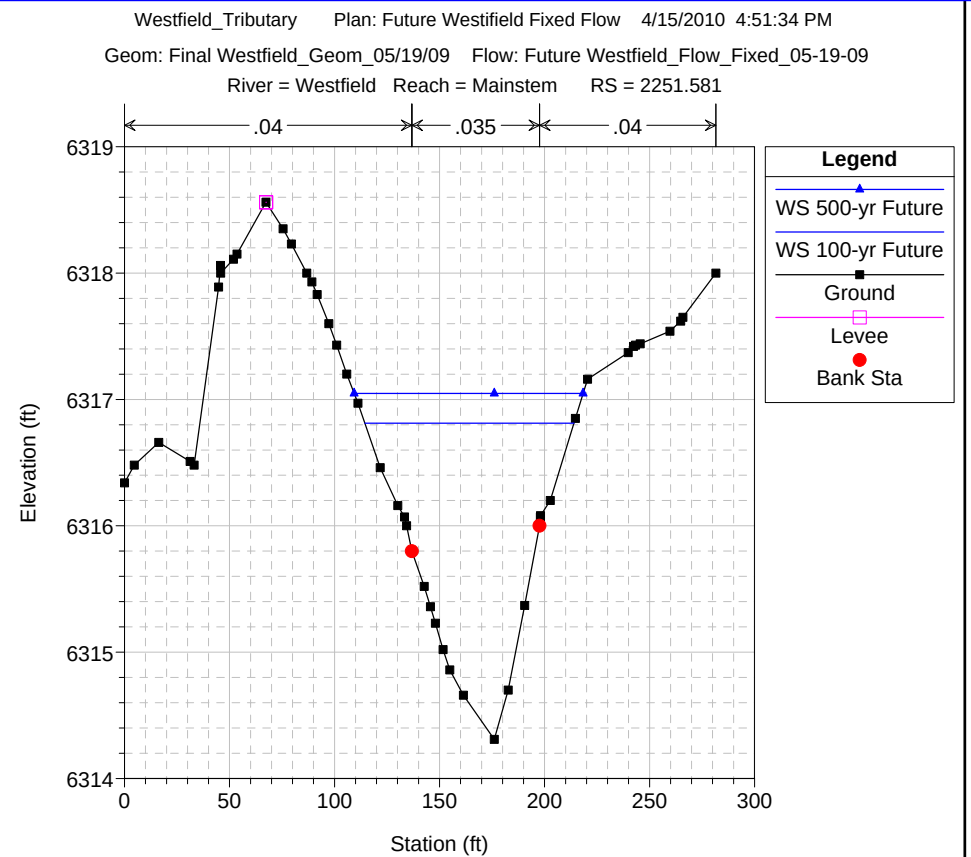
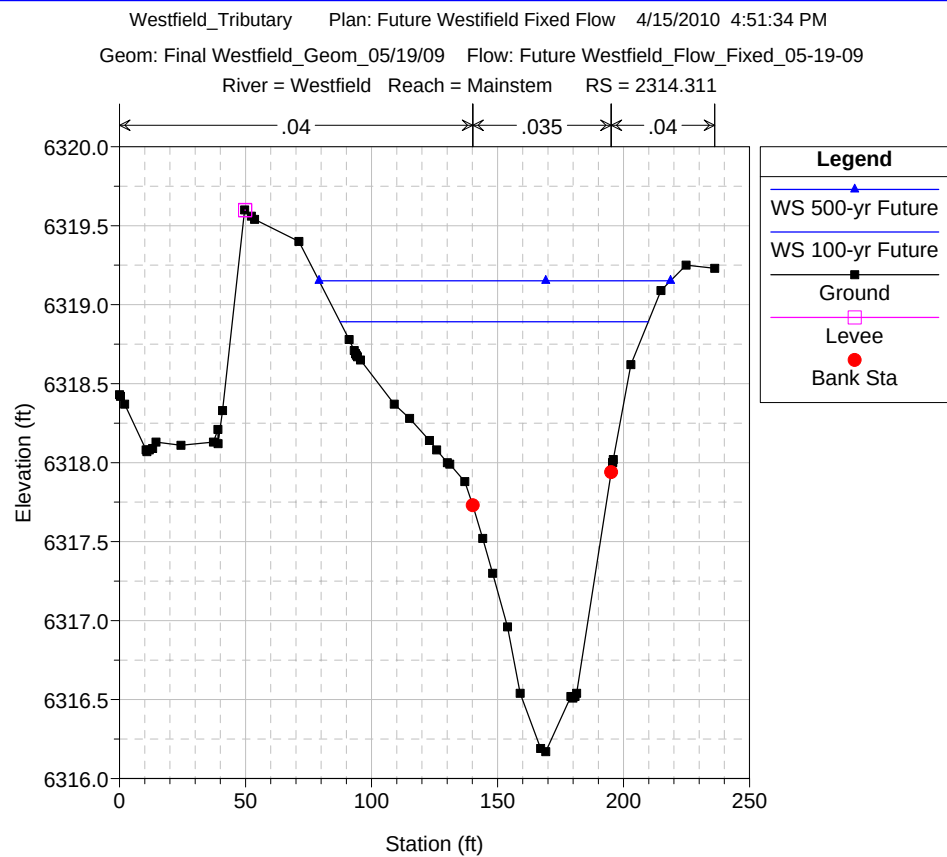
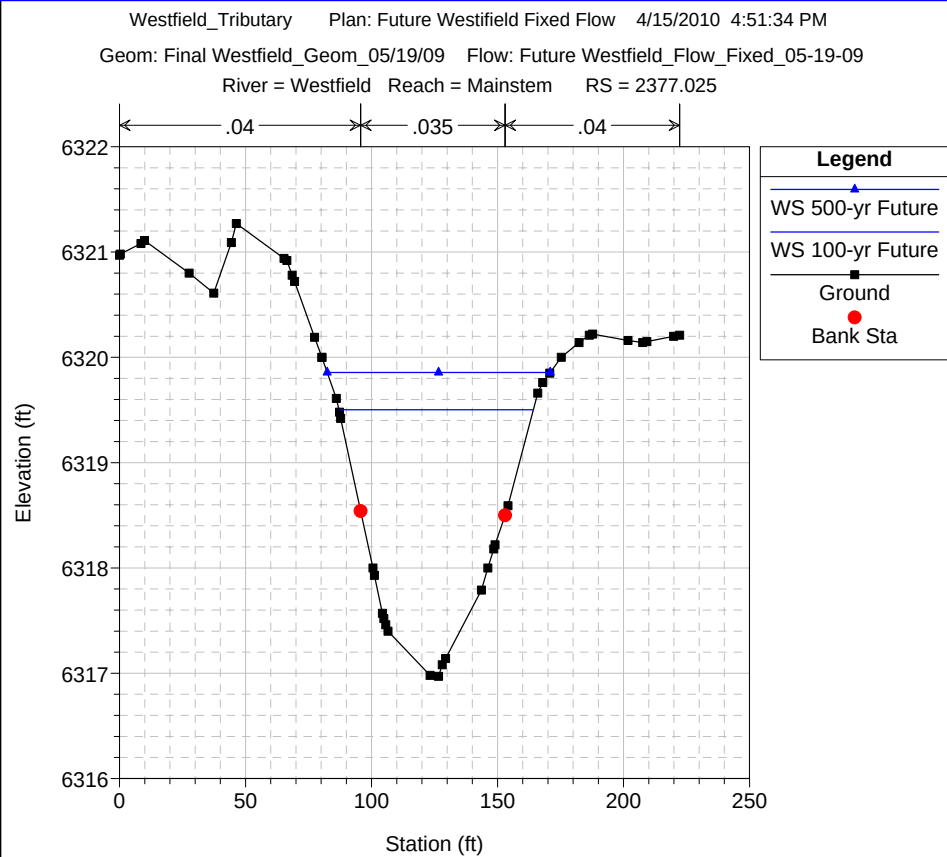
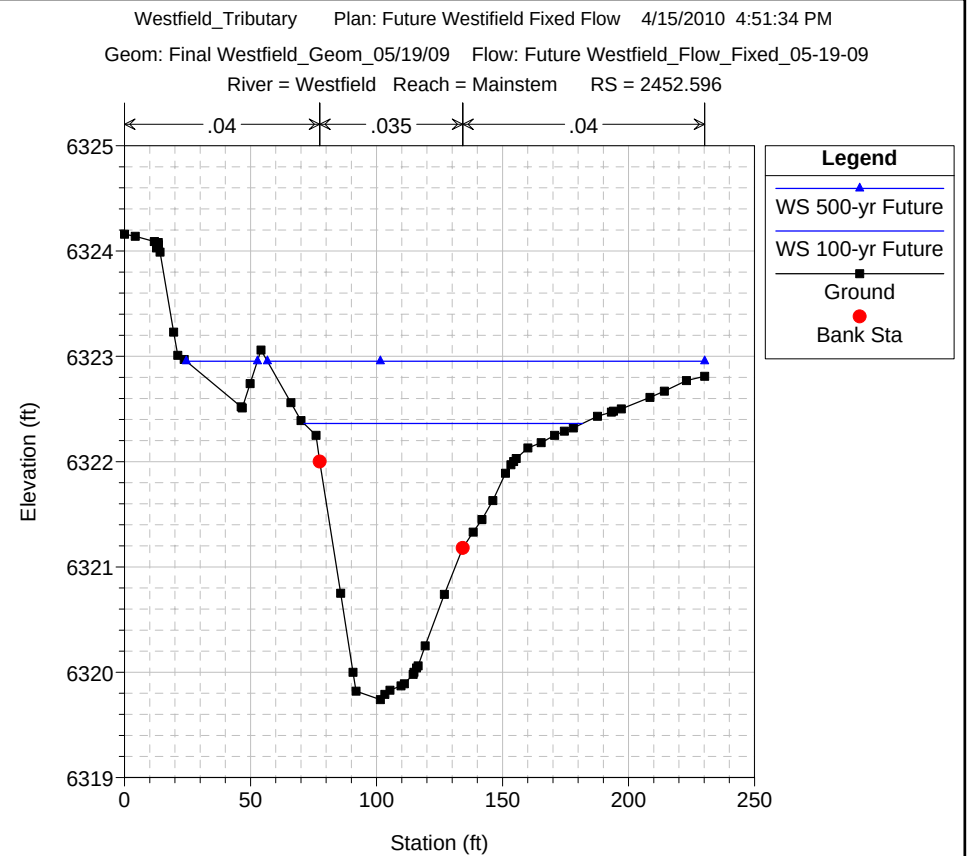
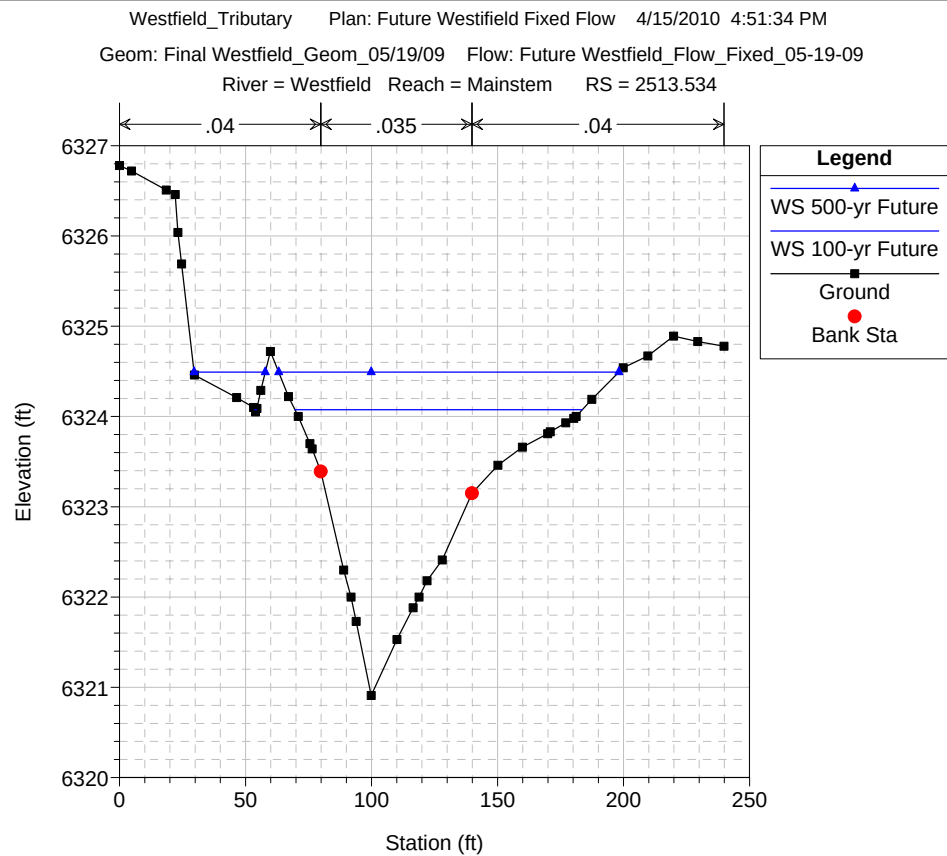
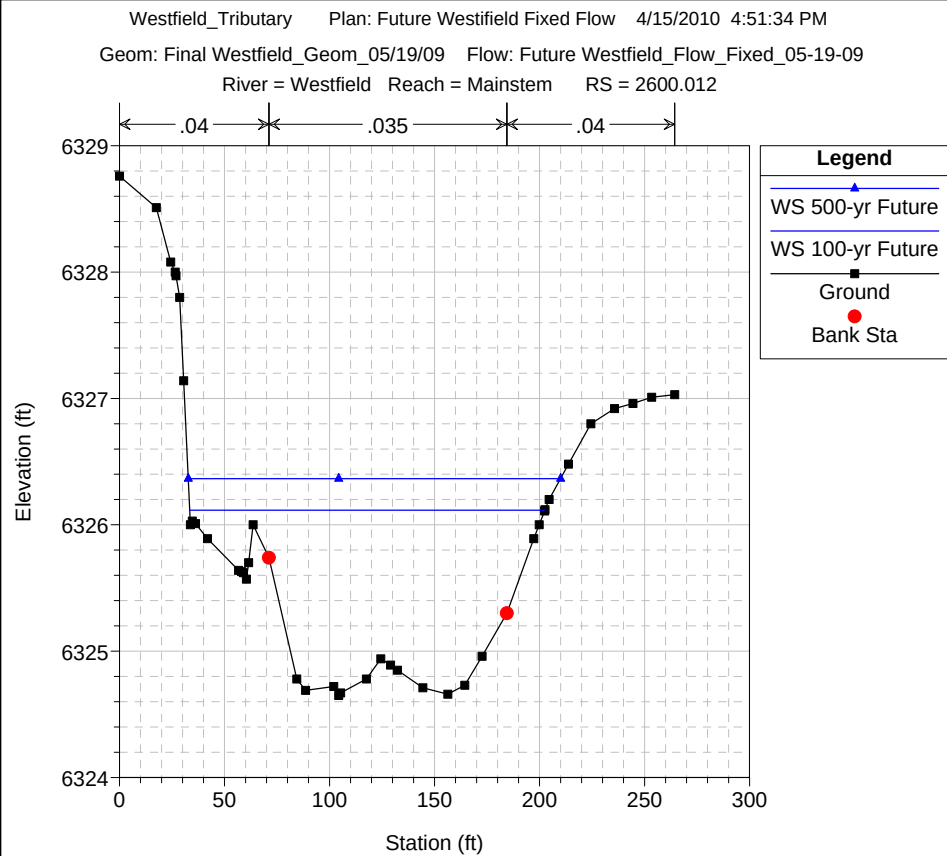
Tables IV-3 through IV-6 shows the floodplain and floodway results for the Omni, Industrial, and Westfield Tributaries. The floodplain and floodway can also be seen on the plan and profile figures presented in Appendix D.

