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Drainage Conformance Letter

for

Maverik
Convenience Store and Fuel Station

Lincoln and Dransfeldt
Parker, Colorado 80138
TOWN OF PARKER # SP21-133

Prepared for:

Maverik, Inc.
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Prepared by:

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DCI Job No. 21-122-0001
March 10, 2023

Engineer's Certification:

I hereby affirm that this report and plan for the drainage design for Maverik Convenience Store and Fuel Station was prepared by me, or under my direct supervision, for the owners thereof, in accordance with the provision for the Town of Parker Storm and Environmental Drainage Design and Technical Criteria Manual, and approved variances and exceptions thereto. I understand that Town of Parker does not and will not assume liability for drainage facilities designed by others.

By: Damon A. Smith, PE
Licensed Professional Engineer
State of Colorado
No. 59516

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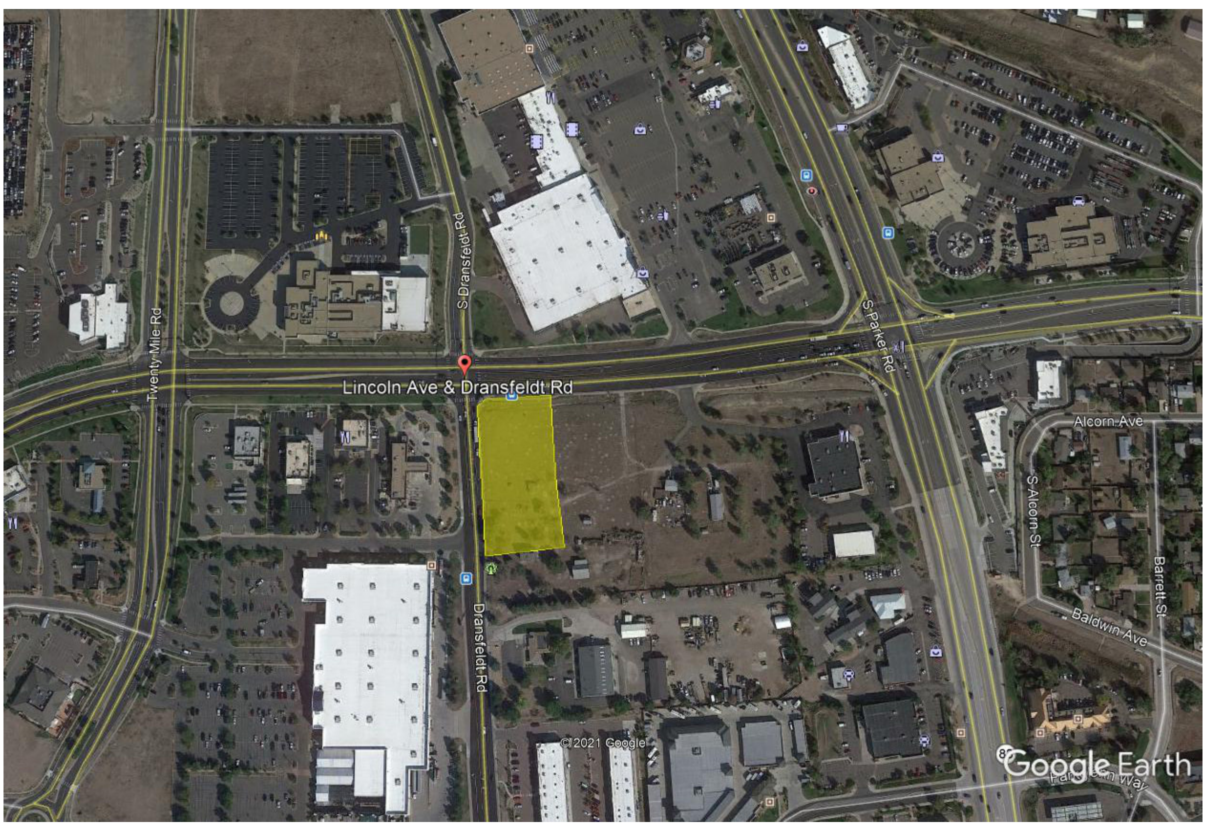
SECTION II. GENERAL LOCATION AND DESCRIPTION

A: GENERAL LOCATION AND DESCRIPTION

The proposed Maverik Development is located within the Town of Parker, within Douglas County in Colorado. Prior to the submittal of the Site Improvement Permit, the site was approved for annexation into the Town boundaries. The proposed 1.30-acre plot of land is comprised of a single parcel that was created under the Lincoln Professional Park Subdivision Filing. The site is situated on the southeast corner of Lincoln Avenue and Dransfeldt Road and is located on what is currently undeveloped former agricultural land.



The Colorado State Land Board maintains records of the Cadastral Survey of the state for purposes of establishing legal property boundaries in proposed land subdivisions. The proposed property is located on the western edge of the A Parcel of land situated in the Northwest Quarter of Section 15, Township 6 South, Range 66 West of the Sixth Principal Meridian, County of Douglas, State of Colorado.

The legal description, as identified on the Subdivision Plat by HKS, is as follows:

COMMENCING AT THE NORTHWEST CORNER OF SAID SECTION 15;

THENCE SOUTH 83°44'03" EAST, A DISTANCE OF 846.76 FEET TO A POINT ON THE SOUTH RIGHT-0F-WAY LINE OF LINCOLN AVENUE AND THE POINT OF BEGINNING;

THENCE NORTH 89°29'OO' EAST ALONG SAID SOUTH RIGHT-OF-WAY LINE, A DISTANCE OF 697.77 FEET TO THE N0RTHTWEST CORNER OF LOT 1, PEASLEE SUBDIVISION FILING NO. 1, RECORDED AT RECEPTION N0. 99104313;

THENCE ALONG THE WEST LINE OF SAID LOT 1 THE FOLLOWING TWO (2) COURSES:

1) SOUTH 00°31'00" EAST, A DISTANCE OF 89.84 FEET;

2) S0UTH 11°49'46" EAST, A DISTANCE OF 225.08 FEET TO THE NORTHWEST CORNER OF LOT 1, PEASLEE SUBDIVISION FILING N0. 2, RECORDED AT RECEPTION NO. 2005085666;

THENCE SOUTH 05°51'51" EAST ALONG THE WEST LINE OF SAID LOT 1, A DISTANCE OF 120.83 FEET TO A POINT ON THE NORTH LINE OF LOT 1B, PARKER PROFESSIONAL PARK FIRST AMENDMENT, RECORDED AT RECEPTION NO. 8725509;

THENCE SOUTH 84°07'50" WEST ALONG SAID NORTH LINE, A DISTANCE OF 24.51 FEET TO THE NORTHEAST CORNER OF LOT 1, BLOCK 1 E.T. TECHNOLOGIES INC., RECORDED AT RECEPTION NO. 2002093991;

THENCE ALONG SAID NORTH LINE THE F0LL0WING TW0 (2) COURSES:

1) SOUTH 82°44'13" WEST, A DISTANCE OF 163.50 FEET;

2) SOUTH 84°51'34" WEST, A DISTANCE OF 577.15 FEET TO A POINT ON THE EAST RIGHT-OF-WAY LINE OF DRANSFELDT ROAD DEEDED BY RECEPTION NO. 2002088425 AND A POINT OF NON-TANGENT CURVATURE:

THENCE ALONG SAID EAST RIGHT-OF-WAY LINE THE FOLLOWING SIX (6) COURSES:

1) ALONG THE ARC OF SAID CURVE TO THE RIGHT AN ARC LENGTH OF 25.79 FEET, SAID CURVE HAVING A RADIUS OF 660.00 FEET, A CENTRAL ANGLE 0F 02°14'19" AND A CHORD WHICH BEARS NORTH 02°20'42" WEST A CHORD DISTANCE OF 25.79 FEET;

2) NORTH 01°13'32" WEST, A DISTANCE OF 165.71 FEET TO A POINT OF CURVATURE;

3) AL0NG THE ARC OF SAID CURVE TO THE LEFT AN ARC LENGTH OF 43.04 FEET, SAID CURVE HAVING A RADIUS OF 740.00 FEET, A CENTRAL ANGLE OF 03°19'57", AND A CHORD WHICH BEARS NORTH 02°53'30" WEST A CHORD DISTANCE OF 43.03 FEET;

4) NORTH 04°33'29" WEST, A DISTANCE OF 217.83 FEET TO A POINT OF CURVATURE;

5) ALONG THE ARC OF SAID CURVE TO THE RIGHT AN ARC LENGTH OF 49.23 FEET, SAID CURVE HAVING A RADIUS OF 3O.00 FEET, A CENTRAL ANGLE OF 94°01'51", AND A CHORD WHICH BEARS NORTH 42°27'27" EAST A CHORD DISTANCE OF 43.89 FEET;

6) NORTH 04°33'21" WEST, A DISTANCE OF 15.04 FEET TO THE POINT OF BEGINNING.

SAID PARCEL CONTAINS 346,479 SQUARE FEET OR 7.95 ACRES, MORE 0R LESS

After the approved annexation in 2021, the proposed development is located within the boundaries of the Town of Parker. The site is bordered by Lincoln Avenue on the north and Dransfeldt Road on the west, both which operates under the jurisdiction of the Douglas County Public Works. Interstate 25, a major thorough-fare for the Denver metro area, is located just 8 miles to the west of the site. As a part of this project, the applicant proposes a new driveway connection on Dransfeldt to align with the existing driveway into Lowe's Home Improvement as well as maintaining the existing driveway access onto Lincoln Avenue, that is currently serving the Walgreens pharmacy and a residential parcel.

Table 6-3. Recommended percentage imperviousness values

Land Use or Surface Characteristics	Percentage Imperviousness (%)
Business:	
Downtown Areas	95
Suburban Areas	75
Residential lots (lot area only):	
Single-family	
2.5 acres or larger	12
0.75 – 2.5 acres	20
0.25 – 0.75 acres	30
0.25 acres or less	45
Apartments	75
Industrial:	
Light areas	80
Heavy areas	90
Parks, cemeteries	10
Playgrounds	25
Schools	55
Railroad yard areas	50
Undeveloped Areas:	
Historic flow analysis	2
Greenbelts, agricultural	2
Off-site flow analysis (when land use not defined)	45
Streets:	
Paved	100
Gravel (packed)	40
Drive and walks	90
Roofs	90
Lawns, sandy soil	2
Lawns, clayey soil	2

The proposed Maverik Convenience Store consists of a new, approximately 5950SF store with a fuel canopy that contains up to 6 fuel islands. The parcel is zoned general commercial with restaurant uses most common in the surrounding properties. The development includes shared driveways and access aisles with the proposed development at Parcel __ that has not yet been defined. A separate parcel has been created to contain the regional drainage for the overall master development. The design of that regional detention was prepared by HKS and is referenced in this report.

B: DESCRIPTION OF PROPERTY

The proposed development consists of 1.30 acres of previously undeveloped property within a larger planned commercial/retail development. The site is currently vacant land that was originally intended as agricultural land but has remained unused for many years. Remnants of a former residence will be demolished as part of this proposed development. A site visit by DCI Engineers in June 2021 saw no visible signs of drainage issues or flooding concerns on the property.

An analysis by DCI Engineers indicates a weighted impervious percentage of 75% for the complete build out of the Maverik parcel. Under the criteria of Table 6-3 from Chapter 1 of the UDFCD (Mile High Flood District) Manual, areas within the development were separated into categories. Roofs were assigned an impervious percentage of 90%, sidewalks and asphalt pavements received a 100% percent categorization, and lawn and landscaped areas were assigned a 2% weighted impervious percentage. The resulting calculations can be found in Appendix A. The weighted impervious percentage is below the same weighted percentage for the existing restaurant space, ensuring that the proposed drainage systems do not exceed the regional capacity. A copy of the calculations identifying land uses for the weighted impervious percentage is included in Appendix A.

