CITY OF SCOTTSDALE

Drainage Reports

CITY OF SCOTTSDALE

MULTI-USE SPORTS FIELDS

NWC of Bell Road & 94th Street

PRELIMINARY

PRELIMINARY DESIGN DATA REPORT

Project No. PA75200538

SEPTEMBER, 2020

Prepared For:

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1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION/BACKGROUND

The purpose of this drainage study is to provide a basis of design for the drainage infrastructure associated with the new Multi-Use Sports Fields at the northwest corner of Bell Road and 94th Street. The proposed complex will primarily consist of six lighted multi-use fields, two parking lots, restroom and maintenance buildings with potable water and sewer connections, sidewalks, offsite street improvements and a non-potable water connection for field and landscaping irrigation purposes. The improvements are located on a 40-acre undisturbed natural desert parcel that is situated within the Lower Desert Environmentally Sensitive Lands (ESL) zoning district. The sports complex site improvements will be designed to meet the drainage and ESL design requirements as outlined in the City of Scottsdale *Design Standards & Policies Manual* (DSPM).

1.2 PROJECT LOCATION

The project is located within the City of Scottsdale on the northwest corner of Bell Road and 94th Street. It is bound by Bell Road on the south, 94th Street on the east, the powerline corridor on the west and the existing Desert Parks Vista Apartments/Corporate Center at DC Ranch on the north. Refer to Figure 1 below for a detailed vicinity map.

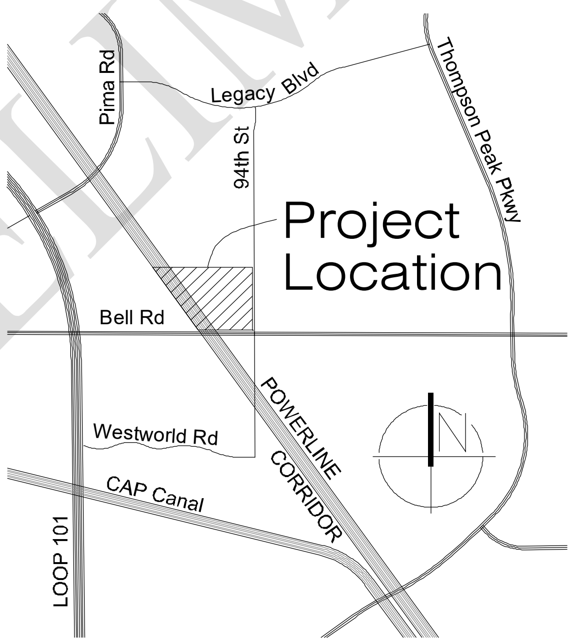


Figure 1: Vicinity Map

2.0 STORM WATER RETENTION BASIN ANALYSIS

2.1 APPROACH

The storm water retention basin system for the new Multi-Use Sports Fields at the northwest corner of Bell Road and 94th Street was designed in accordance with the City of Scottsdale *Drainage Policies and Standards Manual (DSPM).*

The grading plan for the sports fields complex includes, four (4) shallow retention basins as well as one 3-foot deep detention basin. Three of the retention basins are located adjacent to the new 94th Street parking lot with the fourth located on the north end of the main parking lot underneath the powerline corridor. Each of these basins is graded in such a manner as to not appear "basin-like" but instead blend in with the surroundings. The maximum basin depth of these basins is less than 1 foot, with gentler basin side slopes. Each of these basins was also designed to retain the higher of either the first flush volume or the increase in the pre vs post 100-year, 2-hour runoff volume. The largest basin on the project site is located just north of Bell Road, east of the entrance drive. This detention basin is 3.0 feet deep, with 4H:1V side slopes that is partially drained through an outlet pipe to the existing dual 6'x3' concrete box culvert underneath Bell Road. The detention basin was designed to store the first flush volume as well as reduce the post development peak discharges through the existing culvert underneath Bell Road below the pre development peak discharge for the 2-, 10- and 100-year, 6-hour storm events.

2.2 RETENTION BASIN DESIGN

The Multi-Use Sports Fields at the NWC of Bell Road & 94th Street consist of new maintenance and restroom buildings, parking lots, hardscaped plaza area and six new multi-use sand-based sports fields. Excluding the sports fields, the project area is graded into five separate drainage areas, each draining into one of the new retention/detention basins. Since the project area is in an ESL zoning district where disturbance to the natural desert is to be minimized, the four new retention basins were not designed to store the full 100-year, 2-hour runoff volume. Instead they were designed to store the greater of either the 1) first flush volume or 2) the increase in runoff volume from the 100-year, 2-hour storm event from pre-development conditions. Refer to Appendix A for the retention basin design drainage area maps as well as the first flush and increase in pre vs post runoff volume calculation spreadsheets.

A slightly different approach was taken for the design of the basin adjacent to Bell Road at the south side of the project area. This basin was designed to store the first flush volume from the main parking lot as well as the hardscaped plaza areas adjacent to the restroom building. However, instead of storing the increase in runoff volume due to the conversion of natural desert into impervious asphalt, concrete and roof area, the basin was designed as a detention basin with two outlet structures that drain through a 24-inch pipe to the existing dual 6'x3' concrete box culvert underneath Bell Road, just west of the entrance drive. The low-level outlet structure consists of a standard grated catch basin that is located 0.5 feet above the basin bottom and is used to drain the basin volume through a 3-inch orifice plate to a 15-inch drainpipe. The high-level outlet structure consists of a raised grate catch basin that is less susceptible to clogging and is located 2.2 feet above the basin bottom. This basin outlet configuration allows the basin to function as a retention basin up to a depth of 2.2 feet, with the low-level outlet structure serving as a basin bleed-off. During large storm events, once depths of ponding in the basin exceed 2.2 feet, the basin becomes a detention basin with water spilling through the high-level outlet structure. This configuration allows the basin to not only store the first flush runoff volume from the parking lot and plaza areas below a depth of 2.2 feet, but also reduces the post-development peak discharges below the pre-development peak discharges for the 2-, 10- and 100-year, 6-hour storm events. Refer to Appendix A for the retention basin design drainage area map and first flush volume calculations as well as Appendix B for the HEC-1 hydrologic modeling of the existing and proposed conditions.

The six multi-use sports fields were not included in the first flush or pre vs post runoff volume calculations because they will not generate any surface runoff. The two primary reasons why the sports fields will not generate any surface runoff is that they are flat and that the underlying soil stratification consist of a 12-inch sand-based root mix surface layer above a 4-inch gravel layer in conjunction with a subgrade drainage system that consists of 4-inch perforated pipes and a 12-inch drain pipe. The sand-based fields with the advanced subgrade drainage system are designed to infiltrate the rainfall efficiently through the sand layer and convey it through the coarse gravel layer to the 4-inch perforated pipes before eventually discharging it to the spinal 12-inch drain pipe. Since the rainfall onto the fields, as well as the small surrounding pathway and plaza areas that drain into the fields get filtered through the 12-inch sand layer, the water is already naturally filtered and no first flush storage is necessary. The 12-inch subgrade drainpipe is discharged directly to the detention basin high-level outlet structure where it bypasses the detention basin and flows out into the existing Bell Road culvert.

2.3 BASIN DRAIN TIMES

The basins were designed to dispose of the stored runoff volume in the allowable 36 hours. In order to meet this criteria; Double Ring Infiltration Tests were performed at the site of the proposed basins. The three, one-foot deep retention basins adjacent to the 94th Street parking lot were found to infiltrate the stored runoff volume in under 10 hours. A basin drain was not calculated for the new retention basin on the north side of the main parking lot because it is only 0.3 feet deep and the *DSPM* allows for basins less than 0.5 feet to be disposed of through infiltration without showing basin drain time calculations.

The basin drain time for the detention basin was calculated by determining the time required for the low-level bleed off catch basin to lower the depth of ponding in the basin to 6 inches, with the bottom 6-inches calculated to dissipate through infiltration. As can be seen in the proposed conditions model in Appendix B, during the 100-year, 6-hour storm event it takes the 3-inch orifice plate in the low-level grated catch basin a little over 17 hours to drain the basin to a depth of ponding of 0.5 feet. The remaining depth was found to infiltrated into the ground in about 7 hours. Therefore, the total basin drain time for the large detention basin is approximately 24 hours. Refer to Appendix A for the detailed basin drain time calculations.

3.0 HYDROLOGIC ANALYSIS

3.1 APPROACH

The hydrologic analysis for the new Multi-Use Sports Fields at the northwest corner of Bell Road and 94th Street was performed in accordance with the *DSPM* utilizing the hydrologic methods outlined in the Flood Control District of Maricopa County (District) *Drainage Design Manual for Maricopa County – Hydrology (Volume I)*.

There are two separate hydrologic models that are used to determine design peak discharges throughout the project site. The first hydrologic model is the *"Pinnacle Peak South Area Drainage Master Study" (PPS ADMS)* FLO-2D model that was prepared by TY Lin International for the City of Scottsdale in 2013. The primary purpose of the FLO-2D model was to determine the 94th Street Wash inflows that enter the project site. The second hydrologic model is a new HEC-1 model that was developed for the 2-, 10- and 100-year, 6-hour storm events utilizing the Districts' DDMSW software. The HEC-1 model was used to determine additional offsite inflows from the north into the project area as well as peak discharges that are generated within the project site under existing and proposed conditions.

3.2 EXISTING CONDITIONS HEC-1 MODEL

The existing conditions HEC-1 hydrologic model was developed to determine the pre development peak discharges that are currently impacting the project site. In order to determine drainage area boundaries and main flow paths within the project area, as well as the upstream offsite area that contributes to the project area; the latest aerial photography and topographic mapping that was developed for the design of the project was used. To supplement the new topographic mapping, the one-foot contour mapping that was developed for the Pinnacle Peak South ADMS was obtained from the City of Scottsdale and utilized in areas that were developed prior to the contour mapping flight date in 2007. For areas that have been recently developed, any available as-built plans were inspected, and site visits were made to determine the appropriate flow paths and drainage area boundaries.

The existing conditions HEC-1 model includes the entire project area west of the 94th Street Wash. As will be discussed in greater detail in Section 3.4 below, the PPS ADMS FLO-2D model was used to determine the peak discharges for upstream watershed area of the 94th Street Wash. However, two of the offsite watershed areas were included in the existing conditions HEC-1 model. The first one, identified with the HEC-1 Subbasin ID of ODA1 is the contributing drainage area for the 91st Street Wash that crosses the project area on the northwest corner. The second, much smaller one is identified with the HEC-1 Subbasin ID of ODA2 and is the contributing drainage area for the wash that concentrates at the northern boundary of the project area between the Corporate Center at DC Ranch and the Desert Parks Vista Condominium Complex. Refer to the existing condition HEC-1 Schematic and Drainage Area Map in Appendix B for the location and extents of the two main offsite watershed areas.

In addition to the three major washes that enter the site, there are several other minor washes that traverse the project area. These washes generally flow in a southwesterly direction leaving the project area and discharging to the Arizona State Land parcel immediately west of the project boundary and eventually discharging underneath Bell Road through either the dual 6'x3' box culvert at the new entrance drive or the triple barrel 8'x3' concrete box culvert at 91st Street. Refer to Appendix B for the existing conditions HEC-1 Schematic as well as the 100-year, 6-hour HEC-1 hydrologic model. The Digital Data in Appendix E contains the 2-year and 10-year HEC-1 models.

3.3 PROPOSED CONDITIONS HEC-1 MODEL

The proposed conditions HEC-1 hydrologic model was developed by incorporating the proposed Multi-Use Sports Fields improvements into the existing conditions hydrologic model. The offsite drainage area boundaries remained the same, but new onsite drainage area boundaries were drawn based on the grading and drainage design of the main parking lot and the adjacent hardscape plaza areas. Due to the conversion of natural desert to impervious parking and hardscaped area, the runoff volumes, and discharges for the western half of the project area increased significantly under the proposed conditions as compared to the existing conditions. In order to meet the ESL ordinance requirements of keeping the post development peak discharges below the pre-development conditions, the proposed conditions HEC-1 model incorporated a onsite storm drain and detention basin that is used to attenuate the flows before they leave the project site below the existing conditions for the 2-, 10- and 100-year, 6-hour storm events. In addition to the onsite, main parking lot storm drain, an offsite storm drain was included that will convey the offsite flows from the north project boundary through the project site, outletting to the same location where the wash currently leaves the project site. Refer to the proposed conditions HEC-1 Schematic and Drainage Area Map in Appendix B for the updated drainage area boundaries.

Within the project area, the proposed conditions HEC-1 hydrology model serves two primary purposes. The first is to design the main parking lot storm drain and size the new detention basin north of Bell Road to attenuate the increased peak discharges below the existing conditions peak discharges that leave the site. The grading of the main parking lot underneath the powerline corridor was done in such a manner as to crate four shallow sump locations where new storm drain catch basin will intercept the 100-year, 6-hour design peak discharges and convey them through a new storm drain to the detention basin at Bell Road. As was previously mentioned, the detention basin was designed to store the first flush runoff volume from the parking lot area, while larger flows were designed to spill through the high-level outlet structure and discharge through the existing dual 6'x3' Bell Road culvert. Even with the larger contributing drainage area (i.e. under existing conditions, the northwest part of the project area is not part of the drainage area to the dual 6'x3' Bell Road box culvert), the design of the detention basin with its low- and high-level outlet structures achieved the desired goal of reducing the peak discharges through the Bell Road culvert under the proposed conditions to be equal or less than the existing conditions for all three storm events. Refer to the Appendix B for the HEC-1 Schematic and the 100-year, 6-hour HEC-1 model as well as the digital data folder in Appendix E for 2- and 10-year, 6-hour HEC-1 models.

The second purpose of the proposed conditions HEC-1 hydrologic model was to determine the amount of runoff from the multi-use sports fields drainage system during the 2-, 10- and 100-year, 6-hour storm events. Since the sports fields are designed to be flat and consist of a 12-inch deep sand-based root zone mix, there will be no surface runoff generated during even the most intense part of a the 100-year storm event. The first step in determining how much water infiltrates through the sand-based root zone mix was to determine how much available pore space the sand layer has to store water. Recommended root zone mixes for sand-based fields call for the sand to have a total porosity between 35%-55%, this is the total void space between the sand particles. However, not all of this void space has the ability to store water that infiltrates through the surface. The ideal root zone mix has a capillary porosity, which is defined as the amount of the void space that is capable of storing water, between 15%-25%. Assuming an average capillary porosity of 20% we can calculate that the 12-sand layer has a storage capacity of 2.4 inches (12 in x 0.20 = 2.4 in) over the surface area of field. However, this 2.4 inches was further reduced by applying a 50% safety factor to account for potentially wet antecedent moisture conditions that could be a result of recent irrigation or a storm event. Therefore, for the flat portion of the multi-use sports fields an Initial Abstraction value of 1.2 inches was applied to represent the available storage associated with the 20% capillary porosity within the 12-inch sand-based root zone mix.

Once the storm event exceeds the storage capacity of root zone mix associated with the capillary porosity, water will drain into the underlying 4-inch thick gravel layer. The gravel layer has larger voids that are not conducive to "storing" water, but allow it to freely flow through the layer and into the 4-inch diameter perforated pipes that are located 20 feet on center underneath the sports fields. Once water enters these perforated pipes it is conveyed to a 12-inch subgrade drain and out underneath the fields to the high-level outlet structure at the detention basin on the north side of Bell Road. From there the outflow is piped directly to the dual 6'x3' box culvert under Bell Road.

The total open void space within the gravel layer was calculated based on a conservative total porosity of 35%. To utilize the storage capacity within the 4-inch and 12-inch pipes as well as the large open void space in the gravel layer, orifice plates were designed to limit the flow out from underneath each of the fields. The orifice plates will meter the flow out from underneath each of the fields thereby allowing water to pond in the pipes and within the gravel layer to limit the flow out from underneath each one of the fields. This approach to the design of the subgrade drainage system will utilize the full available storage capacity underneath each of the sports fields before discharging directly to the dual 6'x3' Bell Road box culvert via the detention basin outlet pipe.

3.4 94TH STREET WASH HYDROLOGIC ANALYSIS

The 94th Street Wash is located on the east side of the project site, paralleling 94th Street and discharging under Bell Road in a five barrel 8'x3' concrete box culvert. As can be seen in the 94th Street Wash Drainage Area Map in Appendix B, the wash is comprised of a major confluence at the northeast corner of the project site, with two main wash forks entering from the north and three inflows from 94th Street. The main (western) wash fork extends upstream to Legacy Drive, roughly paralleling 94th Street on the west before turning and extending to the existing dual 8'x2' concrete box culvert that penetrates the Reata Wash levee at Hualapai Drive. The secondary (eastern) wash fork extends upstream to 94th Street before meandering through the Desert Haciendas subdivision to a second 10'x2' concrete box culvert that penetrates the Reata Wash levee. The two culvert crossings underneath 94th Street have relatively small drainage areas and do not receive split flows from Reata Wash.

The total contributing drainage area to the 94th Street Wash at the project site is 185 acres. In addition to the runoff form the contributing drainage area, there are the two locations described above where runoff enters the watershed from Reata Wash. There is also one location, just downstream of Hualapai Drive where water from the main 94th Street Wash out of the contributing drainage area during large storm events. To analyze this complex hydrologic condition, the PPS ADMS 100-year, 24-hour FLO-2D model was reviewed and modified to better represent existing flow conditions. Upstream of the project site, within the contributing drainage area to the 94th Street Wash at the project location, the modifications primarily consisted of adjusting grid elevations to 1) prevent flows from breaking out of the two main washes and 2) directing the runoff generated in the contributing drainage area to the correct location based on inspection of contour mapping, aerial photography and as-built plans. These modifications removed any erroneous flow splits or diversions that can easily occur as part of a large regional drainage master study. The modeling of the two Reata Pass levee penetration box culverts was not modified from the original PPS ADMS study model.

As can be seen in the drainage area map in Appendix B, there is approximately 285 cfs that enters the west fork of the 94th Street Wash through the northern dual 8'x2' concrete box culvert from Reata Wash. Just downstream of the culvert, approximately 200 cfs stays within the wash while approximately 85 cfs splits out in a southwesterly direction. With the addition of the runoff from the contributing drainage area, the peak inflow for the west fork of the 94th Street Wash at the project site is 260 cfs. For the east fork of the 94th Street Wash there is approximately 205 cfs that enters through the 10'x2' concrete box culvert from Reata Wash. This flow increases with the addition of the runoff generated from the contributing watershed area to 245 cfs that enters the project site.

The largest inflow from 94th Street occurs through an existing 36-inch pipe culvert that conveys flows from the Desert Haciendas subdivision underneath 94th Street. According to the modified FLO-2D model the peak inflow through the pipe culvert into the 94th Street Wash is 35 cfs. The remaining two inflow locations, just south of the 36-inch culvert, consist of a side-by-side 18-inch storm drain and pipe culvert as well as a scupper that drains the west half of 94th Street to the wash. The contributing drainage area to the existing 18-inch culvert was cut off by the development of the Desert Haciendas subdivision and the runoff generated from the small 94th Street watershed that is intercepted by the 18-inch storm drain and scupper would not have a meaningful impact on the peak discharge of the 94th Street Wash through the project area. Refer to Appendix B for the 94th Street Wash Drainage Area Map that shows the pertinent offsite drainage infrastructure as well as the location of the three main inflows to the 94th Street wash through the project area. Appendix B also includes the three main inflow hydrographs that were obtained from the modified FLO-2D model, which in turn can be found in the Digital Data folder in Appendix E.

The three main inflows were used as direct hydrograph inputs to a two-dimensional HEC-RAS hydraulics model that was used to determine the existing and proposed conditions water surface elevation of the 94th Street Wash through the project area as well as the design of the proposed triple barrel 10'x5' concrete box culvert crossing. The routing of the three main inflows through the project area using the HEC-RAS model resulted in a combined peak discharge through the existing Bell Road culvert of 480 cfs. For a more detailed discussion of the hydraulic analysis refer to Section 5.0 of this report.

4.0 STORM DRAIN DESIGN AND ANALYSIS

Two new storm drains were designed as part of the Multi-Use Sports Fields project. The first is a new storm drain that conveys the offsite flows that enter the project site from the north between the Corporate Center at DC Rand and the Desert Parks Vista Condominium Complex. The second storm drain is a new onsite storm drain that is located along the western edge of the main parking lot underneath the powerline corridor. This storm drain intercepts the runoff from the parking lot and adjacent hardscaped plaza areas with four new combination catch basins and conveys it to the new detention basin on the north side of Bell Road, east of the entrance drive. Refer to Appendix C for the Storm Drain Location Map showing the extents of the offsite and main parking lot storm drains in relationship to the proposed improvements.

4.1 OFFSITE STORM DRAIN DESIGN

The proposed offsite storm drain was designed to intercept the calculated 100-year, 6-hour peak discharge that concentrates at the northern boundary between the Corporate Center at DC Ranch and the Desert Parks Vista Condominiums Complex and convey it through the site to its existing outfall location on the southwest corner of the project area. As can be seen in the HEC-1 model in Appendix B, the design peak discharge at this location is 42 cfs (HEC-1 Subbasin ID: ODA2). Under existing conditions, the alignment of the offsite storm drain roughly follows the alignment of the existing wash that flows through the project area discharging at the same location as the existing wash.

At the upstream end, the offsite storm drain intercepts the design discharge with a headwall inlet. The proposed 36-inch storm drain traverses under the two northern most multi-use fields before entering the main parking lot just west of the new public restroom building. From there it traverses the parking lot at a diagonal before discharging through a headwall outlet structure into the existing wash. Refer to the Storm Drain Location Map in Appendix C for the location of the offsite storm drain as well as the Offsite Storm Drain Hydraulic Grade Line (HGL) design calculations.

4.2 MAIN PARKING LOT STORM DRAIN DESIGN

The main parking lot storm drain was designed to intercept the calculated 100-year, 6-hour peak discharges from the main parking lot underneath the powerline corridor as well as the adjacent hardscape plaza areas. The storm drain starts at the new detention basin north of Bell Road, west of the entrance drive and extends north along the western edge of the new parking lot. The proposed grading of the parking lot was done in such a manner as to create four shallow sumps where four new combination catch basins were designed to intercept the entire 100-year, 6-hour peak discharges from the new parking lot.

The proposed storm drain and catch basins were designed to prevent any water from overtopping the new western curb and flowing into the adjacent Arizona State Land parcel located immediately west of the project area. This approach ensures that all of the runoff generated from the proposed parking lot and adjacent hardscape areas will be intercepted by the new storm drain and routed through the new detention basin before discharging to the existing dual 6'x3' Bell Road box culvert just west of the entrance drive. Refer to the Storm Drain Location Map in Appendix C for the location of the main parking lot storm drain as well as the Storm Drain Hydraulic Grade Line (HGL) design and Catch Basin sizing calculations.

5.0 CULVERT DESIGN & WASH HYDRUALIC ANALYSIS

5.1 APPROACH

The hydraulic analysis for the two main washes that impact the project site was performed using the latest US Army Corps of Engineers HEC-RAS modeling software with two-dimensional surface flow capability. The hydraulic analysis was done in accordance with the City's *DSPM* as well as the District's *Hydraulics Manual.*

The two-dimensional HEC-RAS hydraulics model was developed to determine the water surface elevations for the two main washes that cross the project site. The 94th Street Wash traverses the project site by paralleling 94th Street, while the 91st Street Wash bisects the project site on the northwest corner. In addition to determining the water surface elevations under existing conditions, the two-dimensional hydraulics model was also used to design two new culvert crossings and calculated the proposed conditions water surface elevations.

5.2 94TH STREET WASH HYDRAULIC ANALYSIS

For the existing conditions analysis, the detailed topographic survey that was prepared as part of the project was used to generate the required HEC-RAS geometry files. For the 94th Street Wash model, the HEC-RAS computational domain mesh extends from the Desert Parks Vista Condominium Complex downstream past the existing five barrel 8'x3' Bell Road culvert. The extents of the computational domain incorporate the existing Bell Road culvert as well as the three main inflow locations within the project area for a peak discharge of 480 cfs. The hydraulic analysis of the existing conditions showed that the wash as well as the existing Bell Road box culvert have sufficient capacity to convey the combined peak discharge through the project area, with no water spilling over Bell Road or splitting out of the 94th Street Wash to the west. Refer to the 94th Street Wash Hydraulic Analysis Map in Appendix D for the hydraulic modeling extents, existing conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model.

The design of the Multi-Use Sports Fields and associated infrastructure leaves the 94th Street Wash mostly undisturbed. The main impact to the wash occurs at the proposed culvert crossing from the new 94th Street parking lot to the pathway system that traverses the sports fields. This culvert was designed to convey the entire combined upstream 100-year, 24-hour peak discharge of 480 cfs with no overtopping or spilling into the adjacent fields or into the new parking lot. In order to achieve this, a triple barrel 10'x5' concrete box culvert is recommended to be installed that will have sufficient capacity to convey the entire design peak discharge underneath the pathway connections. Even though the culvert has a 5-foot height, the bottom 12-inches of the culvert will be buried below the existing sandy wash bottom to allow for free movement of sediment though the culvert during storm events. In addition to burring the bottom 12-inches, the proposed 30-foot culvert width will span the exiting 25-foot jurisdictional wash bottom. The combination of burring the bottom 12-inches and spanning the entire sandy wash bottom will make the new culvert less susceptible to clogging due to sediment depositions.

The new culvert will raise the water surface elevations in the wash by about 2.0 feet from 1578.5 feet to 1580.5 feet. However due to the relative steep nature of the area, the increase in water surface elevation only propagates upstream for about 200 feet, well within the project limits. Therefore, the proposed improvements, including the construction of the new culvert crossing will not have a detrimental impact on the water surface elevation upstream within the Desert Parks Vista Condominium Complex or downstream of the Bell Road box culvert. Refer to the 94th Street Culvert Hydraulics Analysis Map in Appendix D for the proposed conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model.

5.3 91ST STREET WASH HYDRAULIC ANALYSIS

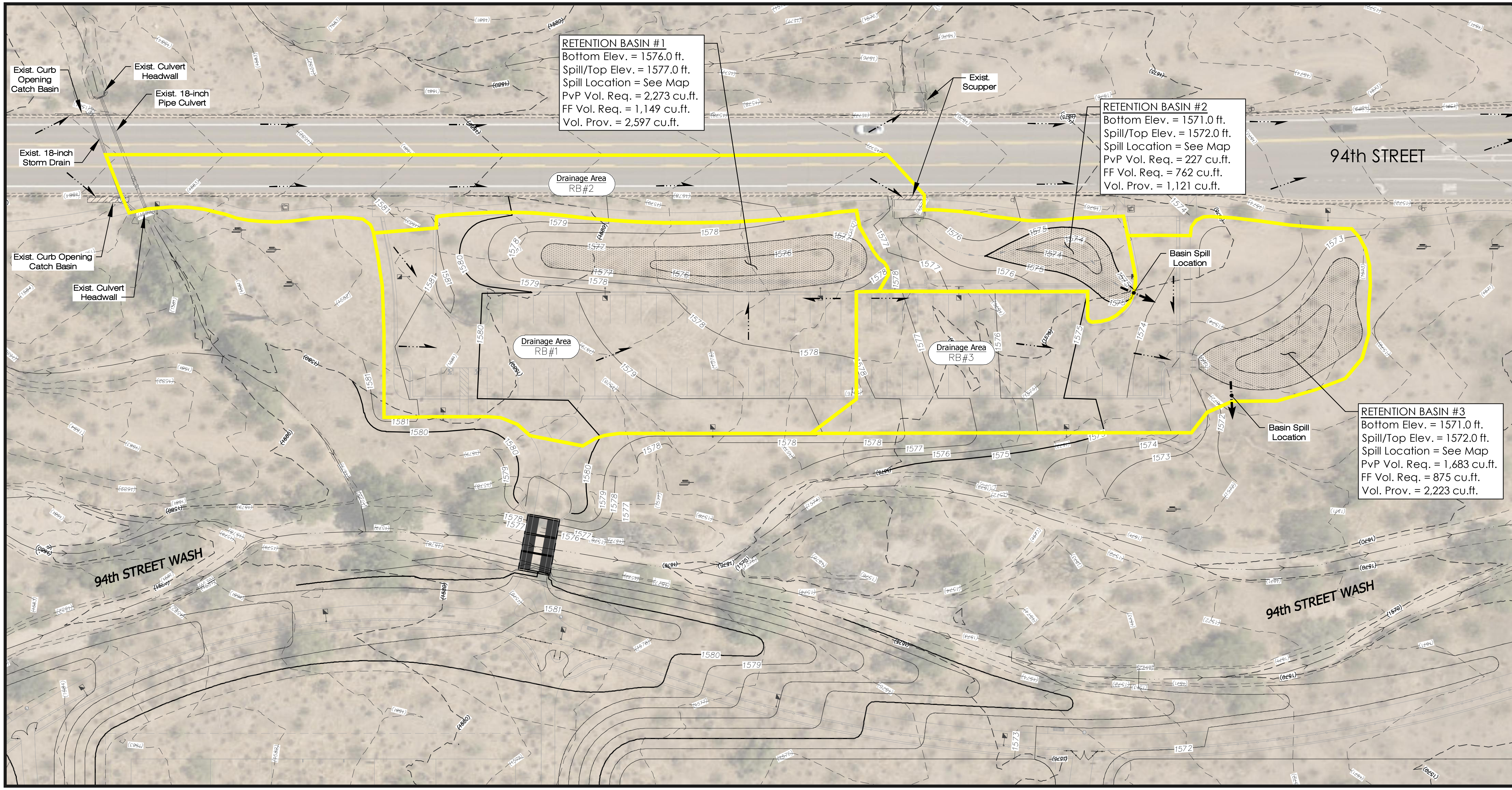
For the 91st Street Wash model, the HEC-RAS computational domain mesh extends upstream and downstream to the boundary of the topographic survey that was obtained as part of the project area. As calculated with the HEC-1 Hydrologic Model, the 100-year, 6-hour design peak discharge for the 91st Street Wash that enters the project site is 140 cfs. The hydraulic analysis of the existing conditions showed that the wash has sufficient capacity to convey the upstream design peak discharge with no water splitting out from the 91st Street Wash. Refer to the 91st Street Wash Hydraulic Analysis Map in Appendix D for the hydraulic modeling extents, existing conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model

The only impact of the Multi-Use Sports Fields project on the 91st Street Wash is the construction of the 91st Street entrance driveway. The driveway which connects the new main parking lot on the northwest corner of the project site to 91st Street must cross the existing jurisdictional wash. Like the 94th Street Wash culvert, this culvert was designed to convey the entire 100-year, 6-hour design peak discharge of 140 cfs. To achieve this, a single 10’x5’ concrete box culvert is recommended to be installed at the wash crossing. To reduce the clogging potential of the new culvert, the bottom foot of the culvert will be buried and the width of the culvert will span the existing 8-foot wide sandy wash bottom to allow for free movement of the sediment through the culvert thereby reducing its clogging potential.

The new crossing will raise the water surface elevations in the wash upstream of the culvert by about 0.8 feet from 1580.0 feet to 1580.8 feet. However due to the relative steep nature of the area, the increase in water surface elevation only propagates upstream for about 50 feet, raising the water surface elevation at the property boundary by less than 0.5 feet and at the existing dual 36-inch culvert outfall by less than 0.1 foot. Therefore, the proposed entrance driveway as well as the new concrete box culvert will span the existing jurisdictional wash bottom and only slightly increase the water surface elevation within 91st Wash in the Corporate Center at DC Ranch. Refer to the 91st Street Culvert Hydraulics Analysis Map in Appendix D for the proposed conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model.