TABLE IV-7
Westfield Floodplain/Floodway Table

				Peak Discharge				Water Surface Elevation				100-YR Floodplain		100-YR Floodway (FW) Data						
Tributary	Reach	River Station	Thalweg Elev. (feet)	Data				Data				Data		(1.0' Rise in WSE)						
				10-YR (cfs)	50-YR (cfs)	100-YR (cfs)	500-YR (cfs)	10-YR (ft)	50-YR (ft)	100-YR (ft)	500-YR (ft)	Top Width (ft)	Energy Grade Line (ft)	Floodway Discharge (cfs)	Floodway Elevation (ft)	Floodway Width (ft)	Dist. Left (ft)	Dist. Right (ft)	Avg. Vel. (ft/sec)	Area (sq-ft)
Westfield	Mainstem	4184	6375.5	216.0	510.0	647.0	885	6376.9	6377.6	6377.9	6378.3	86.1	6378.6	647.0	6377.9	60.0	36.4	23.7	6.8	95
Westfield	Mainstem	3860	6364.4	216.0	510.0	647.0	885	6365.8	6366.5	6366.7	6367.1	54.3	6367.6	647.0	6366.7	49.0	24.5	24.5	7.5	87
Westfield	Mainstem	3733	6358.3	216.0	510.0	647.0	885	6361.1	6363.1	6363.3	6363.6	93.8	6363.3	647.0	6363.5	78.1	39.1	39.1	2.0	324
Westfield	Mainstem	3699	6357.8	216.0	510.0	647.0	885	6361.1	6363.1	6363.3	6363.6	117.5	6363.3	647.0	6363.5	89.3	44.7	44.7	1.6	405
Westfield	Mainstem	3683	6357.6	216.0	510.0	647.0	885	6361.0	6363.1	6363.3	6363.6	236.4	6363.3	647.0	6363.5	120.0	75.0	45.0	1.4	479
Westfield	Mainstem	3662		Private Driveway Crossing																
Westfield	Mainstem	3647	6357.1	216.0	510.0	647.0	885	6359.1	6360.3	6360.7	6361.3	196.7	6360.7	647.0	6360.6	120.0	75.0	45.0	2.4	267
Westfield	Mainstem	3636	6357.0	363.0	707.0	877.0	1185	6358.6	6359.2	6359.5	6359.9	64.4	6360.4	877.0	6359.5	59.0	29.5	29.5	7.7	113
Westfield	Mainstem	3606	6355.2	363.0	707.0	877.0	1185	6356.9	6357.7	6358.1	6358.6	53.1	6359.2	877.0	6358.1	38.1	19.1	19.1	9.1	97
Westfield	Mainstem	3351	6347.1	363.0	707.0	877.0	1185	6348.7	6349.3	6349.6	6350.0	106.3	6349.9	877.0	6350.1	51.1	36.4	14.7	8.0	109
Westfield	Mainstem	3271	6345.5	363.0	707.0	877.0	1185	6347.3	6347.8	6348.0	6348.4	72.6	6348.9	877.0	6348.0	63.5	31.8	31.8	7.7	114
Westfield	Mainstem	3232	6344.3	363.0	707.0	877.0	1185	6347.5	6347.8	6348.0	6348.9	191.9	6348.1	877.0	6348.1	121.4	60.7	60.7	2.8	308
Westfield	Mainstem	3217	6344.2	363.0	707.0	877.0	1185	6347.5	6347.9	6348.0	6348.9	255.7	6348.1	877.0	6348.2	205.3	136.0	69.4	1.8	483
Westfield	Mainstem	3204		Private Driveway Crossing																
Westfield	Mainstem	3185	6342.8	363.0	707.0	877.0	1185	6345.3	6346.5	6347.3	6348.9	388.3	6347.3	877.0	6347.2	197.0	136.0	61.1	1.4	611
Westfield	Mainstem	3170	6341.6	363.0	707.0	877.0	1185	6343.6	6344.6	6345.1	6345.2	244.7	6346.8	877.0	6345.1	70.6	35.3	35.3	10.4	200
Westfield	Mainstem	3138	6340.2	363.0	707.0	877.0	1185	6341.9	6342.6	6342.9	6343.4	130.3	6344.0	877.0	6342.9	59.4	29.7	29.7	8.4	125
Westfield	Mainstem	3079	6338.0	363.0	707.0	877.0	1185	6339.4	6340.0	6340.3	6340.6	107.8	6341.0	877.0	6340.3	61.0	30.5	30.5	7.7	115
Westfield	Mainstem	3014	6336.7	363.0	707.0	877.0	1185	6339.2	6339.5	6339.7	6340.0	340.2	6339.7	877.0	6340.3	148.0	69.4	78.6	2.4	369
Westfield	Mainstem	3000	6336.3	363.0	707.0	877.0	1185	6339.2	6339.5	6339.7	6340.0	344.0	6339.7	877.0	6340.3	148.5	62.1	86.5	2.1	419
Westfield	Mainstem	2983		Private Driveway Crossing																
Westfield	Mainstem	2964	6334.0	363.0	707.0	877.0	1185	6336.4	6337.3	6337.6	6338.3	341.6	6339.0	877.0	6337.6	134.0	48.8	85.2	9.5	201
Westfield	Mainstem	2953	6333.2	363.0	707.0	877.0	1185	6335.1	6335.9	6336.3	6336.7	83.0	6337.7	877.0	6336.3	77.7	43.3	34.4	9.5	178
Westfield	Mainstem	2923	6332.6	363.0	707.0	877.0	1185	6333.9	6334.6	6334.8	6335.2	237.5	6335.6	877.0	6334.8	116.5	82.9	33.6	7.5	118
Westfield	Mainstem	2892	6331.9	363.0	707.0	877.0	1185	6333.3	6333.8	6334.1	6334.4	258.4	6334.7	877.0	6334.2	120.3	86.8	33.5	7.1	123
Westfield	Mainstem	2851	6330.9	363.0	707.0	877.0	1185	6332.3	6332.4	6332.5	6332.8	262.1	6333.0	877.0	6333.2	133.9	95.2	38.7	6.6	133
Westfield	Mainstem	2731	6328.5	363.0	707.0	877.0	1185	6328.8	6329.2	6329.4	6329.5	231.1	6329.7	877.0	6330.3	129.2	97.7	31.4	6.0	146
Westfield	Mainstem	2677	6327.5	363.0	707.0	877.0	1185	6327.8	6328.1	6328.3	6328.5	208.9	6328.7	877.0	6329.3	104.8	68.1	36.7	6.2	141
Westfield	Mainstem	2600	6324.7	363.0	707.0	877.0	1185	6325.5	6325.9	6326.1	6326.4	169.0	6326.6	877.0	6326.1	113.3	56.7	56.7	6.4	138
Westfield	Mainstem	2514	6320.9	363.0	707.0	877.0	1185	6323.1	6323.8	6324.1	6324.5	130.1	6324.8	877.0	6324.0	60.0	30.0	30.0	7.7	113