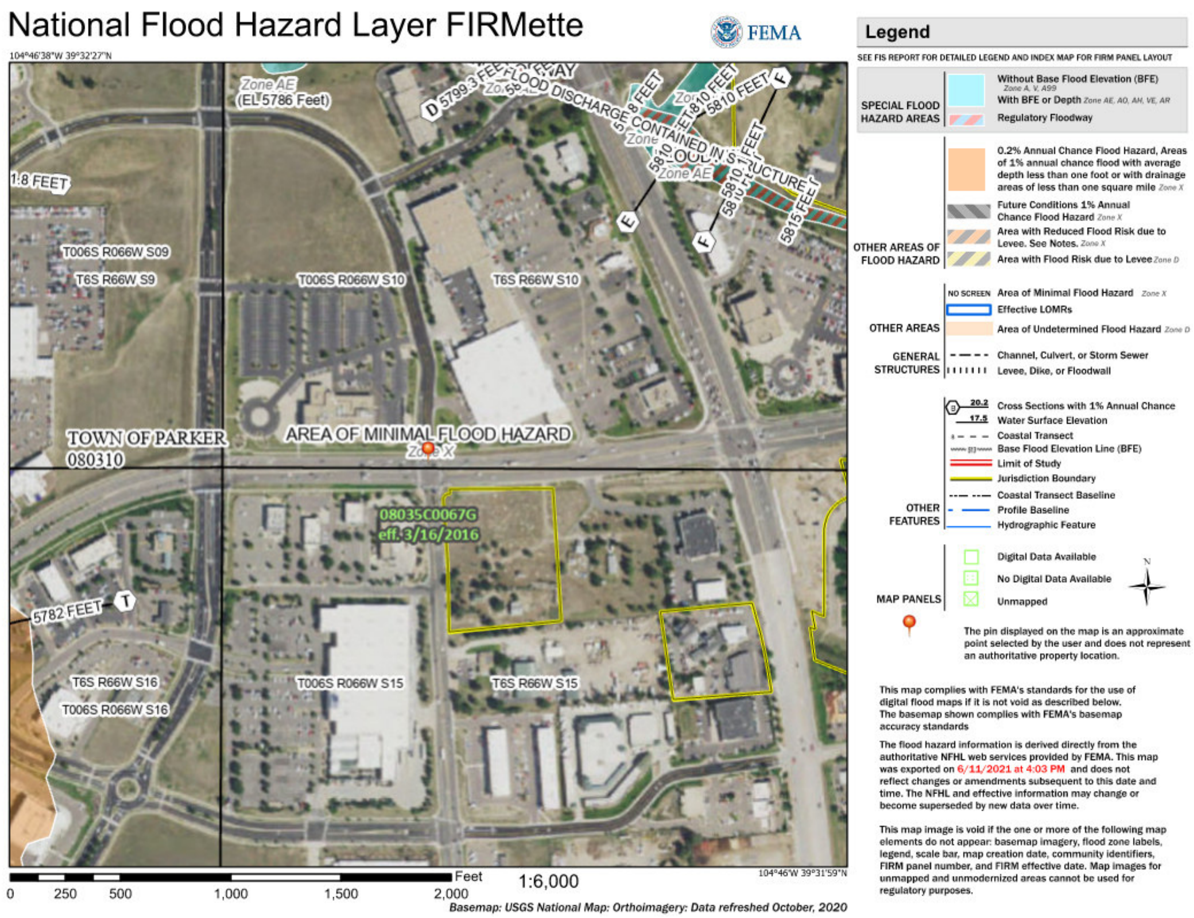
Image 1: Existing Project site facing west along Lincoln

Image 2: Existing site facing north from Dranfeldt

DCI consulted the Natural Resources Conservation Service's (NRCS) Web Soil Survey for a description of local soil types in the area. The soil type across the site is primarily Bresser Sandy Loams loams with slopes in the 1-3% range with a small sample of Sampson Loams. The soils classified as being well-drained and belonging to Hydrologic Group B.

The site lies within the Newlin Gulch basin of the South Platte River (via Cherry Creek). The tributary flows to the northwest through a primarily non-engineered channel (Cherry Creek) as it heads towards its confluence with the South Platte River near downtown Denver.

The proposed Maverik site is located within the area designated as Zone X under FEMA Flood Insurance Rate Map No. 08035C0067G. The map was last modified on March 16, 2016. Zone X indicates that the property is located outside of the are considered to be a special flood hazard zone.



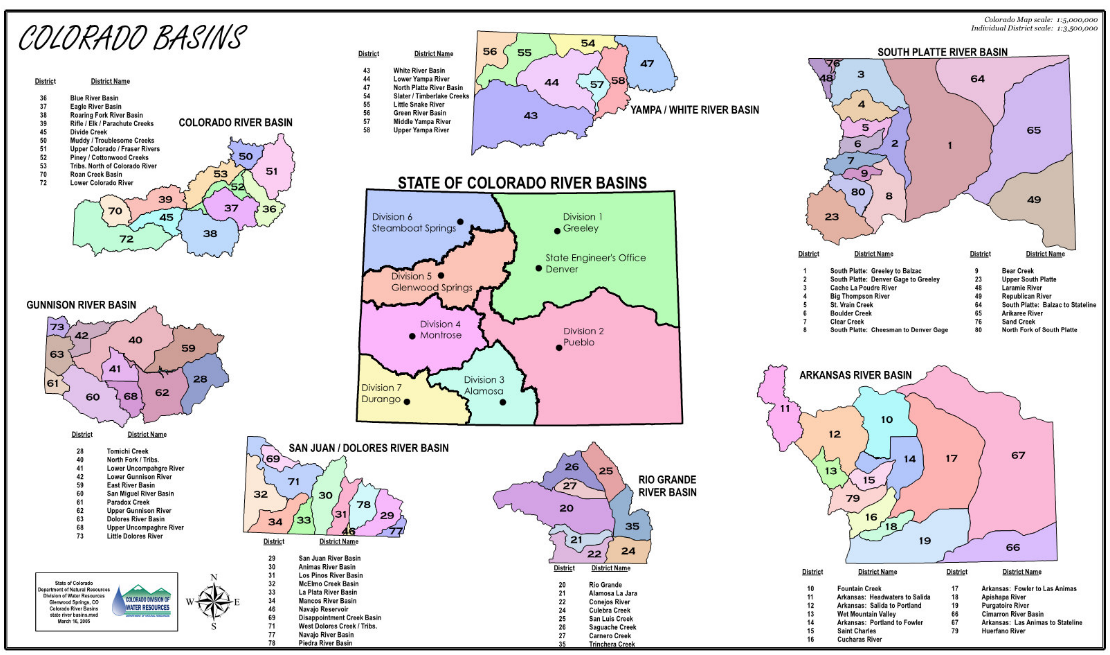
As a part of the proposed development, we do not anticipate any development within a flood plain or any negative effects to the existing floodplain delineations. No additional floodplain permits are required for projects occurring fully outside of the designated floodplain boundaries.

An analysis of the site by DCI Engineers in June of 2021 revealed no signs of current or former irrigation canals or ditches serving the proposed property. There is a drainage swale on the northern boundary of the site that appears to capture runoff from the property and conveys it into the municipal storm drain system at the northwest corner of the proposed development.

SECTION III. DRAINAGE BASINS AND SUB-BASINS

A: MAJOR DRAINAGE BASINS

The State of Colorado is comprised of seven major river basins that are governed by separate divisions within the Colorado Department of Water Resources. The City and County of Denver make up the largest metropolitan city within the South Platte River Basin. The River Basin covers approximately 22,000 square miles in northeastern Colorado and accounts for nearly two-thirds of the state's gross municipal and industrial water demand. Estimates of the total demand for the South Platte River Basin fall between 324,000 and 467,000 acre-feet of water per year.



The South Platte Basin supports a wide range of water needs including municipal, industrial, agricultural as well as important water-dependent ecological and recreational attributes. Coloradoans and tourists regularly enjoy the recreational opportunities provided by the many environmental features of the basin. A South Platte Implementation Plan was developed by HDR in 2015 to identify the unique challenges associated within this river basin.

Within the South Platte River Basin, the site is located within the Newlin Gulch Sub-Basin, a tributary to Cherry Creek. This sub-basin is comprised mostly of a highly urbanized land use with a mix of commercial and residential projects. DCI Engineers analyzed the existing stormwater catchment area using available data from the U.S. Environmental Protection Agency's GeoWaters Viewer. The facility is located within a larger 150-acre catchment that drains into the municipal storm drain system with a discharge into a section of Cherry Creek. Based on Land Use data provided from 2011, this area included approximately 6% High Density Development, 18% Medium Density Development, and 3% Low Density Development. The remaining land uses include a mix of open space, natural waterways, and grasslands. DCI estimated an approximate roughness coefficient of 0.61 for the overall catchment area. In the fully developed condition, we propose a weighted impervious percentage of approximately 75% (per the land use calculations in Appendix A) which is negligible with respect to the greater 150-acre catchment.

Flows from the existing development flow in the northerly and westerly direction towards the existing underground storm drain system with eventual outlet into the Newlin Gulch catchment of Cherry Creek. The project will utilize regional detention to mitigate any increase in flows and maintain the current drainage patterns through the catchment.

B: MINOR DRAINAGE BASINS

Under the proposed improvement plan, this parcel will consist of a 5,950 SF convenience store with six fuel islands under canopy and surface parking. The site will be mostly paved in asphalt with some buffer landscaped areas around the perimeter to match existing grade. At full build-out, the weighted impervious percentage is 75% for the parcel based on the Mile High Flood District Land Use Criteria (Rational Method calculations using UD-Rational software are included in the appendices).

The proposed site has been designed to drain into an underground storm drain system that connects into the regional pond located just west of the proposed parcel. In general, flows will be conveyed over the surface of the asphalt parking lot and along curb and gutter into a Type R inlet at the northeast corner of the site. In addition, the entirety of the fuel canopy is surrounded by a trench drain designed to capture runoff from the parking area before it has the potential to co-mingle with any potential fuel spills underneath the canopies. The intent is to separate "clean" runoff from that which may potentially be contaminated with hydrocarbons. The water from the trench drain will be connected to an oil-water separator to ensure the removal of hydrocarbons before discharging into the detention pond downstream.

The existing site was designed as residential/agricultural use with surface drainage leading in the northwesterly direction towards the existing concrete outfall at the corner of Dransfeldt and Lincoln Avenue. As shown in the photos below, the existing site is currently covered in native grasses with the remains of a small one story residence that will be removed as a part of the project. There is no known underground drainage systems or channelized flow through the site.


Douglas County and the Town of Parker maintain various irrigation channels designed to carry flows to different regions within the county limits. At the time of the proposed development, there are no current *irrigation* channels within the proposed limits of the study. Flows from the surrounding Lincoln Avenue right of way are captured in curb inlets and released into a roadside swale that extends along the northern boundary of this site. Flows are conveyed in this swale to a concrete box culvert that runs beneath Dransfeldt as they make their way westwardly to Cherry Creek. The proposed site does not intend to modify the existing roadside swale in any way.

The off-site flow patterns and paths will not be impacted by the proposed development on the property. The roadside swale discussed above will remain intact for the purpose of draining Lincoln Avenue. Onsite flows will be captured and conveyed into a detention pond before being released at a control rate to the outfall at the Dransfeldt intersection. Similarly, the proposed grades on the site have been designed such that offsite flows will remain within the road-side swale without entering the boundaries of the proposed improvements.

SECTION IV. DRAINAGE DESIGN CRITERIA

A: REGULATIONS

The proposed development was originally part of unincorporated Douglas County, but has since been approved for annexation into the Town of Parker limits. The design of this proposed development is thus subject to the latest stormwater standards of the Town of Parker's Storm Drainage and Environmental Criteria Manual. In accordance with the latest guideline revisions dated February 2014, this project was analyzed for the 5-year and 100-year recurrence intervals.

The 1.30 acre site is well under the threshold required for the use of the Colorado Urban Hydrograph Procedure and thus the site was analyzed using the modified rational method. The proposed area was broken up into smaller sub-drainage basins to represent localized flow-paths and a rational method analysis was conducted on each individual sub-basin using the procedures developed in the Mile High Flood District manual.

The rational method is based on the direct relationship between rainfall and runoff and can be expressed by the equation

$Q = CIA$

In which:

> Q = the maximum rate of runoff (cubic feet per second [cfs])
> C = the runoff coefficient that is the ratio between the runoff volume from an area and the average rainfall depth over a given duration for that area
> I = the average intensity of rainfall for a duration equal to the time of concentration (inches/hour) A = basin area (acres)

The runoff coefficients for each sub-watershed were developed using the UD-Rational Spreadsheet produced by the Mile High Flood District (MHFD/UDFCD). A copy of the Spreadsheet is available in the Appendix.