Appendix A: Retention Basin Design Calculations

Retention Basin Design – Drainage Area Map



94th STREET PARKING LOT

PRE vs. POST & FIRST FLUSH DRAINAGE AREA MAP

100-yr, 2-hr PRE vs. POST & FIRST FLUSH RUNOFF VOLUME SUMMARY TABLE

SUBBASIN	Contributing Drainage Area (sq/ft)	Pre Development Runoff Volume (cu.ft.)	Post Development Runoff Volume (cu.ft.)	Increase in Runoff Volume (cu.ft.)	First Flush Volume (cu.ft.)
RB#1	35,490	3,128	5,400	2,273	1,149
RB#2	24,100	3,355	3,582	227	762
RB#3	27,580	2,430	4,114	1,683	875

LEGEND

- **Drainage Area Boundary**
- **Major Flow Paths**
- **Local Flow Direction**

0 30' 60'
SCALE: 1" = 60'

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Project :

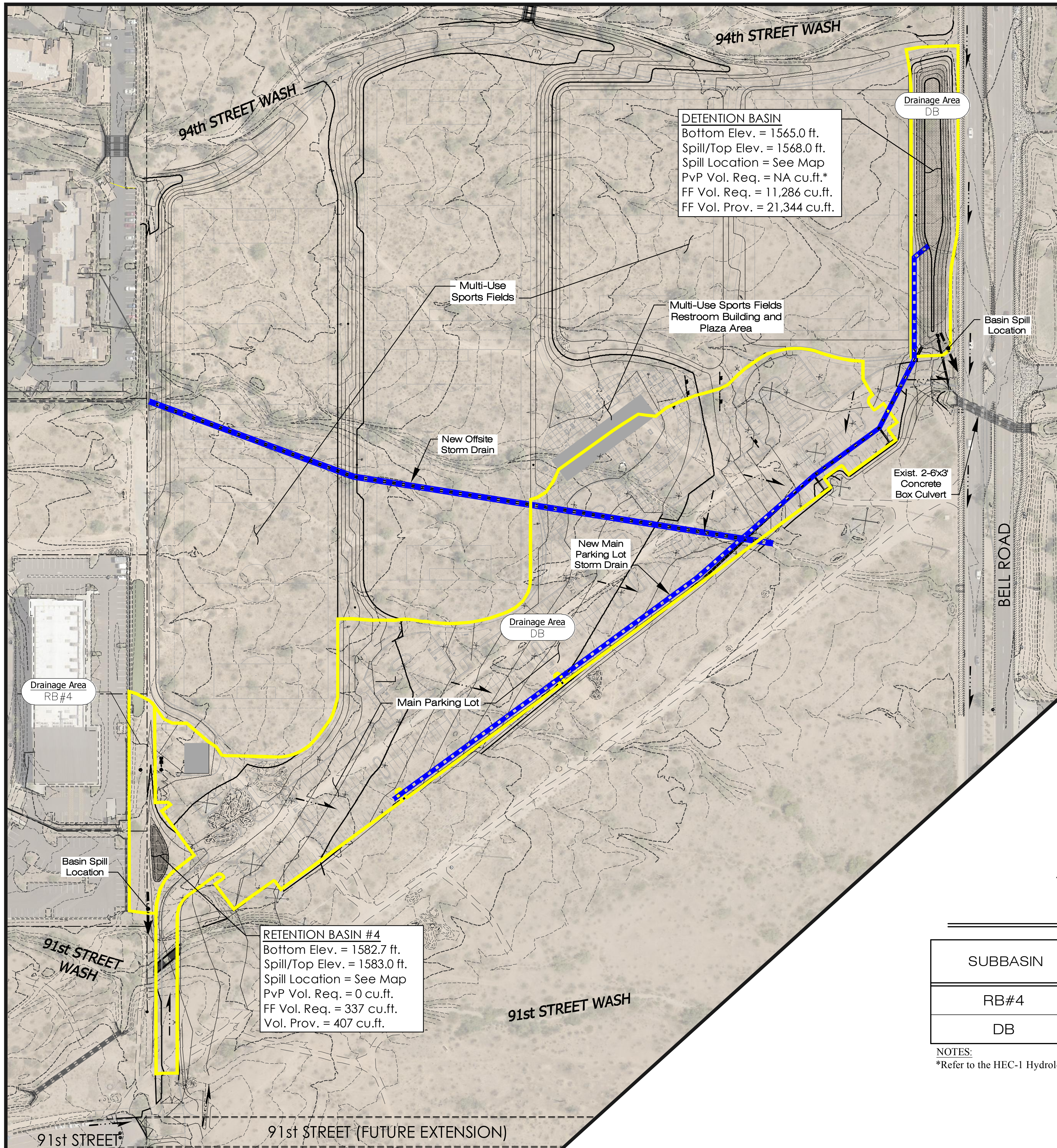
Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :
G&B No. 2003
Issue Date: 05/20
Drawn By: JDH/OK
Checked By: MTG

Sheet Title :
94th Street Parking Lot Drainage Area Map

Sheet Number:
1

10-UP-2020
9/30/2020



MAIN PARKING LOT

PRE vs. POST & FIRST FLUSH DRAINAGE AREA MAP

LEGEND

- **Drainage Area Boundary**
- **Major Flow Paths**
- **Local Flow Direction**
- **New Storm Drain**

0 90' 180'

SCALE: 1" = 180'

100-yr, 2-hr PRE vs. POST & FIRST FLUSH RUNOFF VOLUME SUMMARY TABLE

SUBBASIN	Contributing Drainage Area (sq/ft)	Pre Development Runoff Volume (cu.ft.)	Post Development Runoff Volume (cu.ft.)	Increase in Runoff Volume (cu.ft.)	First Flush Volume (cu.ft.)
RB#4	17,980	1,584	1,584	0	337
DB	329,710	NA*	NA*	NA*	11,286

NOTES:
*Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis

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Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

Main Parking Lot Drainage Area Map

Sheet Number:

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First Flush Volume Calculation

First Flush Volume Calculation

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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North 94th Street Parking Lot First-Flush Volume Calculation (Retention Basin #1)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	23,210	0.95	22,049.5	0.5	918.7
Desert Landscaping	12,280	0.45	5,526.0	0.5	230.3
Total Contributing Drainage Area:	35,490			**Total First Flush Runoff Volume:**	**1,149**

94th Street Scupper First-Flush Volume Calculation (Retention Basin #2)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	14,890	0.95	14,145.5	0.5	589.4
Desert Landscaping	9,210	0.45	4,144.5	0.5	172.7
Total Contributing Drainage Area:	24,100			**Total First Flush Runoff Volume:**	**762**

South 94th Street Parking Lot First-Flush Volume Calculation (Retention Basin #3)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	17,190	0.95	16,330.5	0.5	680.4
Desert Landscaping	10,390	0.45	4,675.5	0.5	194.8
Total Contributing Drainage Area:	27,580			**Total First Flush Runoff Volume:**	**875**

First Flush Volume Calculation

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

Gavan & Barker Inc.
Civil Engineering & Landscape Architecture

North Main Parking Lot First-Flush Volume Calculation (Retention Basin #4)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Desert Landscaping	17,980	0.45	8,091.0	0.5	337.1
Total Contributing Drainage Area:	17,980			**Total First Flush Runoff Volume:**	**337**

Main Parking Lot & Sports Complex Plaza Areas First-Flush Volume Calculation (Retention Basin #5)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	244,990	0.95	232,740.5	0.5	9,697.5
Desert Landscaping	84,720	0.45	38,124.0	0.5	1,588.5
Total Contributing Drainage Area:	329,710			**Total First Flush Runoff Volume:**	**11,286**

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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Pre vs. Post 100-year, 2-hour Runoff Volume Calculation

North 94th Street Parking Lot Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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North 94th Street Parking Lot Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #1)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	35,490	0.45	15,970.5	2.35	3,127.6
Total Contributing Drainage Area:	35,490			**Total Pre Development Runoff Volume**	**3,128**

North 94th Street Parking Lot Post Development 100-yr 2-hr Runoff Volume (Retention Basin #1)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	23,210	0.95	22,049.5	2.35	4,318.0
Desert Landscaping	12,280	0.45	5,526.0	2.35	1,082.2
Total Contributing Drainage Area:	35,490			**Total Post Development Runoff Volume**	**5,400**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**2,273**
Total First Flush Volume Required:	**1,149**
Retention Basin #1 Provided Retention Volume:	2,597

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

94th Street Scupper Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

Gavan & Barker Inc.
Civil Engineering & Landscape Architecture

94th Street Scupper Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #2)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	12,570	0.95	11,941.5	2.35	2,338.5
Undeveloped Desert	11,530	0.45	5,188.5	2.35	1,016.1
Total Contributing Drainage Area:	24,100			**Total Pre Development Runoff Volume**	**3,355**

94th Street Scupper Post Development 100-yr 2-hr Runoff Volume (Retention Basin #2)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	14,890	0.95	14,145.5	2.35	2,770.2
Desert Landscaping	9,210	0.45	4,144.5	2.35	811.6
Total Contributing Drainage Area:	24,100			**Total Post Development Runoff Volume**	**3,582**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**227**
Total First Flush Volume Required:	**762**
Retention Basin #2 Provided Retention Volume:	1,121

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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South 94th Street Parking Lot Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

Gavan & Barker Inc.
Civil Engineering & Landscape Architecture

South 94th Street Parking Lot Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #3)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	27,580	0.45	12,411.0	2.35	2,430.5
Total Contributing Drainage Area:	27,580			**Total Pre Development Runoff Volume**	**2,430**

South 94th Street Parking Lot Post Development 100-yr 2-hr Runoff Volume (Retention Basin #3)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	17,190	0.95	16,330.5	2.35	3,198.1
Desert Landscaping	10,390	0.45	4,675.5	2.35	915.6
Total Contributing Drainage Area:	27,580			**Total Post Development Runoff Volume**	**4,114**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**1,683**
Total First Flush Volume Required:	**875**
Retention Basin #3 Provided Retention Volume:	2,223

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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North Main Parking Lot Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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North Main Parking Lot Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #4)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	17,980	0.45	8,091.0	2.35	1,584.5
Total Contributing Drainage Area:	17,980		**Total Pre Development Runoff Volume**		**1,584**

North Main Parking Lot Post Development 100-yr 2-hr Runoff Volume (Retention Basin #4)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Desert Landscaping	17,980	0.45	8,091.0	2.35	1,584.5
Total Contributing Drainage Area:	17,980		**Total Post Development Runoff Volume**		**1,584**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**0**
Total First Flush Volume Required:	**337**
Retention Basin #4 Provided Retention Volume (0.5 ft Depth):	560

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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Main Parking Lot & Sports Complex Plaza Areas Pre vs Post Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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Main Parking Lot & Sports Complex Plaza Areas Pre 100-yr 2-hr Runoff Volume (Detention Basin)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis					
Total Contributing Drainage Area:	329,710			**Total Pre Development Runoff Volume**	**0**

Main Parking Lot & Sports Complex Plaza Areas Post 100-yr 2-hr Runoff Volume (Detention Basin)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis					
Total Contributing Drainage Area:	329,710			**Total Post Development Runoff Volume**	**0**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume): **0**

Total First Flush Volume Required: **11,286**

Retention Basin #5 Provided Retention Volume~: 21,344

~The provided retention basin #5 volume is calculated to the primary basin outlet spill elevation of 1567.20 ft.

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Retention Basin Drain Time Calculation

Basin Drain Times

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
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Retention Basin	Depth of Ponding (ft)	Tested Perc. Rate* (in/hr)	Safty Factor^	Effective Perc. Rate (ft/hr)	Basin Drain Time (hr)
Retention Basin #1	1.00	2.50	0.5	0.1042	9.6
Retention Basin #2	1.00	2.50	0.5	0.1042	9.6
Retention Basin #3	1.00	3.25	0.5	0.1354	7.4
Retention Basin #4	0.30	*Depth of ponding less than 6 inches, therefore no drain time calculation performed*			
Detention Basin~	0.50	1.75	0.5	0.0729	6.9

*The tested percolation rate was obtained from a Double Ring Infiltration Test performed by Speedie and Associates at the location of the four proposed basins. Refer to the letter by Speedie and Associates in the Digital Data folder for the location and results of the infiltration test.

^In accordance with the *City of Scottsdale Drainage Policies and Standards Manual* a safty factor of 50% was applied to the tested percolation rate to obtain the effective percolation rate.

~Depths of ponding in the detention basin greater than 0.50 feet are drained through the basin bleed-off inlet and 3-inch orifice plate. During the 100-year, 6-hour storm event, the bleed-off inlet will reduce the basin depth to 0.50 feet in approximately 17.3 hours. The additional 0.50 feet, below the bleed-off inlet invert will drain in an additional 6.9 hours, for a total basin drain time of 24.2 hours.

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Appendix B: Hydrologic Analysis

Existing Conditions: HEC-1 Schematic w/100-year, 6-hour Model

LEGEND & HEC-1 SYMBOLOGY

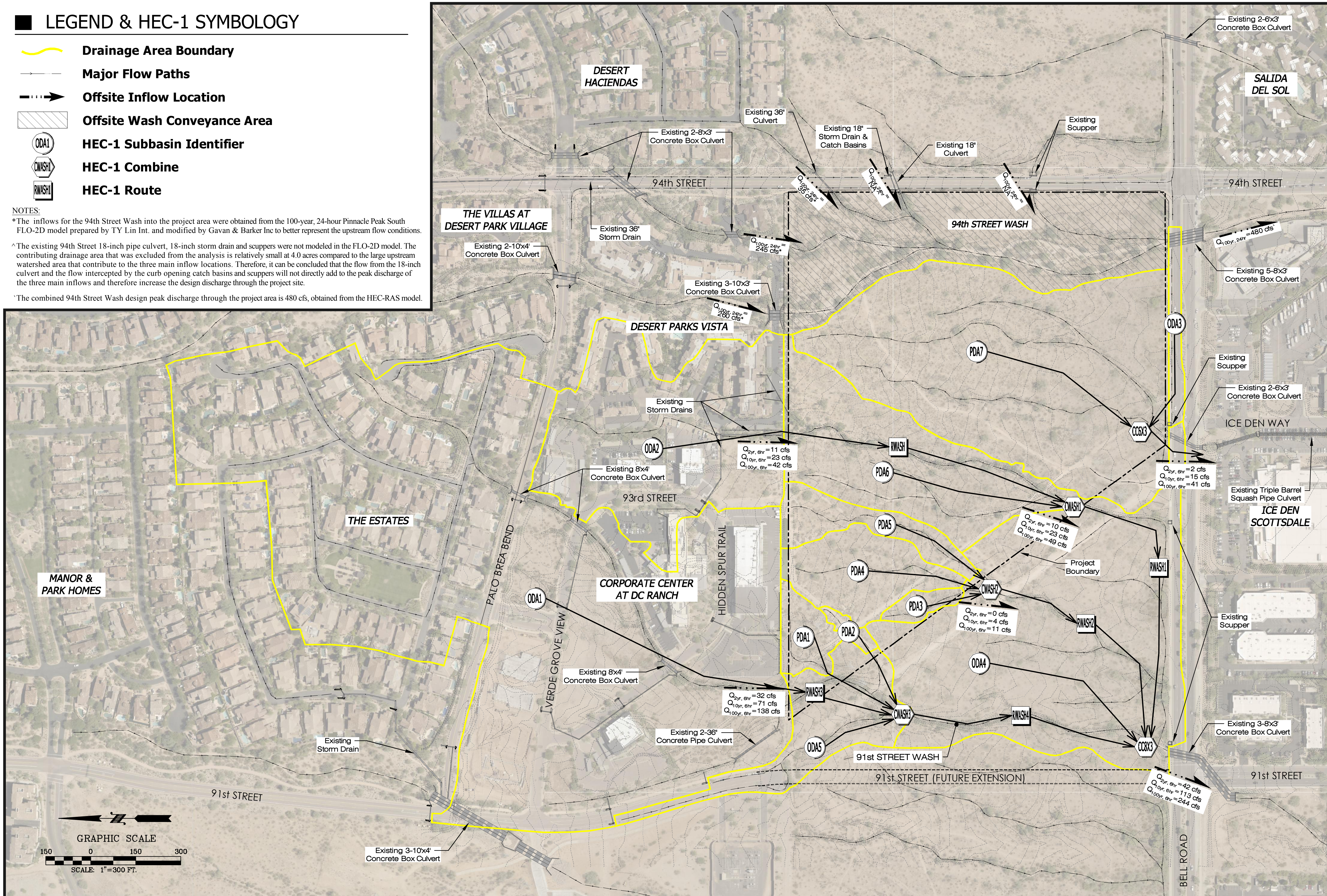
- Drainage Area Boundary
- Major Flow Paths
- Offsite Inflow Location
- Offsite Wash Conveyance Area
- ODA1 HEC-1 Subbasin Identifier
- CWASH1 HEC-1 Combine
- RWASH1 HEC-1 Route

NOTES:

*The inflows for the 94th Street Wash into the project area were obtained from the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin Int. and modified by Gavan & Barker Inc to better represent the upstream flow conditions.

^The existing 94th Street 18-inch pipe culvert, 18-inch storm drain and scuppers were not modeled in the FLO-2D model. The contributing drainage area that was excluded from the analysis is relatively small at 4.0 acres compared to the large upstream watershed area that contribute to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins and scuppers will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

`The combined 94th Street Wash design peak discharge through the project area is 480 cfs, obtained from the HEC-RAS model.



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Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
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Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

Existing Condition HEC-1 Drainage Area Map

Sheet Number:

1

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 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                       *    U.S. ARMY CORPS OF ENGINEERS    *
 *             JUN   1998              *                                        *    HYDROLOGIC ENGINEERING CENTER   *
 *           VERSION 4.1               *                                        *          609 SECOND STREET         *
 *                                     *                                        *       DAVIS, CALIFORNIA 95616      *
 *  RUN DATE   06SEP20  TIME  12:51:11  *                                       *           (916) 756-1104           *
 *                                     *                                        *                                    *
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                              XXXXXXX  XXXX     X        XXXXX  X
                              X     X  X        X               X
                              X     X  X        X     X         X
                              X     X  XXXXXXX   XXXXX         XXX


        THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

        THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
        THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
        NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
        DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
        KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                       HEC-1 INPUT                                          PAGE  1

        LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

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          2         ID        MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
          3         ID        100 YEAR
          4         ID        6 Hour  Storm
          5         ID        Unit Hydrograph: Clark
          6         ID        Storm: Single
          7         ID        06/02/2020
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          8         IT       2  1JAN99       0     360
          9         IO       5
         10         IN      15
                    *
                    *
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10-UP-2020
9/30/2020

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  11           KK    ODA3   BASIN
  12           BA   0.001
  13           PB   2.807
  14           PC   0.000   0.008   0.016   0.025   0.033   0.041   0.050   0.058   0.066   0.074
  15           PC   0.087   0.099   0.118   0.138   0.216   0.377   0.834   0.911   0.931   0.950
  16           PC   0.962   0.972   0.983   0.991   1.000
  17           LG    0.05    0.35    4.03    0.39      95
  18           UC   0.185   0.498
  19           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  20           UA     100
  21           ZW   A=ODA3   B=BASIN  C=FLOW  F=CALC
               *

  22           KK    PDA7   BASIN
  23           BA   0.024
  24           LG    0.35    0.35    4.03    0.43       0
  25           UC   0.257   0.197
  26           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  27           UA     100
  28           ZW   A=PDA7   B=BASIN  C=FLOW  F=CALC
               *

  29           KK   CC6X3 COMBINE
  30           HC       2       2
  31           ZW   A=CC6X3   B=COMBINE  C=FLOW  F=CALC
               *

  32           KK    ODA2   BASIN
  33           BA   0.016
  34           LG    0.19    0.25    4.03    0.59      51
  35           UC   0.142   0.098
  36           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  37           UA     100
  38           ZW   A=ODA2   B=BASIN  C=FLOW  F=CALC
               *
1                                         HEC-1 INPUT                                          PAGE  2

LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  39           KK   RWASH   ROUTE
  40           RS       1    FLOW
  41           RC   0.045   0.030   0.045    1020  0.0168    3.00
  42           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
  43           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
  44           ZW   A=RWASH   B=ROUTE  C=FLOW  F=CALC
               *

  45           KK    PDA6   BASIN
  46           BA   0.010
```

```
 47           LG    0.35    0.35    4.03    0.43       0
 48           UC   0.285   0.336
 49           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 50           UA     100
 51           ZW   A=PDA6   B=BASIN  C=FLOW  F=CALC
              *

 52           KK  CWASH1 COMBINE
 53           HC       2
 54           ZW   A=CWASH1   B=COMBINE  C=FLOW  F=CALC
              *

 55           KK  RWASH1   ROUTE
 56           RS       1    FLOW
 57           RC   0.045   0.030   0.045    1050  0.0087    3.00
 58           RX    0.00   10.00   20.00   30.00   40.00   50.00   60.00   70.00
 59           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 60           ZW   A=RWASH1   B=ROUTE  C=FLOW  F=CAL
              *

 61           KK    PDA5   BASIN
 62           BA   0.003
 63           LG    0.35    0.35    4.03    0.43       0
 64           UC   0.240   0.407
 65           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 66           UA     100
 67           ZW   A=PDA5   B=BASIN  C=FLOW  F=CALC
              *

 68           KK    PDA4   BASIN
 69           BA   0.004
 70           LG    0.35    0.35    4.03    0.43       0
 71           UC   0.205   0.253
 72           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 73           UA     100
 74           ZW   A=PDA4   B=BASIN  C=FLOW  F=CALC
              *

 75           KK    PDA3   BASIN
 76           BA   0.001
 77           LG    0.35    0.35    4.03    0.43       0
 78           UC   0.147   0.206
 79           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 80           UA     100
1                                         HEC-1 INPUT                                        PAGE  3

LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

 81           ZW   A=PDA3   B=BASIN  C=FLOW  F=CALC
              *
```

```
 82           KK  CWASH2 COMBINE
 83           HC       3
 84           ZW   A=CWASH2   B=COMBINE  C=FLOW  F=CALC
              *

 85           KK  RWASH2   ROUTE
 86           RS       1    FLOW
 87           RC   0.045   0.030   0.045     860  0.0128    3.00
 88           RX    0.00   10.00   20.00   25.00   28.00   33.00   43.00   53.00
 89           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 90           ZW   A=RWASH2   B=ROUTE  C=FLOW  F=CAL
              *

 91           KK    PDA2   BASIN
 92           BA   0.001
 93           LG    0.35    0.35    4.03    0.43       0
 94           UC   0.128   0.148
 95           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 96           UA     100
 97           ZW   A=PDA2   B=BASIN  C=FLOW  F=CALC
              *

 98           KK    PDA1   BASIN
 99           BA   0.002
100           LG    0.35    0.35    4.03    0.43       0
101           UC   0.174   0.219
102           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
103           UA     100
104           ZW   A=PDA1   B=BASIN  C=FLOW  F=CALC
              *

105           KK    ODA1   BASIN
106           BA   0.069
107           LG    0.20    0.25    4.03    0.59      48
108           UC   0.247   0.200
109           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
110           UA     100
111           ZW   A=ODA1   B=BASIN  C=FLOW  F=CALC
              *

112           KK  RWASH3   ROUTE
113           RS       1    FLOW
114           RC   0.045   0.030   0.045     450  0.0138    3.00
115           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
116           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
117           ZW   A=RWASH3   B=ROUTE  C=FLOW  F=CAL
              *
```

```
         118           KK    ODA9   BASIN
         119           BA   0.006
         120           LG    0.29    0.35    4.03    0.42      19
         121           UC   0.189   0.211
         122           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
         123           UA     100
         124           ZW   A=ODA9   B=BASIN  C=FLOW  F=CALC
                       *

         125           KK  CWASH3 COMBINE
         126           HC       4
         127           ZW   A=CWASH3   B=COMBINE  C=FLOW  F=CALC
                       *

         128           KK  RWASH4   ROUTE
         129           RS       1    FLOW
         130           RC   0.045   0.030   0.045     870  0.0127    3.00
         131           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
         132           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
         133           ZW   A=RWASH4   B=ROUTE  C=FLOW  F=CAL
                       *

         134           KK    ODA8   BASIN
         135           BA   0.025
         136           LG    0.33    0.35    4.03    0.43       6
         137           UC   0.276   0.184
         138           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
         139           UA     100
         140           ZW   A=ODA8   B=BASIN  C=FLOW  F=CALC
                       *

         141           KK   CC8X3 COMBINE
         142           HC       4
         143           ZW   A=CC8X3   B=COMBINE  C=FLOW  F=CALC
                       *
         144           ZZ
1
                  SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

    11        ODA3
                 .
                 .
    22           .         PDA7
                 .            .
                 .            .
    29       CC6X3............
```

```
                             .
                             .
                32           .        ODA2
                             .           V
                             .           V
                39           .       RWASH
                             .           .
                             .           .
                45           .           .        PDA6
                             .           .           .
                             .           .           .
                52           .      CWASH1............
                             .           V
                             .           V
                55           .      RWASH1
                             .           .
                             .           .
                61           .           .        PDA5
                             .           .           .
                             .           .           .
                68           .           .           .        PDA4
                             .           .           .           .
                             .           .           .           .
                75           .           .           .           .        PDA3
                             .           .           .           .           .
                             .           .           .           .           .
                82           .           .      CWASH2........................
                             .           .           V
                             .           .           V
                85           .           .      RWASH2
                             .           .           .
                             .           .           .
                91           .           .           .        PDA2
                             .           .           .           .
                             .           .           .           .
                98           .           .           .           .        PDA1
                             .           .           .           .           .
                             .           .           .           .           .
               105           .           .           .           .           .        ODA1
                             .           .           .           .           .           V
                             .           .           .           .           .           V
               112           .           .           .           .           .       RWASH3
                             .           .           .           .           .           .
                             .           .           .           .           .           .
               118           .           .           .           .           .           .        ODA9
                             .           .           .           .           .           .           .
                             .           .           .           .           .           .           .
               125           .           .           .      CWASH3....................................
                             .           .           .           V
                             .           .           .           V
               128           .           .           .      RWASH4
```

10-UP-2020
9/30/2020

```
                .           .           .           .
                .           .           .           .
  134           .           .           .           .        ODA8
                .           .           .           .           .
                .           .           .           .           .
  141           .       CC8X3...................................

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                                                    ***************************************
*                                        *                                                    *                                     *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)    *                                                    *    U.S. ARMY CORPS OF ENGINEERS     *
*              JUN   1998                *                                                    *    HYDROLOGIC ENGINEERING CENTER    *
*           VERSION 4.1                  *                                                    *          609 SECOND STREET          *
*                                        *                                                    *       DAVIS, CALIFORNIA 95616       *
*  RUN DATE   06SEP20  TIME  12:51:11    *                                                    *           (916) 756-1104            *
*                                        *                                                    *                                     *
******************************************                                                    ***************************************



                              City of Scottsdale
                              MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
                              100 YEAR
                              6 Hour  Storm
                              Unit Hydrograph: Clark
                              Storm: Single
                              06/02/2020

   9 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

     IT          HYDROGRAPH TIME DATA
                        NMIN           2  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1JAN99  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ         360  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1JAN99  ENDING DATE
                      NDTIME        1158  ENDING TIME
                      ICENT           19  CENTURY MARK

                   COMPUTATION INTERVAL     .03 HOURS
                        TOTAL TIME BASE   11.97 HOURS

          ENGLISH UNITS
               DRAINAGE AREA         SQUARE MILES
               PRECIPITATION DEPTH   INCHES
               LENGTH, ELEVATION     FEET
```

10-UP-2020
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FLOW	CUBIC FEET PER SECOND
STORAGE VOLUME	ACRE-FEET
SURFACE AREA	ACRES
TEMPERATURE	DEGREES FAHRENHEIT

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	ODA3	2.	4.07	0.	0.	0.	.00		
+	HYDROGRAPH AT	PDA7	39.	4.13	3.	1.	1.	.02		
+	2 COMBINED AT	CC6X3	41.	4.13	3.	1.	1.	.03		
+	HYDROGRAPH AT	ODA2	42.	4.03	3.	2.	2.	.02		
+	ROUTED TO	RWASH	40.	4.07	3.	2.	2.	.02		
+									.90	4.07
+	HYDROGRAPH AT	PDA6	12.	4.17	1.	1.	1.	.01		
+	2 COMBINED AT	CWASH1	49.	4.07	4.	2.	2.	.03		
+	ROUTED TO	RWASH1	44.	4.13	4.	2.	2.	.03		
+									.79	4.13
+	HYDROGRAPH AT	PDA5	3.	4.17	0.	0.	0.	.00		
+	HYDROGRAPH AT	PDA4	6.	4.13	0.	0.	0.	.00		
+	HYDROGRAPH AT	PDA3	2.	4.07	0.	0.	0.	.00		
+	3 COMBINED AT	CWASH2	11.	4.13	1.	0.	0.	.01		

```
          ROUTED TO
+                          RWASH2        10.     4.17           1.          0.          0.       .01
+                                                                                                       .56        4.17

          HYDROGRAPH AT
+                            PDA2         2.     4.07           0.          0.          0.       .00

          HYDROGRAPH AT
+                            PDA1         3.     4.10           0.          0.          0.       .00

          HYDROGRAPH AT
+                            ODA1       138.     4.07          14.          7.          7.       .07

          ROUTED TO
+                          RWASH3       136.     4.10          14.          7.          7.       .07
+                                                                                                      1.61        4.10

          HYDROGRAPH AT
+                            ODA9        11.     4.10           1.          0.          0.       .01

          4 COMBINED AT
+                          CWASH3       152.     4.10          15.          8.          8.       .08

          ROUTED TO
+                          RWASH4       148.     4.13          15.          8.          8.       .08
+                                                                                                      1.69        4.13

          HYDROGRAPH AT
+                            ODA8        43.     4.17           3.          2.          2.       .03

          4 COMBINED AT
+                           CC8X3       244.     4.13          23.         12.         12.       .14


 *** NORMAL END OF HEC-1 ***


    -----DSS---ZCLOSE Unit:  71,   File: EC100.DSS
               Pointer Utilization:   .27
               Number of Records:     56
               File Size:    209.4  Kbytes
               Percent Inactive:    .0
```

Proposed Conditions: HEC-1 Schematic w/100-year, 6-hour Model

LEGEND & HEC-1 SYMBOLOGY

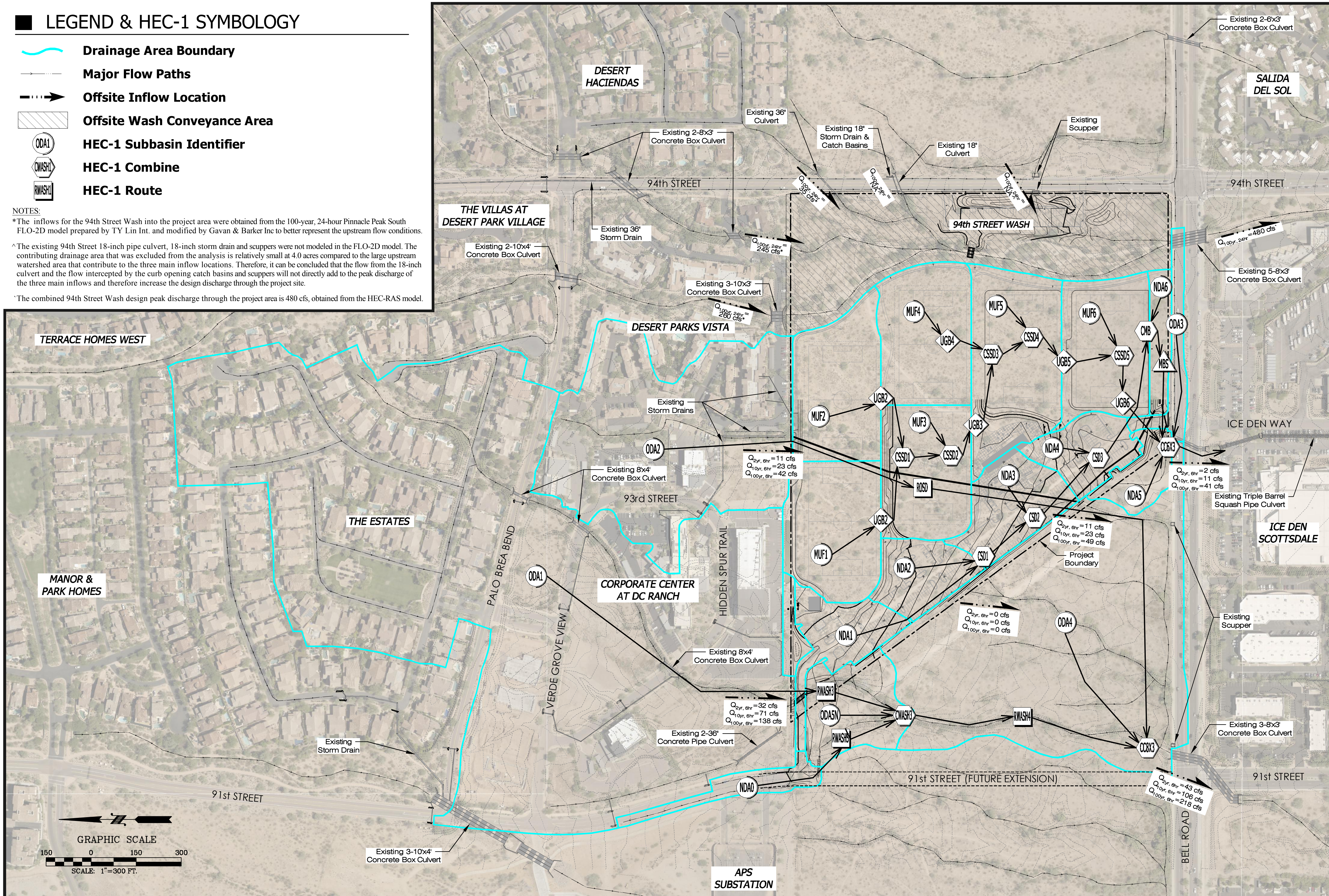
- Drainage Area Boundary
- Major Flow Paths
- Offsite Inflow Location
- Offsite Wash Conveyance Area
- ODA1 — HEC-1 Subbasin Identifier
- CWASH1 — HEC-1 Combine
- RWASH1 — HEC-1 Route

NOTES:

*The inflows for the 94th Street Wash into the project area were obtained from the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin Int. and modified by Gavan & Barker Inc to better represent the upstream flow conditions.

^The existing 94th Street 18-inch pipe culvert, 18-inch storm drain and scuppers were not modeled in the FLO-2D model. The contributing drainage area that was excluded from the analysis is relatively small at 4.0 acres compared to the large upstream watershed area that contribute to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins and scuppers will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

`The combined 94th Street Wash design peak discharge through the project area is 480 cfs, obtained from the HEC-RAS model.



Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032
Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

Proposed Condition HEC-1 Drainage Area Map

Sheet Number:

1

10-UP-2020
9/30/2020

```
1***************************************                                                     ***************************************
 *                                     *                                                     *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                   *    U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998             *                                                     *    HYDROLOGIC ENGINEERING CENTER    *
 *           VERSION 4.1               *                                                     *          609 SECOND STREET          *
 *                                     *                                                     *       DAVIS, CALIFORNIA 95616       *
 * RUN DATE   06SEP20  TIME  12:51:11   *                                                    *           (916) 756-1104            *
 *                                     *                                                     *                                     *
 ***************************************                                                     ***************************************



                                                X     X  XXXXXXX   XXXXX         X
                                                X     X  X        X     X       XX
                                                X     X  X        X              X
                                                XXXXXXX  XXXX     X       XXXXX  X
                                                X     X  X        X              X
                                                X     X  X        X     X        X
                                                X     X  XXXXXXX   XXXXX        XXX


          THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

          THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
          THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
          NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
          DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
          KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                                       HEC-1 INPUT                                                 PAGE  1

          LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

            1            ID        City of Scottsdale
            2            ID        MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
            3            ID        100 YEAR
            4            ID        6 Hour  Storm
            5            ID        Unit Hydrograph: Clark
            6            ID        Storm: Single
            7            ID        06/02/2020
                         *DIAGRAM
            8            IT       2  1JAN99       0     360
            9            IO       5
           10            IN      15
                         *
                         *
```

10-UP-2020
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```
  11           KK    ODA3   BASIN
  12           BA   0.001
  13           PB   2.807
  14           PC   0.000   0.008   0.016   0.025   0.033   0.041   0.050   0.058   0.066   0.074
  15           PC   0.087   0.099   0.118   0.138   0.216   0.377   0.834   0.911   0.931   0.950
  16           PC   0.962   0.972   0.983   0.991   1.000
  17           LG    0.05    0.35    4.03    0.39      95
  18           UC   0.185   0.498
  19           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  20           UA     100
  21           ZW   A=ODA3   B=BASIN  C=FLOW  F=CALC
               *

  22           KK    PDA7   BASIN
  23           BA   0.024
  24           LG    0.35    0.35    4.03    0.43       0
  25           UC   0.257   0.197
  26           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  27           UA     100
  28           ZW   A=PDA7   B=BASIN  C=FLOW  F=CALC
               *

  29           KK   CC6X3 COMBINE
  30           HC       2       2
  31           ZW   A=CC6X3   B=COMBINE  C=FLOW  F=CALC
               *

  32           KK    ODA2   BASIN
  33           BA   0.016
  34           LG    0.19    0.25    4.03    0.59      51
  35           UC   0.142   0.098
  36           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  37           UA     100
  38           ZW   A=ODA2   B=BASIN  C=FLOW  F=CALC
               *
1                                          HEC-1 INPUT                                     PAGE  2

LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  39           KK   RWASH   ROUTE
  40           RS       1    FLOW
  41           RC   0.045   0.030   0.045    1020  0.0168    3.00
  42           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
  43           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
  44           ZW   A=RWASH   B=ROUTE  C=FLOW  F=CALC
               *

  45           KK    PDA6   BASIN
  46           BA   0.010
```

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```
      47           LG    0.35    0.35    4.03    0.43       0
      48           UC   0.285   0.336
      49           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      50           UA     100
      51           ZW   A=PDA6   B=BASIN  C=FLOW  F=CALC
                   *

      52           KK  CWASH1 COMBINE
      53           HC       2
      54           ZW   A=CWASH1   B=COMBINE  C=FLOW  F=CALC
                   *