OMNI, INDUSTRIAL AND WESTFIELD WATERSHED MASTER PLAN

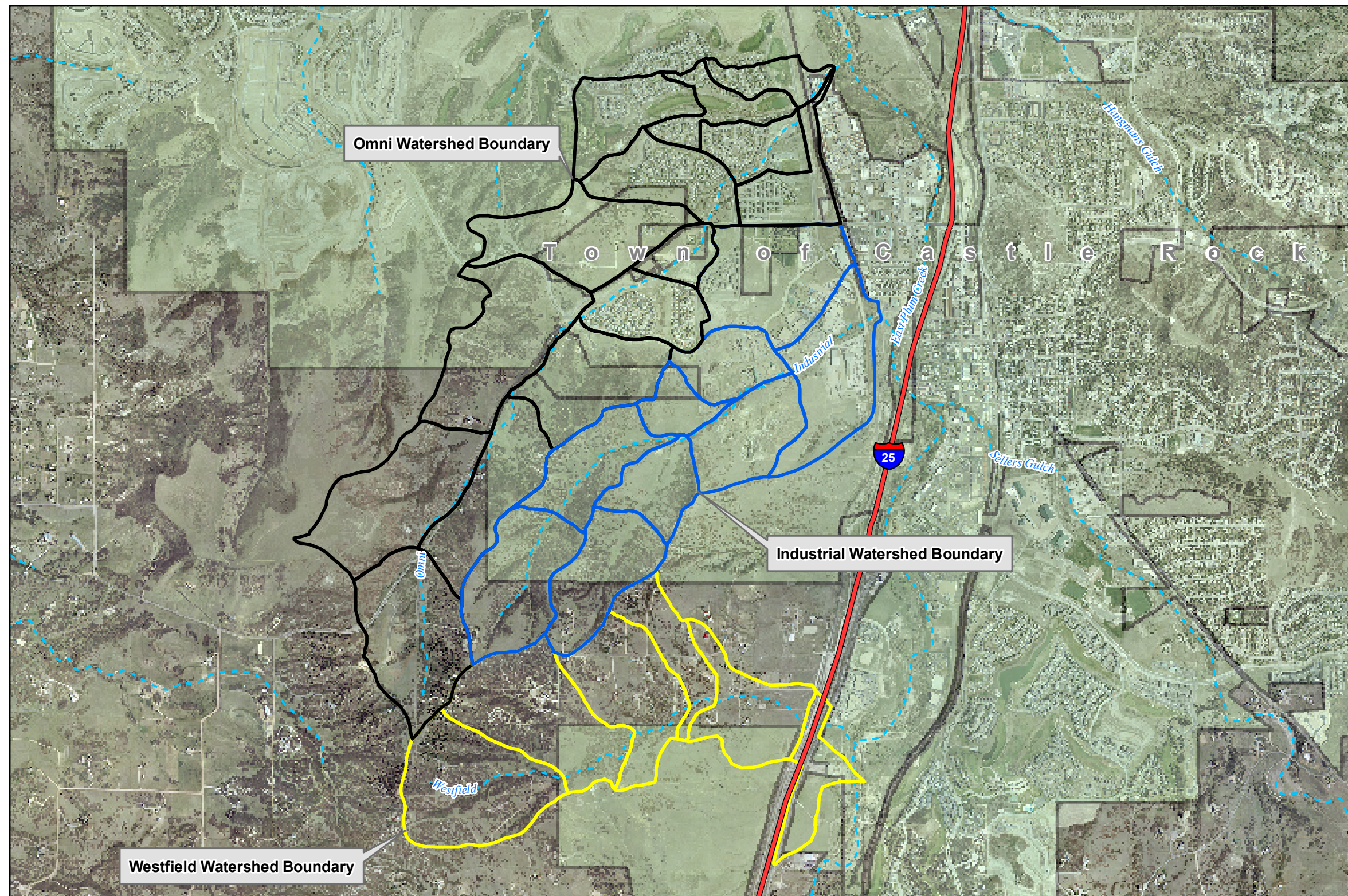
FINAL PRELIMINARY DESIGN REPORT

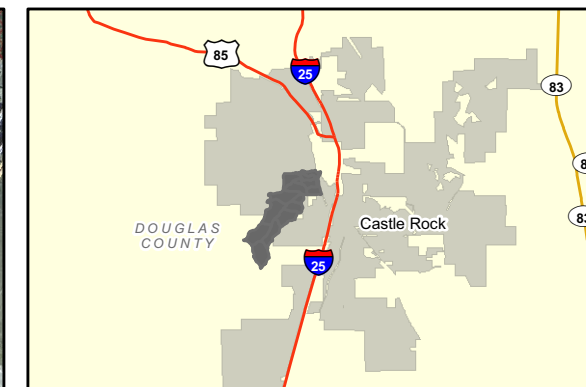
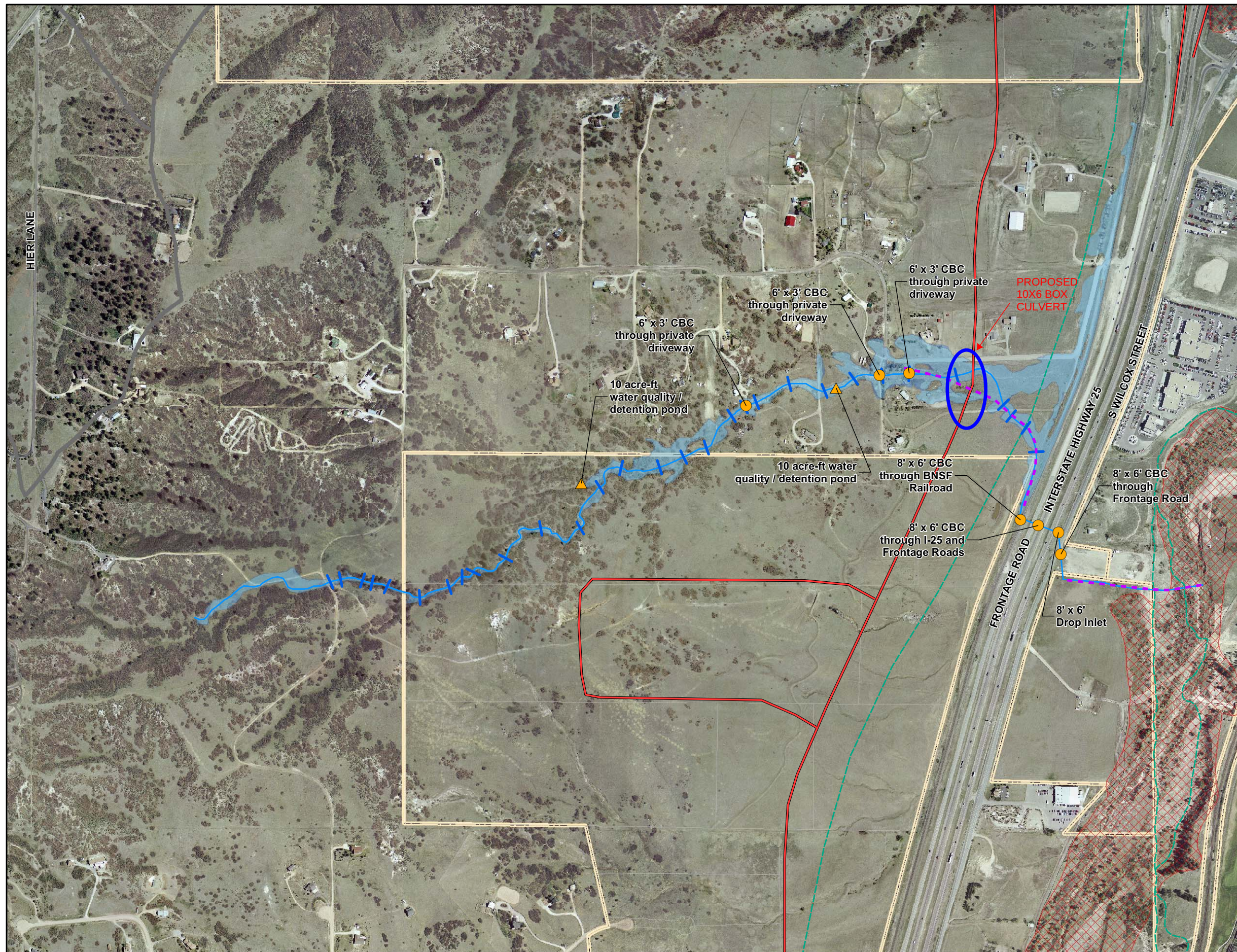
JANUARY 2011

PREPARED FOR:



PREPARED BY:





- LEGEND**
- Combination Detention & Infrastructure
- ▲ Detention
 - Infrastructure
 - - - Channelization
 - Drop Locations
 - ▨ Bank Stabilization
 - Proposed Road Centerline
 - - - Trail
 - Westfield Centerline
 - Westfield 100-YR Floodplain
 - Westfield 500-YR Floodplain
 - ▨ PMJM Habitat
 - Omni
 - ▭ Parcel Boundary
 - ▭ Town of Castle Rock Boundary

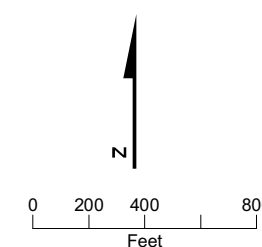


Figure i-6
Omni/Industrial/Westfield
Watershed Master Plan
Preferred Alternative

The floodplain extends outside of the channel banks and enters private industrial property. The floodplain for the remainder of this reach is contained within the channel banks.