The Town of Parker utilizes the criteria set forth by the Mile High Flood District (Previously the Urban Drainage Flood Control District) with respect to hydrology and stormwater detention. For sub-basins smaller than 5 acres, the rational method is an accepted method for determining pre and post developed hydrology.

This particular property is a part of a larger master planned development that has been designed to capture and treat the runoff from both the Maverik parcel and Parcel 2 (yet undetermined). The proposed basin was designed by HKS in 2021 as a full spectrum detention facility capable of capturing and treating all proposed runoff from both parcels. Full spectrum detention involves the storage of runoff volumes into three separate volumes. Volume one is defined as the Water Quality Control Volume, as described in Section E of this report. In addition, Volume 2 consists of the Excess Urban Runoff Volume (EURV) minus the WQCV. The third and final volume reflects the 100-year runoff volume, exclusive of the EURV. This project has provided the appropriate flow summaries to the master developer for incorporation into the overall pond design, but does not address the design of the pond. A copy of that design has been included in the Appendices for reference.

B: DRAINAGE STUDIES, OUTFALL SYSTEMS PLANS, SITE CONSTRAINTS

The Highland Ranch Community is a master planned community located wholly within the boundaries of Douglas County. As part of the development, the Highlands Ranch Metro District works with the County and the Mile High Flood District to maintain a system of storm drain structure that serve the community.

This particular site lies within a larger planned commercial development that is served by a regional detention system designed to capture, convey, and treat stormwater flows for the retail parcels prior to discharge into the Dad Clark Gulch system. This property is part of a larger 248 acre catchment that contributes flows into the Dad Clark Gulch.

The proposed development associated with the Prost Biergarten does not impact the existing drainage patterns or flows across the parcel, or the larger regional basin. Due to the nature of the improvements (replacing impervious patio space and surface parking with impervious outdoor seating area), the total impervious percentage is reduced by 1% over the entire 1.92 acre parcel area.

C: HYDROLOGY

The proposed development is subject to the latest stormwater standards of the Town of Parker's Storm Drainage and Environmental Criteria Manual. The project's contributing area, totaling 1.30 acres, was analyzed as a series of individual sub-basins, each under the five-acre threshold for the use of the Colorado Urban Hydrograph Procedure. In accordance with the latest guidelines, this project was analyzed for the 5-year and 100-year recurrence intervals using the Rational Method.

The runoff coefficients for the parcel were developed using the UD-Rational Spreadsheet produced by the MHFD/UDFCD. The spreadsheet analyzes the existing and proposed conditions at the site relative to sub-watershed size, soil types, site slopes, flow paths, and time of concentrations. A copy of the Spreadsheet is available in the Appendix.

The existing parcel is proposed as a one-story convenience store with surface asphalt parking and a single story fuel canopy that acts as additional roof area. The existing parcel contains 77% impervious area, which includes sidewalks/patio space, roof area, and parking. At full build out, the proposed development is anticipated to be approximately 75% impervious (based on a weighted value in accordance with recommended values in UDFCD Table 6-3).

The hydrological analysis for the site was conducted in accordance with the methodology listed in the Town of Parker's Storm Drainage and Environmental Criteria Manual. The project was analyzed for both minor and major storms for commercial projects. A 5 year-1 hour recurrence interval was selected for use as the minor storm event. For purposes of major storm events, the site was analyzed using the rational method for a 100 year-1 hour recurrence interval. The proposed runoff will maintain its existing drainage patterns towards the outfall on the northwest corner of the site, however, it will be conveyed to that point through a series of underground pipes and an extended detention basin to meter the flows prior to discharge.

The basis of storm drainage and hydrologic design for The Town of Parker is found in Chapter 5 of the Storm Drainage and Environmental Criteria Manual. The town is characterized by a single rainfall zone which can be estimated using point rainfall data. Table 5.1 of the Criteria Manual outlines the proposed 1 hour point rainfall for the Town of Parker at various design intervals. The 1-hour duration rainfall depth for various recurrence intervals has historically been used for the calculation of runoff using the Rational or the CUHP method.

TABLE 5.1
ONE-HOUR POINT RAINFALL

Frequency of Design Event (yr)	One-hour Point Rainfall, P_1 (in)
2	0.99
5	1.39
10	1.64
25	1.98
50	2.31
100	2.60

For the purpose of small urban watersheds less than 160 acres in size, a rainfall intensity duration frequncy curve can be used in association with the Rational Method, to determine rainfall intensity distribution over the period of the 1 hour duration equation RS-1 from the Manual decribes the Intensity with the following equation:

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

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

in which:

I = rainfall intensity (inches per hour)

P_1 = 1-hour point rainfall depth (inches)

T_c = time of concentration (minutes)

TOWN OF PARKER STORM DRAINAGE AND ENVIRONMENTAL CRITERIA MANUAL

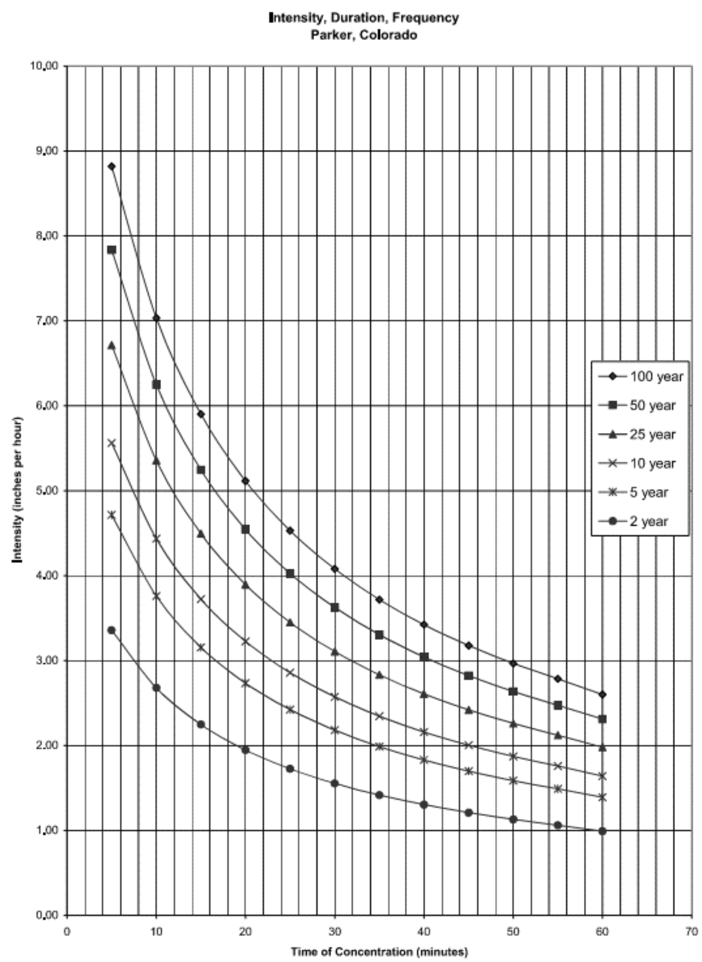


FIGURE 5.1
RAINFALL INTENSITY VERSUS DURATION CURVES FOR PARKER, COLORADO

SECTION 5. HYDROLOGIC CRITERIA 5-5 2014

Both the Town of Parker and Douglas County utilize full spectrum detention to reduce flooding associated with urban development by reducing peak flows across an entire range of storm events up to the 100-year flood event. Full spectrum detention involves the storage of runoff volumes into three separate volumes, depending on the design of the facility. Volume one is defined as the Water Quality Control Volume, as described in Section E of this report. In addition, Volume 2 consists of the Excess Urban Runoff Volume (EURV) minus the

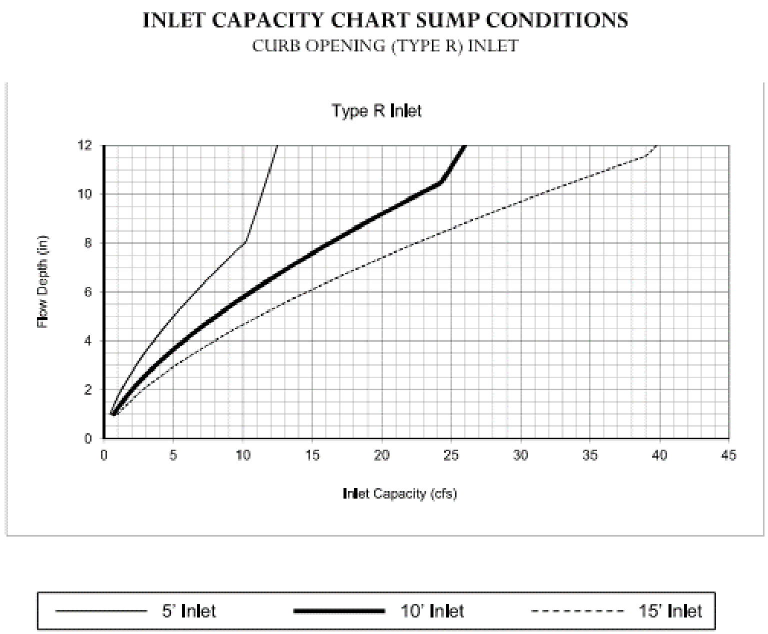
WQCV. The third and final volume reflects the 100-year runoff volume, exclusive of the EURV.

New development and significant redevelopment within the Town of Parker limits are governed by the latest version of the Mile High Flood District Manual and the Town of Parker's Storm Drainage and Environmental Criteria Manual. The drainage system shall account for runoff from both minor and major storm events. The 5-year recurrence interval is utilized as the basis for minor storm events. The design capacity for the development for the major and minor storm events does not include the effects of onsite detention on the peak flows. In all cases, the onsite system has been designed to protect the existing and proposed structures and inhabitants from hazards associated with the 100-year storm event.

This project is being designed as a portion of a greater master planned development with regional detention. The extended detention basin has been sized by HKS according to the flow rates and impervious surfaces presented here. A copy of the pertinent information from the HKS design as relates to this project is included in the Appendix.

D: HYDRAULICS

Typical design of storm drain systems within the Town of Parker utilize the hydraulic loss method to ensure sufficient capacity in the underground systems. Both inlets and piping are sized to adequately convey the 5 year and 100 year storms with a minimum of 1 foot of freeboard between the maximum Hydraulic Grade Line and the finished surface. Type R inlets were selected based on the Town of Parker's Inlet Capacity Chart for sump conditions, shown below.



This project utilizes a combination of overland flow over existing parking lot and underground piping to convey runoff towards the regional detention pond on the west side of the property. Flows travel over parking areas and collect in a series of Type R inlets that collect the runoff into the private underground drainage system, to be maintained by Maverik.

E: WATER QUALITY ENHANCEMENT

The project proposes the continuation of the use of the regional detention pond that is proposed to serve the master commercial development. The detention basin was designed according to the Town's standards for water quality treatment and the design can be found in the document prepared by HKS in 2021-2022. This site is categorized as a tier 3 development based on the Cherry Creek tiered water quality requirements because the total disturbed area is over an acre. Tier 3 water quality requirements will be met per Town of Parker standards (section 8.3): "Tier three new development and redevelopment must install and operate PBMPs that provide WQCV designed and constructed to capture and treat, at a minimum, the 80th percentile runoff event, in accordance with this SDECM and the MANUAL Volume 3".

The Town of Parker falls under the purview of the Phase II Non-Standard Municipal Separate Storm Sewer System (MS4) Permit as administered by the Colorado Water Quality Control Division. The permit coverage, under Permit No. COR-070000, requires the management of stormwater runoff from urban development. In accordance with the MS4

permit for the State of Colorado we assume that this development was originally designed per the guidance of the Mile High Flood District's Urban Storm Drainage Criteria Manual. The Criteria Manual aims to promote a reduction in runoff volume while requiring the treatment of the Water Quality Control Volume prior to the discharge into the municipal storm system.
In addition to the requirements of the Phase II MS4, this project is also subject to the Town's requirements for Grading, Erosion, and Sediment Control as outlined in the County's GESC Manual. A separate GESC Plan has been prepared by DCI Engineers to establish the proposed plan for the protection of the storm drain system from construction sediment loss. Due to the size of the project (over 1 acre), a separate Storm Water Management Plan must be submitted to the Colorado Department of Public Health to comply with the requirements of the NPDES General Construction Permit as well.