      55           KK  RWASH1   ROUTE
      56           RS       1    FLOW
      57           RC   0.045   0.030   0.045    1050  0.0087    3.00
      58           RX    0.00   10.00   20.00   30.00   40.00   50.00   60.00   70.00
      59           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
      60           ZW   A=RWASH1   B=ROUTE  C=FLOW  F=CAL
                   *

      61           KK    PDA5   BASIN
      62           BA   0.003
      63           LG    0.35    0.35    4.03    0.43       0
      64           UC   0.240   0.407
      65           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      66           UA     100
      67           ZW   A=PDA5   B=BASIN  C=FLOW  F=CALC
                   *

      68           KK    PDA4   BASIN
      69           BA   0.004
      70           LG    0.35    0.35    4.03    0.43       0
      71           UC   0.205   0.253
      72           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      73           UA     100
      74           ZW   A=PDA4   B=BASIN  C=FLOW  F=CALC
                   *

      75           KK    PDA3   BASIN
      76           BA   0.001
      77           LG    0.35    0.35    4.03    0.43       0
      78           UC   0.147   0.206
      79           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      80           UA     100
1                                             HEC-1 INPUT                                            PAGE  3

    LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

      81           ZW   A=PDA3   B=BASIN  C=FLOW  F=CALC
                   *
```

```
 82           KK  CWASH2 COMBINE
 83           HC       3
 84           ZW   A=CWASH2   B=COMBINE  C=FLOW  F=CALC
              *

 85           KK  RWASH2   ROUTE
 86           RS       1    FLOW
 87           RC   0.045   0.030   0.045     860  0.0128    3.00
 88           RX    0.00   10.00   20.00   25.00   28.00   33.00   43.00   53.00
 89           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 90           ZW   A=RWASH2   B=ROUTE  C=FLOW  F=CAL
              *

 91           KK    PDA2   BASIN
 92           BA   0.001
 93           LG    0.35    0.35    4.03    0.43       0
 94           UC   0.128   0.148
 95           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 96           UA     100
 97           ZW   A=PDA2   B=BASIN  C=FLOW  F=CALC
              *

 98           KK    PDA1   BASIN
 99           BA   0.002
100           LG    0.35    0.35    4.03    0.43       0
101           UC   0.174   0.219
102           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
103           UA     100
104           ZW   A=PDA1   B=BASIN  C=FLOW  F=CALC
              *

105           KK    ODA1   BASIN
106           BA   0.069
107           LG    0.20    0.25    4.03    0.59      48
108           UC   0.247   0.200
109           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
110           UA     100
111           ZW   A=ODA1   B=BASIN  C=FLOW  F=CALC
              *

112           KK  RWASH3   ROUTE
113           RS       1    FLOW
114           RC   0.045   0.030   0.045     450  0.0138    3.00
115           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
116           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
117           ZW   A=RWASH3   B=ROUTE  C=FLOW  F=CAL
              *
1                                                HEC-1 INPUT                                              PAGE  4

        LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10
```

10-UP-2020
9/30/2020

```
118           KK    ODA9   BASIN
119           BA   0.006
120           LG    0.29    0.35    4.03    0.42      19
121           UC   0.189   0.211
122           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
123           UA     100
124           ZW   A=ODA9   B=BASIN  C=FLOW  F=CALC
              *

125           KK  CWASH3 COMBINE
126           HC       4
127           ZW   A=CWASH3   B=COMBINE  C=FLOW  F=CALC
              *

128           KK  RWASH4   ROUTE
129           RS       1    FLOW
130           RC   0.045   0.030   0.045     870  0.0127    3.00
131           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
132           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
133           ZW   A=RWASH4   B=ROUTE  C=FLOW  F=CAL
              *

134           KK    ODA8   BASIN
135           BA   0.025
136           LG    0.33    0.35    4.03    0.43       6
137           UC   0.276   0.184
138           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
139           UA     100
140           ZW   A=ODA8   B=BASIN  C=FLOW  F=CALC
              *

141           KK   CC8X3 COMBINE
142           HC       4
143           ZW   A=CC8X3   B=COMBINE  C=FLOW  F=CALC
              *
144           ZZ
1
                  SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

    11        ODA3
                 .
                 .
    22           .         PDA7
                 .            .
                 .            .
    29       CC6X3............
```

10-UP-2020
9/30/2020

```
               .
               .
  32           .        ODA2
               .           V
               .           V
  39           .       RWASH
               .           .
               .           .
  45           .           .        PDA6
               .           .           .
               .           .           .
  52           .      CWASH1............
               .           V
               .           V
  55           .      RWASH1
               .           .
               .           .
  61           .           .        PDA5
               .           .           .
               .           .           .
  68           .           .           .        PDA4
               .           .           .           .
               .           .           .           .
  75           .           .           .           .        PDA3
               .           .           .           .           .
               .           .           .           .           .
  82           .           .      CWASH2........................
               .           .           V
               .           .           V
  85           .           .      RWASH2
               .           .           .
               .           .           .
  91           .           .           .        PDA2
               .           .           .           .
               .           .           .           .
  98           .           .           .           .        PDA1
               .           .           .           .           .
               .           .           .           .           .
 105           .           .           .           .           .        ODA1
               .           .           .           .           .           V
               .           .           .           .           .           V
 112           .           .           .           .           .      RWASH3
               .           .           .           .           .           .
               .           .           .           .           .           .
 118           .           .           .           .           .           .        ODA9
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 125           .           .           .      CWASH3....................................
               .           .           .           V
               .           .           .           V
 128           .           .           .      RWASH4
```

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```
                   .           .           .           .
                   .           .           .           .
  134              .           .           .           .        ODA8
                   .           .           .           .           .
                   .           .           .           .           .
  141              .       CC8X3...................................

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                                                    ****************************************
*                                       *                                                    *                                      *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                    *    U.S. ARMY CORPS OF ENGINEERS      *
*              JUN   1998               *                                                    *    HYDROLOGIC ENGINEERING CENTER     *
*           VERSION 4.1                 *                                                    *          609 SECOND STREET           *
*                                       *                                                    *       DAVIS, CALIFORNIA 95616        *
*  RUN DATE   06SEP20  TIME  12:51:11   *                                                    *           (916) 756-1104             *
*                                       *                                                    *                                      *
*****************************************                                                    ****************************************



                                          City of Scottsdale
                                          MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
                                          100 YEAR
                                          6 Hour  Storm
                                          Unit Hydrograph: Clark
                                          Storm: Single
                                          06/02/2020

   9 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

     IT          HYDROGRAPH TIME DATA
                        NMIN           2  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1JAN99  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ         360  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1JAN99  ENDING DATE
                      NDTIME        1158  ENDING TIME
                      ICENT           19  CENTURY MARK

                 COMPUTATION INTERVAL     .03 HOURS
                      TOTAL TIME BASE   11.97 HOURS

        ENGLISH UNITS
             DRAINAGE AREA         SQUARE MILES
             PRECIPITATION DEPTH   INCHES
             LENGTH, ELEVATION     FEET
```

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FLOW	CUBIC FEET PER SECOND
STORAGE VOLUME	ACRE-FEET
SURFACE AREA	ACRES
TEMPERATURE	DEGREES FAHRENHEIT