Eroded Channel in Reach 2 of the Industrial Channel

Reach 3 – 1500' U/S of Atchison Way (Extended) to Confluence with Tributary

Reach 3 of the Industrial Tributary extends from where the flow becomes channelized to just downstream of the large stock pond. Throughout this reach, the channel is poorly defined and shallow overland flow occurs.

There are no infrastructure related problems within this reach. The stock pond at the top of this reach does not release flows to the historic channel.

Within this reach, shallow overland flow occurs with multiple split flow conditions. The stock pond marking the top of this reach releases flows through this spillway to the spillway channel while flows overtopping the pond enter the historic channel. Flows in the spillway channel that do not split to join the historic channel enter private industrial property.

Reach 4 – Confluence with Tributary to approximately 2500' U/S of Confluence with Tributary

Throughout Reach 4, beginning downstream at the large stock pond, the channel is well defined with occasional steep slopes. This reach includes a number of 1-foot head cuts and a few larger head cuts greater than 6 feet. Low flow channel incision is also seen at various locations throughout the reach.

There are no infrastructure related problems within this reach.

There are no flooding related issues within this reach as no properties are impacted by the 100-year floodplain.

Reach 5 – 2000' U/S of Confluence with Tributary to Upstream Study Limit

Throughout reach 5, the channel is poorly defined. At various locations throughout this reach are a few small 2-foot head cuts showing some instability in the stream.

There are no infrastructure related problems within this reach.

There are no flooding related issues within this reach as no properties are impacted by the 100-year floodplain.

Un-named tributary to the Industrial Tributary

Reach 1 – Confluence with mainstem of Industrial Tributary to Upstream Study Limit

The un-named tributary is very similar to the upper two reaches of the Industrial tributary in that the channel is well defined near the stock becoming less defined moving upstream. There is a large, 6-foot head cut a series of small head cuts spanning the distance of the reach.

There are no infrastructure related problems within this reach.

There are no flooding related issues within this reach as no properties are impacted by the 100-year floodplain.

Westfield Study Area

Reach 1 – Outfall to East Plum Creek to Downstream Most Private Driveway

The lower reach of the Westfield Tributary is a poorly defined channel with overland flow occurring in high flow events. This reach crosses two private drives with 2-foot culverts that have a capacity of approximately 22 cfs and that are partially blocked with sediment. These culverts do not pass the 100-year future or 100-year existing flow rates, however being driveways the two culverts are only required to pass the 10-year flow rate. The 2 - 24-inch RCP undersized driveway culverts cause a backwater effect in the floodplain. Also within this reach are two properties impacted by the 100-year floodplain.

In the downstream most reach of the Westfield Tributary, a split flow condition exists with a portion of the flow running north along the railroad ditch to a 72-inch CMP under the Burlington Northern Railroad and a portion of the flow running south along the railroad ditch to a 36-inch CMP outfall under the Burlington Northern Railroad. Neither one of these outfalls are size for the full 100-year future flows. Each culvert pipes has a capacity of 331 cfs and 98 cfs respectively. The predicted North discharge, accounting for the split flow, for the future developed condition is 742 cfs while the south future developed conditions split flow is 317 cfs. This indicates that both culverts that drain the Westfield system are undersized

Reach 2 – Downstream Most Private Driveway to Upstream Most Private Driveway

The channel is well defined throughout reach 2 of the Westfield Tributary. In the lower portion of the reach are 4 - 52-inch by 36-inch elliptical RCPs that are each partially or completely blocked with sediment. The channel upstream of the culverts is severely degraded by development and multiple private driveways with no culvert outlets. At one location of a driveway with no outlet has eroded into a "hole" upstream of the driveway and has collected a substantial amount of debris. The entrance to this reach is large head cut. Continued erosion could affect private structures and property.

Within reach 2 are undersized culverts located at private driveways.

Within this reach, upstream of the undersized culverts, bulges in the floodplain occur. Although this reach is severely degraded and the floodplain is not entirely contained within the channel banks, no properties are at risk from flooding.

Westfield Tributary

Reach 1 – Station 0+00 to Station 36+67

There are substantial Priority 2 recommended alternatives through this reach. Currently a formal outfall channel does not exist for the Westfield channel. As Development progresses it is recommended that a formal outfall east of I-25 to east plum creek be constructed. Infrastructure in the area is also undersized. An 8' x 6' CBC system is recommended to be installed under the BNSF railroad, I-25 and both frontage roads to connect to the newly constructed outfall channel proposed in this master plan. The 8' x 6' CBC system is undersized for 100-year existing peak flows so detention upstream is necessary in combination with the 8' x 6' CBC system.

The Channel should be stabilized with six Priority 2 check structures and two Priority 3 drop structures if required. Finally, addition channelization in the form of a meandering 20 foot bottom width natural channel be constructed to prevent future split flows is recommended upstream from the BNSF railway to station 27+50.

Infrastructure in Private driveways in this reach should be upgraded to 6' x 3' CBC to pass 10-year future flow rates as required by Town and County Criteria.

Reach 2 – Station 36+67 to Station 49+45

The recommended plan for Reach 2 of the Westfield Channel includes three Priority 1 drop structures, and three Priority 2 check structures to stabilize the channel grade. Priority 1 drop structures are recommended to stabilize existing head cuts and downstream of existing infrastructure that may be in danger if channel erosion continues. Priority 3 drop structures should be constructed if Priority 1 and Priority 2 drop structures do not stabilize the channel.

Other Priority 2 recommended improvements within this reach are the construction of two 10 acre-foot detention ponds to improve infrastructure flooding downstream and infrastructure in Private driveways should be upgraded to 6' x 3' CBC to pass 10-year future flow rates as required by Town and County Criteria.

Reach 3 – Station 49+45 to Station 82+99

The recommended plan for Reach 3 of the Westfield Channel includes thirteen Priority 2 check structures, and one Priority 3 drop structure to stabilize the channel grade. Priority 1 structures are recommended to stabilize existing head cuts. Priority 3 drop structures should be constructed if Priority 1 and Priority 2 drop structures do not stabilize the channel.

Infrastructure in Private driveways in this reach should be upgraded to 6' x 3' CBC to pass 10-year future flow rates as required by Town and County Criteria

Reach 4 – Station 82+99 to Station 95+75

The recommended plan for Reach 4 of the Westfield Channel includes one Priority 1 drop structure, and one Priority 2 check structure to stabilize the channel grade. Priority 1 structures are recommended to

stabilize existing head cuts. Priority 3 drop structures should be constructed if Priority 1 and Priority 2 drop structures do not stabilize the channel.

Prioritization and Phasing Plan

Implementation of the proposed Plan will be accomplished in a sequential phased manner based on selected priorities for stabilizing the profile of the channel. Prioritization of the selected plan was accomplished based on the following guidance:

- **Priority 1:** Improvements under this priority are required to protect critical structures and private property or are necessary now to mitigate damage to existing flood control facilities and environmentally sensitive areas. Selected stream improvements under this priority are identified at locations with active head cutting or streambed erosion where the potential for damage is imminent.
- **Priority 2:** Improvements under this priority will be necessary to protect critical structures and private property or to preserve environmentally sensitive areas of the watersheds as development or significant changes to the watershed occur. Selected stream improvements under this priority are located to provide hydraulic controls that address the majority of the anticipated threats associated with long-term channel instability.
- **Priority 3:** Improvements under this priority are long-term improvements. These will help to enhance the existing condition of the corridor and protect structures if conditions warrant. Streams should be monitored for changes or further bank erosion and channel degradation prior to implementing any improvements selected under this priority. If the priority 1 and 2 improvements do not completely stabilize the channel bed or if there are major changes in the watershed then the priority 3 improvements should be considered.