SECTION V. STORMWATER MANAGEMENT FACILITY DESIGN

A: STORMWATER CONVEYANCE FACILITIES

The proposed storm drainage for the site has been designed to maintain compliance with the existing drainage patterns on the site. The proposed improvements have been analyzed by DCI with respect to runoff and we have concluded that the effects on the existing drainage system will be negligible due to the controlled discharge from the engineered detention pond to Newlin Gulch.

The existing drainage path for the site consists of overland flow over native grasses to the road-side swale that is located along the northern perimeter of the site. The swale runs east to west and carries runoff from both the proposed site and Lincoln Avenue into a box culvert that runs westwardly under Dransfeldt. A site visit by DCI Engineers on June 14th, 2021 saw little evidence of standing water, though measurable rainfall was recorded on the site the previous day.

At full build out, the proposed project will mimic the existing drainage conditions. Flows will be directed over the surface of the asphalt parking lot towards new Type R curb inlets situated on the east side of the site. Once captured, the flows will be conveyed back towards the proposed detention pond on the parcel west of this one. The runoff will be detained in the extended detention basin where it will collect with other flows from the neighboring parcel. A metered outflow, using a water quality orifice plate will ensure the adequate treatment of runoff with a WQCV drain time of 40 hours.

Existing Basin EX-11 consists of 1.30 Acres that are entirely landscaped with native grasses for a weighted impervious percentage of 2% based on the MHFD tables. Drainage from the site travels northeasterly into the road-side swale that borders the development along the northern boundary. The average slope across the site is approximately 1.5% as it travels from north to south and approximately 2% as it flows from east to west. Design Point #1

Proposed Basin A: Drainage Basin A has an area of 11,712 SF, or about 0.27 acres. The stormwater sheet flows across the parking lot and to an off-site catch basin, named A1 in the report prepared by HKS. This same inlet captures basin RA6. The MHFD inlet capacity spreadsheet for inlet A1 prepared by HKS is included in the Appendix. The MHFD inlet capacity spreadsheet for inlet A1 from Basin A has also been included. The flow capacity required from basin RA6 is 0.5 cfs for 5-year and 1.0 cfs for 100-year events. The flow capacity required from Basin A is 0.8 cfs for 5-year and 1.8 cfs for 100-year events. The capacity for Inlet A1 is 5.3 cfs for both the 5-year and 100-year events. Therefore, the inlet can take the additional flows from the site. The stormwater is then conveyed to the storm water treatment area to the west. Design Point #1.

Proposed Basin B: Drainage Basin B has an area of 21,999 SF, or about 0.51 acres. The storm water sheet flows across the parking lot and to an on-site catch basin, named as Inlet 1. The MHFD inlet capacity spreadsheet for Inlet 1 is included in the Appendix. The flow capacity required for Inlet 1 from Basin B is 1.74 cfs for 5-year and 3.52 cfs for 100-year events. Basin OS1 also drains to this inlet, which has flows of 0.66 cfs for 5-year and 1.58 cfs for 100-year events. The inlet capacity is 2.7 cfs for 5-year and 5.3 cfs for 100-year events. Therefore, the inlet is sufficient to accommodate both basins, Basin B and Basin OS1. The stormwater is then conveyed via 18" pipe through a series of manholes and to the storm water treatment area to the west. Design Point #2.

Proposed Basin C: Drainage Basin C has an area of 6,424 SF, or about 0.15 acres. The storm water is directed from the canopy directly to the underground conveyance system via downspouts. Design Point #3.

Proposed Basin D: Drainage Basin D has an area of 5,949 SF, or about 0.14 acres. The storm water is directed from the building roof directly to the underground conveyance system via downspouts. Design Point #4.

Proposed Basin OS1: Drainage Basin OS1 has an area of 13,927 SF, or about 0.32 acres. This is an off-site basin and is conveyed onsite via sheet flow where it is captured by Inlet 1 as described above. Design Point #5.

Proposed Basin OS2: Drainage basin OS2 has an area of 5,448 SF, or about 0.13 acres. This is an off-site basin and the stormwater is directed away from the building, and sheet flows into the storm water treatment area to the west. Design Point #6.

Proposed Basin OS3: Drainage basin OS3 has an area of 4,233 SF, or about 0.10 acres. This is an off-site basin and the stormwater is directed away from the building, and sheet flows into the storm water treatment area to the west. Design Point #7.

B: STORMWATER STORAGE FACILITIES

Stormwater detention on the property was designed under the original design of the master development, prepared by HKS in 2021. The property was designed to detain flows within an extended detention pond located within Parcel 1 with a connection to the regional system through the box culvert on the northwest corner of the site. Please note that all assumptions made in the report prepared by HKS as pertains to the Maverik Site reflect what is presented in this letter. The imperviousness assumed in the HKS report and subsequent stormwater contribution from the Maverik site are greater than what is presented in this letter. Therefore, the sizing of the storm water treatment facility for the entirety of the development are adequate and no design change is required.

C: WATER QUALITY ENHANCEMENT BMPs

In order to protect the quality of downstream water bodies, the owner shall implement a comprehensive plan for education and outreach to the individual tenants to promote the benefits of enhanced water quality. An operations and maintenance plan has been developed with this Drainage Report and shall outline the goals, procedures, and processes through which the site shall ensure the protection of water quality for downstream users.

D: FLOODPLAIN MODIFICATIONS

The proposed Maverik site is located within the area designated as Zone X under FEMA Flood Insurance Rate Map No. 08035C0067G. The map was last modified on March 16, 2016. Zone X indicates that the property is located outside of the are considered to be a special flood hazard zone. As a result, there is no modifications to existing floodplains being requested, nor is there any proposed work within a regulatory floodway that would require a floodplain development permit.

SECTION E: ADDITIONAL PERMITTING REQUIREMENTS

In addition to permitting through the Town of Parker Land Development Process, projects within the town limits may be subject to additional permit requirements depending on anticipated scope and location.

Section 404 of the Clean Water Act (CWA) establishes a program to regulate the discharge of dredged or fill material into *waters of the United States*, including wetlands. Activities in waters of the United States regulated under this program include fill for development, water resource projects (such as dams and levees), infrastructure development (such as highways and airports) and mining projects. Section 404 requires a permit before dredged or fill material may be discharged into waters of the United States, unless the activity is exempt from Section 404 regulation (e.g., certain farming and forestry activities).

This project is located outside of the established floodplain and there are no signs of established waters of the United States within the project boundaries. DCI analyzed the National Wetland Inventory database provided by the US Fish and Wildlife Service. The nearest emergent wetland is located approximately ½ mile west of the site along the banks of Cherry Creek.

The proposed development is located within the relatively urbanized Newlin Gulch watershed that encompasses a small portion of the Town of Parker. The proposed site is currently situated on one of the few remaining undeveloped parcels within the larger developed portion of the Town infrastructure. We do not anticipate the need for any additional permitting or monitoring as a result of the Endangered Species Act.

SECTION VI: CONCLUSIONS

A: COMPLIANCE WITH STANDARDS

The proposed Drainage Report for Maverik Convenience Store has been designed to comply with the requirements of the Town of Parker Storm Drainage and Environmental Criteria Manual (SDECM) and the Mile High Flood District Manual.

In accordance with the Town of Parker standards, the site was analyzed by DCI Engineers using a Rational Method analysis for the 5year, 1-hour storm event as well as the 100 year-1 hour major storm event. Rainfall intensities were calculated using user defined input from the DCSDDTCM Table 6-1 into the UD-Rational spreadsheet developed by the Mile High Flood District. The proposed improvements increased the overall weighted impervious percentage from the site by 2% compared to the pre-developed condition. The resulting analysis indicates that the overall development will increase the total runoff from the 7.95 acre site in both the minor and major storm events. A summary of the rational method analysis is included in the Appendix.

B: VARIANCES

DCI does not anticipate the need for any variances from Town of Parker standards for this development. The project scope is limited to the replacement of existing impervious area and will result in a total marginal reduction in the overall runoff. Existing flows from the site are conveyed into a regional detention pond that serves as stormwater detention for the larger commercial development, and as a result will not be impacted by the proposed project.

C: DRAINAGE CONCEPT

The overall drainage concept for the proposed project will have an increase in developed flows. The project scope will capture these flows and direct them through a series of storm sewer pipes and structures and direct those developed flows to the master developed detention pond adjacent to this property. The total impervious percentage is anticipated to increase from 2% to 75%.

Drainage on the site in both the existing and proposed conditions flows from east to west across the surface. Water is conveyed in a series of concrete gutters and ribbon gutters to a final discharge point at a curb inlet at the northeast end of the parking lot. Beyond the curb inlet, water is conveyed through storm sewer pipes to an existing storm manhole at the southwest entrance to the drive isle where it combines with flows from adjacent parcels into a detention system designed by HKS.

SECTION VII: REFERENCES

FEMA DFIRM Map No 08035C0028F and 08035C0036F. Federal Emergency Management Agency, September 30, 2005.

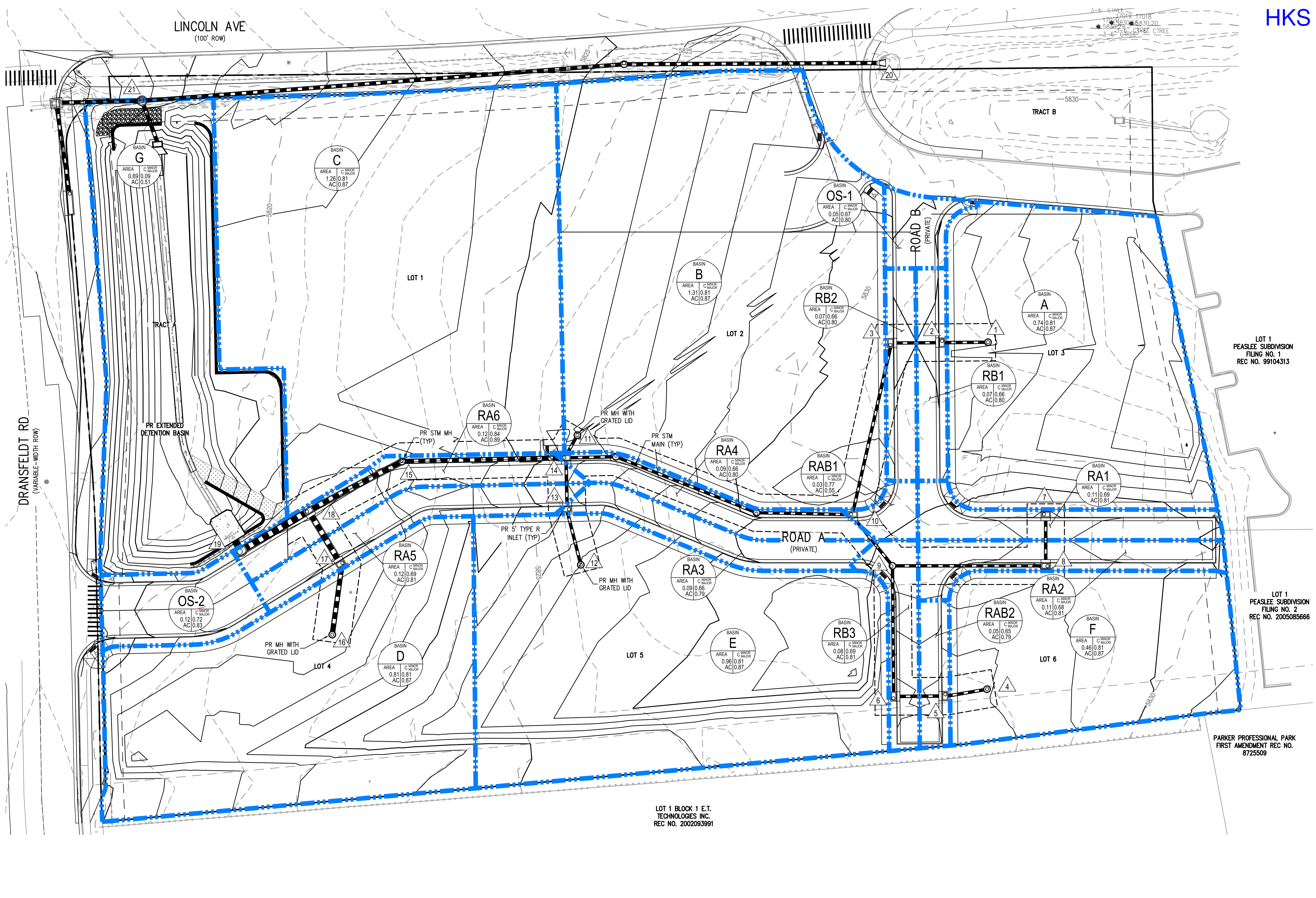
Douglas County Storm Drainage Design and Technical Criteria Manual; Douglas County; Revised May 2019.