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	ODA3	2.	4.07	0.	0.	0.	.00		
HYDROGRAPH AT	PDA7	39.	4.13	3.	1.	1.	.02		
2 COMBINED AT	CC6X3	41.	4.13	3.	1.	1.	.03		
HYDROGRAPH AT	ODA2	42.	4.03	3.	2.	2.	.02		
ROUTED TO	RWASH	40.	4.07	3.	2.	2.	.02		
								.90	4.07
HYDROGRAPH AT	PDA6	12.	4.17	1.	1.	1.	.01		
2 COMBINED AT	CWASH1	49.	4.07	4.	2.	2.	.03		
ROUTED TO	RWASH1	44.	4.13	4.	2.	2.	.03		
								.79	4.13
HYDROGRAPH AT	PDA5	3.	4.17	0.	0.	0.	.00		
HYDROGRAPH AT	PDA4	6.	4.13	0.	0.	0.	.00		
HYDROGRAPH AT	PDA3	2.	4.07	0.	0.	0.	.00		
3 COMBINED AT	CWASH2	11.	4.13	1.	0.	0.	.01		

```
          ROUTED TO
+                    RWASH2       10.    4.17        1.        0.        0.       .01
+                                                                                       .56      4.17

          HYDROGRAPH AT
+                      PDA2        2.    4.07        0.        0.        0.       .00

          HYDROGRAPH AT
+                      PDA1        3.    4.10        0.        0.        0.       .00

          HYDROGRAPH AT
+                      ODA1      138.    4.07       14.        7.        7.       .07

          ROUTED TO
+                    RWASH3      136.    4.10       14.        7.        7.       .07
+                                                                                      1.61      4.10

          HYDROGRAPH AT
+                      ODA9       11.    4.10        1.        0.        0.       .01

          4 COMBINED AT
+                    CWASH3      152.    4.10       15.        8.        8.       .08

          ROUTED TO
+                    RWASH4      148.    4.13       15.        8.        8.       .08
+                                                                                      1.69      4.13

          HYDROGRAPH AT
+                      ODA8       43.    4.17        3.        2.        2.       .03

          4 COMBINED AT
+                     CC8X3      244.    4.13       23.       12.       12.       .14


 *** NORMAL END OF HEC-1 ***


    -----DSS---ZCLOSE Unit:  71,   File: EC100.DSS
               Pointer Utilization:   .27
               Number of Records:     56
               File Size:    209.4  Kbytes
               Percent Inactive:    .0
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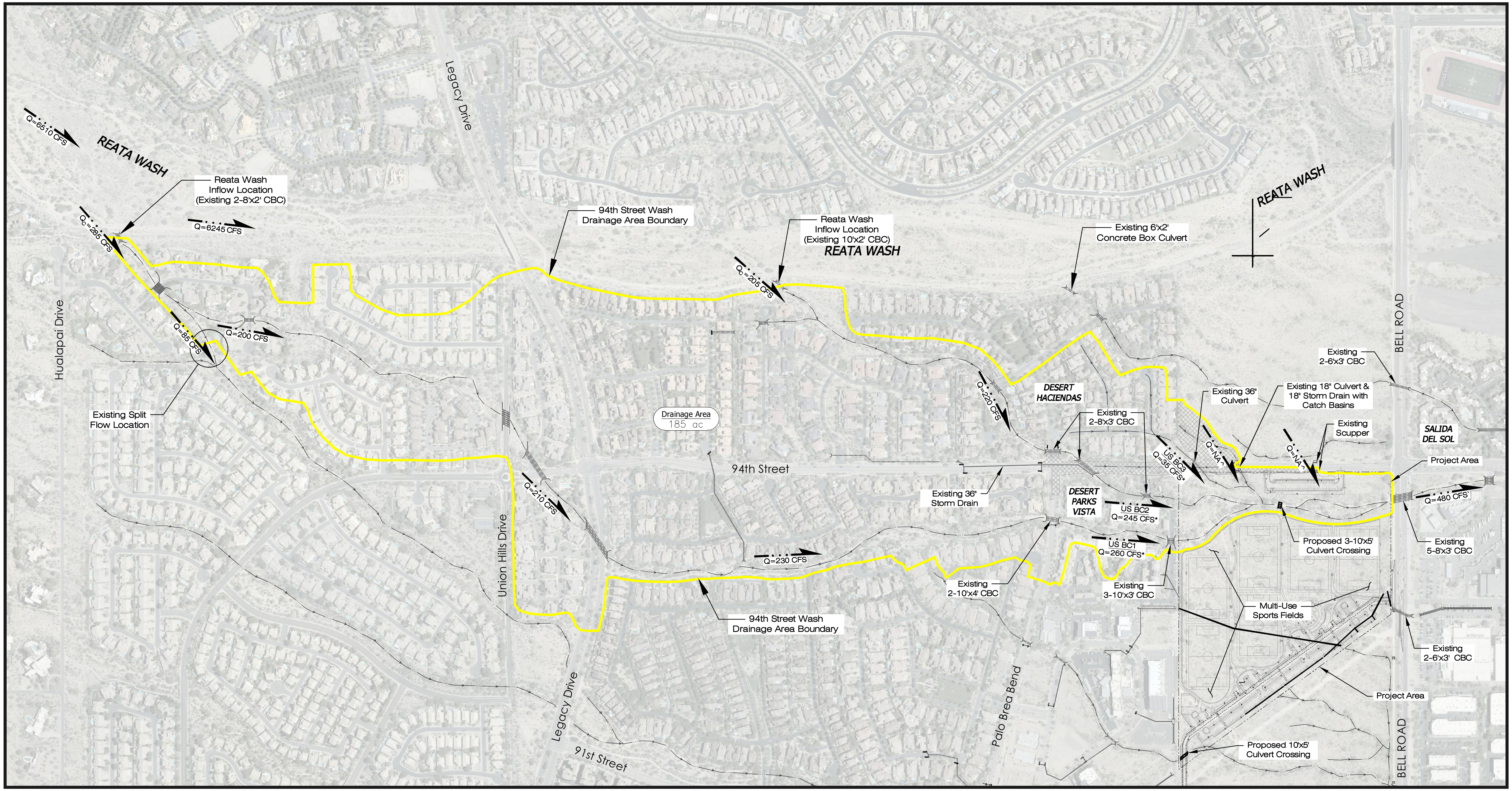
<u>94th Street Wash Drainage Area Map & Inflow Hydrographs</u>



94th STREET WASH

DRAINAGE AREA MAP

LEGEND

- **Drainage Area Boundary**
- **Major Flow Paths**
- Q=210 CFS **Major Peak Flows**
- **Excluded Drainage Area**

0 300' 600'

SCALE: 1" = 600'

NOTES:

*The inflows for the 94th Street Wash into the project area were obtained from the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin International and modified by Gavan & Barker Inc, to better represent the upstream flow conditions. The resulting inflow hydrographs at the three main inflow locations are included in Appendix B and the FLO-2D model is included in the Digital Data folder in Appendix E.

^The existing 94th Street 18-inch pipe culvert, 18-inch storm drain and scuppers were not modeled in the FLO-2D model. The contributing drainage area that was excluded from the analysis is relatively small at 4.0 acres compared to the large upstream watershed area that contribute to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins and scuppers will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

'The combined 94th Street Wash design peak discharge through the project area is 480 cfs, obtained from the HEC-RAS model.

Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032

Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :	
G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

94th Street Wash Drainage Area Map

Sheet Number:

1

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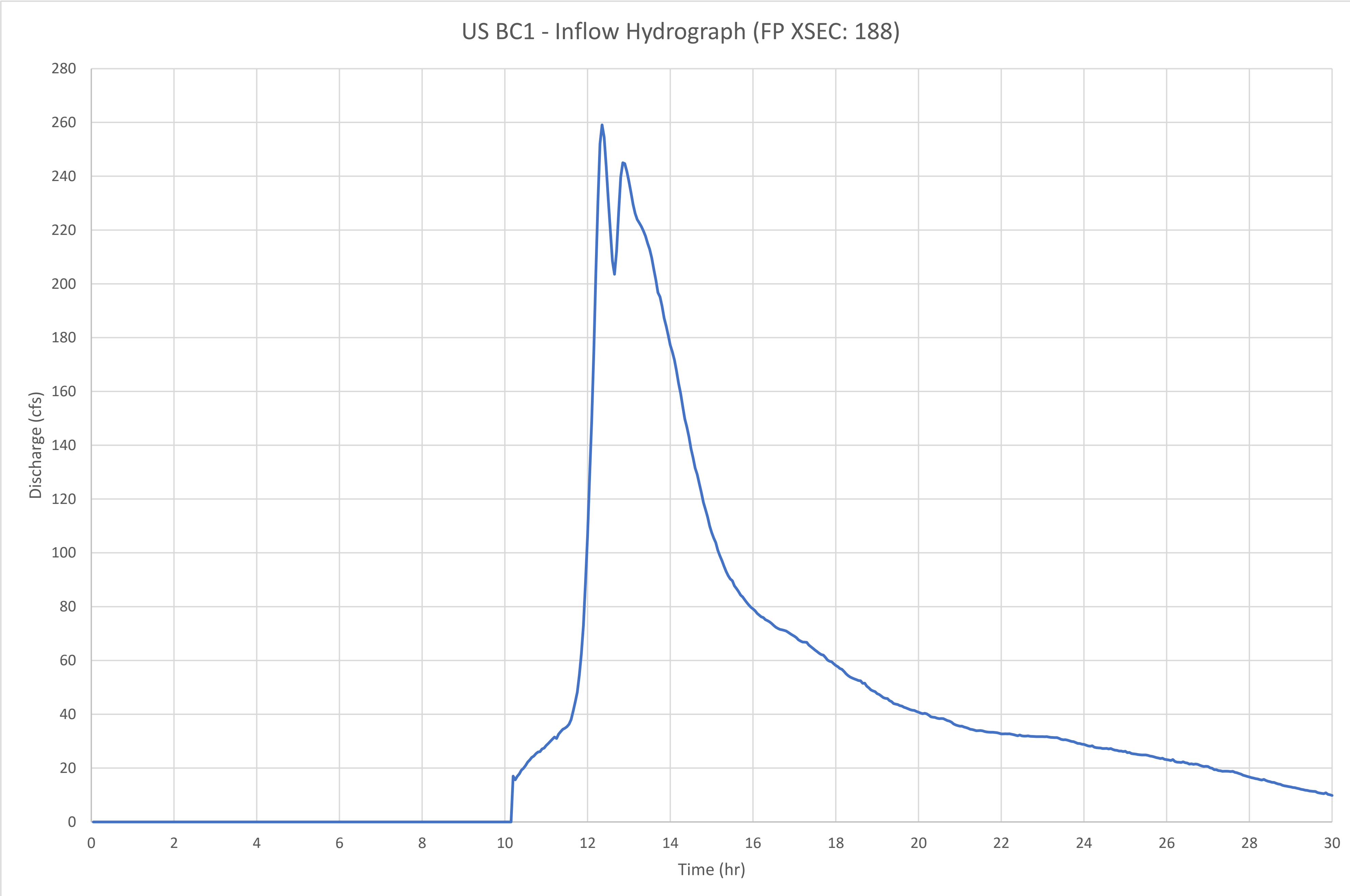

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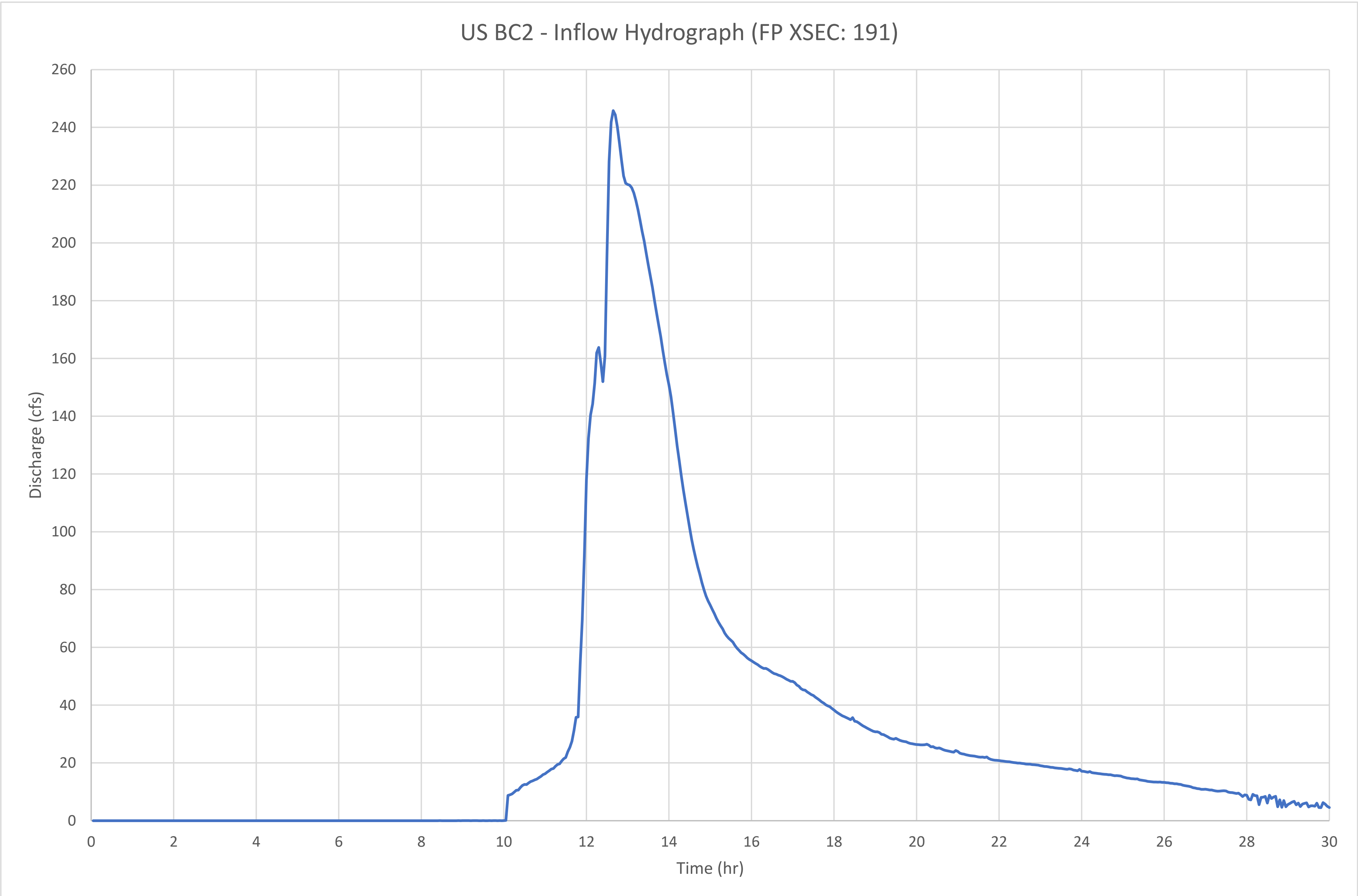

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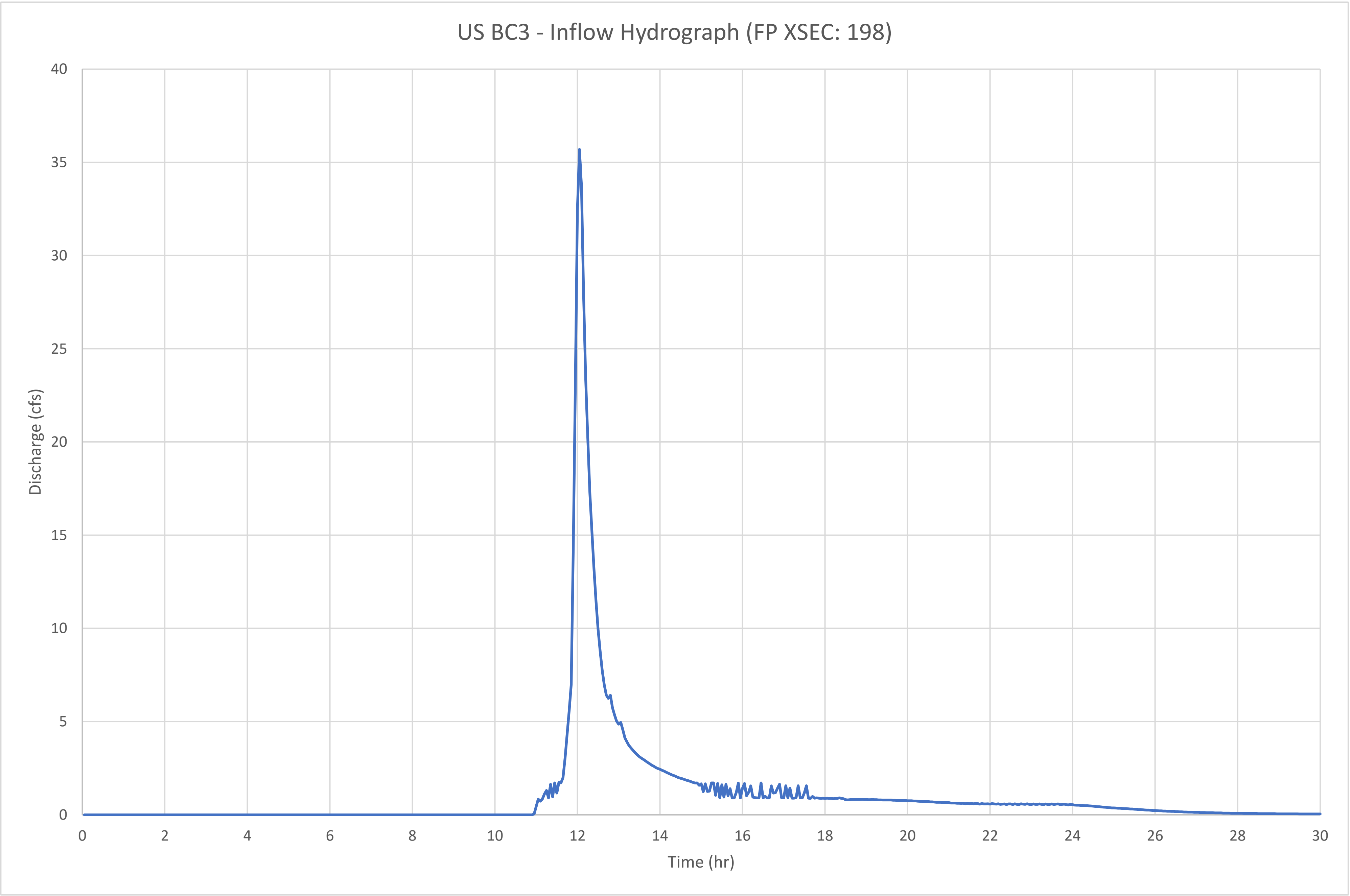

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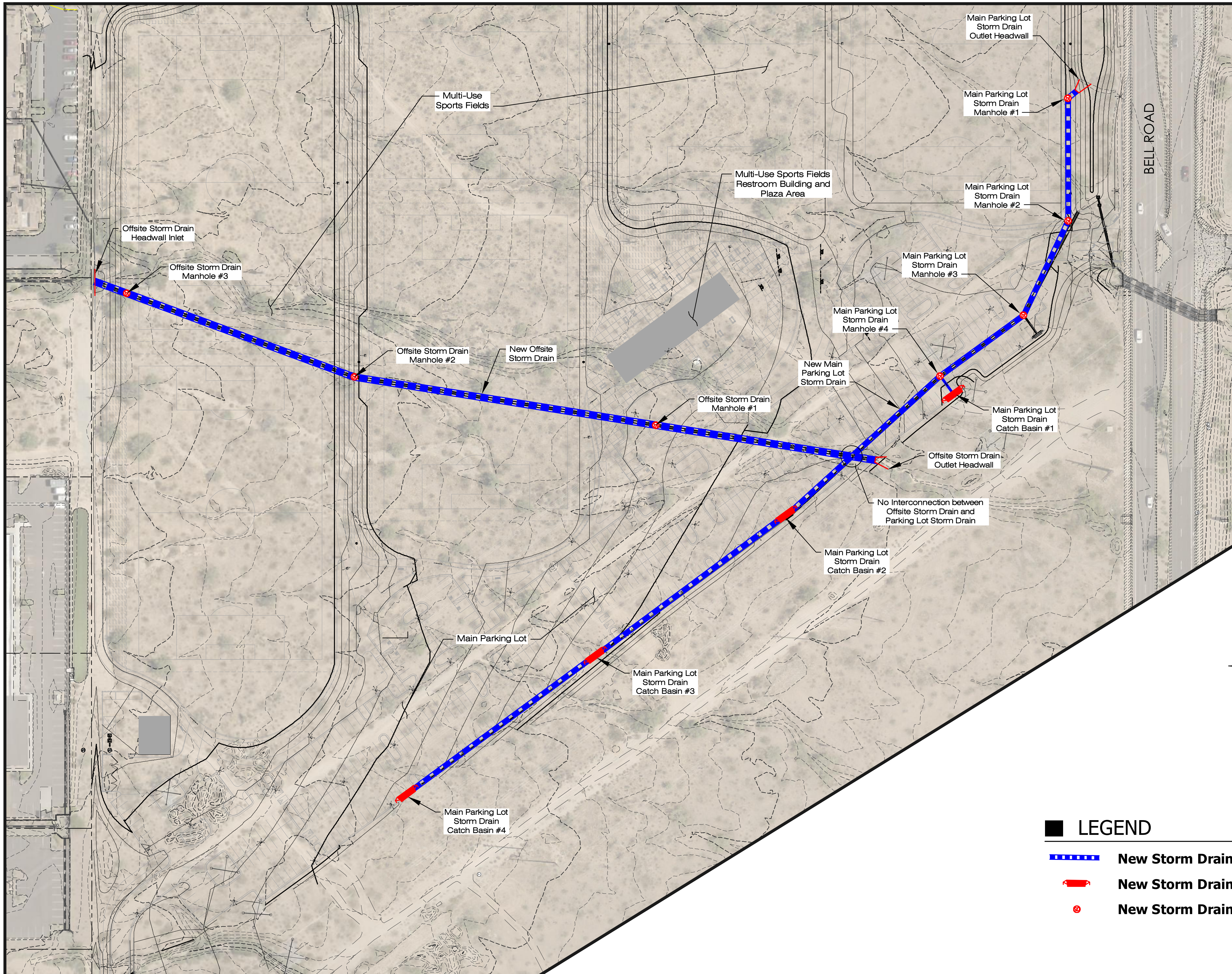
Appendix C: Storm Drain Design Calculations

Storm Drain Design Location Map



0 60' 120'
SCALE: 1" = 120'

LEGEND

- New Storm Drain
- New Storm Drain Catch Basin
- New Storm Drain Manhole

Gavan & Barker Inc.

Gavan & Barker, Inc.
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Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032

Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
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Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

Storm Drain Design Location Map

Sheet Number:
1

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Offsite Storm Drain Design Calculations

Offsite Storm Drain Hydraulic Grade Line (HGL) Summary Table

Location	Type of Headloss	Headloss *(ft)*	HGL Elevation *(ft)*
Headwall Inlet	Junction Loss (Entrance Headloss)^	0.11	1587.70
Offsite MH#3 to Headwall Inlet	Storm Drain Friction Headloss*	0.14	1585.88
Offsite MH#3	Junction Loss (Straight-Through Headloss)^	0.03	1582.40
Offsite MH#2 to Offsite MH#3	Storm Drain Friction Headloss*	1.19	1580.43
Offsite MH#2	Junction Loss (Bend Headloss)	0.04	1575.62
Offsite MH#1 to Offsite MH#2	Storm Drain Friction Headloss	1.46	1575.58
Offsite MH#1	Junction Loss (Straight-Through Headloss)	0.03	1574.12
Outlet HW to Offsite MH#1	Storm Drain Friction Headloss	1.04	1574.09
Outlet Headwall	Junction Loss (Exit Headloss)	0.55	1573.05
		Tailwater Elevation @ Outlet Headwall =	1572.50

* The pipe segments between Offsite MH#2 and the Headwall Inlet are in open channel flow conditions with the hydraulic grade line governed by the 36" Offsite Storm Drain normal depth capacity. Refer to the *Channel Reports* at the end of these calculations for the proposed storm drain normal depth analysis.

^ The hydraulic grade line elevation at Offsite MH#3 and at the Headwall Inlet are governed by the inlet control interception capacity of the 36" Offsite Storm Drain. Refer to the Inlet Control Nomograph calculation at the end of these calculations for the proposed storm drain interception capacity calculation.

Gavan & Barker Inc.

Offsite Storm Drain - Hydraulic Grade Line Calculation

The procedures outlined in Chapter 4 of the Hydraulics Drainage Design Manual for Maricopa County were used in order to compute the Hydraulic Grade Line (HGL) for the Offsite Storm Drain. The equations and figures used henceforth were also taken from the manual.

The new offsite storm drain was designed to intercept the 100-year 6-hour peak discharge of 42 cfs from the ODA2 HEC-1 sub-basin area. The ODA2 sub-basin concentrates at the northern boundary of the project site. Under existing conditions, the flow is conveyed in a natural wash through the project area in the southwesterly direction. Under proposed conditions, the offsite storm drain is designed to intercept the 100-year peak discharge and convey it underneath the new multi-use fields, plaza area and parking lot. The offsite storm drain is discharged in the same location where the existing wash leaves the project site. No additional flows are added to the offsite storm drain from the onsite watershed area. Refer to Appendix B for the HEC-1 Hydrologic Model Results.

Determine Tailwater Elevation:

Since the offiste storm drian discharges into a natural wash, with no backwater effects, the starting tailwater elevation will not be impacted by the hydraulics of the natural wash. Therefore, the soffit elevation of the proposed 36-inch offsite strom drain at the outlet headwall is the starting tailwater elevation. The invert of the 36-inch offsite storm drain is 1569.50 ft, which translates to a starting tailwater elevaton of 1572.50 ft.

$$\boxed{\textbf{Tailwater Elevation @ } \textbf{\textit{Outlet Headwall}} = \textbf{1572.50 ft}}$$

Compute the Storm Drain Outlet Headloss at Outlet Headwall

Exit Loss:

$$h_o = 1.0\frac{V^2}{2g} \quad (Equation\ 4.16)$$

where;

$h_o = Outlet\ Headloss\ at\ Manhole$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42.0}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

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$$h_o = 1.0\frac{V^2}{2g}$$

$$h_o = 1.0\frac{5.94^2}{2\times 32.2}$$

$$h_o = 0.55\,ft$$

$$\boxed{\mathbf{h_o = 0.55}\,\boldsymbol{ft}\ @\ \textbf{Outlet}\ \textbf{\textit{Headwall}}}$$

Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Outlet Headwall to Offsite MH#1)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 267\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$
$$h_f = 0.0039 \times 267$$
$$h_f = 1.04\ ft$$

$$\boxed{\mathbf{h_f = 1.04\ ft\ (Outlet\ Headwall\ to\ Offsite\ MH\#1)}}$$

Compute the Headloss at Manhole (Offsite MH #1)

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{5.94^2}{2 \times 32.2}$$

$$h_{mh} = 0.03\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.03\ ft\ @\ Offsite\ MH\ \#1}}$$

Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Offsite MH#1 to Offsite MH#2)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$$L = 375\ ft$$

$Q = Storm\ Drain\ Design\ Discharge$

$$Q = 42.0\ cfs$$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$$D = 3.0\ ft$$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$

$$h_f = 0.0039 \times 375$$

$$h_f = 1.46\ ft$$

$$\boxed{\mathbf{h_f = 1.46\ ft\ \ (Offsite\ MH\#1\ to\ Offsite\ MH\#2)}}$$

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Compute the Bend Headloss at Manhole (Offsite MH#2)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 11\,°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.08 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.08\ \frac{5.94^2}{2 \times 32.2}$$

$$h_{mh} = 0.04\ ft$$

$\boxed{\mathbf{h_{mh} = 0.04\ ft\ @\ Offsite\ MH\#2}}$

Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Offsite MH#2 to Offsite MH#3)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$$L = 304\ ft$$

$Q = Storm\ Drain\ Design\ Discharge$

$$Q = 42\ cfs$$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$$D = 3.0\ ft$$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$

$$h_f = 0.0039 \times 304$$

$$h_f = 1.19\ ft$$

$$\boxed{\mathbf{h_f = 1.19\ ft\ (Offsite\ MH\#2\ to\ Offsite\ MH\#3)}}$$

Compute the Headloss at Manhole (Offsite MH #3)

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{5.94^2}{2 \times 32.2}$$

$$h_{mh} = 0.03\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.03\ ft\ @\ Offsite\ MH\ \#3}}$$

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Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Offsite MH#3 to Headwall Inlet)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 35\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$

$$h_f = 0.0039 \times 35$$

$$h_f = 0.14\ ft$$

$$\boxed{\mathbf{h_f = 0.14\ ft\ \ (Offsite\ MH\#3\ to\ Headwall\ Inlet)}}$$

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Compute the Entrance Headloss at Headwall (Headwall Inlet)

Entrance Loss:

$$h_i = k_{en}\frac{V^2}{2g} \quad (Equation\ 4.15)$$

where;

$h_i = Headloss\ at\ Pipe\ Entrance$

$k_{en} = Entrance\ Loss\ Coefficient$

$k_{en} = 0.20 \quad (Table\ 5.1)$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_i = k_{en}\frac{V^2}{2g}$$

$$h_i = 0.20\,\frac{5.94^2}{2 \times 32.2}$$

$$h_i = 0.11\ ft$$

$$\boxed{\mathbf{h_i = 0.11\ \mathit{ft}\ @\ Headwall\ \mathit{Inlet}}}$$

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Channel Report

36" Offsite Storm Drain Normal Depth Analysis (Offsite MH#2 to Offsite MH#3)

Circular

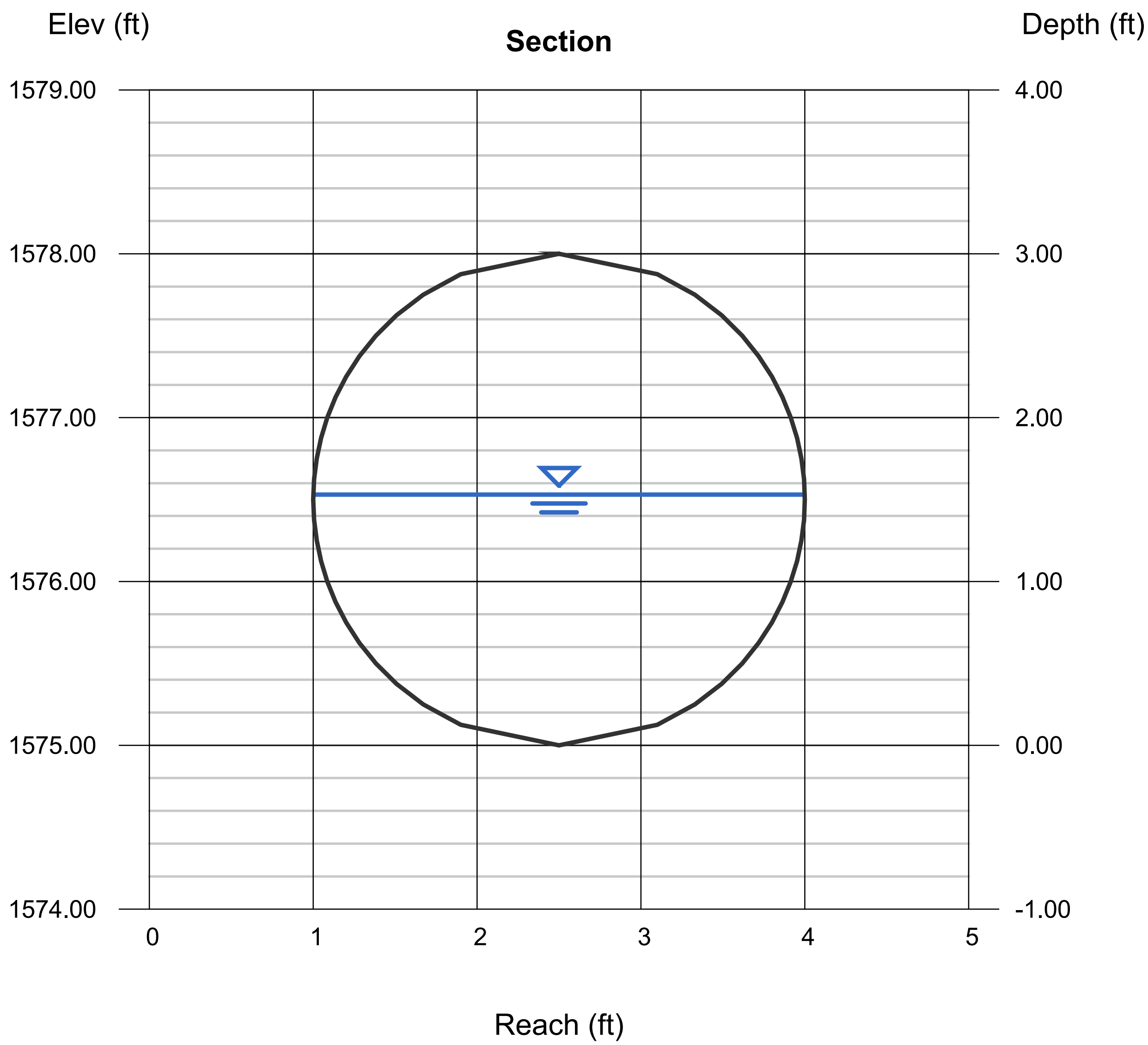
Diameter (ft)	= 3.00
Invert Elev (ft)	= 1575.00
Slope (%)	= 1.50
N-Value	= 0.013

Calculations

Compute by:	Known Q
Known Q (cfs)	= 42.00

Highlighted

Depth (ft)	= 1.53
Q (cfs)	= 42.00
Area (sqft)	= 3.64
Velocity (ft/s)	= 11.53
Wetted Perim (ft)	= 4.78
Crit Depth, Yc (ft)	= 2.12
Top Width (ft)	= 3.00
EGL (ft)	= 3.60



Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. — Monday, Sep 7 2020

36" Offsite Storm Drain Normal Depth Analysis (Offsite MH#3 to Headwall Inlet)

Circular
Diameter (ft) = 3.00

Invert Elev (ft) = 1585.00
Slope (%) = 1.10
N-Value = 0.013

Calculations
Compute by: Known Q
Known Q (cfs) = 42.00

Highlighted
Depth (ft) = 1.68
Q (cfs) = 42.00
Area (sqft) = 4.09
Velocity (ft/s) = 10.27
Wetted Perim (ft) = 5.08
Crit Depth, Yc (ft) = 2.12
Top Width (ft) = 2.98
EGL (ft) = 3.32

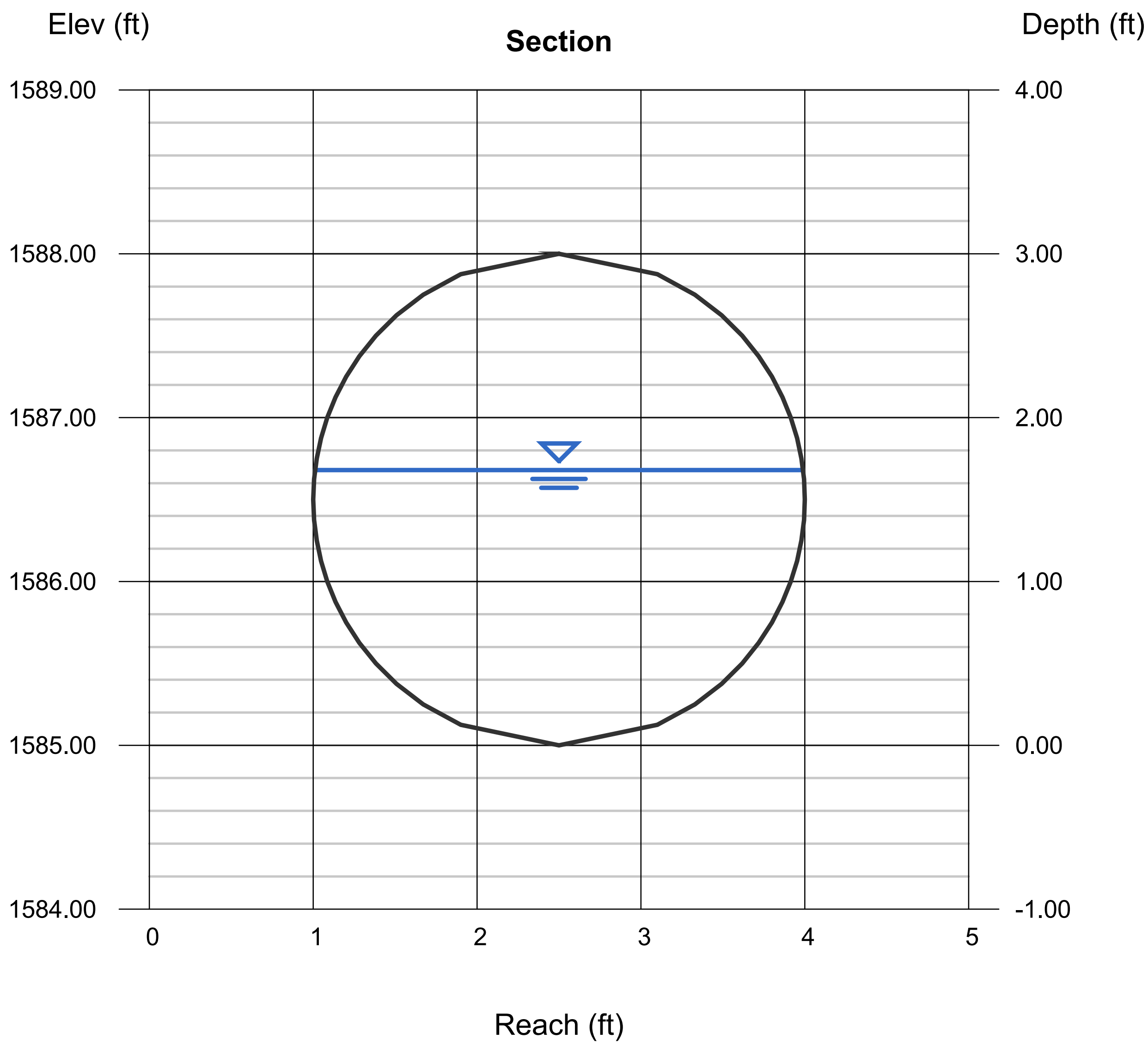


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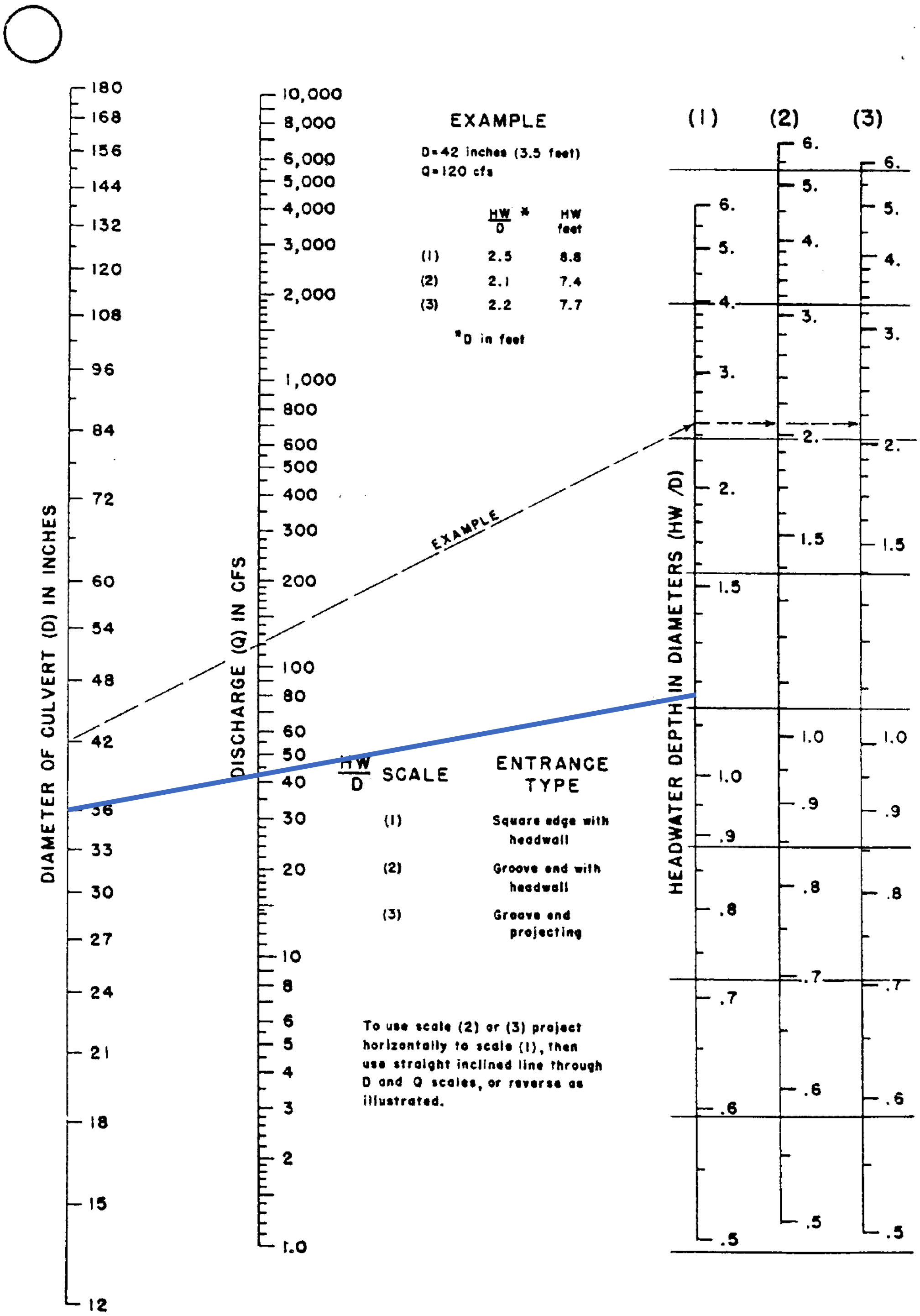
FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)



36-in Storm Drain
Q=42 cfs
D=3.0 ft
HW/D=1.16
Hence;
HW=3.5 ft

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Main Parking Lot Storm Drain Design Calculations

Main Parking Lot Storm Drain Hydraulic Grade Line (HGL) Summary Table

Location	Type of Headloss	Headloss (ft)	HGL Elevation (ft)
CB#4	Junction Loss (Entrance Headloss)^	0.04	1576.00
CB#3 to CB#4	Storm Drain Friction Headloss*	0.92	1575.29
CB#3	Junction Loss (Straight-Through Catch Basin)	0.01	1573.61
CB#2 to CB#3	Storm Drain Friction Headloss	3.03	1573.60
CB#2	Junction Loss (Straight-Through Catch Basin)	0.03	1570.57
MH#4 to CB#2	Storm Drain Friction Headloss	1.74	1570.54
MH#4	Junction Loss (Combined Junction Loss)	0.22	1568.80
MH#3 to MH#4	Storm Drain Friction Headloss	0.18	1568.58
MH#3	Junction Loss (Bend Headloss)	0.02	1568.40
MH#2 to MH#3	Storm Drain Friction Headloss	0.18	1568.38
MH#2	Junction Loss (Bend Headloss)	0.02	1568.20
MH#1 to MH#2	Storm Drain Friction Headloss	0.19	1568.18
MH#1	Junction Loss (Bend Headloss)	0.05	1567.99
Outlet Headwall to MH#1	Storm Drain Friction Headloss	0.02	1567.94
Outlet Headwall	Junction Loss (Exit Headloss)	0.19	1567.92

Tailwater Elevation @ Outlet Headwall = 1567.73

* The pipe segment between CB#3 and CB#4 is in open channel flow conditions with the hydraulic grade line governed by the proposed 18" Storm Drain normal depth capacity. Refer to the *Channel Reports* at the end of these calculations for the proposed storm drain normal depth analysis.

^ The hydraulic grade line elevation at Catch Basin #4 is governed by the inlet control interception capacity of the 18" Storm Drain. Refer to the Inlet Control Nomograph calculation at the end of these calculations for the proposed storm drain interception capacity calculation.

Storm Drain - Hydraulic Grade Line Calculation

The procedures outlined in Chapter 4 of the Hydraulics Drainage Design Manual for Maricopa County were used in order to compute the Hydraulic Grade Line (HGL) for the Offsite Storm Drain. The equations and figures used henceforth were also taken from the manual.

The new storm drain was designed to intercept the 100-year 6-hour peak discharge from the main parking lot and plaza areas and convey it to the proposed detention basin north of Bell Road, just east of the entrance drive. The main parking lot was graded to drain to 4 shallow lot spots where new combination curb opening and grate catch basins will be designed to intercept peak discharge from the upstream contributing drainage area. The proposed storm drain is designed to convey the 100-year, 6-hour peak discharge of 6 cfs from the most upstream Catch Basin #1 (HEC-1 Sub-basin ID: NDA1). Further downstream, the storm drain is designed to convey a combined 11 cfs from the Catch Basin #2 (HEC-1 Combine: CSD1), a combined 19 cfs from Catch Basin #3 (HEC-1 Combine: CSD2) and finally 25 cfs from Manhole #4 (HEC-1 Combine: CSD3). No offsite flows are added to the main parking lot storm drain. Refer to Appendix B for the HEC-1 Hydrologic Model Results.

Determine Tailwater Elevation:

The new storm drain discharges into the proposed new detention basin. The starting tailwater elevation for the design of the strom drain was taken as the stage in the new detention basin at the time the 100-year, 6-hour peak discharge enters the detention basin through the storm drain. From the Proposed Conditions HEC-1 Model it was found that at a time of 4:02 hr a peak flow of 25 cfs enters the basin with the water level in the detention basin at 1567.73 ft. Therefore, the starting tailwater elevation for the proposed storm drain is 1567.73 ft.

$$\boxed{\begin{array}{c}\textbf{Tailwater Elevation @ } \boldsymbol{Outlet\ Headwall} = \textbf{1567.73 ft} \\ (\textit{Water Level in Detention Basin @ Peak Inflow})\end{array}}$$

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Compute the Storm Drain Outlet Headloss at Outlet Headwall

Exit Loss:

$$h_o = 1.0\frac{V^2}{2g} \quad (Equation\ 4.16)$$

where;

$h_o = Outlet\ Headloss\ at\ Manhole$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25.0}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_o = 1.0\frac{V^2}{2g}$$

$$h_o = 1.0\frac{3.54^2}{2 \times 32.2}$$

$$h_o = 0.19\ ft$$

$$\boxed{\mathbf{h_o = 0.19\ \mathit{ft}\ @\ Outlet\ \mathit{Headwall}}}$$

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Compute the Friction Headloss - Proposed 36" Storm Drain (Outlet Headwall to MH#1)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 16\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{3.54^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$$h_f = S_f L$$
$$h_f = 0.0014 \times 16$$
$$h_f = 0.02\ ft$$

$$\boxed{\mathbf{h_f = 0.02\ ft\ \ (Outlet\ Headwall\ to\ MH\#1)}}$$

Gavan & Barker Inc.

Compute the Bend Headloss at Manhole (MH#1)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 39°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.24 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.24\ \frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.05\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.05\ ft\ @\ MH\ \#1}}$$

Compute the Friction Headloss - Proposed 36" Storm Drain (MH#1 to MH#2)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 139\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{3.54^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$$h_f = S_f L$$

$$h_f = 0.0014 \times 139$$

$$h_f = 0.19\ ft$$

$$\boxed{\mathbf{h_f = 0.19\ ft\ \ (MH\#1\ to\ MH\#2)}}$$

Compute the Bend Headloss at Manhole (MH#2)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 25\,°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.12 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.12\ \frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.02\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.02\ ft\ @\ MH\ \#2}}$$

Compute the Friction Headloss - Proposed 36" Storm Drain (MH#2 to MH#3)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$$L = 127\ ft$$

$Q = Storm\ Drain\ Design\ Discharge$

$$Q = 25\ cfs$$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$$D = 3.0\ ft$$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$$h_f = S_f L$$

$$h_f = 0.0014 \times 127$$

$$h_f = 0.18\ ft$$

$$\boxed{\mathbf{h_f = 0.18\ ft\ \ (MH\#2\ to\ MH\#3)}}$$

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Compute the Bend Headloss at Manhole (MH#3)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 28\,°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.12 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.12\ \frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.02\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.02\ ft\ @\ MH\ \#3}}$$

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Compute the Friction Headloss - Proposed 36" Storm Drain (MH#3 to MH#4)

$h_f = S_f L \quad (Equation\ 4.6)$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 126\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{3.54^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$h_f = S_f L$

$h_f = 0.0014 \times 126$

$h_f = 0.18\ ft$

$$\boxed{\mathbf{h_f = 0.18\ ft\ (MH\#3\ to\ MH\#4)}}$$

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Compute the Combined Headloss at Manhole (MH#4)

At this junction, compute the headloss associated with the Straight-Through Manhole and the lateral inflow at the manhole. The combined headloss is the total headloss at this Manhole (MH#4).

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.01\ ft$$

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Junction Loss (Lateral Inflow):

$$h_j = \frac{2\left(Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta\right)}{\left(A_1 + A_2\right) g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad \left(Equation\, 4.10b\right)$$

where;

$h_j = Headloss\ at\ Junction\ with\ lateral\ inflow$

$A_1 = Upstream\ Flow\ Area$

$$A_1 = \pi \frac{d^2}{4} = \pi \frac{2.0^2}{4} = 3.14\ ft^2$$

$A_2 = Downstream\ Flow\ Area$

$$A_2 = \pi \frac{d^2}{4} = \pi \frac{3.0^2}{4} = 7.07\ ft^2$$

$Q_1 = Upstream\ Flow\ Rate$

$Q_1 = 19\ cfs$

$Q_2 = Downstream\ Flow\ Rate$

$Q_2 = 25\ cfs$

$Q_3 = Q_2 - Q_1 = Lateral\ Flow\ Rate$

$Q_3 = 6\ cfs$

NOTE: The lateral flow rate represents the flow rate in the 18-inch Catch Basin #1 connector pipe.

$V_1 = Upstream\ Flow\ Velocity$

$$V_1 = \frac{Q_1}{A_1} = \frac{19}{3.14} = 6.05\ \frac{ft}{s}$$

$V_2 = Downstream\ Flow\ Velocity$

$$V_2 = \frac{Q_2}{A_2} = \frac{25}{7.07} = 3.54\ \frac{ft}{s}$$

$V_3 = Lateral\ Flow\ Velocity$

$$V_3 = \frac{Q_3}{A_3} = \frac{6}{\pi \times \left(\frac{1.50^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$\theta = Angle\ between\ lateral\ and\ main\ storm\ drain\ line$

$\theta = 90\,^\circ \ \left(Figure\, 4.7\right)$

$$h_j = \frac{2\left(25 \times 3.54 - 19 \times 6.05 - 6 \times 3.40 \times \cos(90)\right)}{(3.14 + 7.07)\, 32.2} + \frac{6.05^2}{2 \times 32.2} - \frac{3.54^2}{2 \times 32.2}$$

$h_j = 0.21\ ft$

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Total Combined Headloss at Manhole (MH#4)

$$h_{mh_{TOTAL}} = h_{mh} + h_j$$

$$h_{mh_{TOTAL}} = 0.01 + 0.21$$

$$h_{mh_{TOTAL}} = 0.22\, ft$$

$$\boxed{\mathbf{h_{mh} = 0.22\, \mathit{ft} \text{ @ } MH\#4}}$$

Compute the Friction Headloss - Proposed 24" Storm Drain (MH#4 to CB#2)

$$h_f = S_f L \quad \left(Equation\ 4.6\right)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 249\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 19\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 2.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad \left(Equation\ 4.4\right)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{19}{\pi \times \left(\frac{2.0^2}{4}\right)} = 6.05\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad \left(Table\ 4.1 - Smooth\ Plastic\ Pipe\right)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{2.0}{4} = 0.500\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{6.05^2}{2 \times 32.2 \times 0.500^{\frac{4}{3}}} = 0.0070$$

$$h_f = S_f L$$

$$h_f = 0.0070 \times 248$$

$$h_f = 1.74\ ft$$

$$\boxed{\mathbf{h_f = 1.74\ ft\ \ (MH\#4\ to\ CB\#2)}}$$

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Compute the Headloss at Catch Basin (CB #2)

Straight-Through Catch Basin Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

$where;$

$h_{mh} = Headloss\ at\ catch\ basin$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 11\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{11}{\pi \times \left(\frac{1.5^2}{4}\right)} = 6.22\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{6.22^2}{2 \times 32.2}$$

$$h_{mh} = 0.03\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.03\ ft\ @\ CB\#2}}$$

Compute the Friction Headloss - Proposed 18" Storm Drain (CB#2 to CB#3)

$h_f = S_f L \quad (Equation\ 4.6)$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 278\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 11\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{11}{\pi \times \left(\frac{1.5^2}{4}\right)} = 6.22\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{1.5}{4} = 0.375\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{6.22^2}{2 \times 32.2 \times 0.375^{\frac{4}{3}}} = 0.0109$$

$h_f = S_f L$

$h_f = 0.0109 \times 278$

$h_f = 3.03\ ft$

$$\boxed{\mathbf{h_f = 3.03\ ft\ \ (CB\#2\ to\ CB\#3)}}$$

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Compute the Headloss at Catch Basin (CB #3)

Straight-Through Catch Basin Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

$where;$

$h_{mh} = Headloss\ at\ catch\ basin$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 6\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{6}{\pi \times \left(\frac{1.5^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{3.40^2}{2 \times 32.2}$$

$$h_{mh} = 0.01\ ft$$

$$\boxed{\mathbf{h}_{mh} = \mathbf{0.01}\ ft\ @\ CB\#3}$$

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Compute the Friction Headloss - Proposed 18" Storm Drain (CB#3 to CB#4)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$$L = 278\ ft$$

$Q = Storm\ Drain\ Design\ Discharge$

$$Q = 6\ cfs$$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$$D = 1.5\ ft$$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{6}{\pi \times \left(\frac{1.5^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{1.5}{4} = 0.375\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{3.40^2}{2 \times 32.2 \times 0.375^{\frac{4}{3}}} = 0.0033$$

$$h_f = S_f L$$

$$h_f = 0.0033 \times 278$$

$$h_f = 0.92\ ft$$

$$\boxed{\mathbf{h_f = 0.92\ ft\ \ (CB\#3\ to\ CB\#4)}}$$

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Compute the Entrance Headloss at Catch Basin (CB#4)

Entrance Loss:

$$h_i = k_{en}\frac{V^2}{2g} \quad (Equation\ 4.15)$$

where;

$h_i = Headloss\ at\ Pipe\ Entrance$

$k_{en} = Entrance\ Loss\ Coefficient$

$k_{en} = 0.20 \quad (Table\ 5.1)$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{6}{\pi \times \left(\frac{1.5^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$$h_i = k_{en}\frac{V^2}{2g}$$

$$h_i = 0.20\ \frac{3.40^2}{2 \times 32.2}$$

$$h_i = 0.04\ ft$$

$$\boxed{\mathbf{h_i = 0.04\ ft\ @\ CB\#4}}$$

Channel Report

18" Storm Drain Normal Depth Analysis (CB#3 to CB#4)

Circular
Diameter (ft) = 1.50

Invert Elev (ft) = 1575.00
Slope (%) = 1.10
N-Value = 0.013

Calculations
Compute by: Known Q
Known Q (cfs) = 6.00

Highlighted
Depth (ft) = 0.79
Q (cfs) = 6.000
Area (sqft) = 0.94
Velocity (ft/s) = 6.35
Wetted Perim (ft) = 2.44
Crit Depth, Yc (ft) = 0.95
Top Width (ft) = 1.50
EGL (ft) = 1.42

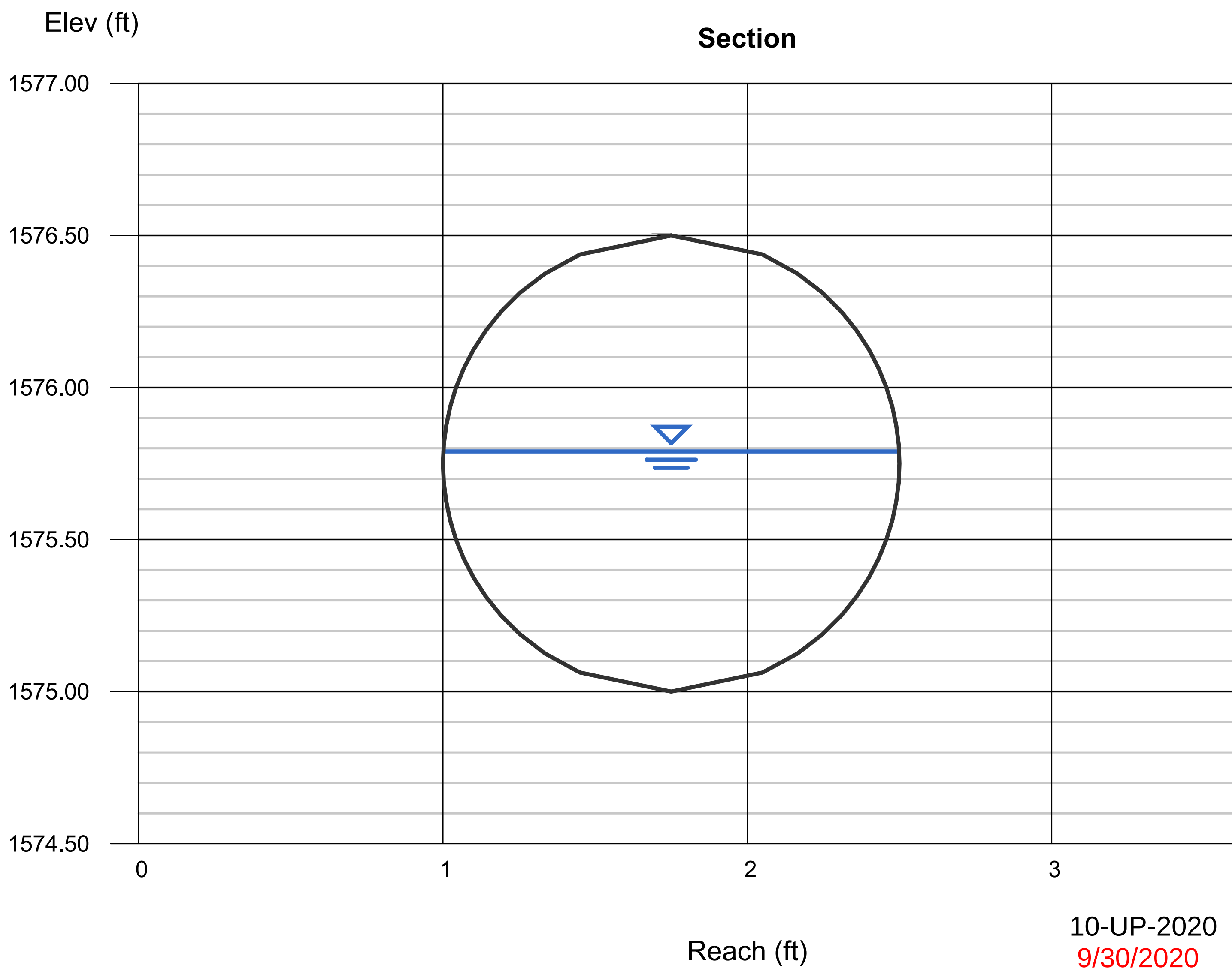

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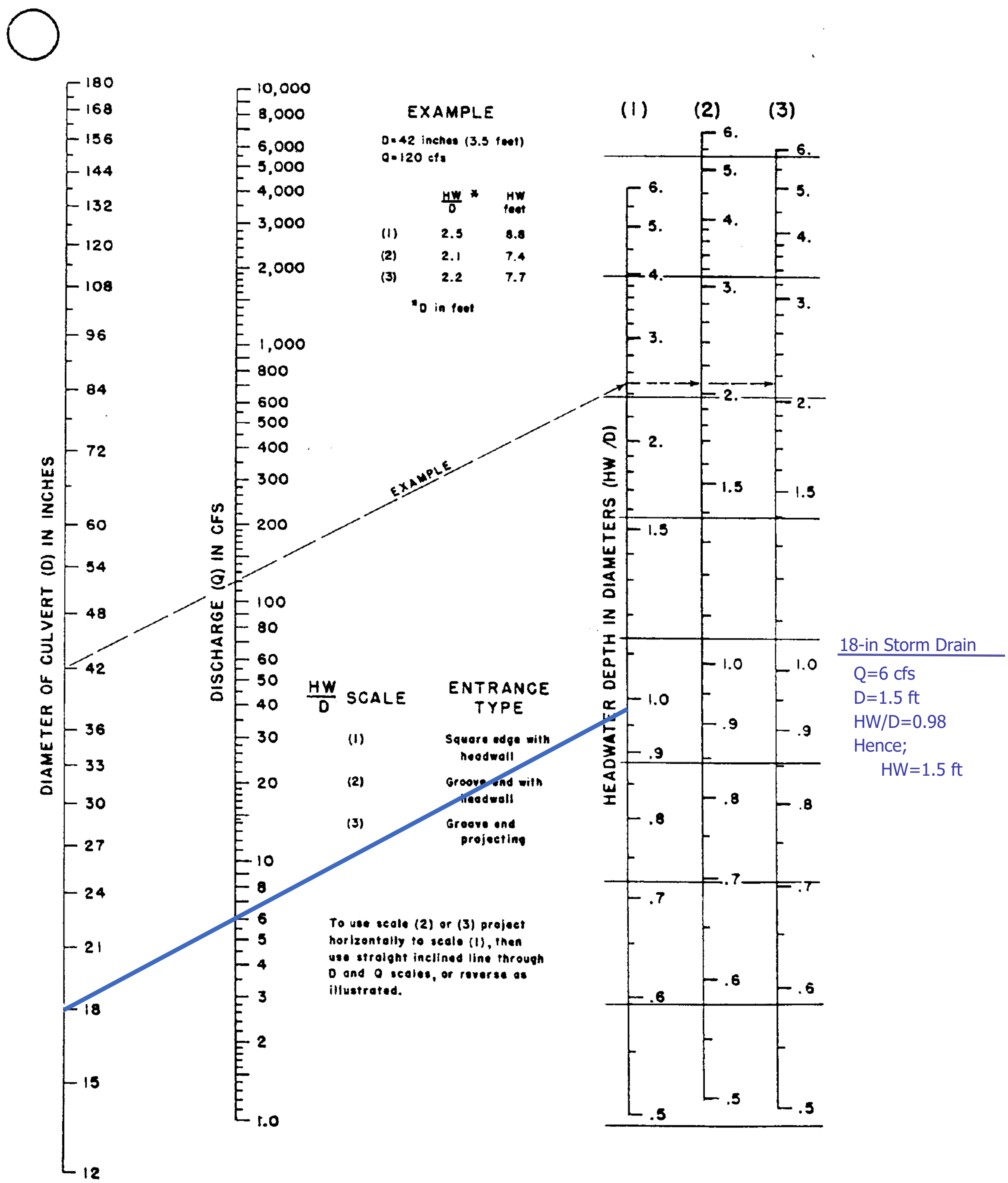
FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)



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Main Parking Lot Storm Drain Inlet Sizing Calculations

Gavan & Barker Inc.

Main Parking Lot – Catch Basin Design Calculations

The main parking at the Multi-Use Fields NWC of Bell Road & 94th Street sports complex is a ¼ mile long, linear parking lot located underneath the powerline corridor. The parking long is graded to drain to four shallow sumps that are drained by four catch basins connected to the proposed parking lot storm drain. The storm drain is discharged to the new detention basin located north of Bell Road, east of the entrance drive. The four sumps and corresponding catch basins are located approximately equidistant to each other, each designed to intercept the 100-year, 6-hour peak discharge from the parking lot and adjacent plaza area.

The four proposed inlets are located along the western edge of the main parking lot and are identified as Catch Basin #1 (CB#1) as the southernmost inlet, with Catch Basin #4 (CB#4) being the northernmost inlet. The remaining two inlets, Catch Basin #2 (CB#2) and Catch Basin #3 (CB#3) located in the middle of the parking lot. The 100-year, 6-hour design peak discharges and the corresponding HEC-1 Sub-Basin IDs for each inlet are as follows:

- Catch Basin #1 (CB#1) – Q_{100}=6 cfs (NDA4)
- Catch Basin #2 (CB#2) – Q_{100}=8 cfs (NDA3)
- Catch Basin #3 (CB#3) – Q_{100}=5 cfs (NDA2)
- Catch Basin #4 (CB#4) – Q_{100}=6 cfs (NDA1)

Refer to the Storm Drain Design Location Map at the beginning of these calculations for the exact location of the proposed catch basins as well as Appendix B for the HEC-1 Hydrologic Model.

It is recommended to install a City of Phoenix Type "Q" (Triple) combination catch basin (Std. Det. No P1572) with a total curb opening and grate length of 10 feet at each of the four sump locations. As can be seen in the following catch basin design calculations, the proposed catch basins have the capacity to intercept the entire 100-year, 6-hour design peak discharge. With the proposed catch basins located on the west curb line of the proposed parking lot, the additional benefit of using the Type "Q" catch basins is that it has the maintenance basin underneath the gutter, which will not protrude behind the back of curb into the fill slope

Gavan & Barker Inc.

Catch Basin Design @ CB#1

Determine if Catch Basin operates as a Weir or as an Orifice:

$$d \le h \quad \Rightarrow \textit{Weir Flow}$$
$$h \ge d \ge 1.4h \quad \Rightarrow \textit{Transitional Flow}$$
$$d \ge 1.4h \quad \Rightarrow \textit{Orifice Flow}$$

where;

$d = \textit{Depth of Sump at Proposed Catch Basin Inlet} = \textit{Spill Elev} - \textit{Weir Elev}$

$\textit{Spill Elev} = \textit{Low Point in the Curb Elevation at Catch Basin} = 1570.10\ ft$

$\textit{Weir Elev} = \textit{Catch Basin Weir Elevation} = \textit{Top of Curb Elevation at Catch Basin} - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$\textit{Top of Curb Elevation at Catch Basin} = 1570.10\ ft$

$\textit{Weir Elev} = 1570.10\ ft - 4.75\ in = 1569.70\ ft$

$d = \textit{Spill Elev} - \textit{Lip Elev}$

$d = 1570.10 - 1569.70$

$\left[d = 0.40\ ft\right]$

$h = \textit{Height of Curb Opening portion of the Catch Basin}$

$h = 5\ in$

$\left[h = 0.42\ ft\right]$

$d < h$

$0.40\ ft < 0.42\ ft$

$\boxed{\textbf{Weir Flow}}$

Gavan & Barker Inc.

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w \left(L_f + 1.8W\right) d^{1.5} \quad \text{(Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics)}$$

where;

$Q_i = \textit{Combination Basin Flow Interception Capacity}$

$C_w = \textit{Weir Flow Coefficient} = 2.3$

$d = \textit{Depth of Sump at Proposed Catch Basin} = 0.40$

$L = \textit{Length of Proposed Combination Catch Basin} = 10.0\ ft$

$L_f = \textit{Effective Length of Catch Basin} = C_f \times L$

$C_f = \textit{Clogging Factor} = 0.80$ (Table 6.8 - FCDMC Drainage Policies and Standards Manual)

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = \textit{Width of Depressed Gutter} = 4.3\ ft \left(\textit{Modified per COP Std Det P1572}\right)$

$$Q_i = C_w \left(L_f + 1.8W\right) d^{1.5}$$

$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.40^{1.5}$$

$$Q_i = 9.16$$

$$\boxed{\mathbf{Q_i = 9\ cfs}}$$

The proposed City of Phoenix Type "Q" combination catch basin (Std. Det. P1572) at CB #1 has an interception capacity of 9 cfs, which is greater than the 100-year, 6-hour design peak discharge of 6 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

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Connector Pipe Sizing:

Determine the total (friction + inlet) headloss:

$$h_f = S_f L \quad \left(\text{Equation 4.6 - Drainage Design Manual for Maricopa County, Hydraulics}\right)$$

$$h_i = \left(1 + k_{en}\right)\left(\frac{V^2}{2g}\right)$$

Try a 18 inch (d = 1.50 ft) Pipe:

$$S_f = Pipe\ Friction\ Slope = K\frac{V^2}{2gR^{\frac{4}{3}}} \left(\text{Equation 4.4 - Design Manual for Maricopa County, Hydraulics}\right)$$

$$K = Roughness\ Coefficient = \frac{2gn^2}{2.21} \left(\text{Equation 4.5 - Design Manual for Maricopa County, Hydraulics}\right)$$