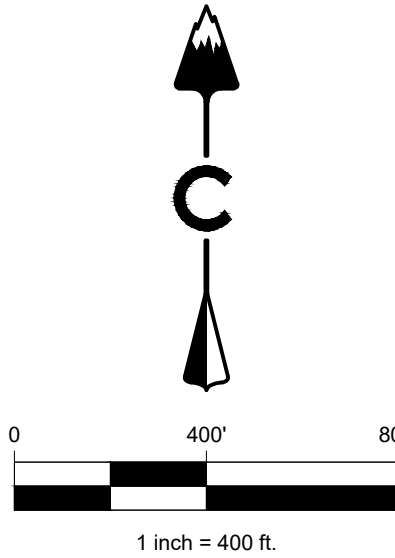
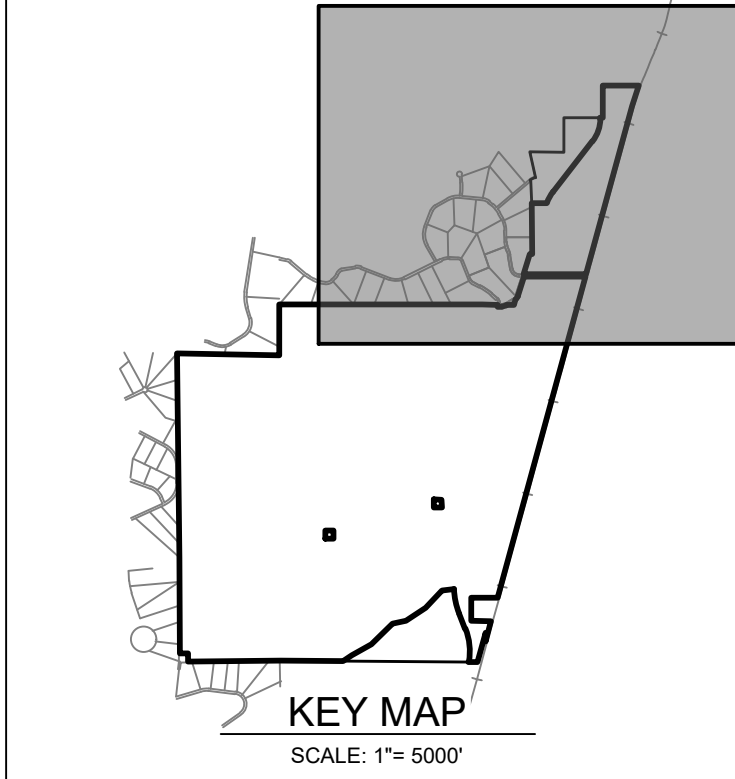
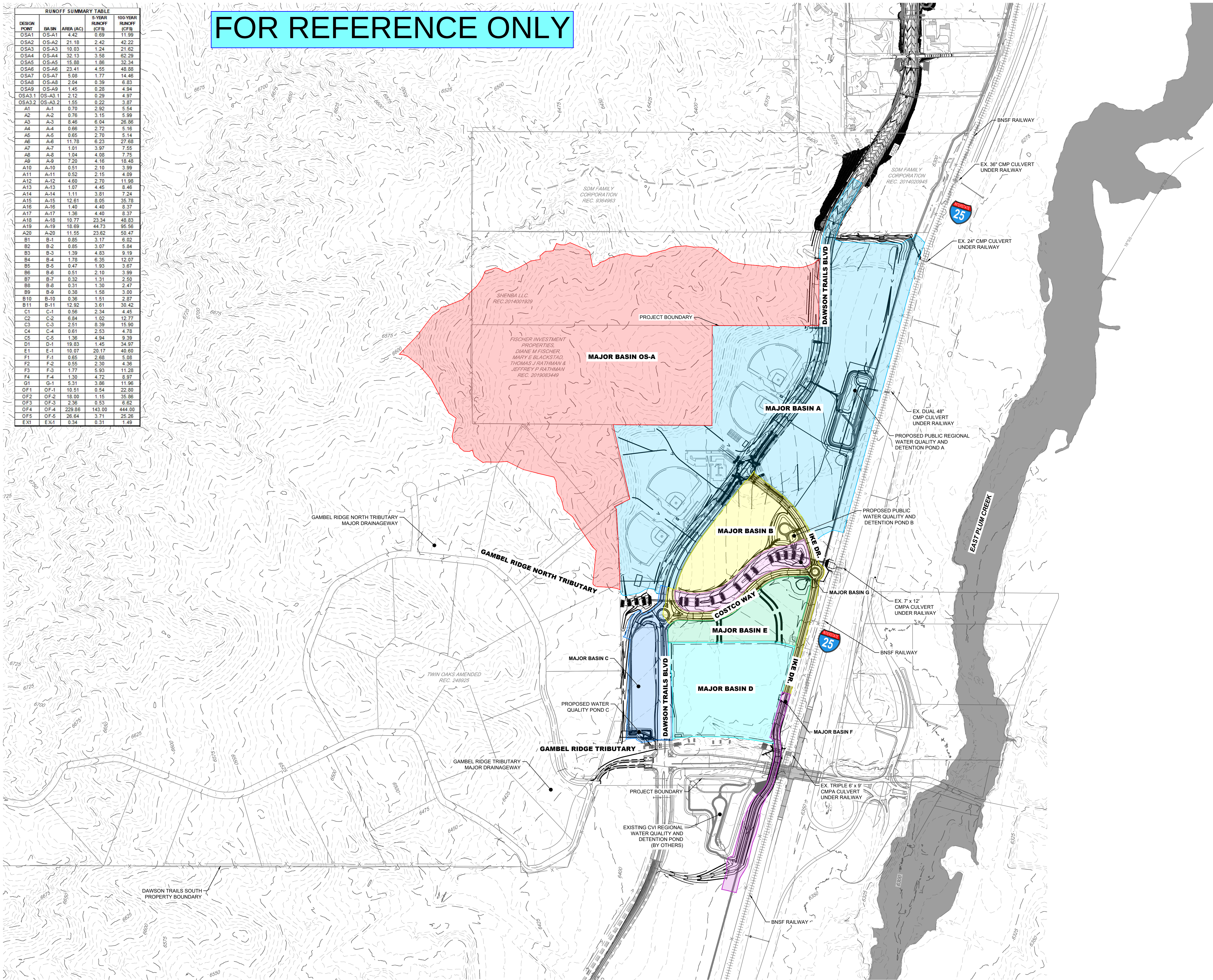
The following **Table VI-1** through **VI-5** presents the selected implementation priorities For the Omni Watershed, Omni Tributary, Industrial Watershed, Industrial Tributary and Westfield Tributary.

TABLE VI-1
Implementation Priority for the Omni Watershed

Priority	Improvement	Reach
1	4' Drop Structure at Station 10+99	1
1	4' Drop Structure at Station 42+68	2
1	4' Drop Structure at Station 34+54	2
1	Bank Stabilization from Station 42+94 to Station 39+50	2
1	3 – 7' x 4' CBC at Wolfensberger Road	2
1	Add Additional 66" RCP at Red Hawk Drive	2
1	6.2 acre-ft Detention Pond	2
1	3 – 36" RCP Equalizer Culverts for Detention Pond	2
1	4' Drop Structure at Station 53+60	3
1	Bank Stabilization from Station 52+50 to Station 45+00	3
1	Raise Channel Invert from Station 55+00 to Station 44+00	3

RUNOFF SUMMARY TABLE				
DESIGN POINT	Basin	Area (Ac)	5-Year Runoff (CFS)	100-Year Runoff (CFS)
OSA1	OS-A1	4.42	0.89	11.99
OSA2	OS-A2	21.18	2.42	42.22
OSA3	OS-A3	10.03	1.24	23.62
OSA4	OS-A4	32.13	3.58	62.28
OSA5	OS-A5	15.88	1.86	32.34
OSA6	OS-A6	23.41	4.55	48.88
OSA7	OS-A7	5.08	1.77	14.46
OSA8	OS-A8	2.04	0.39	6.83
OSA9	OS-A9	1.45	0.29	4.94
OSA3.1	OS-A3.1	2.12	0.29	4.97
OSA3.2	OS-A3.2	1.56	0.22	3.87
A1	A-1	0.70	2.92	5.54
A2	A-2	0.76	3.15	5.99
A3	A-3	8.46	6.04	26.86
A4	A-4	0.66	2.72	5.16
A5	A-5	0.65	2.70	5.14
A6	A-6	11.78	6.23	27.68
A7	A-7	1.01	3.97	7.55
A8	A-8	1.04	4.08	7.75
A9	A-9	7.20	4.16	18.48
A10	A-10	0.51	2.10	3.99
A11	A-11	0.52	2.15	4.09
A12	A-12	4.60	2.70	11.88
A13	A-13	1.07	4.45	8.46
A14	A-14	1.11	3.81	7.24
A15	A-15	12.61	8.05	35.78
A16	A-16	1.40	4.48	8.37
A17	A-17	1.36	4.40	8.37
A18	A-18	10.77	23.34	48.83
A19	A-19	18.89	44.73	95.56
A20	A-20	11.55	23.62	50.47
B1	B-1	0.85	3.17	6.02
B2	B-2	0.85	3.07	5.84
B3	B-3	1.39	4.83	9.19
B4	B-4	1.78	6.35	12.07
B5	B-5	0.47	1.93	3.67
B6	B-6	0.51	2.10	3.99
B7	B-7	0.32	1.31	2.50
B8	B-8	0.31	1.30	2.47
B9	B-9	0.38	1.58	3.00
B10	B-10	0.36	1.51	2.87
B11	B-11	12.92	3.91	30.42
C1	C-1	0.56	2.34	4.45
C2	C-2	6.84	1.92	12.77
C3	C-3	2.51	8.39	15.90
C4	C-4	0.61	2.53	4.78
C5	C-5	1.38	4.94	9.39
D1	D-1	19.83	1.45	34.97
E1	E-1	10.07	20.17	40.60
F1	F-1	0.65	2.68	5.08
F2	F-2	0.55	2.30	4.36
F3	F-3	1.77	5.93	11.28
F4	F-4	1.30	4.72	8.97
G1	G-1	5.31	3.86	11.96
OF1	OF-1	10.51	0.54	22.80
OF2	OF-2	18.00	1.15	35.86
OF3	OF-3	2.36	0.53	6.62
OF4	OF-4	229.86	143.00	444.00
OF5	OF-5	26.64	3.71	25.26
EX1	EX-1	0.34	0.31	1.48

FOR REFERENCE ONLY



LEGEND

- PROPOSED
- BASIN DESIGNATION
 - 5-YEAR RUNOFF COEFFICIENTS
 - 100-YEAR RUNOFF COEFFICIENTS
 - DESIGN POINT
 - DIRECTIONAL FLOW ARROW
 - EMERGENCY OVERFLOW ROUTE
 - PROPOSED DRAINAGE BASIN
 - EXISTING DRAINAGE BASIN
 - PROPOSED MAJOR CONTOUR
 - PROPOSED MINOR CONTOUR
 - EXISTING MAJOR CONTOUR
 - EXISTING MINOR CONTOUR
 - EXISTING RAILWAY
 - EXISTING FENCE
 - EXISTING PROPERTY BOUNDARY
 - PROPOSED RIGHT OF WAY (R.O.W.)
 - PROPOSED CENTERLINE
 - PROPOSED PROJECT BOUNDARY
 - PROPOSED STORM & STUB OUT
 - EXISTING STORM & STUB OUT
 - STORM MANHOLES
 - STORM INLETS
 - FES, FOREBAY, & TRICKLE CHANNEL
 - OUTLET STRUCTURE
 - CRUSHER FINES
 - MAINTENANCE ACCESS
 - RIPRAP
 - 100-YEAR FEMA FLOODPLAIN LIMITS
 - 100-YEAR TOWN FLOODPLAIN
 - 100-YEAR TOWN FLOODPLAIN LIMITS
 - 100-YEAR TOWN FLOODPLAIN
 - FUTURE CURB & GUTTER
 - MAJOR BASIN A
 - MAJOR BASIN B
 - MAJOR BASIN C
 - MAJOR BASIN D
 - MAJOR BASIN E
 - MAJOR BASIN F
 - MAJOR BASIN G
 - MAJOR BASIN OS-A