National Wetlands Inventory Online Mapper; US Fish and Wildlife Service; April 2021

Urban Storm Drainage Criteria Manual-Volumes 1-3; Urban Drainage and Flood Control District (UDFCD); Revised March 2017.

SECTION VIII: APPENDICES

HKS DRAINAGE MAP



DIRECT RUNOFF SUMMARY TABLE			
BASIN	AREA (AC)	Q_5 (CFS)	Q_{100} (CFS)
A	0.74	2.79	5.67
B	1.31	4.73	9.64
C	1.26	4.60	9.36
D	0.81	2.84	5.79
E	0.96	3.30	6.72
F	0.46	1.73	3.52
G	0.69	0.22	2.34
RAB1	0.03	0.11	0.22
RAB2	0.05	0.14	0.32
RB1	0.07	0.20	0.47
RB2	0.07	0.20	0.47
RB3	0.08	0.25	0.55
RA1	0.11	0.34	0.76
RA2	0.11	0.34	0.77
RA3	0.09	0.29	0.66
RA4	0.09	0.29	0.66
RA5	0.12	0.37	0.83
RA6	0.12	0.48	0.97
OFF-SITE			
OS-1	0.05	0.16	0.35
OS-2	0.12	0.40	0.88

NO CHANGES ARE TO BE MADE TO THIS DRAWING WITHOUT WRITTEN PERMISSION OF HARRIS KOCHER SMITH

FILEPATH: K:\200829\ENGINEERING\DRAINAGE\DRAINAGE PLAN.DWG LAYOUT: LAYOUT1
6 XREFs: e-base, e-legal, e-util, p-base, p-legal, p-util
PLOTTED: THU 10/20/22 2:46:12P BY: BRAD WEAVERLING

Know what's below. Call before you dig.

SCALE: 1" = 30'

DESIGNED BY: ORM
CHECKED BY: RCP
DRAWN BY: ORM

ISSUE DATE: 07-30-2021

DATE	REVISION COMMENTS
11-19-2021	PER TOWN OF PARKER COMMENTS
04-08-2022	PER TOWN OF PARKER COMMENTS
07-21-2022	PER TOWN OF PARKER COMMENTS
10-20-2022	PER TOWN OF PARKER COMMENTS

HARRIS KOCHER SMITH
1120 Lincoln Street, Suite 1000
Denver, Colorado 80203
P: 303.623.6300 F: 303.623.6311
HarrisKocherSmith.com

PLAZA STREET PARTNERS

LINCOLN & DRANSFELDT
DRAINAGE MAP

PROJECT #: 200829
SHEET NUMBER
1
1 OF 1

HKS NARRATIVE

The proposed drainage will follow these existing patterns. There are proposed widening improvements along Dransfeldt Road. The existing drainage patterns on Dransfeldt Road will be maintained and will be routed to the existing storm infrastructure, which be ultimately routed to the existing downstream drainage facility.

The following section describes in detail the on-site drainage basins.

Basin A (0.74 acres) consists of commercial area, with an assumed impervious of 95%. Runoff from Basin A will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. The runoff will be connected to the proposed storm system at Design Point 1.

Basin B (1.31 acres) consists of commercial area, with an assumed impervious of 95%. Runoff from Basin B will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. A proposed manhole with a grated lid has been provided for a future connection. The runoff will be connected to the proposed storm system at Design Point 11.

Basin C (1.26 acres) consists of commercial area, with an assumed impervious of 95%. Runoff from Basin C will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. The runoff will be connected to the proposed storm system at Design Point 15.

Basin D (0.81 acres) consists of commercial area, with an assumed impervious of 95%. Runoff from Basin D will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. A proposed manhole with a grated lid has been provided for a future connection. The runoff will be connected to the proposed storm system at Design Point 16.

Basin E (0.96 acres) consists of commercial area, with a calculated impervious of 95%. Runoff from Basin E will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. A proposed manhole with a grated lid has been provided for a future connection. The runoff will be connected to the proposed storm system at Design Point 12.

Basin F (0.46 acres) consists of commercial area, with an assumed impervious of 95%. Runoff from Basin F will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. A proposed manhole with a grated lid has been provided for a future connection. The runoff will be connected to the proposed storm system at Design Point 4.

Basin G (0.69 acres) consists of the proposed extended detention basin, with an assumed impervious of 7%. Runoff from this basin will be routed from the outlet structure to the existing storm infrastructure.

Basin RAB1 (0.03 acres) consists of portions of proposed Roads A & B, with a calculated impervious of 90%. Runoff from Basin RAB1 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet A6 at Design Point 10.

Basin RAB2 (0.05 acres) consists of portions of proposed Road B, with a calculated impervious of 75%. Runoff from Basin RAB2 will runoff to the proposed curb and gutter and will be routed to the proposed

HKS NARRATIVE

detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet D3 at Design Point 5.

Basin RB1 (0.07 acres) consists of a portion of proposed Road B, with a calculated impervious of 76%. Runoff from Basin RB1 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet F2 at Design Point 2.

Basin RB2 (0.07 acres) consists of a portion of proposed Road B, with a calculated impervious of 76%. Runoff from Basin RB2 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet F1 at Design Point 3.

Basin RB3 (0.08 acres) consists of a portion of proposed Roads A & B, with a calculated impervious of 80%. Runoff from Basin RB3 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet D2 at Design Point 6.

Basin RA1 (0.11 acres) consists of a portion of proposed Road A, with a calculated impervious of 80%. Runoff from Basin RA1 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet J2 at Design Point 7.

Basin RA2 (0.11 acres) consists of a portion proposed Road A, with a calculated impervious of 79%. Runoff from Basin RA2 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet J1 at Design Point 8.

Basin RA3 (0.09 acres) consists of a portion of proposed Road A, with a calculated impervious of 76%. Runoff from Basin RA3 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet C1 at Design Point 13.

Basin RA4 (0.09 acres) consists of a portion of proposed Road A, with a calculated impervious of 76%. Runoff from Basin RA4 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet A3 at Design Point 14.

Basin RA5 (0.12 acres) consists of a portion of proposed Road A, with a calculated impervious of 79%. Runoff from Basin RA5 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet B1 at Design Point 17.

Basin RA6 (0.12 acres) consists of a portion of proposed Road A, with a calculated impervious of 99%. Runoff from Basin RA6 will runoff to the proposed curb and gutter and will be routed to the proposed detention basin via the proposed storm infrastructure. Runoff from this basin will be captured by Inlet A1 at Design Point 18.

HKS CALCULATIONS

Project Name:	Lincoln & Dransfeldt
	Composite C-Value Computations
	Post-Development
Project No:	200829
Date:	07/30/21
Revised:	10/19/22
Design by:	ORM
Checked by:	RCP
NRCS Soil Group	C/D

BASIN	TOTAL AREA (ACRES)	ROOFS (90%)	CONCRETE DRIVES & WALKS (90%)	STREETS (100%)	LANDSCAPE AREA (2%)	PERCENT IMPERVIOUS	C_5=	C_{100}=
A	0.74					95%	0.81	0.87
B	1.31					95%	0.81	0.87
C	1.26					95%	0.81	0.87
D	0.81					95%	0.81	0.87
E	0.96					95%	0.81	0.87
F	0.46					95%	0.81	0.87
G	0.69	0.00	0.04	0.00	0.65	7%	0.09	0.51
RAB1	0.03	0.00	0.00	0.02	0.00	90%	0.77	0.85
RAB2	0.05	0.00	0.00	0.03	0.01	75%	0.65	0.79
RB1	0.07	0.00	0.00	0.05	0.02	76%	0.66	0.80
RB2	0.07	0.00	0.00	0.05	0.02	76%	0.66	0.80
RB3	0.08	0.00	0.00	0.06	0.02	80%	0.69	0.81
RA1	0.11	0.00	0.00	0.08	0.02	80%	0.69	0.81
RA2	0.11	0.00	0.00	0.08	0.02	79%	0.68	0.81
RA3	0.09	0.00	0.00	0.07	0.02	76%	0.66	0.79
RA4	0.09	0.00	0.00	0.07	0.02	76%	0.66	0.80
RA5	0.12	0.00	0.00	0.09	0.02	79%	0.69	0.81
RA6	0.12	0.00	0.01	0.11	0.00	99%	0.84	0.89
OFF-SITE TO EX POND								
EX-1	0.43	0.00	0.00	0.17	0.26	41%	0.37	0.65
EX-2	2.38					100%	0.86	0.89
OFF-SITE RUNOFF								
OS-1	0.05	0.00	0.00	0.04	0.01	77%	0.67	0.80
OS-2	0.12	0.00	0.01	0.09	0.02	84%	0.72	0.83
Total Runoff Off-Site	0.17							
Total Treated	**7.15**					**81%**	**0.70**	**0.82**

*Percent Impervious for Basins A-F determined from USDCM Vol 1 Table 6-3

Runoff Coefficient Equations (from USDCM Vol. 1)

NCRS Soil Group	C/D
5-year	0.82(i)+0.035
100-year	0.41(i)+0.484

HKS CALCULATIONS

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RA3	0.09	0.29	0.66
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OFF-SITE			
OS-1	0.05	0.16	0.35
OS-2	0.12	0.40	0.88

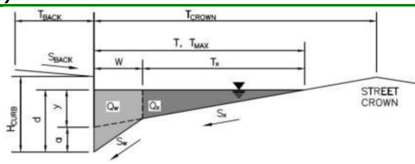
HKS CALCULATIONS

MHFD-Inlet, Version 5.01 (April 2021)

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

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

Project: LINCOLN & DRANSFELDT
Inlet ID: INLET A1 (BASIN RA6)



Gutter Geometry:

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	5.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =	0.015	ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =	0.013	
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	18.5	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	18.5	18.5	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

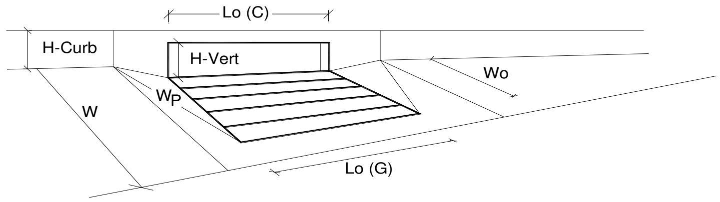
MINOR STORM Allowable Capacity is based on Depth Criterion
MAJOR STORM Allowable Capacity is based on Depth Criterion