$$n = Manning's\ roughness\ coefficient = 0.013\ (\text{Typ})$$

$$g = Acceleration\ due\ to\ Gravity = 32.2\ \frac{ft}{s^2}$$

$$V = Velocity\ in\ Pipe = \frac{Q}{A}$$

$$Q = Flow\ Rate\ in\ Pipe = 6\ cfs$$

$$A = Cross-Section\ area\ of\ Pipe = \pi\frac{d^2}{4} = \pi\frac{1.50^2}{4} = 1.77\ ft^2$$

$$R_h = Hydraulic\ Radius = \frac{A}{W_p} = \frac{\pi\frac{d^2}{4}}{\pi d} = \frac{d}{4} = \frac{1.50}{4} = 0.375\ ft$$

$$L = Length\ of\ Pipe = 28\ ft$$

$$k_{en} = Entrance\ Loss\ Coefficient = 0.2 \left(\text{Table 5.1 - Design Manual for Maricopa County, Hydraulics}\right)$$

$$K = \frac{2gn^2}{2.21} = \frac{2(32.2)(0.013)^2}{2.21}$$

$$[K = 0.0049]$$

$$V = \frac{Q}{A} = \frac{6}{1.77}$$

$$\left[V = 3.39\ \frac{ft}{s}\right]$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = (0.0049)\frac{3.39^2}{2(32.2)(0.375)^{\frac{4}{3}}}$$

$$\left[S_f = 0.0032\right]$$

Friction Headloss:

$$h_f = S_f L$$
$$h_f = (0.0032)(28)$$
$$\left[h_f = 0.09\,ft\right]$$

Inlet and Manhole Headloss:

$$h_i = \left(1 + k_{en}\right)\left(\frac{V^2}{2g}\right)$$
$$h_i = (1 + 0.2)\left(\frac{3.39^2}{2(32.2)}\right)$$
$$\left[h_i = 0.21\,ft\right]$$

Total Headloss:

$$h_t = h_f + h_i$$
$$h_t = 0.09 + 0.21$$
$$h_t = 0.30$$

$$\boxed{\boldsymbol{h_t = 0.3\,ft}}$$

Available Head: h_a

Upstream HW Elevation: 1569.10 *ft* (Six inches below the proposed catch basin gutter elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" below the gutter elevation at the inlet. The gutter elevation of Catch Basin #1 is 1569.60 ft.

Downstream HW Elevation: 1568.80 *ft* (Storm Drain HGL at Connection Manhole (MH#4))

$$h_a = Upstream\ HW - Downstream\ HW$$
$$h_a = 1569.10 - 1568.80$$
$$h_a = 0.30$$

$$\boxed{\boldsymbol{h_a = 0.3\,ft}}$$

The available head is equal to the total catch basin connector pipe headloss, therefore:

The 18-inch connector pipes have sufficient capacity to convey the intercepted flow

Gavan & Barker Inc.

Catch Basin Design @ CB#2

Determine if Catch Basin operates as a Weir or as an Orifice:

$$d \le h \quad \Rightarrow Weir\ Flow$$
$$h \ge d \ge 1.4h \quad \Rightarrow Transitional\ Flow$$
$$d \ge 1.4h \quad \Rightarrow Orifice\ Flow$$

where;

$$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin\ Inlet = Spill\ Elev - Weir\ Elev$$

$$Spill\ Elev = Low\ Point\ in\ the\ Curb\ Elevation\ at\ Catch\ Basin = 1573.50\ ft$$

$$Weir\ Elev = Catch\ Basin\ Weir\ Elevation = Top\ of\ Curb\ Elevation\ at\ Catch\ Basin - 4.75\ in$$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$$Top\ of\ Curb\ Elevation\ at\ Catch\ Basin = 1573.50\ ft$$

$$Weir\ Elev = 1573.50\ ft - 4.75\ in = 1573.10\ ft$$

$$d = Spill\ Elev - Lip\ Elev$$

$$d = 1573.50 - 1573.10$$

$$\left[d = 0.40\ ft\right]$$

$$h = Height\ of\ Curb\ Opening\ portion\ of\ the\ Catch\ Basin$$

$$h = 5\ in$$

$$\left[h = 0.42\ ft\right]$$

$$d < h$$
$$0.40\ ft < 0.42\ ft$$

$\boxed{\textbf{Weir Flow}}$

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w \left(L_f + 1.8W\right) d^{1.5} \quad \left(\text{Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics}\right)$$

where;

$Q_i = Combination\ Basin\ Flow\ Interception\ Capacity$

$C_w = Weir\ Flow\ Coefficient = 2.3$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin = 0.40$

$L = Length\ of\ Proposed\ Combination\ Catch\ Basin = 10.0\ ft$

$L_f = Effective\ Length\ of\ Catch\ Basin = C_f \times L$

$C_f = Clogging\ Factor = 0.80 \left(\text{Table 6.8 - FCDMC Drainage Policies and Standards Manual}\right)$

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = Width\ of\ Depressed\ Gutter = 4.3\ ft \left(Modified\ per\ COP\ Std\ Det\ P1572\right)$

$$Q_i = C_w \left(L_f + 1.8W\right) d^{1.5}$$

$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.40^{1.5}$$

$$Q_i = 9.16$$

$$\boxed{\mathbf{Q_i = 9\ cfs}}$$

The proposed City of Phoenix Type "Q" combination catch basin (Std. Det. P1572) at CB #2 has an interception capacity of 9 cfs, which is greater than the 100-year, 6-hour design peak discharge of 8 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Catch Basin Design @ CB#3

Determine if Catch Basin operates as a Weir or as an Orifice:

$d \leq h \qquad \Rightarrow Weir\ Flow$

$h \geq d \geq 1.4h \qquad \Rightarrow Transitional\ Flow$

$d \geq 1.4h \qquad \Rightarrow Orifice\ Flow$

$where;$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin\ Inlet = Spill\ Elev - Weir\ Elev$

$Spill\ Elev = Low\ Point\ in\ the\ sump,\ gutter\ elevation\ at\ landscaped\ island = 1575.40\ ft$

$Weir\ Elev = Catch\ Basin\ Weir\ Elevation = Top\ of\ Curb\ Elevation\ at\ Catch\ Basin - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$Top\ of\ Curb\ Elevation\ at\ Catch\ Basin = 1575.50\ ft$

$Weir\ Elev = 1575.50\ ft - 4.75\ in = 1575.10\ ft$

$d = Spill\ Elev - Lip\ Elev$

$d = 1575.40 - 1575.10$

$\left[d = 0.30\ ft\right]$

$h = Height\ of\ Curb\ Opening\ portion\ of\ the\ Catch\ Basin$

$h = 5\ in$

$\left[h = 0.42\ ft\right]$

$d < h$

$0.30\ ft < 0.42\ ft$

$\boxed{\textbf{Weir Flow}}$

Gavan & Barker Inc.

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w\left(L_f + 1.8W\right)d^{1.5} \quad \left(\text{Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics}\right)$$

where;

$Q_i = Combination\ Basin\ Flow\ Interception\ Capacity$

$C_w = Weir\ Flow\ Coefficient = 2.3$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin = 0.30$

$L = Length\ of\ Proposed\ Combination\ Catch\ Basin = 10.0\ ft$

$L_f = Effective\ Length\ of\ Catch\ Basin = C_f \times L$

$C_f = Clogging\ Factor = 0.80\ \left(\text{Table 6.8 - FCDMC Drainage Policies and Standards Manual}\right)$

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = Width\ of\ Depressed\ Gutter = 4.3\ ft\ \left(Modified\ per\ COP\ Std\ Det\ P1572\right)$

$$Q_i = C_w\left(L_f + 1.8W\right)d^{1.5}$$

$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.30^{1.5}$$

$$Q_i = 5.95$$

$$\boxed{\mathbf{Q_i = 6\ cfs}}$$

The proposed City of Phoenix Type "Q" combination catch basin (Std. Det. P1572) at CB #3 has an interception capacity of 6 cfs, which is greater than the 100-year, 6-hour design peak discharge of 5 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Gavan & Barker Inc.

Catch Basin Design @ CB#4

Determine if Catch Basin operates as a Weir or as an Orifice:

$$d \le h \quad \Rightarrow \textit{Weir Flow}$$
$$h \ge d \ge 1.4h \quad \Rightarrow \textit{Transitional Flow}$$
$$d \ge 1.4h \quad \Rightarrow \textit{Orifice Flow}$$

where;

$d = \textit{Depth of Sump at Proposed Catch Basin Inlet} = \textit{Spill Elev} - \textit{Weir Elev}$

$\textit{Spill Elev} = \textit{Low Point in the sump, gutter elevation at landscaped island} = 1578.90\ ft$

$\textit{Weir Elev} = \textit{Catch Basin Weir Elevation} = \textit{Top of Curb Elevation at Catch Basin} - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$\textit{Top of Curb Elevation at Catch Basin} = 1579.00\ ft$

$\textit{Weir Elev} = 1579.00\ ft - 4.75\ in = 1578.60\ ft$

$d = \textit{Spill Elev} - \textit{Lip Elev}$

$d = 1578.90 - 1578.60$

$[d = 0.30\ ft]$

$h = \textit{Height of Curb Opening portion of the Catch Basin}$

$h = 5\ in$

$[h = 0.42\ ft]$

$d < h$

$0.30\ ft < 0.42\ ft$

Weir Flow

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Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w \left(L_f + 1.8W\right) d^{1.5}$$ (Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics)

where;

$Q_i = Combination\ Basin\ Flow\ Interception\ Capacity$

$C_w = Weir\ Flow\ Coefficient = 2.3$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin = 0.30$

$L = Length\ of\ Proposed\ Combination\ Catch\ Basin = 10.0\ ft$

$L_f = Effective\ Length\ of\ Catch\ Basin = C_f \times L$

$C_f = Clogging\ Factor = 0.80$ (Table 6.8 - FCDMC Drainage Policies and Standards Manual)

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = Width\ of\ Depressed\ Gutter = 4.3\ ft$ $\left(Modified\ per\ COP\ Std\ Det\ P1572\right)$

$$Q_i = C_w \left(L_f + 1.8W\right) d^{1.5}$$

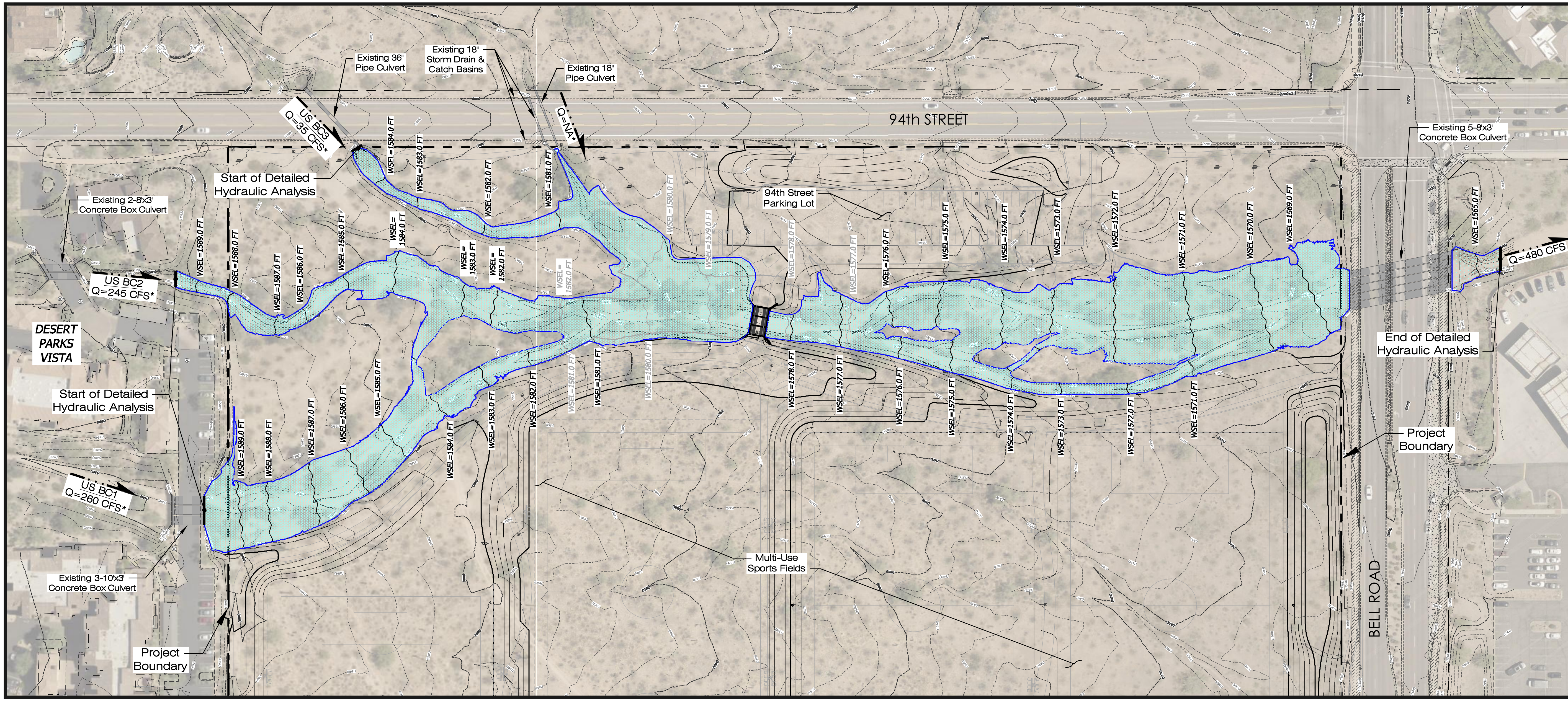
$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.30^{1.5}$$

$$Q_i = 5.95$$

$$\boxed{\mathbf{Q_i = 6\ cfs}}$$

The proposed City of Phoenix Type “Q” combination catch basin (Std. Det. P1572) at CB #4 has an interception capacity of 6 cfs, which is greater than the 100-year, 6-hour design peak discharge of 6 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Appendix D: Culvert Design & Wash Hydraulic Analysis



LEGEND

Proposed Condition Inundation Boundary

Proposed Condition Water Surface Elevation

Existing Condition Inundation Boundary

Existing Condition Water Surface Elevation

Q=260 CFS **Max Peak Discharge & Flow Direction**

94th STREET WASH

Culvert Hydraulic Analysis

NOTES:

*The 94st Street Wash design discharges upstream of the project area were determined with the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin International and modified to reflect current flow conditions by Gavan & Barker Inc. as part of this project. Refer to Appendix B for the inflow hydrographs and Appendix E for the modified FLO-2D model.

There are three main wash inflows that enter the project site. The two largest ones enter from the Desert Parks Vista Condominium Complex, just north of the project site and include the west fork that has a peak discharge of 260 cfs and the east fork with a peak discharge of 245 cfs. The third, smaller inflow enters underneath 94th Street through a 36-inch pipe culvert and has a peak discharge of 35 cfs. There is a fourth inflow location across 94th Street that consists of two curb opening catch basins that drain 94th Street and a 18-inch pipe culvert that conveys flows underneath the street. However, the contributing drainage area to that 18-inch culvert and two curb opening catch basins is small compared to the large upstream watershed areas that contributed to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

N

0 60' 120'

SCALE: 1" = 120'

Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032

Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :	
G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

94th Street Wash Culvert Hydraulic Analysis

Sheet Number:

1

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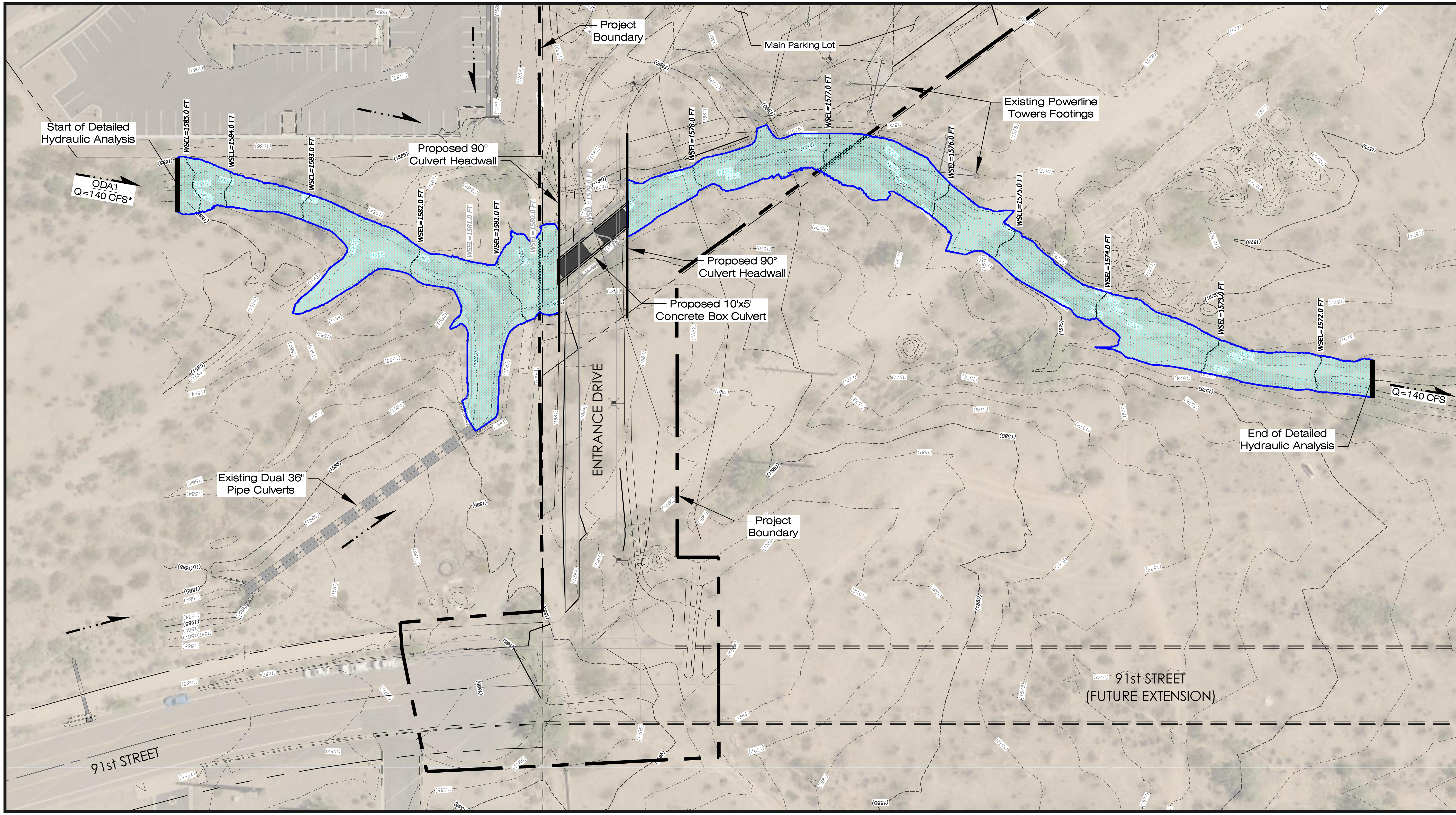

LEGEND

- Proposed Condition Inundation Boundary
- Proposed Condition Water Surface Elevation
- Existing Condition Inundation Boundary
- Existing Condition Water Surface Elevation
- Q=140 CFS — Max Peak Discharge & Flow Direction

91st STREET WASH

Hydraulic Analysis

NOTES:

*The 91st Street Wash 100-year, 6-hour design discharge upstream of the proposed culvert is 140 cfs. For hydraulic modeling purposes, the hydrograph associated with the design discharge was applied to the main 91st Street was, however it includes the flow from the exiting dual 36-inch pipe culvert and the buisiness park northeast of the proposed culvert crossing. Refer to the HEC-1 Hydrologic Modeling in Appendix ZZZ for the hydrologic results as well as a detailed drainage area map showing the contributing drainage area to the proposed culvert.

0 30' 60'

SCALE: 1" = 60'

Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
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Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

91st Street Wash Culvert Hydraulic Analysis

Sheet Number:

2

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Appendix E: Digital Data

If pdf copy, click HERE to download the Digital Data

[Digital Data CD]

Please plan on providing a complete color hard copy of the report with next case submittal with full size exhibits.

CITY OF SCOTTSDALE

MULTI-USE SPORTS FIELDS

NWC of Bell Road & 94th Street

Stormwater review by Richard Anderson

PRELIMINARY DESIGN DATA REPORT

Based on direction from planning, this review was to be treated as a conceptual submittal, similar to a zoning case, and our review of the report was consistent with a zoning level case or conceptual submittal review and did not consist of a more detailed review of some aspects of the report (such as the HEC-1 model). A zoning level review is a 50% level of design and analysis review. The submittal is acceptable for the zoning level submittal. Additionally, based on direction from planning, there will be a subsequent submittal on the project which will be similar to the development review submittal a 75% level of design and analysis review. This review does, however, identify issues we will need addressed or need more information on in the subsequent case submittal. The report will need to be updated to meet the 75% requirement at the next case submittal.

Project No. PA75200538

SEPTEMBER, 2020

Prepared For:

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Job No. 2003

10 UP 2020
09/15/20

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1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION/BACKGROUND

The purpose of this drainage study is to provide a basis of design for the drainage infrastructure associated with the new Multi-Use Sports Fields at the northwest corner of Bell Road and 94th Street. The proposed complex will primarily consist of six lighted multi-use fields, two parking lots, restroom and maintenance buildings with potable water and sewer connections, sidewalks, offsite street improvements and a non-potable water connection for field and landscaping irrigation purposes. The improvements are located on a 40-acre undisturbed natural desert parcel that is situated within the Lower Desert Environmentally Sensitive Lands (ESL) zoning district. The sports complex site improvements will be designed to meet the drainage and ESL design requirements as outlined in the City of Scottsdale *Design Standards & Policies Manual* (DSPM).

1.2 PROJECT LOCATION

The project is located within the City of Scottsdale on the northwest corner of Bell Road and 94th Street. It is bound by Bell Road on the south, 94th Street on the east, the powerline corridor on the west and the existing Desert Parks Vista Apartments/Corporate Center at DC Ranch on the north. Refer to Figure 1 below for a detailed vicinity map.

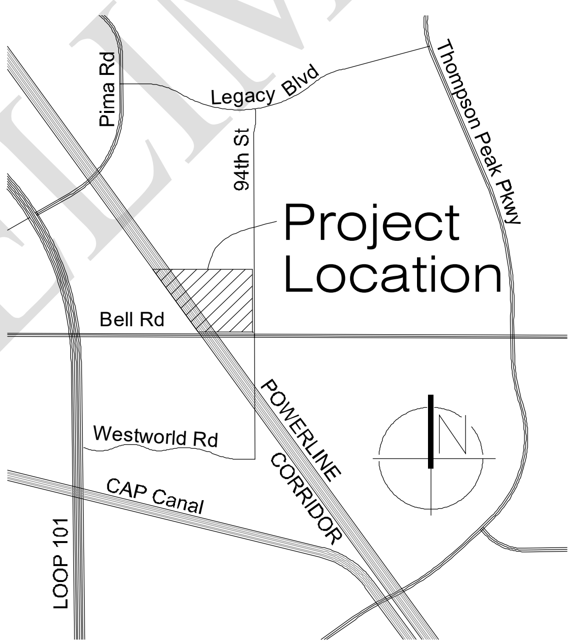

Figure 1: Vicinity Map

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2.0 STORM WATER RETENTION BASIN ANALYSIS

2.1 APPROACH

The storm water retention basin system for the new Multi-Use Sports Fields at the northwest corner of Bell Road and 94th Street was designed in accordance with the City of Scottsdale *Drainage Policies and Standards Manual (DSPM).*

The grading plan for the sports fields complex includes, four (4) shallow retention basins as well as one 3-foot deep detention basin. Three of the retention basins are located adjacent to the new 94th Street parking lot with the fourth located on the north end of the main parking lot underneath the powerline corridor. Each of these basins is graded in such a manner as to not appear "basin-like" but instead blend in with the surroundings. The maximum basin depth of these basins is less than 1 foot, with gentler basin side slopes. Each of these basins was also designed to retain the higher of either the first flush volume or the increase in the pre vs post 100-year, 2-hour runoff volume. The largest basin on the project site is located just north of Bell Road, east of the entrance drive. This detention basin is 3.0 feet deep, with 4H:1V side slopes that is partially drained through an outlet pipe to the existing dual 6'x3' concrete box culvert underneath Bell Road. The detention basin was designed to store the first flush volume as well as reduce the post development peak discharges through the existing culvert underneath Bell Road below the pre development peak discharge for the 2-, 10- and 100-year, 6-hour storm events.

2.2 RETENTION BASIN DESIGN

The Multi-Use Sports Fields at the NWC of Bell Road & 94th Street consist of new maintenance and restroom buildings, parking lots, hardscaped plaza area and six new multi-use sand-based sports fields. Excluding the sports fields, the project area is graded into five separate drainage areas, each draining into one of the new retention/detention basins. Since the project area is in an ESL zoning district where disturbance to the natural desert is to be minimized, the four new retention basins were not designed to store the full 100-year, 2-hour runoff volume. Instead they were designed to store the greater of either the 1) first flush volume or 2) the increase in runoff volume from the 100-year, 2-hour storm event from pre-development conditions. Refer to Appendix A for the retention basin design drainage area maps as well as the first flush and increase in pre vs post runoff volume calculation spreadsheets.

A slightly different approach was taken for the design of the basin adjacent to Bell Road at the south side of the project area. This basin was designed to store the first flush volume from the main parking lot as well as the hardscaped plaza areas adjacent to the restroom building. However, instead of storing the increase in runoff volume due to the conversion of natural desert into impervious asphalt, concrete and roof area, the basin was designed as a detention basin with two outlet structures that drain through a 24-inch pipe to the existing dual 6'x3' concrete box culvert underneath Bell Road, just west of the entrance drive. The low-level outlet structure consists of a standard grated catch basin that is located 0.5 feet above the basin bottom and is used to drain the basin volume through a 3-inch orifice plate to a 15-inch drainpipe. The high-level outlet structure consists of a raised grate catch basin that is less susceptible to clogging and is located 2.2 feet above the basin bottom. This basin outlet configuration allows the basin to function as a retention basin up to a depth of 2.2 feet, with the low-level outlet structure serving as a basin bleed-off. During large storm events, once depths of ponding in the basin exceed 2.2 feet, the basin becomes a detention basin with water spilling through the high-level outlet structure. This configuration allows the basin to not only store the first flush runoff volume from the parking lot and plaza areas below a depth of 2.2 feet, but also reduces the post-development peak discharges below the pre-development peak discharges for the 2-, 10- and 100-year, 6-hour storm events. Refer to Appendix A for the retention basin design drainage area map and first flush volume calculations as well as Appendix B for the HEC-1 hydrologic modeling of the existing and proposed conditions.

The six multi-use sports fields were not included in the first flush or pre vs post runoff volume calculations because they will not generate any surface runoff. The two primary reasons why the sports fields will not generate any surface runoff is that they are flat and that the underlying soil stratification consist of a 12-inch sand-based root mix surface layer above a 4-inch gravel layer in conjunction with a subgrade drainage system that consists of 4-inch perforated pipes and a 12-inch drain pipe. The sand-based fields with the advanced subgrade drainage system are designed to infiltrate the rainfall efficiently through the sand layer and convey it through the coarse gravel layer to the 4-inch perforated pipes before eventually discharging it to the spinal 12-inch drain pipe. Since the rainfall onto the fields, as well as the small surrounding pathway and plaza areas that drain into the fields get filtered through the 12-inch sand layer, the water is already naturally filtered and no first flush storage is necessary. The 12-inch subgrade drainpipe is discharged

I do not understand how the fields are not contributing runoff to the pre versus post analysis and how they do not generate any runoff. From a conservation perspective the water must go somewhere and the fields inlcude an underdrain system that all outlet into the large basin. This will need to be resolved as part of the next case submittal as it is part of the pre versus post requriement.

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directly to the detention basin high-level outlet structure where it bypasses the detention basin and flows out into the existing Bell Road culvert.

2.3 BASIN DRAIN TIMES

The basins were designed to dispose of the stored runoff volume in the allowable 36 hours. In order to meet this criteria; Double Ring Infiltration Tests were performed at the site of the proposed basins. The three, one-foot deep retention basins adjacent to the 94th Street parking lot were found to infiltrate the stored runoff volume in under 10 hours. A basin drain was not calculated for the new retention basin on the north side of the main parking lot because it is only 0.3 feet deep and the *DSPM* allows for basins less than 0.5 feet to be disposed of through infiltration without showing basin drain time calculations.

The basin drain time for the detention basin was calculated by determining the time required for the low-level bleed off catch basin to lower the depth of ponding in the basin to 6 inches, with the bottom 6-inches calculated to dissipate through infiltration. As can be seen in the proposed conditions model in Appendix B, during the 100-year, 6-hour storm event it takes the 3-inch orifice plate in the low-level grated catch basin a little over 17 hours to drain the basin to a depth of ponding of 0.5 feet. The remaining depth was found to infiltrated into the ground in about 7 hours. Therefore, the total basin drain time for the large detention basin is approximately 24 hours. Refer to Appendix A for the detailed basin drain time calculations.

3.0 HYDROLOGIC ANALYSIS

3.1 APPROACH

The hydrologic analysis for the new Multi-Use Sports Fields at the northwest corner of Bell Road and 94th Street was performed in accordance with the *DSPM* utilizing the hydrologic methods outlined in the Flood Control District of Maricopa County (District) *Drainage Design Manual for Maricopa County – Hydrology (Volume I)*.

There are two separate hydrologic models that are used to determine design peak discharges throughout the project site. The first hydrologic model is the *"Pinnacle Peak South Area Drainage Master Study" (PPS ADMS)* FLO-2D model that was prepared by TY Lin International for the City of Scottsdale in 2013. The primary purpose of the FLO-2D model was to determine the 94th Street Wash inflows that enter the project site. The second hydrologic model is a new HEC-1 model that was developed for the 2-, 10- and 100-year, 6-hour storm events utilizing the Districts' DDMSW software. The HEC-1 model was used to determine additional offsite inflows from the north into the project area as well as peak discharges that are generated within the project site under existing and proposed conditions.

3.2 EXISTING CONDITIONS HEC-1 MODEL

The existing conditions HEC-1 hydrologic model was developed to determine the pre development peak discharges that are currently impacting the project site. In order to determine drainage area boundaries and main flow paths within the project area, as well as the upstream offsite area that contributes to the project area; the latest aerial photography and topographic mapping that was developed for the design of the project was used. To supplement the new topographic mapping, the one-foot contour mapping that was developed for the Pinnacle Peak South ADMS was obtained from the City of Scottsdale and utilized in areas that were developed prior to the contour mapping flight date in 2007. For areas that have been recently developed, any available as-built plans were inspected, and site visits were made to determine the appropriate flow paths and drainage area boundaries.

The existing conditions HEC-1 model includes the entire project area west of the 94th Street Wash. As will be discussed in greater detail in Section 3.4 below, the PPS ADMS FLO-2D model was used to determine the peak discharges for upstream watershed area of the 94th Street Wash. However, two of the offsite watershed areas were included in the existing conditions HEC-1 model. The first one, identified with the HEC-1 Subbasin ID of ODA1 is the contributing drainage area for the 91st Street Wash that crosses the project area on the northwest corner. The second, much smaller one is identified with the HEC-1 Subbasin ID of ODA2 and is the contributing drainage area for the wash that concentrates at the northern boundary of the project area between the Corporate Center at DC Ranch and the Desert Parks Vista Condominium Complex. Refer to the existing condition HEC-1 Schematic and Drainage Area Map in Appendix B for the location and extents of the two main offsite watershed areas.

In addition to the three major washes that enter the site, there are several other minor washes that traverse the project area. These washes generally flow in a southwesterly direction leaving the project area and discharging to the Arizona State Land parcel immediately west of the project boundary and eventually discharging underneath Bell Road through either the dual 6'x3' box culvert at the new entrance drive or the triple barrel 8'x3' concrete box culvert at 91st Street. Refer to Appendix B for the existing conditions HEC-1 Schematic as well as the 100-year, 6-hour HEC-1 hydrologic model. The Digital Data in Appendix E contains the 2-year and 10-year HEC-1 models.

3.3 PROPOSED CONDITIONS HEC-1 MODEL

The proposed conditions HEC-1 hydrologic model was developed by incorporating the proposed Multi-Use Sports Fields improvements into the existing conditions hydrologic model. The offsite drainage area boundaries remained the same, but new onsite drainage area boundaries were drawn based on the grading and drainage design of the main parking lot and the adjacent hardscape plaza areas. Due to the conversion of natural desert to impervious parking and hardscaped area, the runoff volumes, and discharges for the western half of the project area increased significantly under the proposed conditions as compared to the existing conditions. In order to meet the ESL ordinance requirements of keeping the post development peak discharges below the pre-development conditions, the proposed conditions HEC-1 model incorporated a onsite storm drain and detention basin that is used to attenuate the flows before they leave the project site below the existing conditions for the 2-, 10- and 100-year, 6-hour storm events. In addition to the onsite, main parking lot storm drain, an offsite storm drain was included that will convey the offsite flows from the north project boundary through the project site, outletting to the same location where the wash currently leaves the project site. Refer to the proposed conditions HEC-1 Schematic and Drainage Area Map in Appendix B for the updated drainage area boundaries.

Within the project area, the proposed conditions HEC-1 hydrology model serves two primary purposes. The first is to design the main parking lot storm drain and size the new detention basin north of Bell Road to attenuate the increased peak discharges below the existing conditions peak discharges that leave the site. The grading of the main parking lot underneath the powerline corridor was done in such a manner as to crate four shallow sump locations where new storm drain catch basin will intercept the 100-year, 6-hour design peak discharges and convey them through a new storm drain to the detention basin at Bell Road. As was previously mentioned, the detention basin was designed to store the first flush runoff volume from the parking lot area, while larger flows were designed to spill through the high-level outlet structure and discharge through the existing dual 6'x3' Bell Road culvert. Even with the larger contributing drainage area (i.e. under existing conditions, the northwest part of the project area is not part of the drainage area to the dual 6'x3' Bell Road box culvert), the design of the detention basin with its low- and high-level outlet structures achieved the desired goal of reducing the peak discharges through the Bell Road culvert under the proposed conditions to be equal or less than the existing conditions for all three storm events. Refer to the Appendix B for the HEC-1 Schematic and the 100-year, 6-hour HEC-1 model as well as the digital data folder in Appendix E for 2- and 10-year, 6-hour HEC-1 models.

The second purpose of the proposed conditions HEC-1 hydrologic model was to determine the amount of runoff from the multi-use sports fields drainage system during the 2-, 10- and 100-year, 6-hour storm events. Since the sports fields are designed to be flat and consist of a 12-inch deep sand-based root zone mix, there will be no surface runoff generated during even the most intense part of a the 100-year storm event. The first step in determining how much water infiltrates through the sand-based root zone mix was to determine how much available pore space the sand layer has to store water. Recommended root zone mixes for sand-based fields call for the sand to have a total porosity between 35%-55%, this is the total void space between the sand particles. However, not all of this void space has the ability to store water that infiltrates through the surface. The ideal root zone mix has a capillary porosity, which is defined as the amount of the void space that is capable of storing water, between 15%-25%. Assuming an average capillary porosity of 20% we can calculate that the 12-sand layer has a storage capacity of 2.4 inches (12 in x 0.20 = 2.4 in) over the surface area of field. However, this 2.4 inches was further reduced by applying a 50% safety factor to account for potentially wet antecedent moisture conditions that could be a result of recent irrigation or a storm event. Therefore, for the flat portion of the multi-use sports fields an Initial Abstraction value of 1.2 inches was applied to represent the available storage associated with the 20% capillary porosity within the 12-inch sand-based root zone mix.

Once the storm event exceeds the storage capacity of root zone mix associated with the capillary porosity, water will drain into the underlying 4-inch thick gravel layer. The gravel layer has larger voids that are not conducive to "storing" water, but allow it to freely flow through the layer and into the 4-inch diameter perforated pipes that are located 20 feet on center underneath the sports fields. Once water enters these perforated pipes it is conveyed to a 12-inch subgrade drain and out underneath the fields to the high-level outlet structure at the detention basin on the north side of Bell Road. From there the outflow is piped directly to the dual 6'x3' box culvert under Bell Road.

The total open void space within the gravel layer was calculated based on a conservative total porosity of 35%. To utilize the storage capacity within the 4-inch and 12-inch pipes as well as the large open void space in the gravel layer, orifice plates were designed to limit the flow out from underneath each of the fields. The orifice plates will meter the flow out from underneath each of the fields thereby allowing water to pond in the pipes and within the gravel layer to limit the flow out from underneath each one of the fields. This approach to the design of the subgrade drainage system will utilize the full available storage capacity underneath each of the sports fields before discharging directly to the dual 6'x3' Bell Road box culvert via the detention basin outlet pipe.