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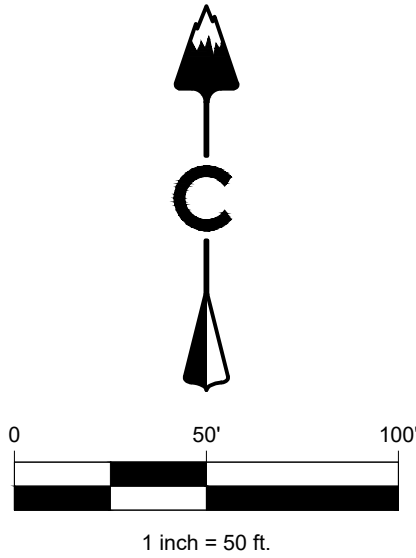
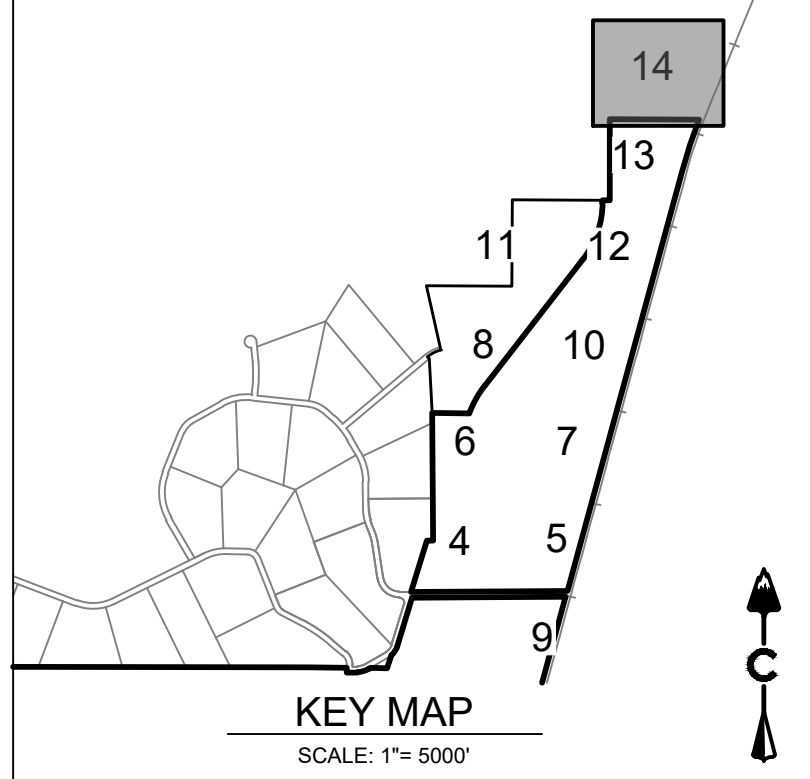
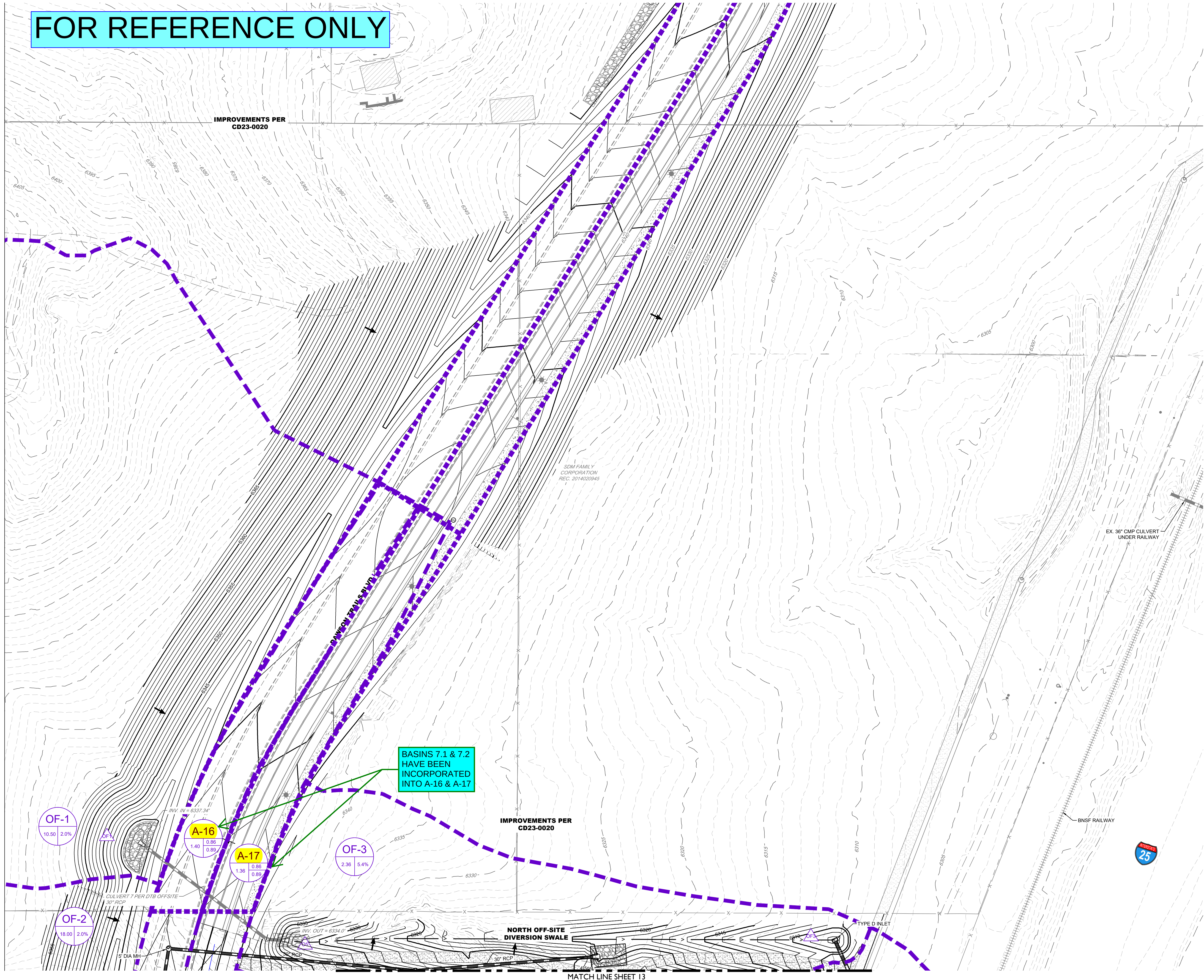
DAWSON TRAILS BLVD (SOUTH)- FILING 1 INFRASTRUCTURE
TOWN OF CASTLE ROCK, COLORADO
PHASE III DRAINAGE REPORT
PROPOSED OVERALL DRAINAGE MAP



DESIGNED BY: EKT
DRAWN BY: EKT
CHECKED BY: BG

JOB NO.
20-224
SHEET
3 OF 14

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LEGEND

- PROPOSED**
- BASIN DESIGNATION
 - 5-YEAR RUNOFF COEFFICIENTS
 - 100-YEAR RUNOFF COEFFICIENTS
 - DESIGN POINT
 - DIRECTIONAL FLOW ARROW
 - EMERGENCY OVERFLOW ROUTE
 - PROPOSED DRAINAGE BASIN
 - EXISTING DRAINAGE BASIN
 - PROPOSED MAJOR CONTOUR
 - PROPOSED MINOR CONTOUR
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 - FUTURE CURB & GUTTER
 - MAJOR BASIN A
 - MAJOR BASIN B
 - MAJOR BASIN C
 - MAJOR BASIN D
 - MAJOR BASIN E
 - MAJOR BASIN F
 - MAJOR BASIN G
 - MAJOR BASIN OS-A

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DESCRIPTION

DAWSON TRAILS BLVD (SOUTH)- FILING 1 INFRASTRUCTURE

TOWN OF CASTLE ROCK, COLORADO

PHASE III DRAINAGE REPORT

DETAILED DRAINAGE MAP (11 OF 11)