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

HKS CALCULATIONS

INLET IN A SUMP OR SAG LOCATION

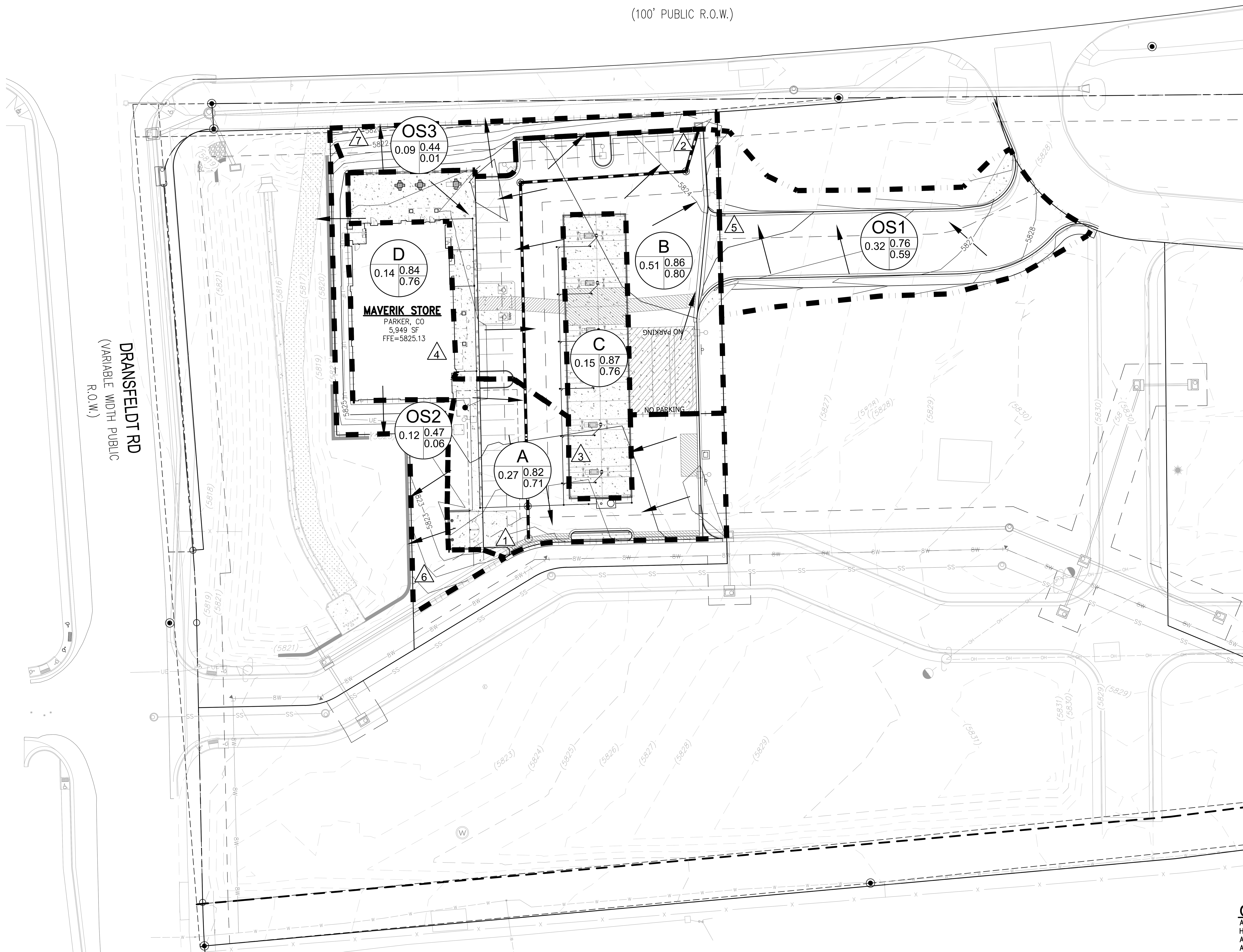
MHFD-Inlet, Version 5.01 (April 2021)



Design Information (Input)	CDOT Type R Curb Opening		MINOR	MAJOR	
Type of Inlet		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	6.0	inches
Grate Information			MINOR	MAJOR	☐ Override Depths
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
Area Opening 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) =	5.00	5.00	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	
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	0.33	ft
Combination Inlet Performance Reduction Factor for Long Inlets		$RF_{Combination}$ =	0.76	0.76	
Curb Opening Performance Reduction Factor for Long Inlets		RF_{Curb} =	1.00	1.00	
Grated Inlet Performance Reduction Factor for Long Inlets		RF_{Grate} =	N/A	N/A	
			MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)		$\mathbf{Q_a}$ =	**5.3**	**5.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms(>Q PEAK)		$Q_{PEAK\ REQUIRED}$ =	0.5	1.0	cfs

LINCOLN PROFESSIONAL PARK FILING NO. 1 LOT 1
MAVERIK INC. STORE

SITUATED IN THE NORTHWEST $\frac{1}{4}$ OF SECTIOON 15,
TOWNSHIP 6 SOUTH, RANGE 66 WEST OF THE 6TH P.M.
TOWN OF PARKER, COUNTY OF DOUGLAS, STATE OF COLORADO



DRAINAGE LEGEND

- BASIN ID
- 100–YEAR COMPOSITE RUNOFF COEFFICIENT
- 5–YEAR COMPOSITE RUNOFF COEFFICIENT
- BASIN SIZE (ACRES)
- DRAINAGE FLOW DIRECTION
- DESIGN POINT
- DRAINAGE BOUNDARY (EXISTING CONDITION)
- DRAINAGE BOUNDARY (PROPOSED CONDITIONS)
- 100YR WSE
- EXIST. MINOR CONTOUR
- 5950 EXIST. MAJOR CONTOUR
- PROP. MINOR CONTOUR
- 5950 PROP. MAJOR CONTOUR

N

GRAPHIC SCALE

30 0 10 30

(IN FEET)
1 inch = 30 ft.

VERTICAL DATUM/BENCHMARK:

BENCHMARK IS THE NW CORNER SECTION 15, T6S S R66W, BEING A 3.5" ALUMINUM CAP LS 19003 1999 IN RANGE BOX,

ELEVATION=5806.79' NAVD 88.

CONTRACTOR NOTE:

ALL EXISTING UTILITIES SHOWN ON PLANS ARE TO BE VERIFIED HORIZONTALLY AND VERTICALLY PRIOR TO ANY CONSTRUCTION. ALL EXISTING FEATURES INCLUDING BURIED UTILITIES ARE SHOWN AS INDICATED ON RECORD MAPS AND SURVEYS FURNISHED BY OTHERS. WE ASSUME NO LIABILITY FOR THE ACCURACY OF THOSE RECORDS AND SURVEYS. CONTACT THE UTILITY OWNER/AGENCY FOR THE FINAL LOCATION OF EXISTING UTILITIES IN AREAS CRITICAL TO CONSTRUCTION.

COLORADO 811

PROJECT NUMBER

21-122-0001

ISSUE DATE:

OCTOBER 26, 2022

REVISIONS:

No.	Date	Description
.	.	.

NOTE:
Bid documents should not be separated or issued as partial sets to subcontractors. Bidders are responsible for all portions of the documents that pertain to work covered by sub-bids. Bidder assumes full responsibility for error or misinterpretations resulting from partial sets of Bidding Documents by itself or any sub-bidder.

Conflicting information or errors found in the construction documents should be brought to the attention of the architect immediatly. In the event of a conflict in the drawings, bidder should not assume the least expensive option will meet the project requirements.

MAVERIK

MAVERIK INC. STORE
LINCOLN AVE. AND DRANSFELDT RD.
PARKER, CO

DCI ENGINEERS
1331 17TH STREET • SUITE 605
DENVER, COLORADO 80202
PHONE: (720) 439-4700
WEBSITE: www.dci-engineers.com
CIVIL / STRUCTURAL

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DRAINAGE PLAN

Plot Date: 2023-03-09 File Location: O:\1200-Denver\DCI-Civil\Projects\2021\21122-0001 Maverik Lincoln and Dransfeldt\21-122-0001DRN.dwg

PROPOSED LAND USE CALCULATIONS

DCI Engineers
1331 17th Street
Suite 605
Denver, Colorado 80202

Maverik Lincoln and Dransfeldt
Maverik Inc.
Lincoln Avenue and Dransfeldt
Parker, CO

PROPOSED LAND USE CALCULATIONS																	
Design Point	Tributary Basins	Landscape Area (SF)	Artificial Turf (SF)	Pvmt (SF)	Roof (SF)	Total Contributing Area (Ac)	Landscape (Ac)	Runoff Coeff	Artificial Turf (Ac)	Runoff Coeff	Pavement (Ac)	Runoff Coeff	Roof (Ac)	Runoff Coeff	Weighted Impervious %	Discharges To:
1	A	1914	0	9798	0	0.27	0.04	2.00	0.00	60.00	0.22	100.00	0.00	90.00	84.0	Pavement surface flow to Offsite CB "Inlet A1"
2	B	1385	0	20614	0	0.51	0.03	2.00	0.00	60.00	0.47	100.00	0.00	90.00	93.8	Pavement surface flow to Onsite CB "Inlet 1"
3	C	0	0	0	6424	0.15	0.00	2.00	0.00	60.00	0.00	100.00	0.15	90.00	90.0	Fueling canopy to downspouts
4	D	0	0	0	5949	0.14	0.00	2.00	0.00	60.00	0.00	100.00	0.14	90.00	90.0	Store roof to downspouts
5	OS1	4141	0	9786	0	0.32	0.10	2.00	0.00	60.00	0.22	100.00	0.00	90.00	70.9	Offsite pavement surface flow to Onsite CB "Inlet 1"
6	OS2	5063	0	376	0	0.12	0.12	2.00	0.00	60.00	0.01	100.00	0.00	90.00	8.8	Offsite surface flow to swale
7	OS3	4022	0	0	0	0.09	0.09	2.00	0.00	60.00	0.00	100.00	0.00	90.00	2.0	Offsite surface flow to swale
	Sum:					1.59										

Total Weighted Impervious Area:

Sum of Area, A:	1.59		
Sum of Landscape Areas:	0.4	x 2%	0.007587236
Sum of Pavement:	0.93	x 100%	0.931450872
Sum of Artificial Turf	0.00	x60%	0
Sum of Rooftop:	0.28	x 90%	0.255640496
Sum / Total Area	1.594857668		0.749

PROPOSED RUNOFF SUMMARY TABLE

Washington
Oregon
California
Texas
Alaska
Colorado
Montana
www.dci-engineers.com

DCI Engineers
1331 17th Street
Suite 605
Denver, Colorado 80202

Maverik Lincoln and Dransfeldt
Maverik Inc.
Lincoln Avenue and Dransfeldt
Parker, CO

PROPOSED CONDITIONS RUNOFF SUMMARY TABLE						
Design Point	**Tributary Basins**	**Contributing Area (Ac)**	**Weighted Impervious %**	**5 Yr Peak Discharge (CFS)**	**100 Yr Peak Discharge (CFS)**	**Discharges To:**
1	A	0.27	83.98	0.83	1.80	Pavement surface flow to Offsite CB "Inlet A1"
2	B	0.51	93.83	1.74	3.52	Pavement surface flow to Onsite CB "Inlet 1"
3	C	0.15	90.00	0.54	1.12	Fueling canopy to downspouts
4	D	0.14	90.00	0.50	1.04	Store roof to downspouts
5	OS1	0.32	70.86	0.66	1.58	Offsite pavement surface flow to Onsite CB "Inlet 1"
6	OS2	0.12	8.77	0.03	0.39	Offsite surface flow to swale
7	OS3	0.09	2.00	0.00	0.28	Offsite surface flow to swale
		TOTAL	**74.91%**	**4.31**	**9.73**	

PROPOSED RATIONAL METHOD CALCULATIONS

Calculation of Peak Runoff using Rational Method

Designer: Connor Collins
Company: DCI Engineers
Date: 10/25/2022
Project: Maverik Lincoln and Dransfeldt
Location: Parker, CO

Version 2.00 released May 2017

Cells of this color are for required user-input
Cells of this color are for optional override values
Cells of this color are for calculated results based on overrides

$$t_i = \frac{0.395(1.1 - C_5)\sqrt{L_i}}{S_i^{0.33}}$$

$$t_t = \frac{L_t}{60K\sqrt{S_t}} = \frac{L_t}{60V_t}$$

$$\text{Computed } t_c = t_i + t_t$$

$$\text{Regional } t_c = (26 - 17i) + \frac{L_t}{60(14i + 9)\sqrt{S_t}}$$

$t_{minimum} = 5$ (urban)
$t_{minimum} = 10$ (non-urban)

$$\text{Selected } t_c = \max\{t_{minimum}, \min(\text{Computed } t_c, \text{Regional } t_c)\}$$

Select UDFCD location for NOAA Atlas 14 Rainfall Depths from the pulldown list OR enter your own depths obtained from the NOAA website (click this link)

	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
1-hour rainfall depth, P1 (in) =	0.99	1.39	1.64	1.98	2.31	2.60	3.14

	a	b	c
Rainfall Intensity Equation Coefficients =	28.50	10.00	0.786

$$I(in/hr) = \frac{a * P_1}{(b + t_c)^c}$$

$$Q(cfs) = CIA$$

Subcatchment Name	Area (ac)	NRCS Hydrologic Soil Group	Percent Imperviousness	Runoff Coefficient, C 2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
A	0.27	B	84.0	0.68	0.71	0.73	0.78	0.80	0.82	0.84
B	0.51	B	93.8	0.77	0.80	0.81	0.84	0.85	0.86	0.88
C	0.15	B	90.0	0.74	0.76	0.78	0.81	0.83	0.84	0.87
D	0.14	B	90.0	0.74	0.76	0.78	0.81	0.83	0.84	0.87
OS1	0.32	B	70.9	0.56	0.59	0.63	0.69	0.72	0.76	0.80
OS2	0.12	B	8.8	0.05	0.06	0.13	0.30	0.38	0.47	0.57
OS3	0.09	B	2.0	0.01	0.01	0.07	0.26	0.34	0.44	0.54