3.4 94TH STREET WASH HYDROLOGIC ANALYSIS

The 94th Street Wash is located on the east side of the project site, paralleling 94th Street and discharging under Bell Road in a five barrel 8'x3' concrete box culvert. As can be seen in the 94th Street Wash Drainage Area Map in Appendix B, the wash is comprised of a major confluence at the northeast corner of the project site, with two main wash forks entering from the north and three inflows from 94th Street. The main (western) wash fork extends upstream to Legacy Drive, roughly paralleling 94th Street on the west before turning and extending to the existing dual 8'x2' concrete box culvert that penetrates the Reata Wash levee at Hualapai Drive. The secondary (eastern) wash fork extends upstream to 94th Street before meandering through the Desert Haciendas subdivision to a second 10'x2' concrete box culvert that penetrates the Reata Wash levee. The two culvert crossings underneath 94th Street have relatively small drainage areas and do not receive split flows from Reata Wash.

The total contributing drainage area to the 94th Street Wash at the project site is 185 acres. In addition to the runoff form the contributing drainage area, there are the two locations described above where runoff enters the watershed from Reata Wash. There is also one location, just downstream of Hualapai Drive where water from the main 94th Street Wash out of the contributing drainage area during large storm events. To analyze this complex hydrologic condition, the PPS ADMS 100-year, 24-hour FLO-2D model was reviewed and modified to better represent existing flow conditions. Upstream of the project site, within the contributing drainage area to the 94th Street Wash at the project location, the modifications primarily consisted of adjusting grid elevations to 1) prevent flows from breaking out of the two main washes and 2) directing the runoff generated in the contributing drainage area to the correct location based on inspection of contour mapping, aerial photography and as-built plans. These modifications removed any erroneous flow splits or diversions that can easily occur as part of a large regional drainage master study. The modeling of the two Reata Pass levee penetration box culverts was not modified from the original PPS ADMS study model.

As can be seen in the drainage area map in Appendix B, there is approximately 285 cfs that enters the west fork of the 94th Street Wash through the northern dual 8'x2' concrete box culvert from Reata Wash. Just downstream of the culvert, approximately 200 cfs stays within the wash while approximately 85 cfs splits out in a southwesterly direction. With the addition of the runoff from the contributing drainage area, the peak inflow for the west fork of the 94th Street Wash at the project site is 260 cfs. For the east fork of the 94th Street Wash there is approximately 205 cfs that enters through the 10'x2' concrete box culvert from Reata Wash. This flow increases with the addition of the runoff generated from the contributing watershed area to 245 cfs that enters the project site.

The largest inflow from 94th Street occurs through an existing 36-inch pipe culvert that conveys flows from the Desert Haciendas subdivision underneath 94th Street. According to the modified FLO-2D model the peak inflow through the pipe culvert into the 94th Street Wash is 35 cfs. The remaining two inflow locations, just south of the 36-inch culvert, consist of a side-by-side 18-inch storm drain and pipe culvert as well as a scupper that drains the west half of 94th Street to the wash. The contributing drainage area to the existing 18-inch culvert was cut off by the development of the Desert Haciendas subdivision and the runoff generated from the small 94th Street watershed that is intercepted by the 18-inch storm drain and scupper would not have a meaningful impact on the peak discharge of the 94th Street Wash through the project area. Refer to Appendix B for the 94th Street Wash Drainage Area Map that shows the pertinent offsite drainage infrastructure as well as the location of the three main inflows to the 94th Street wash through the project area. Appendix B also includes the three main inflow hydrographs that were obtained from the modified FLO-2D model, which in turn can be found in the Digital Data folder in Appendix E.

The three main inflows were used as direct hydrograph inputs to a two-dimensional HEC-RAS hydraulics model that was used to determine the existing and proposed conditions water surface elevation of the 94th Street Wash through the project area as well as the design of the proposed triple barrel 10'x5' concrete box culvert crossing. The routing of the three main inflows through the project area using the HEC-RAS model resulted in a combined peak discharge through the existing Bell Road culvert of 480 cfs. For a more detailed discussion of the hydraulic analysis refer to Section 5.0 of this report.

4.0 STORM DRAIN DESIGN AND ANALYSIS

Two new storm drains were designed as part of the Multi-Use Sports Fields project. The first is a new storm drain that conveys the offsite flows that enter the project site from the north between the Corporate Center at DC Rand and the Desert Parks Vista Condominium Complex. The second storm drain is a new onsite storm drain that is located along the western edge of the main parking lot underneath the powerline corridor. This storm drain intercepts the runoff from the parking lot and adjacent hardscaped plaza areas with four new combination catch basins and conveys it to the new detention basin on the north side of Bell Road, east of the entrance drive. Refer to Appendix C for the Storm Drain Location Map showing the extents of the offsite and main parking lot storm drains in relationship to the proposed improvements.

4.1 OFFSITE STORM DRAIN DESIGN

The proposed offsite storm drain was designed to intercept the calculated 100-year, 6-hour peak discharge that concentrates at the northern boundary between the Corporate Center at DC Ranch and the Desert Parks Vista Condominiums Complex and convey it through the site to its existing outfall location on the southwest corner of the project area. As can be seen in the HEC-1 model in Appendix B, the design peak discharge at this location is 42 cfs (HEC-1 Subbasin ID: ODA2). Under existing conditions, the alignment of the offsite storm drain roughly follows the alignment of the existing wash that flows through the project area discharging at the same location as the existing wash.

At the upstream end, the offsite storm drain intercepts the design discharge with a headwall inlet. The proposed 36-inch storm drain traverses under the two northern most multi-use fields before entering the main parking lot just west of the new public restroom building. From there it traverses the parking lot at a diagonal before discharging through a headwall outlet structure into the existing wash. Refer to the Storm Drain Location Map in Appendix C for the location of the offsite storm drain as well as the Offsite Storm Drain Hydraulic Grade Line (HGL) design calculations.

4.2 MAIN PARKING LOT STORM DRAIN DESIGN

The main parking lot storm drain was designed to intercept the calculated 100-year, 6-hour peak discharges from the main parking lot underneath the powerline corridor as well as the adjacent hardscape plaza areas. The storm drain starts at the new detention basin north of Bell Road, west of the entrance drive and extends north along the western edge of the new parking lot. The proposed grading of the parking lot was done in such a manner as to create four shallow sumps where four new combination catch basins were designed to intercept the entire 100-year, 6-hour peak discharges from the new parking lot.

The proposed storm drain and catch basins were designed to prevent any water from overtopping the new western curb and flowing into the adjacent Arizona State Land parcel located immediately west of the project area. This approach ensures that all of the runoff generated from the proposed parking lot and adjacent hardscape areas will be intercepted by the new storm drain and routed through the new detention basin before discharging to the existing dual 6'x3' Bell Road box culvert just west of the entrance drive. Refer to the Storm Drain Location Map in Appendix C for the location of the main parking lot storm drain as well as the Storm Drain Hydraulic Grade Line (HGL) design and Catch Basin sizing calculations.

5.0 CULVERT DESIGN & WASH HYDRUALIC ANALYSIS

5.1 APPROACH

The hydraulic analysis for the two main washes that impact the project site was performed using the latest US Army Corps of Engineers HEC-RAS modeling software with two-dimensional surface flow capability. The hydraulic analysis was done in accordance with the City's *DSPM* as well as the District's *Hydraulics Manual.*

The two-dimensional HEC-RAS hydraulics model was developed to determine the water surface elevations for the two main washes that cross the project site. The 94th Street Wash traverses the project site by paralleling 94th Street, while the 91st Street Wash bisects the project site on the northwest corner. In addition to determining the water surface elevations under existing conditions, the two-dimensional hydraulics model was also used to design two new culvert crossings and calculated the proposed conditions water surface elevations.

5.2 94TH STREET WASH HYDRAULIC ANALYSIS

For the existing conditions analysis, the detailed topographic survey that was prepared as part of the project was used to generate the required HEC-RAS geometry files. For the 94th Street Wash model, the HEC-RAS computational domain mesh extends from the Desert Parks Vista Condominium Complex downstream past the existing five barrel 8'x3' Bell Road culvert. The extents of the computational domain incorporate the existing Bell Road culvert as well as the three main inflow locations within the project area for a peak discharge of 480 cfs. The hydraulic analysis of the existing conditions showed that the wash as well as the existing Bell Road box culvert have sufficient capacity to convey the combined peak discharge through the project area, with no water spilling over Bell Road or splitting out of the 94th Street Wash to the west. Refer to the 94th Street Wash Hydraulic Analysis Map in Appendix D for the hydraulic modeling extents, existing conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model.

The design of the Multi-Use Sports Fields and associated infrastructure leaves the 94th Street Wash mostly undisturbed. The main impact to the wash occurs at the proposed culvert crossing from the new 94th Street parking lot to the pathway system that traverses the sports fields. This culvert was designed to convey the entire combined upstream 100-year, 24-hour peak discharge of 480 cfs with no overtopping or spilling into the adjacent fields or into the new parking lot. In order to achieve this, a triple barrel 10'x5' concrete box culvert is recommended to be installed that will have sufficient capacity to convey the entire design peak discharge underneath the pathway connections. Even though the culvert has a 5-foot height, the bottom 12-inches of the culvert will be buried below the existing sandy wash bottom to allow for free movement of sediment though the culvert during storm events. In addition to burring the bottom 12-inches, the proposed 30-foot culvert width will span the exiting 25-foot jurisdictional wash bottom. The combination of burring the bottom 12-inches and spanning the entire sandy wash bottom will make the new culvert less susceptible to clogging due to sediment depositions.

The new culvert will raise the water surface elevations in the wash by about 2.0 feet from 1578.5 feet to 1580.5 feet. However due to the relative steep nature of the area, the increase in water surface elevation only propagates upstream for about 200 feet, well within the project limits. Therefore, the proposed improvements, including the construction of the new culvert crossing will not have a detrimental impact on the water surface elevation upstream within the Desert Parks Vista Condominium Complex or downstream of the Bell Road box culvert. Refer to the 94th Street Culvert Hydraulics Analysis Map in Appendix D for the proposed conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model.

5.3 91ST STREET WASH HYDRAULIC ANALYSIS

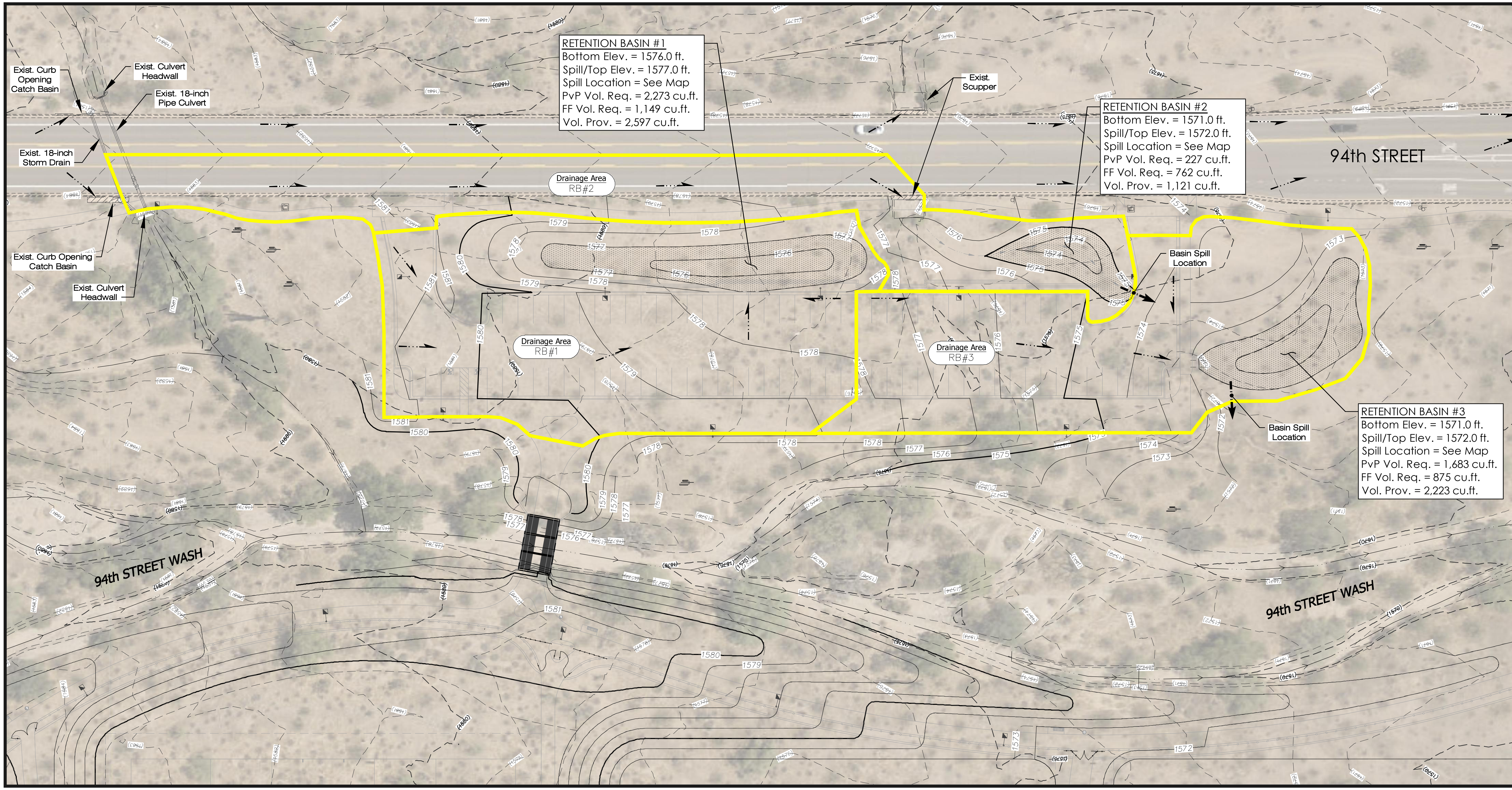
For the 91st Street Wash model, the HEC-RAS computational domain mesh extends upstream and downstream to the boundary of the topographic survey that was obtained as part of the project area. As calculated with the HEC-1 Hydrologic Model, the 100-year, 6-hour design peak discharge for the 91st Street Wash that enters the project site is 140 cfs. The hydraulic analysis of the existing conditions showed that the wash has sufficient capacity to convey the upstream design peak discharge with no water splitting out from the 91st Street Wash. Refer to the 91st Street Wash Hydraulic Analysis Map in Appendix D for the hydraulic modeling extents, existing conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model

The only impact of the Multi-Use Sports Fields project on the 91st Street Wash is the construction of the 91st Street entrance driveway. The driveway which connects the new main parking lot on the northwest corner of the project site to 91st Street must cross the existing jurisdictional wash. Like the 94th Street Wash culvert, this culvert was designed to convey the entire 100-year, 6-hour design peak discharge of 140 cfs. To achieve this, a single 10'x5' concrete box culvert is recommended to be installed at the wash crossing. To reduce the clogging potential of the new culvert, the bottom foot of the culvert will be buried and the width of the culvert will span the existing 8-foot wide sandy wash bottom to allow for free movement of the sediment through the culvert thereby reducing its clogging potential.

The new crossing will raise the water surface elevations in the wash upstream of the culvert by about 0.8 feet from 1580.0 feet to 1580.8 feet. However due to the relative steep nature of the area, the increase in water surface elevation only propagates upstream for about 50 feet, raising the water surface elevation at the property boundary by less than 0.5 feet and at the existing dual 36-inch culvert outfall by less than 0.1 foot. Therefore, the proposed entrance driveway as well as the new concrete box culvert will span the existing jurisdictional wash bottom and only slightly increase the water surface elevation within 91st Wash in the Corporate Center at DC Ranch. Refer to the 91st Street Culvert Hydraulics Analysis Map in Appendix D for the proposed conditions inundation boundary and corresponding water surface elevations as well as Appendix E for the HEC-RAS two-dimensional hydraulics model.

Appendix A: Retention Basin Design Calculations

<u>Retention Basin Design – Drainage Area Map</u>



94th STREET PARKING LOT

PRE vs. POST & FIRST FLUSH DRAINAGE AREA MAP

100-yr, 2-hr PRE vs. POST & FIRST FLUSH RUNOFF VOLUME SUMMARY TABLE

SUBBASIN	Contributing Drainage Area (sq/ft)	Pre Development Runoff Volume (cu.ft.)	Post Development Runoff Volume (cu.ft.)	Increase in Runoff Volume (cu.ft.)	First Flush Volume (cu.ft.)
RB#1	35,490	3,128	5,400	2,273	1,149
RB#2	24,100	3,355	3,582	227	762
RB#3	27,580	2,430	4,114	1,683	875

LEGEND

- **Drainage Area Boundary**
- **Major Flow Paths**
- **Local Flow Direction**

0 30' 60'
SCALE: 1" = 60'

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Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

94th Street Parking Lot Drainage Area Map

Sheet Number:

1

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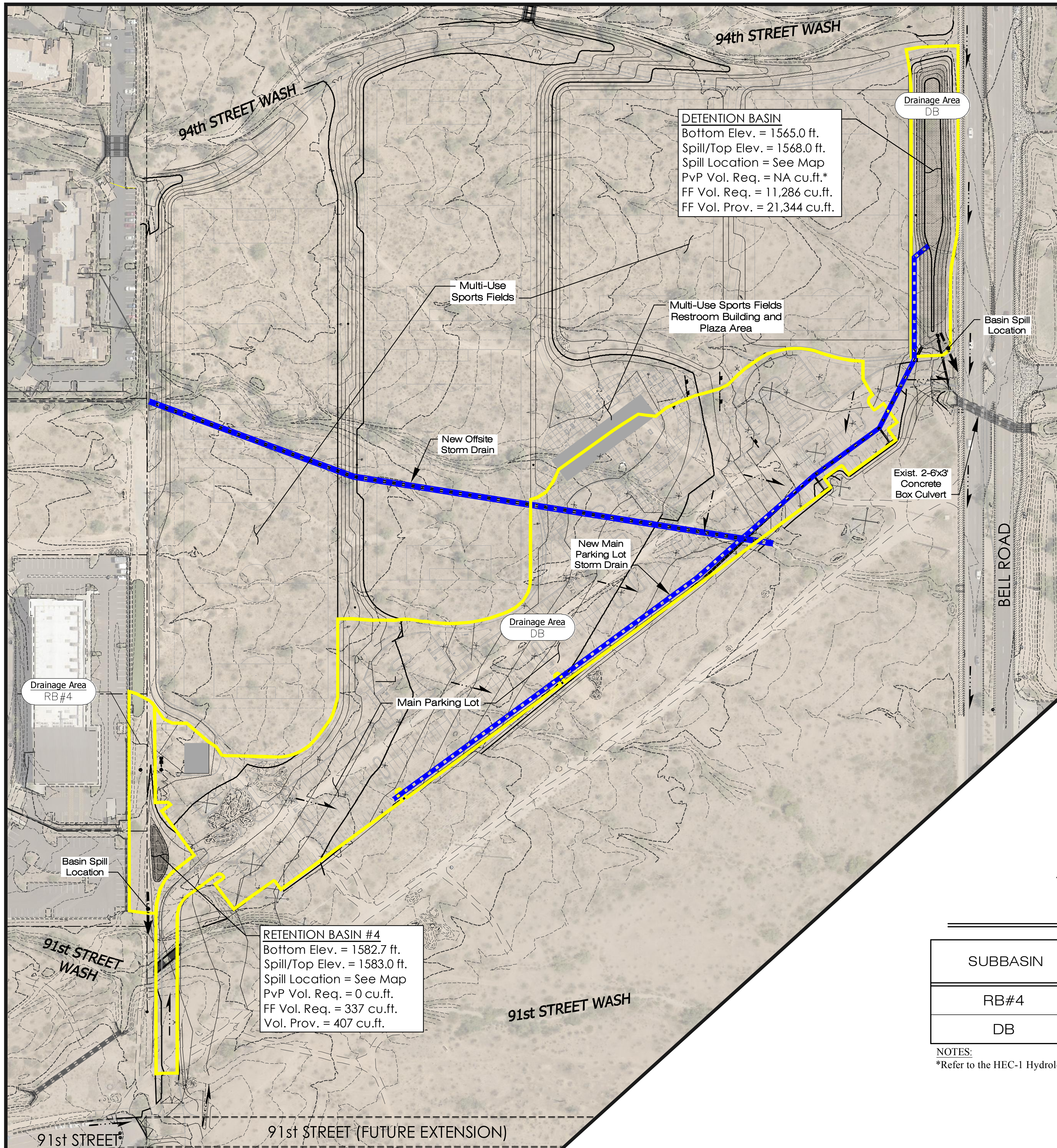

MAIN PARKING LOT

PRE vs. POST & FIRST FLUSH DRAINAGE AREA MAP

LEGEND

- **Drainage Area Boundary**
- **Major Flow Paths**
- **Local Flow Direction**
- **New Storm Drain**

0 90' 180'
SCALE: 1" = 180'

100-yr, 2-hr PRE vs. POST & FIRST FLUSH RUNOFF VOLUME SUMMARY TABLE

SUBBASIN	Contributing Drainage Area (sq/ft)	Pre Development Runoff Volume (cu.ft.)	Post Development Runoff Volume (cu.ft.)	Increase in Runoff Volume (cu.ft.)	First Flush Volume (cu.ft.)
RB#4	17,980	1,584	1,584	0	337
DB	329,710	NA*	NA*	NA*	11,286

NOTES:
*Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis

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Civil Engineering
Landscape Architecture

Project :
Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

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First Flush Volume Calculation

First Flush Volume Calculation

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

Gavan & Barker Inc.
Civil Engineering & Landscape Architecture

North 94th Street Parking Lot First-Flush Volume Calculation (Retention Basin #1)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	23,210	0.95	22,049.5	0.5	918.7
Desert Landscaping	12,280	0.45	5,526.0	0.5	230.3
Total Contributing Drainage Area:	35,490			**Total First Flush Runoff Volume:**	**1,149**

94th Street Scupper First-Flush Volume Calculation (Retention Basin #2)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	14,890	0.95	14,145.5	0.5	589.4
Desert Landscaping	9,210	0.45	4,144.5	0.5	172.7
Total Contributing Drainage Area:	24,100			**Total First Flush Runoff Volume:**	**762**

South 94th Street Parking Lot First-Flush Volume Calculation (Retention Basin #3)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	17,190	0.95	16,330.5	0.5	680.4
Desert Landscaping	10,390	0.45	4,675.5	0.5	194.8
Total Contributing Drainage Area:	27,580			**Total First Flush Runoff Volume:**	**875**

First Flush Volume Calculation

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

Gavan & Barker Inc.
Civil Engineering & Landscape Architecture

North Main Parking Lot First-Flush Volume Calculation (Retention Basin #4)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Desert Landscaping	17,980	0.45	8,091.0	0.5	337.1
Total Contributing Drainage Area:	17,980			**Total First Flush Runoff Volume:**	**337**

Main Parking Lot & Sports Complex Plaza Areas First-Flush Volume Calculation (Retention Basin #5)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	244,990	0.95	232,740.5	0.5	9,697.5
Desert Landscaping	84,720	0.45	38,124.0	0.5	1,588.5
Total Contributing Drainage Area:	329,710			**Total First Flush Runoff Volume:**	**11,286**

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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Pre vs. Post 100-year, 2-hour Runoff Volume Calculation

North 94th Street Parking Lot Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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North 94th Street Parking Lot Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #1)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	35,490	0.45	15,970.5	2.35	3,127.6
Total Contributing Drainage Area:	35,490		**Total Pre Development Runoff Volume**		**3,128**

North 94th Street Parking Lot Post Development 100-yr 2-hr Runoff Volume (Retention Basin #1)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	23,210	0.95	22,049.5	2.35	4,318.0
Desert Landscaping	12,280	0.45	5,526.0	2.35	1,082.2
Total Contributing Drainage Area:	35,490		**Total Post Development Runoff Volume**		**5,400**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**2,273**
Total First Flush Volume Required:	**1,149**
Retention Basin #1 Provided Retention Volume:	2,597

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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94th Street Scupper Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

Gavan & Barker Inc.
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94th Street Scupper Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #2)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	12,570	0.95	11,941.5	2.35	2,338.5
Undeveloped Desert	11,530	0.45	5,188.5	2.35	1,016.1
Total Contributing Drainage Area:	24,100			**Total Pre Development Runoff Volume**	**3,355**

94th Street Scupper Post Development 100-yr 2-hr Runoff Volume (Retention Basin #2)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	14,890	0.95	14,145.5	2.35	2,770.2
Desert Landscaping	9,210	0.45	4,144.5	2.35	811.6
Total Contributing Drainage Area:	24,100			**Total Post Development Runoff Volume**	**3,582**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**227**
Total First Flush Volume Required:	**762**
Retention Basin #2 Provided Retention Volume:	1,121

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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South 94th Street Parking Lot Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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South 94th Street Parking Lot Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #3)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	27,580	0.45	12,411.0	2.35	2,430.5
Total Contributing Drainage Area:	27,580			**Total Pre Development Runoff Volume**	**2,430**

South 94th Street Parking Lot Post Development 100-yr 2-hr Runoff Volume (Retention Basin #3)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	17,190	0.95	16,330.5	2.35	3,198.1
Desert Landscaping	10,390	0.45	4,675.5	2.35	915.6
Total Contributing Drainage Area:	27,580			**Total Post Development Runoff Volume**	**4,114**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**1,683**
Total First Flush Volume Required:	**875**
Retention Basin #3 Provided Retention Volume:	2,223

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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North Main Parking Lot Pre vs Post 100-year, 2-hour Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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North Main Parking Lot Pre Development 100-yr 2-hr Runoff Volume (Retention Basin #4)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	17,980	0.45	8,091.0	2.35	1,584.5
Total Contributing Drainage Area:	17,980			**Total Pre Development Runoff Volume**	**1,584**

North Main Parking Lot Post Development 100-yr 2-hr Runoff Volume (Retention Basin #4)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Desert Landscaping	17,980	0.45	8,091.0	2.35	1,584.5
Total Contributing Drainage Area:	17,980			**Total Post Development Runoff Volume**	**1,584**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**0**
Total First Flush Volume Required:	**337**
Retention Basin #4 Provided Retention Volume (0.5 ft Depth):	560

^The first-flush rainfall depth was obtained from **Section 4-1.201 (C)(2)(a)** of the *City of Scottsdale Drainage Policies and Standards Manual.*

*The runoff coefficients were obtained from **Figure 4-1.5** of the *City of Scottsdale Drainage Policies and Standards Manual.*

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Main Parking Lot & Sports Complex Plaza Areas Pre vs Post Runoff Volume Calculations

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
Gavan & Barker No. 2003
City of Scottsdale Contract No.: 2020-068-COS

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Main Parking Lot & Sports Complex Plaza Areas Pre 100-yr 2-hr Runoff Volume (Detention Basin)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis					
Total Contributing Drainage Area:	329,710			**Total Pre Development Runoff Volume**	**0**

Main Parking Lot & Sports Complex Plaza Areas Post 100-yr 2-hr Runoff Volume (Detention Basin)

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis					
Total Contributing Drainage Area:	329,710			**Total Post Development Runoff Volume**	**0**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**0**
Total First Flush Volume Required:	**11,286**
Retention Basin #5 Provided Retention Volume~:	21,344

~The provided retention basin #5 volume is calculated to the primary basin outlet spill elevation of 1567.20 ft.

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Retention Basin Drain Time Calculation

Basin Drain Times

Multi-Use Sports Fields
NWC of Bell Road & 94th Street
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Retention Basin	Depth of Ponding (ft)	Tested Perc. Rate* (in/hr)	Safty Factor^	Effective Perc. Rate (ft/hr)	Basin Drain Time (hr)
Retention Basin #1	1.00	2.50	0.5	0.1042	9.6
Retention Basin #2	1.00	2.50	0.5	0.1042	9.6
Retention Basin #3	1.00	3.25	0.5	0.1354	7.4
Retention Basin #4	0.30	*Depth of ponding less than 6 inches, therefore no drain time calculation performed*			
Detention Basin~	0.50	1.75	0.5	0.0729	6.9

*The tested percolation rate was obtained from a Double Ring Infiltration Test performed by Speedie and Associates at the location of the four proposed basins. Refer to the letter by Speedie and Associates in the Digital Data folder for the location and results of the infiltration test.

^In accordance with the *City of Scottsdale Drainage Policies and Standards Manual* a safty factor of 50% was applied to the tested percolation rate to obtain the effective percolation rate.

~Depths of ponding in the detention basin greater than 0.50 feet are drained through the basin bleed-off inlet and 3-inch orifice plate. During the 100-year, 6-hour storm event, the bleed-off inlet will reduce the basin depth to 0.50 feet in approximately 17.3 hours. The additional 0.50 feet, below the bleed-off inlet invert will drain in an additional 6.9 hours, for a total basin drain time of 24.2 hours.

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Appendix B: Hydrologic Analysis

Existing Conditions: HEC-1 Schematic w/100-year, 6-hour Model

LEGEND & HEC-1 SYMBOLOGY

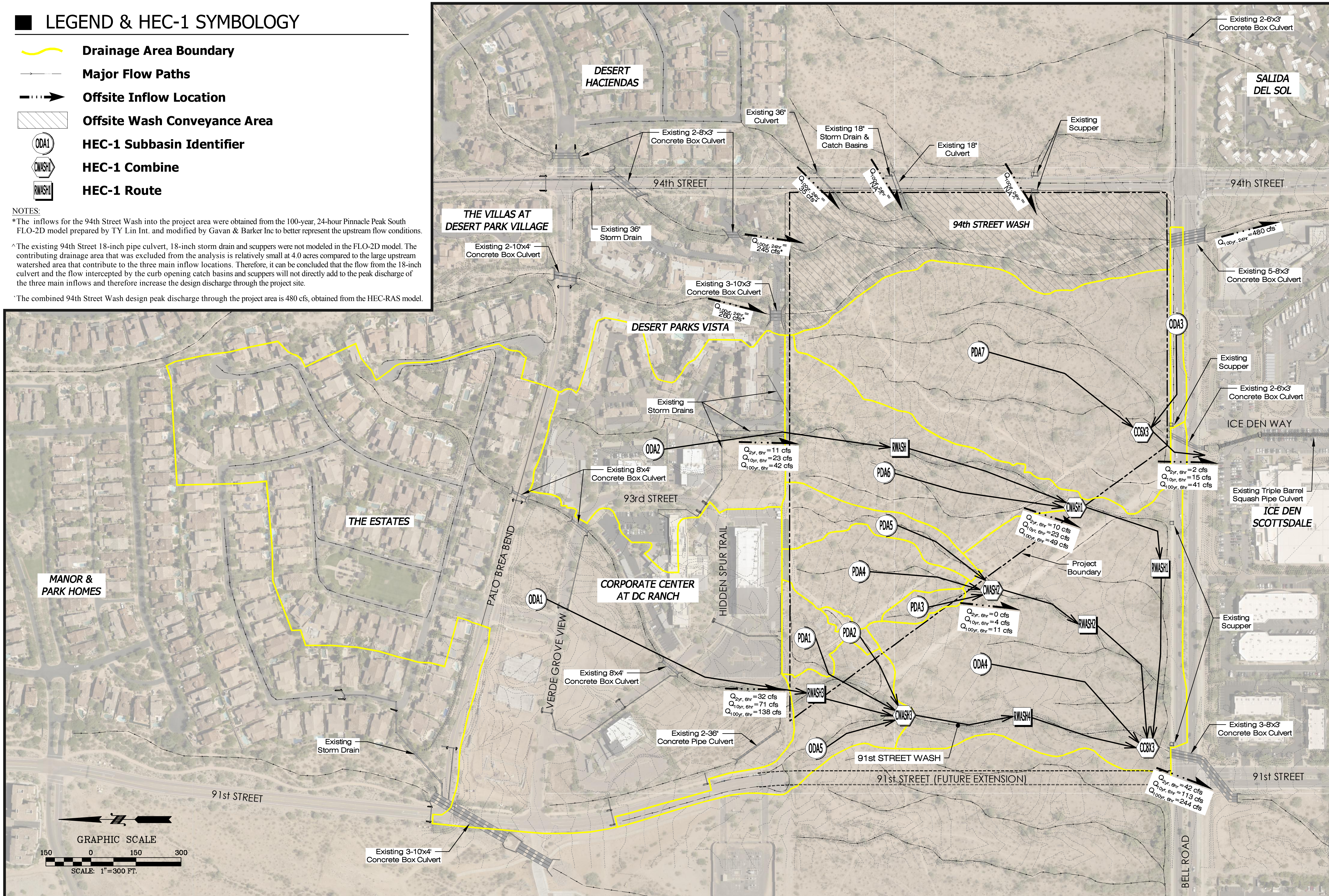
- **Drainage Area Boundary**
- **Major Flow Paths**
- **Offsite Inflow Location**
- **Offsite Wash Conveyance Area**
- ODA1 **HEC-1 Subbasin Identifier**
- CWASH1 **HEC-1 Combine**
- RWASH1 **HEC-1 Route**

NOTES:

*The inflows for the 94th Street Wash into the project area were obtained from the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin Int. and modified by Gavan & Barker Inc to better represent the upstream flow conditions.

^The existing 94th Street 18-inch pipe culvert, 18-inch storm drain and scuppers were not modeled in the FLO-2D model. The contributing drainage area that was excluded from the analysis is relatively small at 4.0 acres compared to the large upstream watershed area that contribute to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins and scuppers will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

`The combined 94th Street Wash design peak discharge through the project area is 480 cfs, obtained from the HEC-RAS model.



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Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

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Sheet Title :

Existing Condition HEC-1 Drainage Area Map

Sheet Number:

1

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1***************************************                                   ***************************************
 *                                     *                                   *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                 *    U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998             *                                   *    HYDROLOGIC ENGINEERING CENTER    *
 *            VERSION 4.1              *                                   *          609 SECOND STREET          *
 *                                     *                                   *       DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   06SEP20  TIME  12:51:11 *                                   *           (916) 756-1104            *
 *                                     *                                   *                                     *
 ***************************************                                   ***************************************


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                    X     X  X        X     X         XX
                    X     X  X        X                X
                    XXXXXXX  XXXX     X        XXXXX   X
                    X     X  X        X                X
                    X     X  X        X     X          X
                    X     X  XXXXXXX   XXXXX          XXX


 THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

 THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
 THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
 NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
 DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
 KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
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 LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1           ID        City of Scottsdale
   2           ID        MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
   3           ID        100 YEAR
   4           ID        6 Hour  Storm
   5           ID        Unit Hydrograph: Clark
   6           ID        Storm: Single
   7           ID        06/02/2020
               *DIAGRAM
   8           IT       2  1JAN99       0     360
   9           IO       5
  10           IN      15
               *
               *
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  11           KK    ODA3   BASIN
  12           BA   0.001
  13           PB   2.807
  14           PC   0.000   0.008   0.016   0.025   0.033   0.041   0.050   0.058   0.066   0.074
  15           PC   0.087   0.099   0.118   0.138   0.216   0.377   0.834   0.911   0.931   0.950
  16           PC   0.962   0.972   0.983   0.991   1.000
  17           LG    0.05    0.35    4.03    0.39      95
  18           UC   0.185   0.498
  19           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  20           UA     100
  21           ZW   A=ODA3   B=BASIN  C=FLOW  F=CALC
               *

  22           KK    PDA7   BASIN
  23           BA   0.024
  24           LG    0.35    0.35    4.03    0.43       0
  25           UC   0.257   0.197
  26           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  27           UA     100
  28           ZW   A=PDA7   B=BASIN  C=FLOW  F=CALC
               *

  29           KK   CC6X3 COMBINE
  30           HC       2       2
  31           ZW   A=CC6X3   B=COMBINE  C=FLOW  F=CALC
               *

  32           KK    ODA2   BASIN
  33           BA   0.016
  34           LG    0.19    0.25    4.03    0.59      51
  35           UC   0.142   0.098
  36           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  37           UA     100
  38           ZW   A=ODA2   B=BASIN  C=FLOW  F=CALC
               *
1                                     HEC-1 INPUT                                            PAGE  2

LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


  39           KK   RWASH   ROUTE
  40           RS       1    FLOW
  41           RC   0.045   0.030   0.045    1020  0.0168    3.00
  42           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
  43           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
  44           ZW   A=RWASH   B=ROUTE  C=FLOW  F=CALC
               *

  45           KK    PDA6   BASIN
  46           BA   0.010
```