DESIGNED BY: EKT
DRAWN BY: EKT
CHECKED BY: BG

JOB NO.
20-224
SHEET
14 OF 14

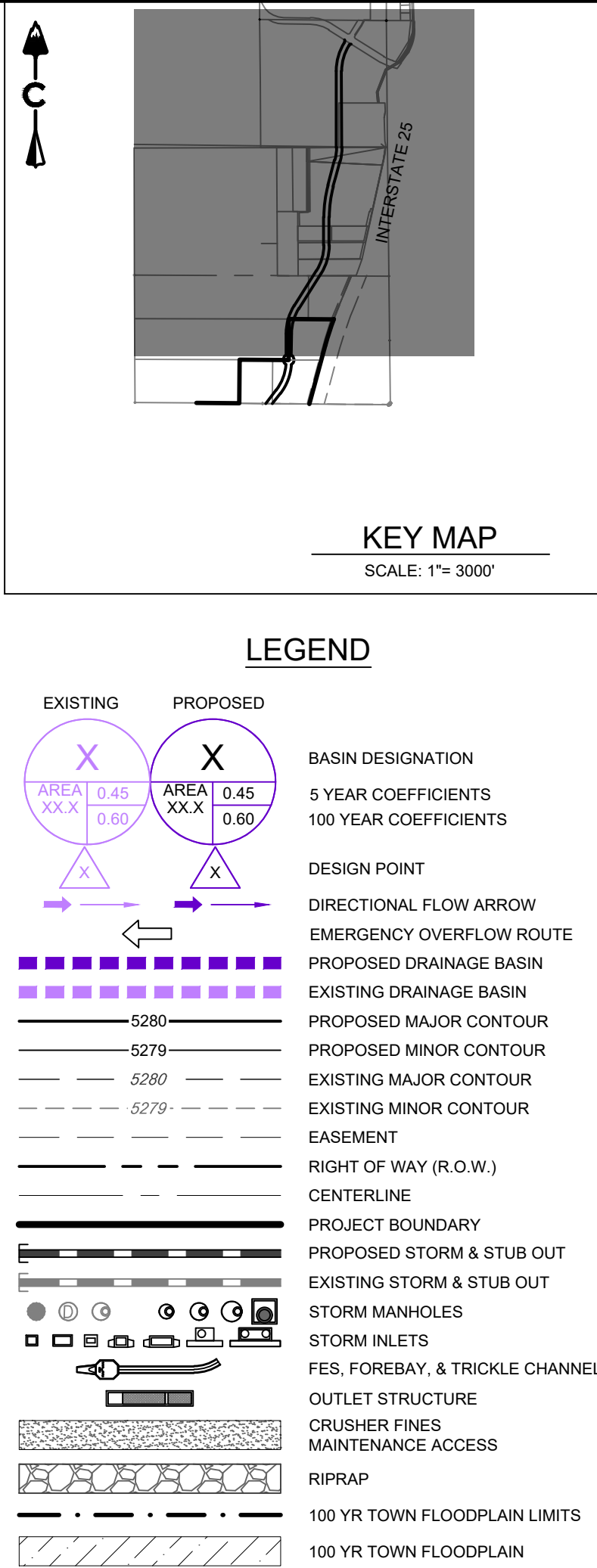
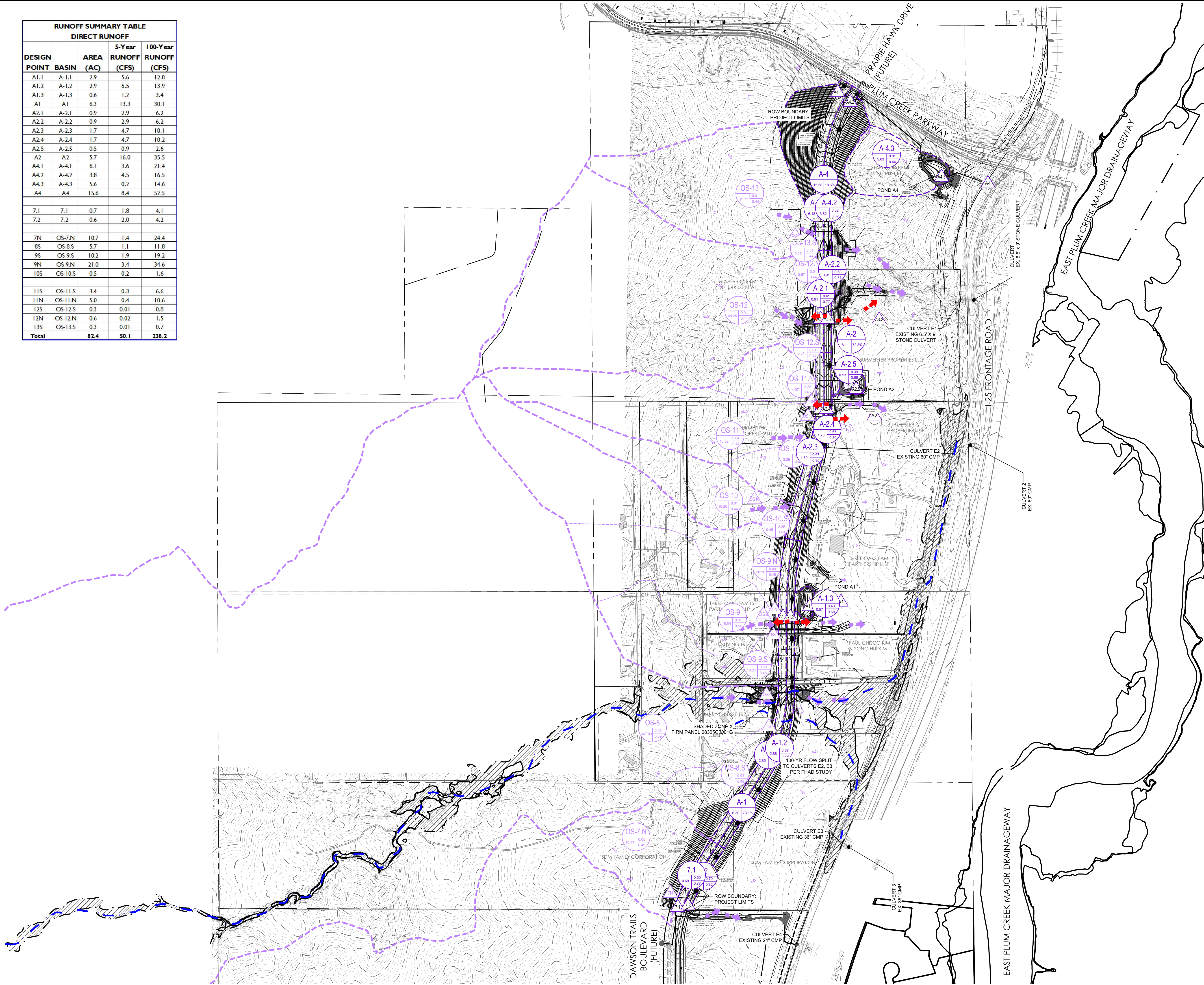
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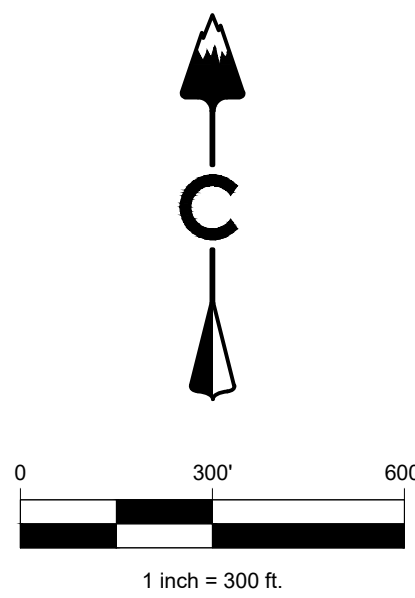
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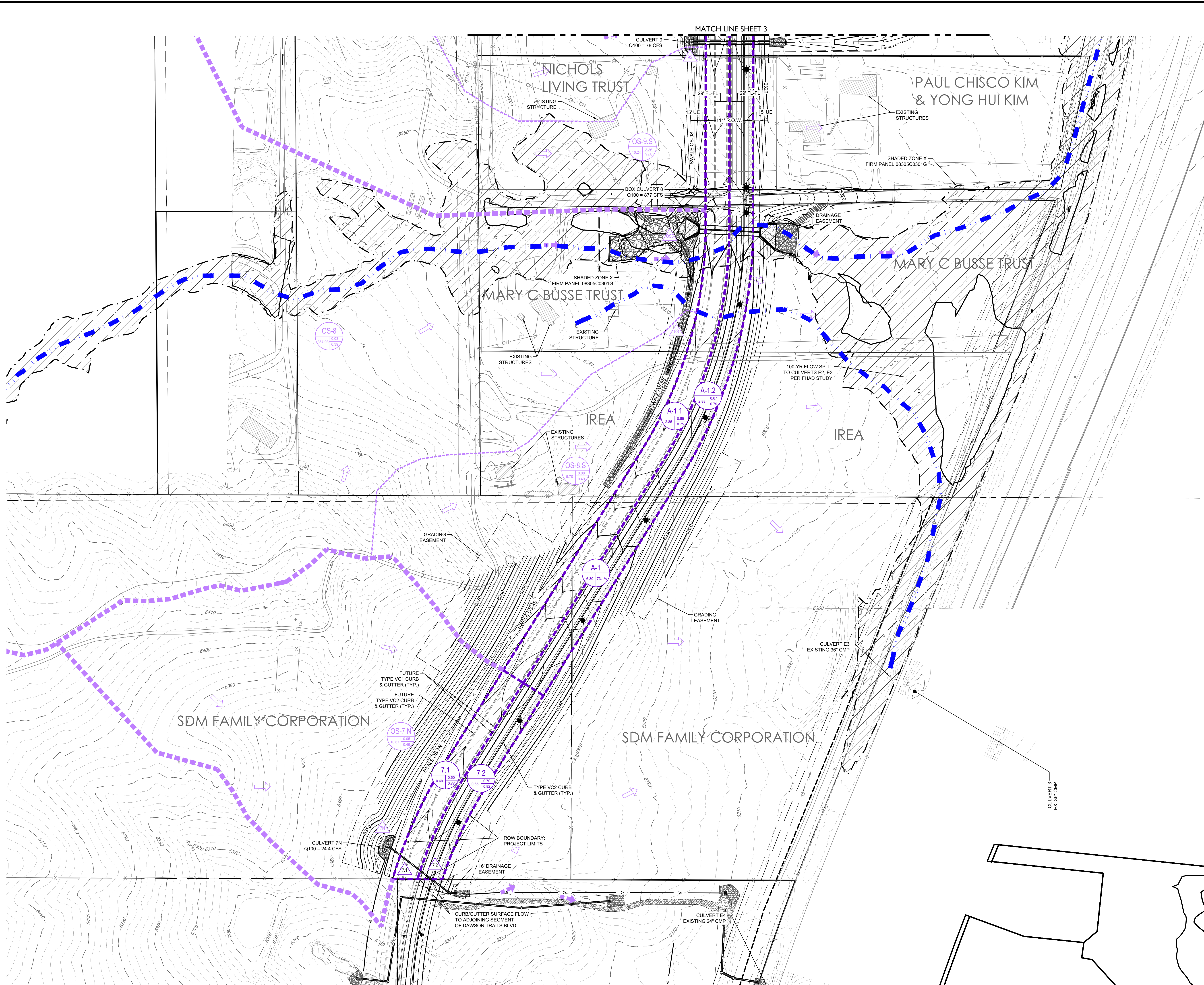
DRAINAGE MAPS

RUNOFF SUMMARY TABLE				
DIRECT RUNOFF				
DESIGN POINT	BASIN	AREA (AC)	5-Year RUNOFF (CFS)	100-Year RUNOFF (CFS)
A1.1	A-1.1	2.9	5.6	12.8
A1.2	A-1.2	2.9	6.5	13.9
A1.3	A-1.3	0.6	1.2	3.4
A1	A1	6.3	13.3	30.1
A2.1	A-2.1	0.9	2.9	6.2
A2.2	A-2.2	0.9	2.9	6.2
A2.3	A-2.3	1.7	4.7	10.1
A2.4	A-2.4	1.7	4.7	10.2
A2.5	A-2.5	0.5	0.9	2.6
A2	A2	5.7	16.0	35.5
A4.1	A-4.1	6.1	3.6	21.4
A4.2	A-4.2	3.8	4.5	16.5
A4.3	A-4.3	5.6	0.2	14.6
A4	A4	15.6	8.4	52.5
7.1	7.1	0.7	1.8	4.1
7.2	7.2	0.6	2.0	4.2
7N	OS-7.N	10.7	1.4	24.4
8S	OS-8.S	5.7	1.1	11.8
9S	OS-9.S	10.2	1.9	19.2
9N	OS-9.N	21.0	3.4	34.6
10S	OS-10.S	0.5	0.2	1.6
11S	OS-11.S	3.4	0.3	6.6
11N	OS-11.N	5.0	0.4	10.6
12S	OS-12.S	0.3	0.01	0.8
12N	OS-12.N	0.6	0.02	1.5
13S	OS-13.S	0.3	0.01	0.7
Total		82.4	50.1	238.2



- NOTES:**
1. FEMA MAPPED FLOOD AREAS SHOWN ON DRAINAGE MAP ARE DELINEATED ON FIRM PANEL 080835C0301G.