Subcatchment Name	Overland (Initial) Flow Time: Overland Flow Length L_i (ft)	U/S Elevation (ft) (Optional)	D/S Elevation (ft) (Optional)	Overland Flow Slope S_i (ft/ft)	Overland Flow Time t_i (min)	Channelized (Travel) Flow Time: Channelized Flow Length L_t (ft)	U/S Elevation (ft) (Optional)	D/S Elevation (ft) (Optional)	Channelized Flow Slope S_t (ft/ft)	NRCS Conveyance Factor K	Channelized Flow Velocity V_t (ft/sec)	Channelized Flow Time t_t (min)
A	137.00			0.020	6.57	0.00			0.010	20	2.00	0.00
B	215.00			0.015	6.96	0.00			0.010	20	2.00	0.00
C	20.00			0.010	2.71	0.00			0.010	20	2.00	0.00
D	20.00			0.010	2.71	0.00			0.010	20	2.00	0.00
OS1	220.00			0.015	11.96	0.00			0.010	20	2.00	0.00
OS2	25.00			0.250	3.24	0.00			0.010	20	2.00	0.00
OS3	40.00			0.167	4.91	0.00			0.010	20	2.00	0.00

Subcatchment Name	Time of Concentration: Computed t_c (min)	Regional t_c (min)	Selected t_c (min)
A	6.57	11.72	6.57
B	6.96	10.05	6.96
C	2.71	10.70	5.00
D	2.71	10.70	5.00
OS1	11.96	13.95	11.96
OS2	3.24	24.50	10.00
OS3	4.91	25.66	10.00

Subcatchment Name	Rainfall Intensity, I (in/hr) 2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr	Peak Flow, Q (cfs) 2-yr	5-yr	10-yr	25-yr	50-yr	100-yr	500-yr
A	3.10	4.36	5.14	6.21	7.24	8.15	9.85	0.57	0.83	1.02	1.30	1.56	1.80	2.24
B	3.05	4.28	5.05	6.10	7.11	8.01	9.67	1.20	1.74	2.10	2.61	3.09	3.52	4.34
C	3.36	4.71	5.56	6.72	7.84	8.82	10.65	0.37	0.54	0.65	0.82	0.98	1.12	1.38
D	3.36	4.71	5.56	6.72	7.84	8.82	10.65	0.35	0.50	0.61	0.77	0.91	1.04	1.29
OS1	2.49	3.49	4.12	4.98	5.81	6.54	7.89	0.44	0.66	0.83	1.11	1.34	1.58	2.01
OS2	2.68	3.76	4.44	5.36	6.25	7.03	8.50	0.02	0.03	0.07	0.20	0.28	0.39	0.58
OS3	2.68	3.76	4.44	5.36	6.25	7.03	8.50	0.00	0.00	0.03	0.13	0.19	0.28	0.42

PROPOSED INLETS INPUT DATA

MHFD-Inlet, Version 5.02 (August 2022)

INLET MANAGEMENT

Worksheet Protected

INLET NAME	Inlet 1	Inlet A1
Site Type (Urban or Rural)		
Inlet Application (Street or Area)	STREET	STREET
Hydraulic Condition	In Sump	In Sump
Inlet Type	CDOT Type R Curb Opening	CDOT Type R Curb Opening

USER-DEFINED INPUT

User-Defined Design Flows

Minor Q_{Known} (cfs)	2.4	0.8
Major Q_{Known} (cfs)	5.1	1.8

Bypass (Carry-Over) Flow from Upstream

Inlets must be organized from upstream (left) to downstream (right) in order for byp

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

Watershed Characteristics

Subcatchment Area (acres)	0.75	0.27
Percent Impervious	92.6	93.8
NRCS Soil Type	B	B

Watershed Profile

Overland Slope (ft/ft)	0.015	0.020
Overland Length (ft)	215	137
Channel Slope (ft/ft)	0.000	0.000
Channel Length (ft)	0	0

Minor Storm Rainfall Input

Design Storm Return Period, T_r (years)	5	5
One-Hour Precipitation, P_1 (inches)	1.39	1.39

Major Storm Rainfall Input

Design Storm Return Period, T_r (years)	100	100
One-Hour Precipitation, P_1 (inches)	2.6	2.6

CALCULATED OUTPUT

Minor Total Design Peak Flow, Q (cfs)	**2.4**	**0.8**
Major Total Design Peak Flow, Q (cfs)	**5.1**	**1.8**
Minor Flow Bypassed Downstream, Q_b (cfs)	N/A	N/A
Major Flow Bypassed Downstream, Q_b (cfs)	N/A	N/A

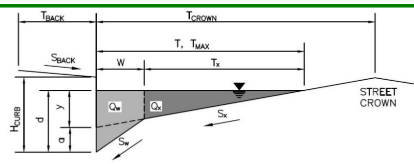
PROPOSED INLET 1 CAPACITY

MHFD-Inlet, Version 5.02 (August 2022)

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

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

Project:
Inlet ID: **Inlet 1**



Gutter Geometry:

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	0.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =		ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =		
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	215.0	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.015	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.013	

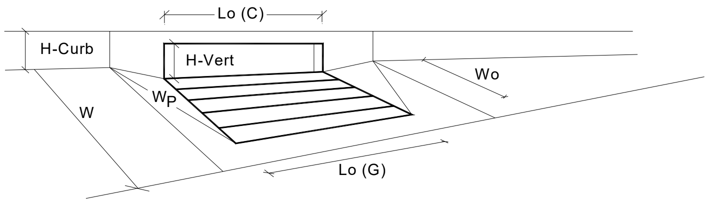
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	16.0	24.0	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	6.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is not applicable to Sump Condition MAJOR STORM Allowable Capacity is not applicable to Sump Condition	Q_{allow} =	**SUMP**	**SUMP**	**cfs**

PROPOSED INLET 1 CAPACITY

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 =	4.5	6.0	inches
Grate Information		MINOR	MAJOR	Override Depths
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) =	5.00	5.00	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	
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} =	0.21	0.33	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} =	1.00	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	Q_a =	**2.7**	**5.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)	$Q_{PEAK\ REQUIRED}$ =	2.4	5.1	cfs

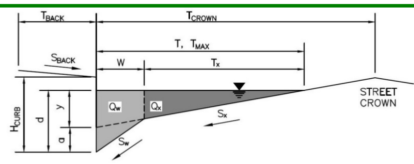
PROPOSED INLET A1 CAPACITY

MHFD-Inlet, Version 5.02 (August 2022)

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

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

Project:
Inlet ID: **Inlet A1**



Gutter Geometry:

Maximum Allowable Width for Spread Behind Curb	T_{BACK} =	0.0	ft
Side Slope Behind Curb (leave blank for no conveyance credit behind curb)	S_{BACK} =		ft/ft
Manning's Roughness Behind Curb (typically between 0.012 and 0.020)	n_{BACK} =		
Height of Curb at Gutter Flow Line	H_{CURB} =	6.00	inches
Distance from Curb Face to Street Crown	T_{CROWN} =	18.5	ft
Gutter Width	W =	2.00	ft
Street Transverse Slope	S_X =	0.020	ft/ft
Gutter Cross Slope (typically 2 inches over 24 inches or 0.083 ft/ft)	S_W =	0.083	ft/ft
Street Longitudinal Slope - Enter 0 for sump condition	S_O =	0.000	ft/ft
Manning's Roughness for Street Section (typically between 0.012 and 0.020)	n_{STREET} =	0.016	

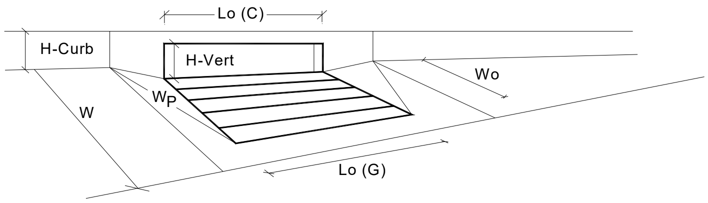
		Minor Storm	Major Storm	
Max. Allowable Spread for Minor & Major Storm	T_{MAX} =	18.5	18.5	ft
Max. Allowable Depth at Gutter Flowline for Minor & Major Storm	d_{MAX} =	6.0	12.0	inches
Check boxes are not applicable in SUMP conditions		☐	☐	

		Minor Storm	Major Storm	
MINOR STORM Allowable Capacity is not applicable to Sump Condition MAJOR STORM Allowable Capacity is not applicable to Sump Condition	**Q_{allow} =**	**SUMP**	**SUMP**	**cfs**

PROPOSED INLET A1 CAPACITY

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	6.0	inches
Grate Information		MINOR	MAJOR	Override Depths
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) =	5.00	5.00	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	
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	0.33	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} =	1.00	1.00	
Combination Inlet Performance Reduction Factor for Long Inlets	$RF_{Combination}$ =	N/A	N/A	
		MINOR	MAJOR	
Total Inlet Interception Capacity (assumes clogged condition)	**Q_a =**	**5.3**	**5.3**	**cfs**
Inlet Capacity IS GOOD for Minor and Major Storms (>Q Peak)	$Q_{PEAK\ REQUIRED}$ =	0.8	1.8	cfs

5-yr Energy Grade Line and Hydraulic Grade Line

Instructions: *Enter values in blue boxes. Spreadsheet calculates values in yellow boxes*

NOTE: Point 1A must be the outfall or else the most downstream point for the length of storm sewer for which the HGL and EGL are being calculated.

Pipe$_{\text{LINE 1}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s10} =	5822.54	ft
Pipe Diameter, D_{10} =	18	in	Surface Elevation, E_{s11} =	5821.95	ft
Invert Elevation, E_{i10} =	5817.63	ft	Length of Pipe$_{10\text{-}11}$, $L_{10\text{-}11}$ =	19.2	ft
Invert Elevation, E_{i11} =	5817.53	ft	Stormwater Flow Rate, Q_{10} =	3.44	cfs
Manning Roughness, n_{full} =	0.013				

Calculation of Normal Depth in Pipe$_{10\text{-}11}$

Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	31.99	ft
Pipe Slope, $S_{10\text{-}11}$ =	0.0052	ft/ft	Normal Depth of Flow, $y_{10\text{-}11}$ =	9.65	in
Cross-Sect. Area, A =	0.96	ft^2	Depth of Flow/Diam, y/D =	0.54	
Wetted Perimeter, P =	2.46	ft	Manning Roughness, n =	0.0161	
Hydraulic Radius, R =	0.39	ft	Flow Velocity, V_{11} =	3.57	ft/sec

NOTE1: The normal depth for pipe 10-11 is determined by an automated iterative calculation in cells L18:Y26 to the right.

NOTE2: If "#NUM" appears in cell H17 the most likely cause is that the stormwater flow rate entered in cell H10 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

HGL1A =	5818.33	ft	**EGL1A** =	5818.53	ft
HGL1B =	5818.43	ft	**EGL1B** =	5818.63	ft

5-yr Energy Grade Line and Hydraulic Grade Line

Pipe$_{\text{LINE 2}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s9} =	5825.19	ft
Pipe Diameter, D_9 =	18	in	Length of Pipe$_{5\text{-}6}$, $L_{9\text{-}10}$ =	191.67	ft
Invert Elevation, E_{i9} =	5818.8	ft	Stormwater Flow Rate, Q_9 =	2.4	cfs
Manning Roughness, n_{full} =	0.013		Manhole head loss Coeff., K_{10} =	1.8	

Calculation of Normal Depth in Pipe$_{9\text{-}10}$

			Downstream Invert Elev. =	5817.63	ft
Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	20.62	ft
Pipe Slope, $S_{9\text{-}10}$ =	0.0061	ft/ft	Normal Depth of Flow, $y_{9\text{-}10}$ =	7.59	in
Cross-Sect. Area, A =	0.71	ft^2	Depth of Flow/Diam, y/D =	0.42	
Wetted Perimeter, P =	2.12	ft	Manning Roughness, n =	0.0165	
Hydraulic Radius, R =	0.33	ft	Manhole Head Loss, Δh_{M10} =	0.3211	ft
Flow Velocity, V_{10} =	3.39	ft/sec	(for flow at normal depth in pipe 9-10)		

NOTE1: Iterative calculation of the normal depth for pipe 9-10 is shown at the right.