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     47           LG    0.35    0.35    4.03    0.43       0
     48           UC   0.285   0.336
     49           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
     50           UA     100
     51           ZW   A=PDA6   B=BASIN  C=FLOW  F=CALC
                  *

     52           KK  CWASH1 COMBINE
     53           HC       2
     54           ZW   A=CWASH1   B=COMBINE  C=FLOW  F=CALC
                  *

     55           KK  RWASH1   ROUTE
     56           RS       1    FLOW
     57           RC   0.045   0.030   0.045    1050  0.0087    3.00
     58           RX    0.00   10.00   20.00   30.00   40.00   50.00   60.00   70.00
     59           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
     60           ZW   A=RWASH1   B=ROUTE  C=FLOW  F=CAL
                  *

     61           KK    PDA5   BASIN
     62           BA   0.003
     63           LG    0.35    0.35    4.03    0.43       0
     64           UC   0.240   0.407
     65           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
     66           UA     100
     67           ZW   A=PDA5   B=BASIN  C=FLOW  F=CALC
                  *

     68           KK    PDA4   BASIN
     69           BA   0.004
     70           LG    0.35    0.35    4.03    0.43       0
     71           UC   0.205   0.253
     72           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
     73           UA     100
     74           ZW   A=PDA4   B=BASIN  C=FLOW  F=CALC
                  *

     75           KK    PDA3   BASIN
     76           BA   0.001
     77           LG    0.35    0.35    4.03    0.43       0
     78           UC   0.147   0.206
     79           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
     80           UA     100
1                                               HEC-1 INPUT                                           PAGE  3

   LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

     81           ZW   A=PDA3   B=BASIN  C=FLOW  F=CALC
                  *
```

```
  82           KK  CWASH2 COMBINE
  83           HC       3
  84           ZW   A=CWASH2   B=COMBINE  C=FLOW  F=CALC
               *

  85           KK  RWASH2   ROUTE
  86           RS       1    FLOW
  87           RC   0.045   0.030   0.045     860  0.0128    3.00
  88           RX    0.00   10.00   20.00   25.00   28.00   33.00   43.00   53.00
  89           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
  90           ZW   A=RWASH2   B=ROUTE  C=FLOW  F=CAL
               *

  91           KK    PDA2   BASIN
  92           BA   0.001
  93           LG    0.35    0.35    4.03    0.43       0
  94           UC   0.128   0.148
  95           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  96           UA     100
  97           ZW   A=PDA2   B=BASIN  C=FLOW  F=CALC
               *

  98           KK    PDA1   BASIN
  99           BA   0.002
 100           LG    0.35    0.35    4.03    0.43       0
 101           UC   0.174   0.219
 102           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 103           UA     100
 104           ZW   A=PDA1   B=BASIN  C=FLOW  F=CALC
               *

 105           KK    ODA1   BASIN
 106           BA   0.069
 107           LG    0.20    0.25    4.03    0.59      48
 108           UC   0.247   0.200
 109           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 110           UA     100
 111           ZW   A=ODA1   B=BASIN  C=FLOW  F=CALC
               *

 112           KK  RWASH3   ROUTE
 113           RS       1    FLOW
 114           RC   0.045   0.030   0.045     450  0.0138    3.00
 115           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
 116           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 117           ZW   A=RWASH3   B=ROUTE  C=FLOW  F=CAL
               *
```

```
         118           KK    ODA9   BASIN
         119           BA   0.006
         120           LG    0.29    0.35    4.03    0.42      19
         121           UC   0.189   0.211
         122           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
         123           UA     100
         124           ZW   A=ODA9   B=BASIN  C=FLOW  F=CALC
                       *

         125           KK  CWASH3 COMBINE
         126           HC       4
         127           ZW   A=CWASH3   B=COMBINE  C=FLOW  F=CALC
                       *

         128           KK  RWASH4   ROUTE
         129           RS       1    FLOW
         130           RC   0.045   0.030   0.045     870  0.0127    3.00
         131           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
         132           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
         133           ZW   A=RWASH4   B=ROUTE  C=FLOW  F=CAL
                       *

         134           KK    ODA8   BASIN
         135           BA   0.025
         136           LG    0.33    0.35    4.03    0.43       6
         137           UC   0.276   0.184
         138           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
         139           UA     100
         140           ZW   A=ODA8   B=BASIN  C=FLOW  F=CALC
                       *

         141           KK   CC8X3 COMBINE
         142           HC       4
         143           ZW   A=CC8X3   B=COMBINE  C=FLOW  F=CALC
                       *
         144           ZZ
1
                  SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

    11        ODA3
                 .
                 .
    22           .         PDA7
                 .            .
                 .            .
    29       CC6X3............
```

```
            .
            .
 32         .        ODA2
            .           V
            .           V
 39         .       RWASH
            .           .
            .           .
 45         .           .        PDA6
            .           .           .
            .           .           .
 52         .      CWASH1............
            .           V
            .           V
 55         .      RWASH1
            .           .
            .           .
 61         .           .        PDA5
            .           .           .
            .           .           .
 68         .           .           .        PDA4
            .           .           .           .
            .           .           .           .
 75         .           .           .           .        PDA3
            .           .           .           .           .
            .           .           .           .           .
 82         .           .      CWASH2........................
            .           .           V
            .           .           V
 85         .           .      RWASH2
            .           .           .
            .           .           .
 91         .           .           .        PDA2
            .           .           .           .
            .           .           .           .
 98         .           .           .           .        PDA1
            .           .           .           .           .
            .           .           .           .           .
105         .           .           .           .           .        ODA1
            .           .           .           .           .           V
            .           .           .           .           .           V
112         .           .           .           .           .      RWASH3
            .           .           .           .           .           .
            .           .           .           .           .           .
118         .           .           .           .           .           .        ODA9
            .           .           .           .           .           .           .
            .           .           .           .           .           .           .
125         .           .           .      CWASH3....................................
            .           .           .           V
            .           .           .           V
128         .           .           .      RWASH4
```

```
               .           .           .           .
               .           .           .           .
 134           .           .           .           .        ODA8
               .           .           .           .           .
               .           .           .           .           .
 141           .       CC8X3...................................

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                                                  ***************************************
 *                                        *                                                  *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)    *                                                  *    U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998                *                                                  *    HYDROLOGIC ENGINEERING CENTER    *
 *           VERSION 4.1                  *                                                  *          609 SECOND STREET          *
 *                                        *                                                  *       DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   06SEP20  TIME  12:51:11    *                                                  *           (916) 756-1104            *
 *                                        *                                                  *                                     *
 *****************************************                                                  ***************************************



                              City of Scottsdale
                              MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
                              100 YEAR
                              6 Hour  Storm
                              Unit Hydrograph: Clark
                              Storm: Single
                              06/02/2020

    9 IO         OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

      IT         HYDROGRAPH TIME DATA
                        NMIN           2  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1JAN99  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ         360  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1JAN99  ENDING DATE
                      NDTIME        1158  ENDING TIME
                      ICENT           19  CENTURY MARK

                   COMPUTATION INTERVAL     .03 HOURS
                        TOTAL TIME BASE   11.97 HOURS

           ENGLISH UNITS
                DRAINAGE AREA         SQUARE MILES
                PRECIPITATION DEPTH   INCHES
                LENGTH, ELEVATION     FEET
```

10-UP-2020
09/15/20

FLOW	CUBIC FEET PER SECOND
STORAGE VOLUME	ACRE-FEET
SURFACE AREA	ACRES
TEMPERATURE	DEGREES FAHRENHEIT

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
	HYDROGRAPH AT									
+		ODA3	2.	4.07	0.	0.	0.	.00		
	HYDROGRAPH AT									
+		PDA7	39.	4.13	3.	1.	1.	.02		
	2 COMBINED AT									
+		CC6X3	41.	4.13	3.	1.	1.	.03		
	HYDROGRAPH AT									
+		ODA2	42.	4.03	3.	2.	2.	.02		
	ROUTED TO									
+		RWASH	40.	4.07	3.	2.	2.	.02		
+									.90	4.07
	HYDROGRAPH AT									
+		PDA6	12.	4.17	1.	1.	1.	.01		
	2 COMBINED AT									
+		CWASH1	49.	4.07	4.	2.	2.	.03		
	ROUTED TO									
+		RWASH1	44.	4.13	4.	2.	2.	.03		
+									.79	4.13
	HYDROGRAPH AT									
+		PDA5	3.	4.17	0.	0.	0.	.00		
	HYDROGRAPH AT									
+		PDA4	6.	4.13	0.	0.	0.	.00		
	HYDROGRAPH AT									
+		PDA3	2.	4.07	0.	0.	0.	.00		
	3 COMBINED AT									
+		CWASH2	11.	4.13	1.	0.	0.	.01		

```
           ROUTED TO
+                           RWASH2        10.    4.17           1.          0.          0.         .01
+                                                                                                              .56        4.17

           HYDROGRAPH AT
+                             PDA2         2.    4.07           0.          0.          0.         .00

           HYDROGRAPH AT
+                             PDA1         3.    4.10           0.          0.          0.         .00

           HYDROGRAPH AT
+                             ODA1       138.    4.07          14.          7.          7.         .07

           ROUTED TO
+                           RWASH3       136.    4.10          14.          7.          7.         .07
+                                                                                                             1.61        4.10

           HYDROGRAPH AT
+                             ODA9        11.    4.10           1.          0.          0.         .01

           4 COMBINED AT
+                           CWASH3       152.    4.10          15.          8.          8.         .08

           ROUTED TO
+                           RWASH4       148.    4.13          15.          8.          8.         .08
+                                                                                                             1.69        4.13

           HYDROGRAPH AT
+                             ODA8        43.    4.17           3.          2.          2.         .03

           4 COMBINED AT
+                            CC8X3       244.    4.13          23.         12.         12.         .14


 *** NORMAL END OF HEC-1 ***


    -----DSS---ZCLOSE Unit:  71,   File: EC100.DSS
               Pointer Utilization:   .27
               Number of Records:     56
               File Size:    209.4  Kbytes
               Percent Inactive:    .0
```

Proposed Conditions: HEC-1 Schematic w/100-year, 6-hour Model

LEGEND & HEC-1 SYMBOLOGY

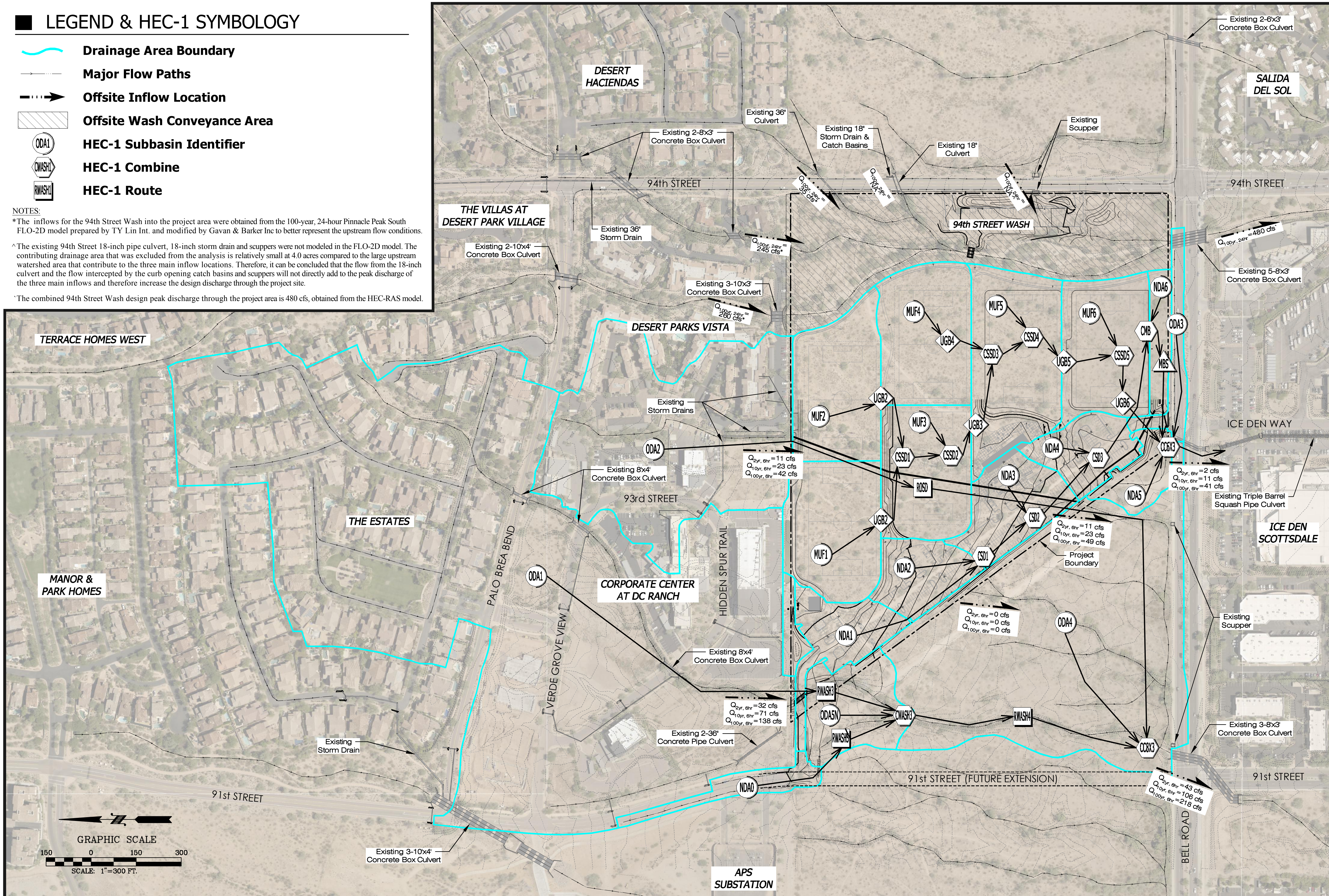
- Drainage Area Boundary
- Major Flow Paths
- Offsite Inflow Location
- Offsite Wash Conveyance Area
- ODA1 — HEC-1 Subbasin Identifier
- CWASH1 — HEC-1 Combine
- RWASH1 — HEC-1 Route

NOTES:

*The inflows for the 94th Street Wash into the project area were obtained from the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin Int. and modified by Gavan & Barker Inc to better represent the upstream flow conditions.

^The existing 94th Street 18-inch pipe culvert, 18-inch storm drain and scuppers were not modeled in the FLO-2D model. The contributing drainage area that was excluded from the analysis is relatively small at 4.0 acres compared to the large upstream watershed area that contribute to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins and scuppers will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

`The combined 94th Street Wash design peak discharge through the project area is 480 cfs, obtained from the HEC-RAS model.



Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032

Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

Proposed Condition HEC-1 Drainage Area Map

Sheet Number:

1

10-UP-2020
09/15/20

```
1**************************************                                                          **************************************
 *                                    *                                                          *                                    *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                                        *   U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998              *                                                        *   HYDROLOGIC ENGINEERING CENTER    *
 *            VERSION 4.1               *                                                        *        609 SECOND STREET           *
 *                                    *                                                          *     DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   06SEP20  TIME  12:51:11   *                                                       *         (916) 756-1104             *
 *                                    *                                                          *                                    *
 **************************************                                                          **************************************


                              X     X  XXXXXXX   XXXXX           X
                              X     X  X        X     X         XX
                              X     X  X        X                X
                              XXXXXXX  XXXX     X        XXXXX   X
                              X     X  X        X                X
                              X     X  X        X     X          X
                              X     X  XXXXXXX   XXXXX          XXX


           THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

           THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
           THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
           NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
           DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
           KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1                                                        HEC-1 INPUT                                                    PAGE  1

          LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

            1            ID        City of Scottsdale
            2            ID        MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
            3            ID        100 YEAR
            4            ID        6 Hour  Storm
            5            ID        Unit Hydrograph: Clark
            6            ID        Storm: Single
            7            ID        06/02/2020
                         *DIAGRAM
            8            IT       2  1JAN99       0     360
            9            IO       5
           10            IN      15
                         *
                         *
```

10-UP-2020
09/15/20

```
   11           KK    ODA3   BASIN
   12           BA   0.001
   13           PB   2.807
   14           PC   0.000   0.008   0.016   0.025   0.033   0.041   0.050   0.058   0.066   0.074
   15           PC   0.087   0.099   0.118   0.138   0.216   0.377   0.834   0.911   0.931   0.950
   16           PC   0.962   0.972   0.983   0.991   1.000
   17           LG    0.05    0.35    4.03    0.39      95
   18           UC   0.185   0.498
   19           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
   20           UA     100
   21           ZW   A=ODA3   B=BASIN  C=FLOW  F=CALC
                *

   22           KK    PDA7   BASIN
   23           BA   0.024
   24           LG    0.35    0.35    4.03    0.43       0
   25           UC   0.257   0.197
   26           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
   27           UA     100
   28           ZW   A=PDA7   B=BASIN  C=FLOW  F=CALC
                *

   29           KK   CC6X3 COMBINE
   30           HC       2       2
   31           ZW   A=CC6X3   B=COMBINE  C=FLOW  F=CALC
                *

   32           KK    ODA2   BASIN
   33           BA   0.016
   34           LG    0.19    0.25    4.03    0.59      51
   35           UC   0.142   0.098
   36           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
   37           UA     100
   38           ZW   A=ODA2   B=BASIN  C=FLOW  F=CALC
                *
```



```
 LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10


   39           KK   RWASH   ROUTE
   40           RS       1    FLOW
   41           RC   0.045   0.030   0.045    1020  0.0168    3.00
   42           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
   43           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
   44           ZW   A=RWASH   B=ROUTE  C=FLOW  F=CALC
                *

   45           KK    PDA6   BASIN
   46           BA   0.010
```

```
   47           LG    0.35    0.35    4.03    0.43       0
   48           UC   0.285   0.336
   49           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
   50           UA     100
   51           ZW   A=PDA6   B=BASIN  C=FLOW  F=CALC
                *

   52           KK  CWASH1 COMBINE
   53           HC       2
   54           ZW   A=CWASH1   B=COMBINE  C=FLOW  F=CALC
                *

   55           KK  RWASH1   ROUTE
   56           RS       1    FLOW
   57           RC   0.045   0.030   0.045    1050  0.0087    3.00
   58           RX    0.00   10.00   20.00   30.00   40.00   50.00   60.00   70.00
   59           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
   60           ZW   A=RWASH1   B=ROUTE  C=FLOW  F=CAL
                *

   61           KK    PDA5   BASIN
   62           BA   0.003
   63           LG    0.35    0.35    4.03    0.43       0
   64           UC   0.240   0.407
   65           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
   66           UA     100
   67           ZW   A=PDA5   B=BASIN  C=FLOW  F=CALC
                *

   68           KK    PDA4   BASIN
   69           BA   0.004
   70           LG    0.35    0.35    4.03    0.43       0
   71           UC   0.205   0.253
   72           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
   73           UA     100
   74           ZW   A=PDA4   B=BASIN  C=FLOW  F=CALC
                *

   75           KK    PDA3   BASIN
   76           BA   0.001
   77           LG    0.35    0.35    4.03    0.43       0
   78           UC   0.147   0.206
   79           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
   80           UA     100
```



```
 LINE           ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   81           ZW   A=PDA3   B=BASIN  C=FLOW  F=CALC
                *
```

```
 82           KK  CWASH2 COMBINE
 83           HC       3
 84           ZW   A=CWASH2   B=COMBINE  C=FLOW  F=CALC
              *

 85           KK  RWASH2   ROUTE
 86           RS       1    FLOW
 87           RC   0.045   0.030   0.045     860  0.0128    3.00
 88           RX    0.00   10.00   20.00   25.00   28.00   33.00   43.00   53.00
 89           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 90           ZW   A=RWASH2   B=ROUTE  C=FLOW  F=CAL
              *

 91           KK    PDA2   BASIN
 92           BA   0.001
 93           LG    0.35    0.35    4.03    0.43       0
 94           UC   0.128   0.148
 95           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
 96           UA     100
 97           ZW   A=PDA2   B=BASIN  C=FLOW  F=CALC
              *

 98           KK    PDA1   BASIN
 99           BA   0.002
100           LG    0.35    0.35    4.03    0.43       0
101           UC   0.174   0.219
102           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
103           UA     100
104           ZW   A=PDA1   B=BASIN  C=FLOW  F=CALC
              *

105           KK    ODA1   BASIN
106           BA   0.069
107           LG    0.20    0.25    4.03    0.59      48
108           UC   0.247   0.200
109           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
110           UA     100
111           ZW   A=ODA1   B=BASIN  C=FLOW  F=CALC
              *

112           KK  RWASH3   ROUTE
113           RS       1    FLOW
114           RC   0.045   0.030   0.045     450  0.0138    3.00
115           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
116           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
117           ZW   A=RWASH3   B=ROUTE  C=FLOW  F=CAL
              *
```

```
           118           KK    ODA9   BASIN
           119           BA   0.006
           120           LG    0.29    0.35    4.03    0.42      19
           121           UC   0.189   0.211
           122           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
           123           UA     100
           124           ZW   A=ODA9   B=BASIN  C=FLOW  F=CALC
                         *

           125           KK  CWASH3 COMBINE
           126           HC       4
           127           ZW   A=CWASH3   B=COMBINE  C=FLOW  F=CALC
                         *

           128           KK  RWASH4   ROUTE
           129           RS       1    FLOW
           130           RC   0.045   0.030   0.045     870  0.0127    3.00
           131           RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
           132           RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
           133           ZW   A=RWASH4   B=ROUTE  C=FLOW  F=CAL
                         *

           134           KK    ODA8   BASIN
           135           BA   0.025
           136           LG    0.33    0.35    4.03    0.43       6
           137           UC   0.276   0.184
           138           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
           139           UA     100
           140           ZW   A=ODA8   B=BASIN  C=FLOW  F=CALC
                         *

           141           KK   CC8X3 COMBINE
           142           HC       4
           143           ZW   A=CC8X3   B=COMBINE  C=FLOW  F=CALC
                         *
           144           ZZ
1
                  SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

    11        ODA3
                 .
                 .
    22           .        PDA7
                 .           .
                 .           .
    29       CC6X3............
```

```
                             .
                             .
                32           .        ODA2
                             .           V
                             .           V
                39           .       RWASH
                             .           .
                             .           .
                45           .           .        PDA6
                             .           .           .
                             .           .           .
                52           .      CWASH1............
                             .           V
                             .           V
                55           .      RWASH1
                             .           .
                             .           .
                61           .           .        PDA5
                             .           .           .
                             .           .           .
                68           .           .           .        PDA4
                             .           .           .           .
                             .           .           .           .
                75           .           .           .           .        PDA3
                             .           .           .           .           .
                             .           .           .           .           .
                82           .           .      CWASH2........................
                             .           .           V
                             .           .           V
                85           .           .      RWASH2
                             .           .           .
                             .           .           .
                91           .           .           .        PDA2
                             .           .           .           .
                             .           .           .           .
                98           .           .           .           .        PDA1
                             .           .           .           .           .
                             .           .           .           .           .
               105           .           .           .           .           .        ODA1
                             .           .           .           .           .           V
                             .           .           .           .           .           V
               112           .           .           .           .           .      RWASH3
                             .           .           .           .           .           .
                             .           .           .           .           .           .
               118           .           .           .           .           .           .        ODA9
                             .           .           .           .           .           .           .
                             .           .           .           .           .           .           .
               125           .           .           .      CWASH3....................................
                             .           .           .           V
                             .           .           .           V
               128           .           .           .      RWASH4
```

```
                 .           .           .           .
                 .           .           .           .
  134            .           .           .           .        ODA8
                 .           .           .           .           .
                 .           .           .           .           .
  141            .       CC8X3...................................