KEY MAP
SCALE: 1"= 3000'

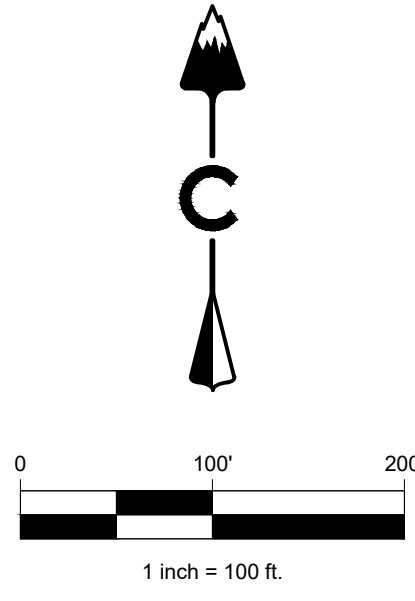
LEGEND

EXISTING	PROPOSED	
		BASIN DESIGNATION
		5 YEAR COEFFICIENTS
		100 YEAR COEFFICIENTS
		DESIGN POINT
		DIRECTIONAL FLOW ARROW
		EMERGENCY OVERFLOW ROUTE
		PROPOSED DRAINAGE BASIN
		EXISTING DRAINAGE BASIN
		PROPOSED MAJOR CONTOUR
		PROPOSED MINOR CONTOUR
		EXISTING MAJOR CONTOUR
		EXISTING MINOR CONTOUR
		EASEMENT
		RIGHT OF WAY (R.O.W.)
		CENTERLINE
		PROJECT BOUNDARY
		PROPOSED STORM & STUB OUT
		EXISTING STORM & STUB OUT
		STORM MANHOLES
		STORM INLETS
		FES, FOREBAY, & TRICKLE CHANNEL
		OUTLET STRUCTURE
		CRUSHER FINES
		MAINTENANCE ACCESS
		RIPRAP
		100 YR TOWN FLOODPLAIN LIMITS
		100 YR TOWN FLOODPLAIN

NOTES:

- FEMA MAPPED FLOOD AREAS SHOWN ON DRAINAGE MAP ARE DELINEATED ON FIRM PANEL 080835C0301G.

RUNOFF SUMMARY TABLE				
DIRECT RUNOFF				
DESIGN POINT	BASIN	AREA (AC)	5-Year RUNOFF (CFS)	100-Year RUNOFF (CFS)
A1.1	A-1.1	2.9	5.6	12.8
A1.2	A-1.2	2.9	6.5	13.9
A1.3	A-1.3	0.6	1.2	3.4
A1	A1	6.3	13.3	30.1
A2.1	A-2.1	0.9	2.9	6.2
A2.2	A-2.2	0.9	2.9	6.2
A2.3	A-2.3	1.7	4.7	10.1
A2.4	A-2.4	1.7	4.7	10.2
A2.5	A-2.5	0.5	0.9	2.6
A2	A2	5.7	16.0	35.5
A4.1	A-4.1	6.1	3.6	21.4
A4.2	A-4.2	3.8	4.5	16.5
A4.3	A-4.3	5.6	0.2	14.6
A4	A4	15.6	8.4	52.5
7.1	7.1	0.7	1.8	4.1
7.2	7.2	0.6	2.0	4.2
7N	OS-7.N	10.7	1.4	24.4
8S	OS-8.S	5.7	1.1	11.8
9S	OS-9.S	10.2	1.9	19.2
9N	OS-9.N	21.0	3.4	34.6
10S	OS-10.S	0.5	0.2	1.6
11S	OS-11.S	3.4	0.3	6.6
11N	OS-11.N	5.0	0.4	10.6
12S	OS-12.S	0.3	0.01	0.8
12N	OS-12.N	0.6	0.02	1.5
13S	OS-13.S	0.3	0.01	0.7
Total		82.4	50.1	238.2



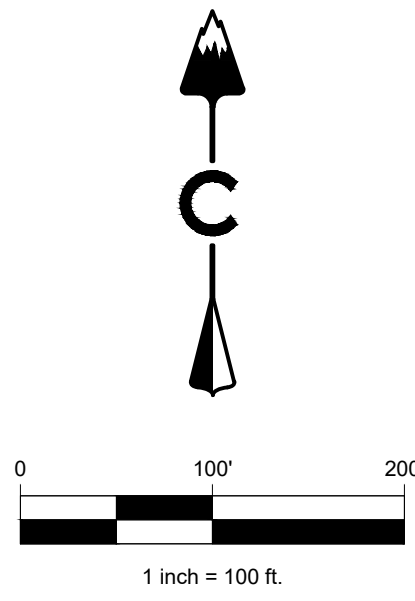
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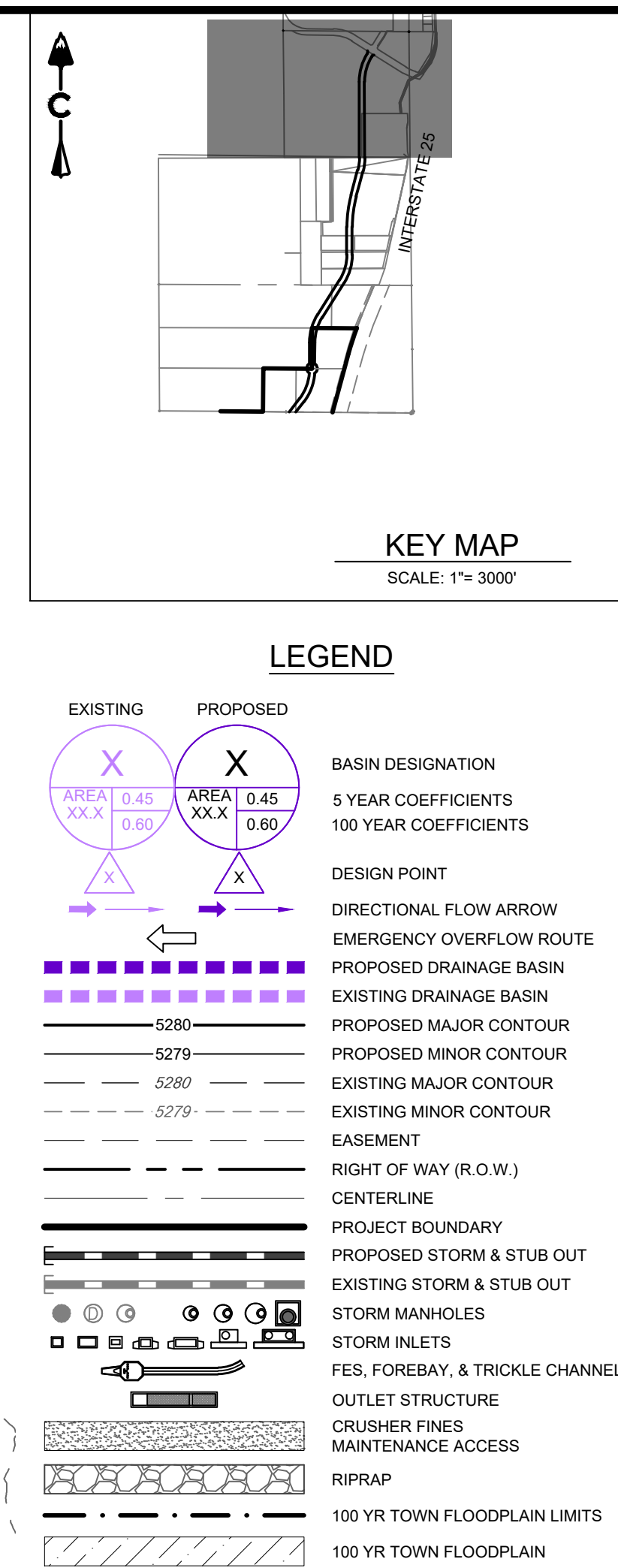
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DAWSON TRAILS - PRAIRIE HAWK NORTH
TOWN OF CASTLE ROCK, COLORADO
PHASE III DRAINAGE REPORT
DETAILED DRAINAGE PLAN

DESIGNED BY: CJ
DRAWN BY: CJ
CHECKED BY: MG

JOB NO.
20-224
SHEET
2 OF 4





NOTES:

1. FEMA MAPPED FLOOD AREAS SHOWN ON DRAINAGE MAP ARE DELINEATED ON FIRM PANEL 080835C0301G.

RUNOFF SUMMARY TABLE				
DIRECT RUNOFF				
DESIGN POINT	BASIN	AREA (AC)	5-Year RUNOFF (CFS)	100-Year RUNOFF (CFS)
A1.1	A-1.1	2.9	5.6	12.8
A1.2	A-1.2	2.9	6.5	13.9
A1.3	A-1.3	0.6	1.2	3.4
A1	A1	6.3	13.3	30.1
A2.1	A-2.1	0.9	2.9	6.2
A2.2	A-2.2	0.9	2.9	6.2
A2.3	A-2.3	1.7	4.7	10.1
A2.4	A-2.4	1.7	4.7	10.2
A2.5	A-2.5	0.5	0.9	2.6
A2	A2	5.7	16.0	35.5
A4.1	A-4.1	6.1	3.6	21.4
A4.2	A-4.2	3.8	4.5	16.5
A4.3	A-4.3	5.6	0.2	14.6
A4	A4	15.6	8.4	52.5
7.1	7.1	0.7	1.8	4.1
7.2	7.2	0.6	2.0	4.2
7N	OS-7.N	10.7	1.4	24.4
8S	OS-8.S	5.7	1.1	11.8
9S	OS-9.S	10.2	1.9	19.2
9N	OS-9.N	21.0	3.4	34.6
10S	OS-10.S	0.5	0.2	1.6
11S	OS-11.S	3.4	0.3	6.6
11N	OS-11.N	5.0	0.4	10.6
12S	OS-12.S	0.3	0.01	0.8
12N	OS-12.N	0.6	0.02	1.5
13S	OS-13.S	0.3	0.01	0.7
Total		82.4	50.1	238.2