NOTE2: If "#NUM" appears in cell H47 the most likely cause is that the stormwater flow rate entered in cell H41 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

Depth of Flow, y_{10} = (just upstream of manhole 10)	9.97	in	Circ. Segment Height, h =	0.67	ft
			Depth of Flow/Diam, y/D =	0.55	
Central Angle, θ =	2.93	radians	Cross-Sect. Area, A =	1.00	ft^2
Flow Velocity, V_9 =	2.39	ft/sec	Distance, L_1 =	19	ft
HGL2 =	5819.63	ft	**EGL2** =	5819.72	ft

5-yr Energy Grade Line and Hydraulic Grade Line

Pipe$_{\text{LINE 3}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s8}=	5823.82	ft
Pipe Diameter, D_8 =	18	in	Length of Pipe$_{8\text{-}9}$, $L_{8\text{-}9}$ =	98.24	ft
Invert Elevation, E_{i8} =	5819.58	ft	Stormwater Flow Rate, Q_8 =	2.4	cfs
Manning Roughness, n_{full} =	0.013		Manhole head loss Coeff., K_9 =	1.8	

Calculation of Normal Depth in Pipe$_{8\text{-}9}$

			Downstream Invert Elev. =	5818.80	ft
Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	18.08	
Pipe Slope, $S_{8\text{-}9}$ =	0.0079	ft/ft	Normal Depth of Flow, $y_{8\text{-}9}$ =	7.08	in
Cross-Sect. Area, A =	0.65	ft^2	Depth of Flow/Diam, y/D =	0.39	
Wetted Perimeter, P =	2.03	ft	Manning Roughness, n =	0.0166	
Hydraulic Radius, R =	0.32	ft	Manhole Head Loss, Δh_{M9} =	0.3868	ft
Flow Velocity, V_9 =	3.72	ft/sec	(for flow at normal depth in pipe 8-9)		

NOTE1: Iterative calculation of the normal depth for pipe 8-9 is shown at the right.

NOTE2: If "#NUM" appears in cell H81 the most likely cause is that the stormwater flow rate entered in cell H75 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

Depth of Flow, y_9 = (just upstream of manhole 9)	7.98	in	Circ. Segment Height, h =	0.66	ft
			Depth of Flow/Diam, y/D =	0.44	
Central Angle, θ =	2.91	radians	Cross-Sect. Area, A =	0.76	ft^2
Flow Velocity, V_8 =	3.17	ft/sec	Cumulative Distance, L_2 =	211	ft
HGL3 =	5820.24	ft	**EGL3**	5820.40	ft

5-yr Energy Grade Line and Hydraulic Grade Line

$\text{Pipe}_{\text{LINE 4}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s7} =	5824.07	ft
Pipe Diameter, D_7 =	18	in	Length of $\text{Pipe}_{7\text{-}8}$, $L_{7\text{-}8}$ =	27.69	ft
Invert Elevation, E_{i7} =	5819.82	ft	Stormwater Flow Rate, Q_7 =	2.4	cfs
Manning Roughness, n_{full} =	0.013		Manhole head loss Coeff., K_9 =	0	

Calculation of Normal Depth in $\text{Pipe}_{7\text{-}8}$

			Downstream Invert Elev. =	5819.58	ft
Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	17.30	
Pipe Slope, $S_{7\text{-}8}$ =	0.0087	ft/ft	Normal Depth of Flow, $y_{7\text{-}8}$ =	6.92	in
Cross-Sect. Area, A =	0.63	ft^2	Depth of Flow/Diam, y/D =	0.38	
Wetted Perimeter, P =	2.01	ft	Manning Roughness, n =	0.0166	
Hydraulic Radius, R =	0.31	ft	Manhole Head Loss, Δh_{M8} =	0.0000	ft
Flow Velocity, V_8 =	3.84	ft/sec	(for flow at normal depth in pipe 7-8)		

NOTE1: Iterative calculation of the normal depth for pipe 7-8 is shown at the right.

NOTE2: If "#NUM" appears in cell H115 the most likely cause is that the stormwater flow rate entered in cell H109 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

Depth of Flow, y_8 = (just upstream of manhole 8)	7.08	in	Circ. Segment Height, h =	0.59	ft
			Depth of Flow/Diam, y/D =	0.39	
Central Angle, θ =	2.71	radians	Cross-Sect. Area, A =	0.65	ft^2
Flow Velocity, V_7 =	3.72	ft/sec	Cumulative Distance, L_3 =	309	ft
HGL4 =	5820.41	ft	**EGL4**	5820.62	ft

100-yr Energy Grade Line and Hydraulic Grade Line

Instructions: *Enter values in blue boxes. Spreadsheet calculates values in yellow boxes*

<u>NOTE</u>: Point 1A must be the outfall or else the most downstream point for the length of storm sewer for which the HGL and EGL are being calculated.

$\text{Pipe}_{\text{LINE 1}}$ — Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s10} =	5822.54	ft
Pipe Diameter, D_{10} =	18	in	Surface Elevation, E_{s11} =	5821.95	ft
Invert Elevation, E_{i10} =	5817.63	ft	Length of $\text{Pipe}_{10\text{-}11}$, $L_{10\text{-}11}$ =	19.2	ft
Invert Elevation, E_{i11} =	5817.53	ft	Stormwater Flow Rate, Q_{10} =	7.26	cfs
Manning Roughness, n_{full} =	0.013				

Calculation of Normal Depth in $\text{Pipe}_{10\text{-}11}$

Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	67.52	ft
Pipe Slope, $S_{10\text{-}11}$ =	0.0052	ft/ft	Normal Depth of Flow, $y_{10\text{-}11}$ =	15.47	in
Cross-Sect. Area, A =	1.62	ft^2	Depth of Flow/Diam, y/D =	0.86	
Wetted Perimeter, P =	3.56	ft	Manning Roughness, n =	0.0141	
Hydraulic Radius, R =	0.45	ft	Flow Velocity, V_{11} =	4.49	ft/sec

NOTE1: The normal depth for pipe 10-11 is determined by an automated iterative calculation in cells L18:Y26 to the right.

NOTE2: If "#NUM" appears in cell H17 the most likely cause is that the stormwater flow rate entered in cell H10 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

HGL1A =	5818.82	ft	**EGL1A** =	5819.13	ft
HGL1B =	5818.92	ft	**EGL1B** =	5819.23	ft

100-yr Energy Grade Line and Hydraulic Grade Line

Pipe$_{\text{LINE 2}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s9} =	5825.19	ft
Pipe Diameter, D_9 =	18	in	Length of Pipe$_{5\text{-}6}$, $L_{9\text{-}10}$ =	191.67	ft
Invert Elevation, E_{i9} =	5818.8	ft	Stormwater Flow Rate, Q_9 =	5.1	cfs
Manning Roughness, n_{full} =	0.013		Manhole head loss Coeff., K_{10} =	1.8	

Calculation of Normal Depth in Pipe$_{9\text{-}10}$

			Downstream Invert Elev. =	5817.63	ft
Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	43.81	ft
Pipe Slope, $S_{9\text{-}10}$ =	0.0061	ft/ft	Normal Depth of Flow, $y_{9\text{-}10}$ =	11.56	in
Cross-Sect. Area, A =	1.20	ft^2	Depth of Flow/Diam, y/D =	0.64	
Wetted Perimeter, P =	2.79	ft	Manning Roughness, n =	0.0156	
Hydraulic Radius, R =	0.43	ft	Manhole Head Loss, Δh_{M10} =	0.5060	ft
Flow Velocity, V_{10} =	4.25	ft/sec	(for flow at normal depth in pipe 9-10)		

NOTE1: Iterative calculation of the normal depth for pipe 9-10 is shown at the right.

NOTE2: If "#NUM" appears in cell H47 the most likely cause is that the stormwater flow rate entered in cell H41 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

Depth of Flow, y_{10} = (just upstream of manhole 10)	15.98	in	Circ. Segment Height, h =	0.17	ft
			Depth of Flow/Diam, y/D =	0.89	
Central Angle, θ =	1.37	radians	Cross-Sect. Area, A =	1.66	ft^2
Flow Velocity, V_9 =	3.08	ft/sec	Distance, L_1 =	19	ft
HGL2 =	5820.13	ft	**EGL2** =	5820.28	ft

100-yr Energy Grade Line and Hydraulic Grade Line

Pipe$_{\text{LINE 3}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s8}=	5823.82	ft
Pipe Diameter, D_8 =	18	in	Length of Pipe$_{8\text{-}9}$, $L_{8\text{-}9}$ =	98.24	ft
Invert Elevation, E_{i8} =	5819.58	ft	Stormwater Flow Rate, Q_8 =	5.1	cfs
Manning Roughness, n_{full} =	0.013		Manhole head loss Coeff., K_9 =	1.8	

Calculation of Normal Depth in Pipe$_{8\text{-}9}$

			Downstream Invert Elev. =	5818.80	ft
Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	38.41	
Pipe Slope, $S_{8\text{-}9}$ =	0.0079	ft/ft	Normal Depth of Flow, $y_{8\text{-}9}$ =	10.70	in
Cross-Sect. Area, A =	1.10	ft^2	Depth of Flow/Diam, y/D =	0.59	
Wetted Perimeter, P =	2.64	ft	Manning Roughness, n =	0.0159	
Hydraulic Radius, R =	0.41	ft	Manhole Head Loss, Δh_{M9} =	0.6067	ft
Flow Velocity, V_9 =	4.66	ft/sec	(for flow at normal depth in pipe 8-9)		

NOTE1: Iterative calculation of the normal depth for pipe 8-9 is shown at the right.

NOTE2: If "#NUM" appears in cell H81 the most likely cause is that the stormwater flow rate entered in cell H75 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

Depth of Flow, y_9 = (just upstream of manhole 9)	12.17	in	Circ. Segment Height, h =	0.49	ft
			Depth of Flow/Diam, y/D =	0.68	
Central Angle, θ =	2.42	radians	Cross-Sect. Area, A =	1.27	ft^2
Flow Velocity, V_8 =	4.01	ft/sec	Cumulative Distance, L_2 =	211	ft
HGL3 =	5820.59	ft	**EGL3**	5820.84	ft

100-yr Energy Grade Line and Hydraulic Grade Line

Pipe$_{\text{LINE 4}}$

Inputs: (enter values in blue boxes)

			Surface Elevation, E_{s7} =	5824.07	ft
Pipe Diameter, D_7 =	18	in	Length of Pipe$_{7\text{-}8}$, $L_{7\text{-}8}$ =	27.69	ft
Invert Elevation, E_{i7} =	5819.82	ft	Stormwater Flow Rate, Q_7 =	5.1	cfs
Manning Roughness, n_{full} =	0.013		Manhole head loss Coeff., K_9 =	0	

Calculation of Normal Depth in Pipe$_{7\text{-}8}$

			Downstream Invert Elev. =	5819.58	ft
Pipe radius, r =	0.75	ft	$Q/(1.49S^{1/2})$ =	36.77	
Pipe Slope, $S_{7\text{-}8}$ =	0.0087	ft/ft	Normal Depth of Flow, $y_{7\text{-}8}$ =	10.44	in
Cross-Sect. Area, A =	1.06	ft^2	Depth of Flow/Diam, y/D =	0.58	
Wetted Perimeter, P =	2.60	ft	Manning Roughness, n =	0.0159	
Hydraulic Radius, R =	0.41	ft	Manhole Head Loss, Δh_{M8} =	0.0000	ft
Flow Velocity, V_8 =	4.80	ft/sec	(for flow at normal depth in pipe 7-8)		

NOTE1: Iterative calculation of the normal depth for pipe 7-8 is shown at the right.

NOTE2: If "#NUM" appears in cell H115 the most likely cause is that the stormwater flow rate entered in cell H109 is too large for the pipe of specified diameter and slope to carry when flowing full. The pipe diameter and/or slope must be increased to carry the specified stormwater flow rate.

Depth of Flow, y_8 = (just upstream of manhole 8)	10.70	in	Circ. Segment Height, h =	0.61	ft
			Depth of Flow/Diam, y/D =	0.59	
Central Angle, θ =	2.76	radians	Cross-Sect. Area, A =	1.10	ft^2
Flow Velocity, V_7 =	4.66	ft/sec	Cumulative Distance, L_3 =	309	ft
HGL4 =	5820.71	ft	**EGL4**	5821.05	ft