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                                                           ***************************************
*                                         *                                                          *                                     *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)     *                                                          *     U.S. ARMY CORPS OF ENGINEERS    *
*              JUN   1998                 *                                                          *     HYDROLOGIC ENGINEERING CENTER   *
*           VERSION 4.1                   *                                                          *           609 SECOND STREET         *
*                                         *                                                          *        DAVIS, CALIFORNIA 95616      *
*  RUN DATE   06SEP20  TIME  12:51:11     *                                                          *            (916) 756-1104           *
*                                         *                                                          *                                     *
*****************************************                                                            ***************************************



                                    City of Scottsdale
                                    MULTI-USE FIELDS - Multi-Use Sports Fields NWC Bell Rd & 94th St
                                    100 YEAR
                                    6 Hour  Storm
                                    Unit Hydrograph: Clark
                                    Storm: Single
                                    06/02/2020

   9 IO          OUTPUT CONTROL VARIABLES
                       IPRNT           5  PRINT CONTROL
                       IPLOT           0  PLOT CONTROL
                       QSCAL          0.  HYDROGRAPH PLOT SCALE

     IT          HYDROGRAPH TIME DATA
                        NMIN           2  MINUTES IN COMPUTATION INTERVAL
                       IDATE      1JAN99  STARTING DATE
                       ITIME        0000  STARTING TIME
                          NQ         360  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1JAN99  ENDING DATE
                      NDTIME        1158  ENDING TIME
                      ICENT           19  CENTURY MARK

                   COMPUTATION INTERVAL      .03 HOURS
                        TOTAL TIME BASE    11.97 HOURS

         ENGLISH UNITS
              DRAINAGE AREA          SQUARE MILES
              PRECIPITATION DEPTH    INCHES
              LENGTH, ELEVATION      FEET
```

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09/15/20

FLOW	CUBIC FEET PER SECOND
STORAGE VOLUME	ACRE-FEET
SURFACE AREA	ACRES
TEMPERATURE	DEGREES FAHRENHEIT

1

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	ODA3	2.	4.07	0.	0.	0.	.00		
+	HYDROGRAPH AT	PDA7	39.	4.13	3.	1.	1.	.02		
+	2 COMBINED AT	CC6X3	41.	4.13	3.	1.	1.	.03		
+	HYDROGRAPH AT	ODA2	42.	4.03	3.	2.	2.	.02		
+	ROUTED TO	RWASH	40.	4.07	3.	2.	2.	.02		
+									.90	4.07
+	HYDROGRAPH AT	PDA6	12.	4.17	1.	1.	1.	.01		
+	2 COMBINED AT	CWASH1	49.	4.07	4.	2.	2.	.03		
+	ROUTED TO	RWASH1	44.	4.13	4.	2.	2.	.03		
+									.79	4.13
+	HYDROGRAPH AT	PDA5	3.	4.17	0.	0.	0.	.00		
+	HYDROGRAPH AT	PDA4	6.	4.13	0.	0.	0.	.00		
+	HYDROGRAPH AT	PDA3	2.	4.07	0.	0.	0.	.00		
+	3 COMBINED AT	CWASH2	11.	4.13	1.	0.	0.	.01		

ROUTED TO									
+	RWASH2	10.	4.17	1.	0.	0.	.01		
+								.56	4.17
HYDROGRAPH AT									
+	PDA2	2.	4.07	0.	0.	0.	.00		
HYDROGRAPH AT									
+	PDA1	3.	4.10	0.	0.	0.	.00		
HYDROGRAPH AT									
+	ODA1	138.	4.07	14.	7.	7.	.07		
ROUTED TO									
+	RWASH3	136.	4.10	14.	7.	7.	.07		
+								1.61	4.10
HYDROGRAPH AT									
+	ODA9	11.	4.10	1.	0.	0.	.01		
4 COMBINED AT									
+	CWASH3	152.	4.10	15.	8.	8.	.08		
ROUTED TO									
+	RWASH4	148.	4.13	15.	8.	8.	.08		
+								1.69	4.13
HYDROGRAPH AT									
+	ODA8	43.	4.17	3.	2.	2.	.03		
4 COMBINED AT									
+	CC8X3	244.	4.13	23.	12.	12.	.14		

*** NORMAL END OF HEC-1 ***

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-----DSS---ZCLOSE Unit:  71,   File: EC100.DSS
           Pointer Utilization:    .27
           Number of Records:     56
           File Size:    209.4  Kbytes
           Percent Inactive:    .0
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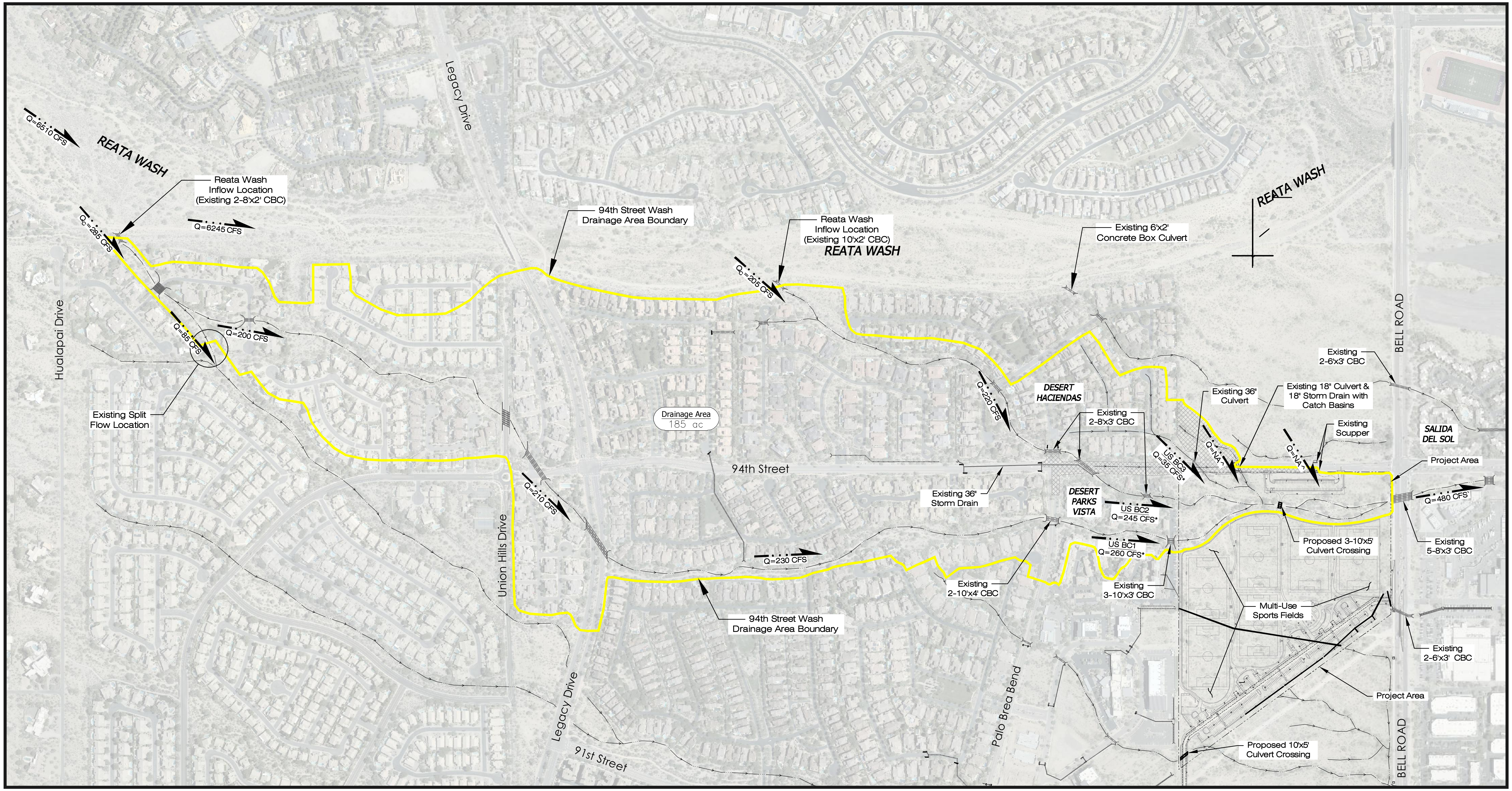
94th Street Wash Drainage Area Map & Inflow Hydrographs



94th STREET WASH

DRAINAGE AREA MAP

LEGEND

- **Drainage Area Boundary**
- **Major Flow Paths**
- Q=210 CFS **Major Peak Flows**
- **Excluded Drainage Area**

0 300' 600'

SCALE: 1'' = 600'

NOTES:

*The inflows for the 94th Street Wash into the project area were obtained from the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin International and modified by Gavan & Barker Inc, to better represent the upstream flow conditions. The resulting inflow hydrographs at the three main inflow locations are included in Appendix B and the FLO-2D model is included in the Digital Data folder in Appendix E.

^The existing 94th Street 18-inch pipe culvert, 18-inch storm drain and scuppers were not modeled in the FLO-2D model. The contributing drainage area that was excluded from the analysis is relatively small at 4.0 acres compared to the large upstream watershed area that contribute to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins and scuppers will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

`The combined 94th Street Wash design peak discharge through the project area is 480 cfs, obtained from the HEC-RAS model.

Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032

Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

94th Street Wash Drainage Area Map

Sheet Number:

1

1 of 1

10-UP-2020

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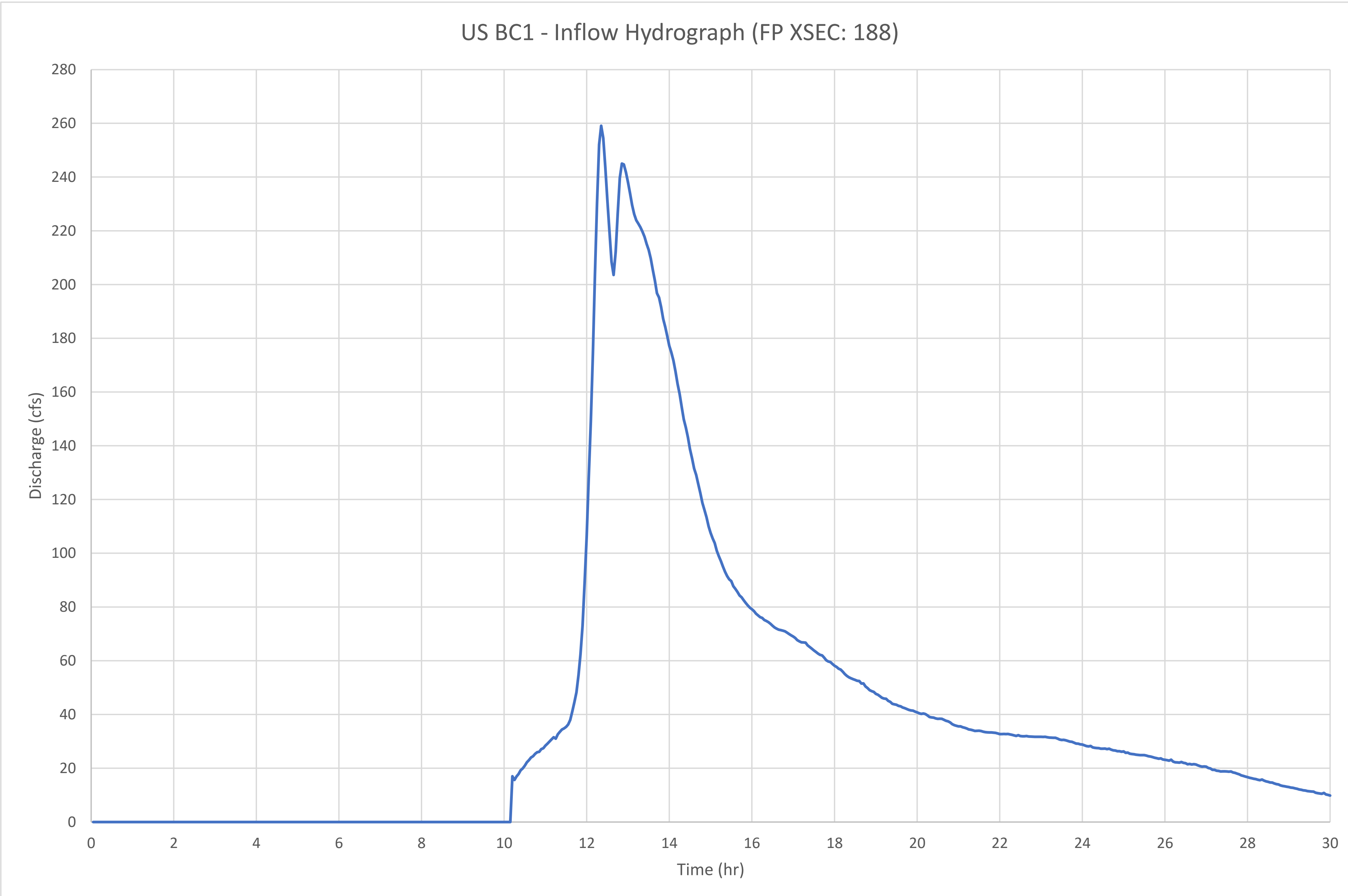
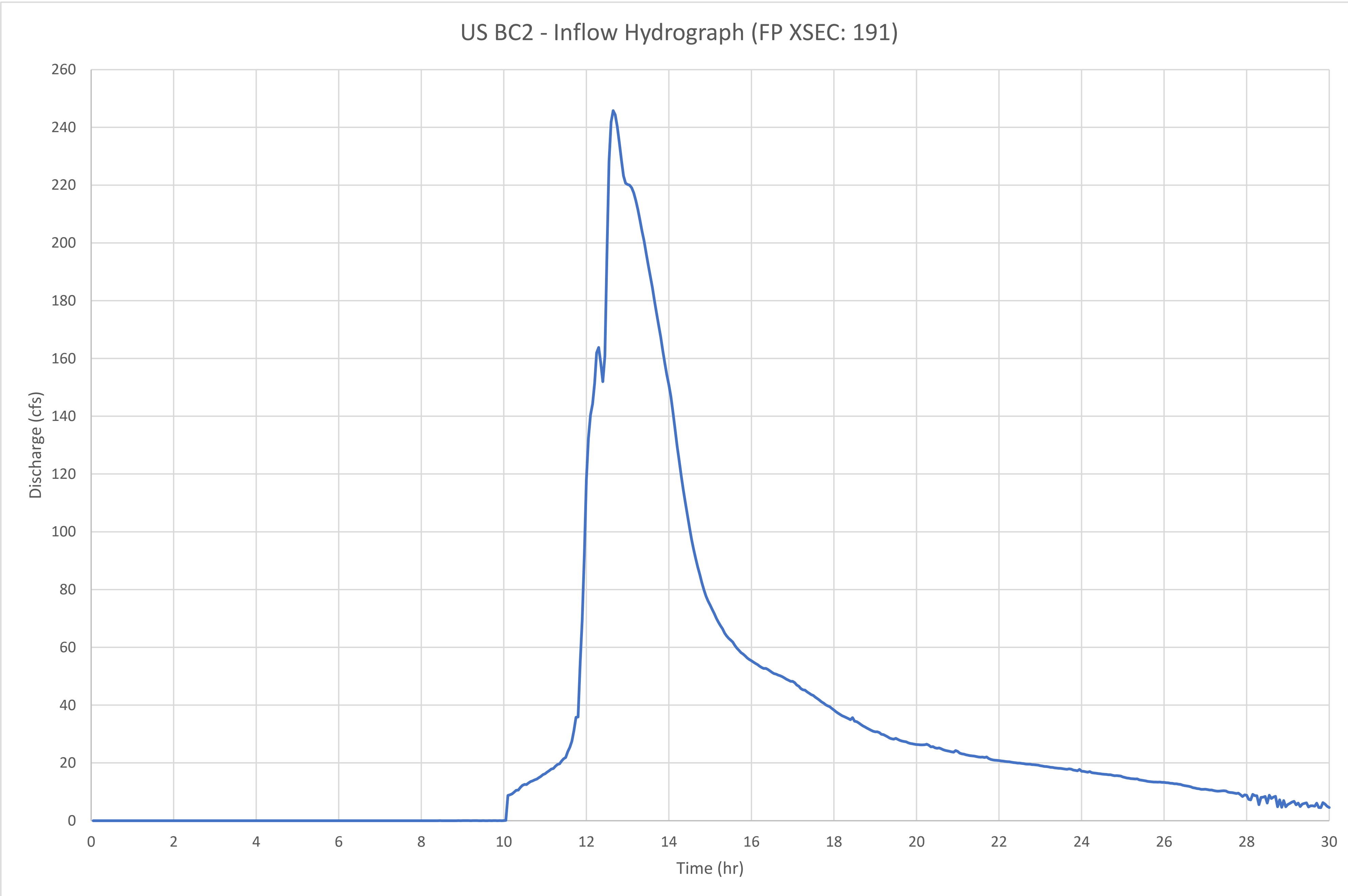


10-UP-2020
09/15/20



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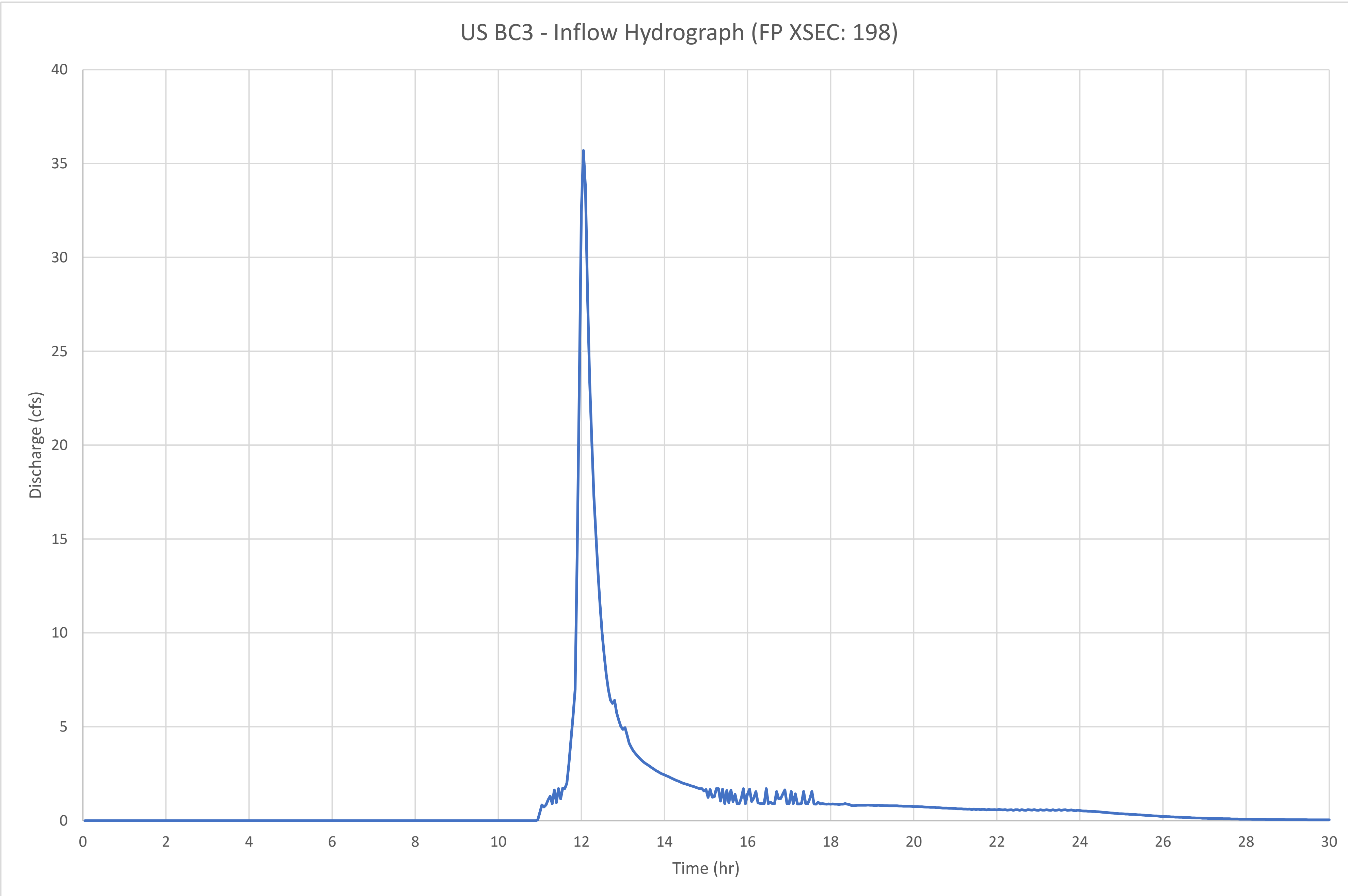
09/15/20



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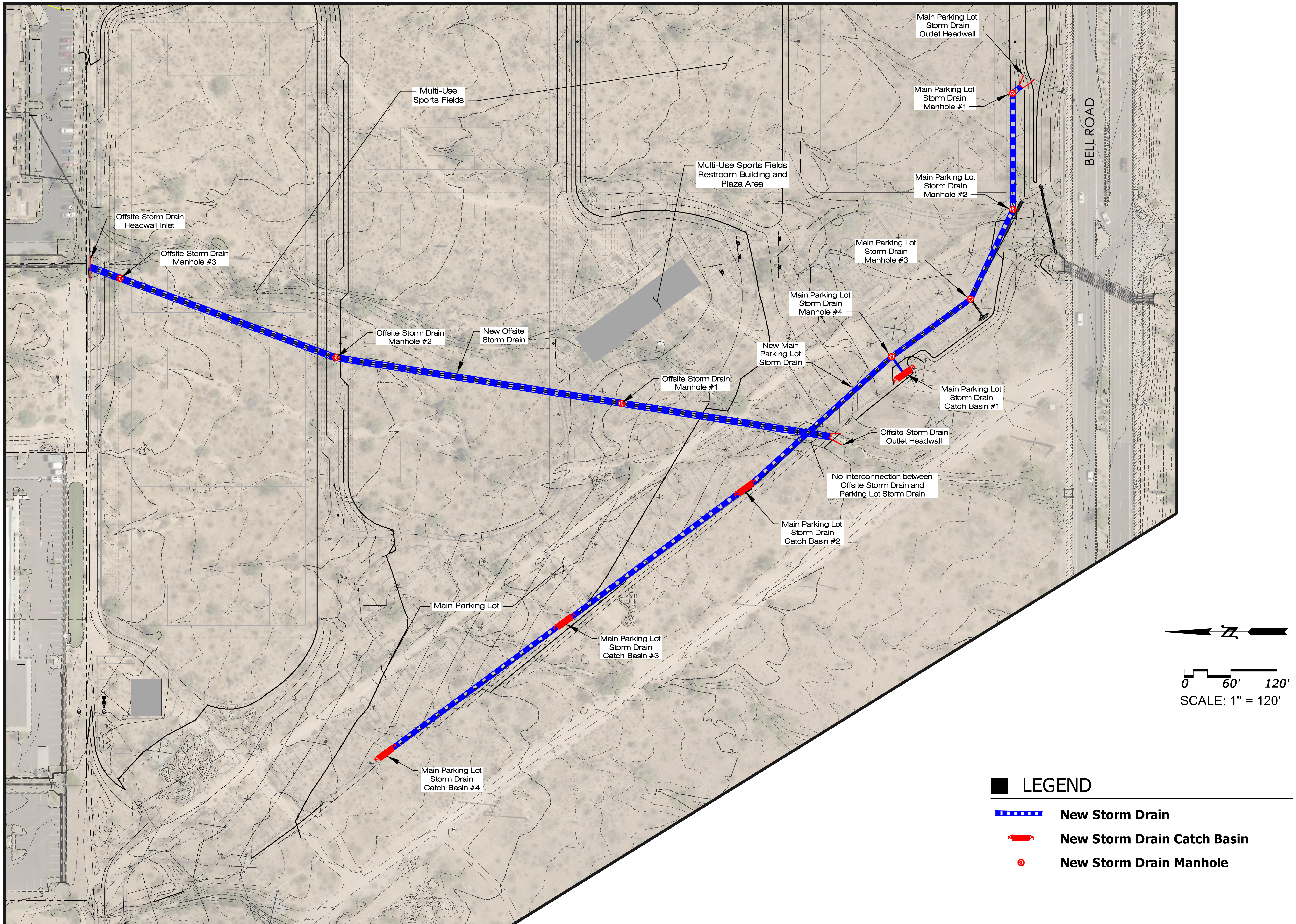
Appendix C: Storm Drain Design Calculations

Storm Drain Design Location Map



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Multi-Use Sports Fields NWC Bell Rd & 94th St

CITY OF SCOTTSDALE
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Sheet Title :

Storm Drain Design Location Map

Sheet Number:

1

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09/15/20

Offsite Storm Drain Design Calculations

Offsite Storm Drain Hydraulic Grade Line (HGL) Summary Table

Location	Type of Headloss	Headloss *(ft)*	HGL Elevation *(ft)*
Headwall Inlet	Junction Loss (Entrance Headloss)^	0.11	1587.70
Offsite MH#3 to Headwall Inlet	Storm Drain Friction Headloss*	0.14	1585.88
Offsite MH#3	Junction Loss (Straight-Through Headloss)^	0.03	1582.40
Offsite MH#2 to Offsite MH#3	Storm Drain Friction Headloss*	1.19	1580.43
Offsite MH#2	Junction Loss (Bend Headloss)	0.04	1575.62
Offsite MH#1 to Offsite MH#2	Storm Drain Friction Headloss	1.46	1575.58
Offsite MH#1	Junction Loss (Straight-Through Headloss)	0.03	1574.12
Outlet HW to Offsite MH#1	Storm Drain Friction Headloss	1.04	1574.09
Outlet Headwall	Junction Loss (Exit Headloss)	0.55	1573.05
		Tailwater Elevation @ Outlet Headwall =	1572.50

* The pipe segments between Offsite MH#2 and the Headwall Inlet are in open channel flow conditions with the hydraulic grade line governed by the 36" Offsite Storm Drain normal depth capacity. Refer to the *Channel Reports* at the end of these calculations for the proposed storm drain normal depth analysis.

^ The hydraulic grade line elevation at Offsite MH#3 and at the Headwall Inlet are governed by the inlet control interception capacity of the 36" Offsite Storm Drain. Refer to the Inlet Control Nomograph calculation at the end of these calculations for the proposed storm drain interception capacity calculation.

Offsite Storm Drain - Hydraulic Grade Line Calculation

The procedures outlined in Chapter 4 of the Hydraulics Drainage Design Manual for Maricopa County were used in order to compute the Hydraulic Grade Line (HGL) for the Offsite Storm Drain. The equations and figures used henceforth were also taken from the manual.

The new offsite storm drain was designed to intercept the 100-year 6-hour peak discharge of 42 cfs from the ODA2 HEC-1 sub-basin area. The ODA2 sub-basin concentrates at the northern boundary of the project site. Under existing conditions, the flow is conveyed in a natural wash through the project area in the southwesterly direction. Under proposed conditions, the offsite storm drain is designed to intercept the 100-year peak discharge and convey it underneath the new multi-use fields, plaza area and parking lot. The offsite storm drain is discharged in the same location where the existing wash leaves the project site. No additional flows are added to the offsite storm drain from the onsite watershed area. Refer to Appendix B for the HEC-1 Hydrologic Model Results.

Determine Tailwater Elevation:

Since the offiste storm drian discharges into a natural wash, with no backwater effects, the starting tailwater elevation will not be impacted by the hydraulics of the natural wash. Therefore, the soffit elevation of the proposed 36-inch offsite strom drain at the outlet headwall is the starting tailwater elevation. The invert of the 36-inch offsite storm drain is 1569.50 ft, which translates to a starting tailwater elevaton of 1572.50 ft.

Tailwater Elevation @ *Outlet Headwall* = 1572.50 ft

Compute the Storm Drain Outlet Headloss at Outlet Headwall

Exit Loss:

$$h_o = 1.0\frac{V^2}{2g} \quad (Equation\ 4.16)$$

where;

$h_o = Outlet\ Headloss\ at\ Manhole$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42.0}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_o = 1.0\frac{V^2}{2g}$$

$$h_o = 1.0\frac{5.94^2}{2\times 32.2}$$

$$h_o = 0.55\,ft$$

$$\boxed{\mathbf{h_o = 0.55}\,\boldsymbol{ft}\ @\ \textbf{Outlet}\ \textbf{\textit{Headwall}}}$$

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Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Outlet Headwall to Offsite MH#1)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 267\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$h_f = S_f L$

$h_f = 0.0039 \times 267$

$h_f = 1.04\ ft$

$$\boxed{\mathbf{h_f = 1.04\ ft\ (Outlet\ Headwall\ to\ Offsite\ MH\#1)}}$$

Compute the Headloss at Manhole (Offsite MH #1)

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{5.94^2}{2 \times 32.2}$$

$$h_{mh} = 0.03\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.03\ ft\ @\ Offsite\ MH\ \#1}}$$

Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Offsite MH#1 to Offsite MH#2)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 375\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$

$$h_f = 0.0039 \times 375$$

$$h_f = 1.46\ ft$$

$$\boxed{\mathbf{h_f = 1.46\ ft\ \ (Offsite\ MH\#1\ to\ Offsite\ MH\#2)}}$$

Gavan & Barker Inc.

Compute the Bend Headloss at Manhole (Offsite MH#2)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 11\ °$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.08 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.08\ \frac{5.94^2}{2 \times 32.2}$$

$$h_{mh} = 0.04\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.04\ ft\ @\ Offsite\ MH\#2}}$$

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Gavan & Barker Inc.

Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Offsite MH#2 to Offsite MH#3)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$$L = 304\ ft$$

$Q = Storm\ Drain\ Design\ Discharge$

$$Q = 42\ cfs$$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$$D = 3.0\ ft$$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$

$$h_f = 0.0039 \times 304$$

$$h_f = 1.19\ ft$$

$$\boxed{\mathbf{h_f = 1.19\ ft \quad (Offsite\ MH\#2\ to\ Offsite\ MH\#3)}}$$

Compute the Headloss at Manhole (Offsite MH #3)

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{5.94^2}{2 \times 32.2}$$

$$h_{mh} = 0.03\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.03\ ft\ @\ Offsite\ MH\ \#3}}$$

Gavan & Barker Inc.

Compute the Friction Headloss - Proposed 36" Offsite Storm Drain (Offsite MH#3 to Headwall Inlet)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 35\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0039$$

$$h_f = S_f L$$

$$h_f = 0.0039 \times 35$$

$$h_f = 0.14\ ft$$

$$\boxed{\mathbf{h_f = 0.14\ ft\ \ (Offsite\ MH\#3\ to\ Headwall\ Inlet)}}$$

Gavan & Barker Inc.

Compute the Entrance Headloss at Headwall (Headwall Inlet)

Entrance Loss:

$$h_i = k_{en}\frac{V^2}{2g} \quad (Equation\ 4.15)$$

where;

$h_i = Headloss\ at\ Pipe\ Entrance$

$k_{en} = Entrance\ Loss\ Coefficient$

$k_{en} = 0.20 \quad (Table\ 5.1)$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 5.94\ \frac{ft}{s}$$

$$h_i = k_{en}\frac{V^2}{2g}$$

$$h_i = 0.20\ \frac{5.94^2}{2 \times 32.2}$$

$$h_i = 0.11\ ft$$

$$\boxed{\mathbf{h_i = 0.11}\ \boldsymbol{ft}\ @\ \mathbf{Headwall}\ \boldsymbol{Inlet}}$$

Channel Report

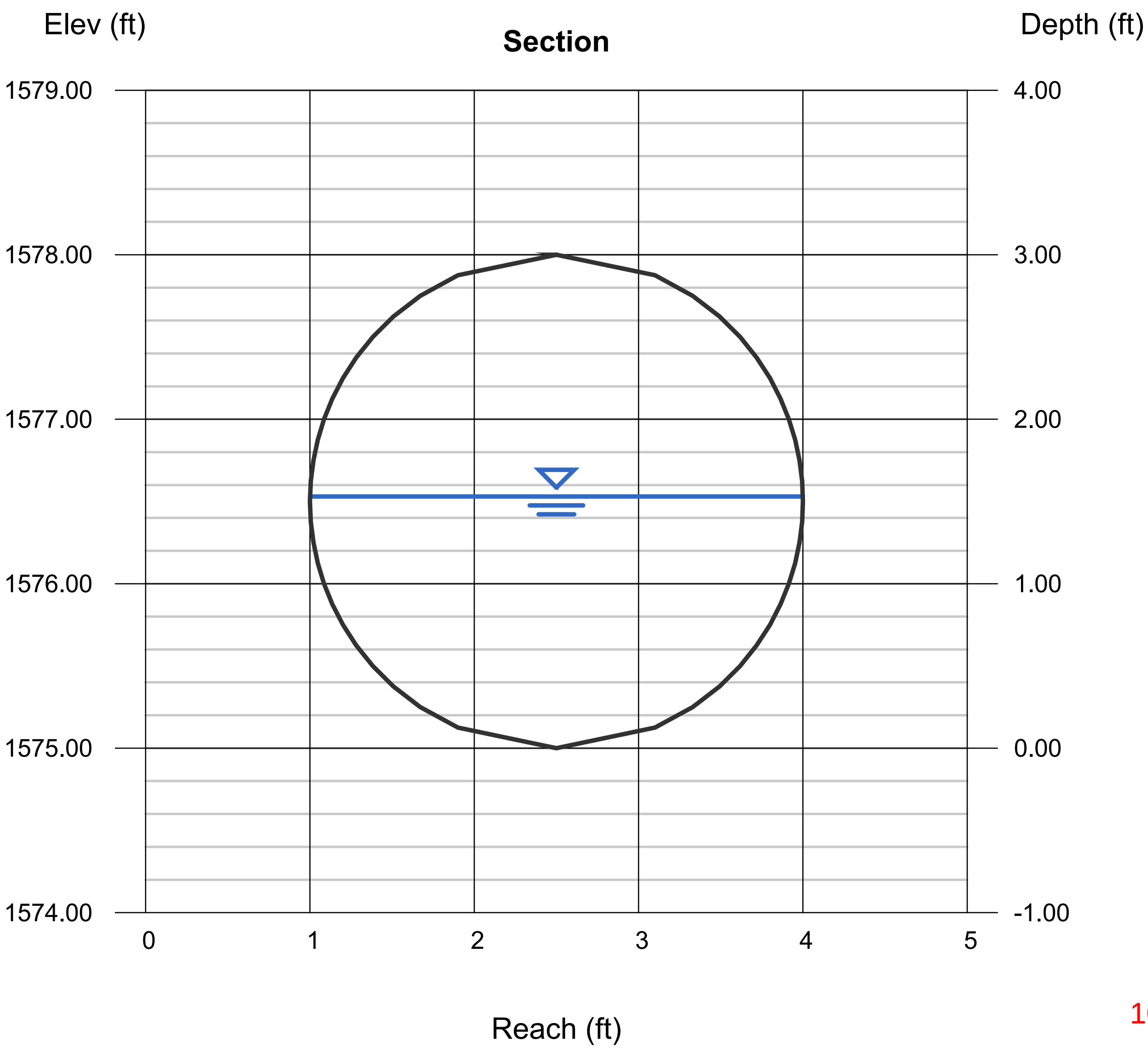
36" Offsite Storm Drain Normal Depth Analysis (Offsite MH#2 to Offsite MH#3)

Circular
Diameter (ft) = 3.00

Invert Elev (ft) = 1575.00
Slope (%) = 1.50
N-Value = 0.013

Calculations
Compute by: Known Q
Known Q (cfs) = 42.00

Highlighted
Depth (ft) = 1.53
Q (cfs) = 42.00
Area (sqft) = 3.64
Velocity (ft/s) = 11.53
Wetted Perim (ft) = 4.78
Crit Depth, Yc (ft) = 2.12
Top Width (ft) = 3.00
EGL (ft) = 3.60



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Channel Report

36" Offsite Storm Drain Normal Depth Analysis (Offsite MH#3 to Headwall Inlet)

Circular	
Diameter (ft)	= 3.00
Invert Elev (ft)	= 1585.00
Slope (%)	= 1.10
N-Value	= 0.013
Calculations	
Compute by:	Known Q
Known Q (cfs)	= 42.00

Highlighted	
Depth (ft)	= 1.68
Q (cfs)	= 42.00
Area (sqft)	= 4.09
Velocity (ft/s)	= 10.27
Wetted Perim (ft)	= 5.08
Crit Depth, Yc (ft)	= 2.12
Top Width (ft)	= 2.98
EGL (ft)	= 3.32

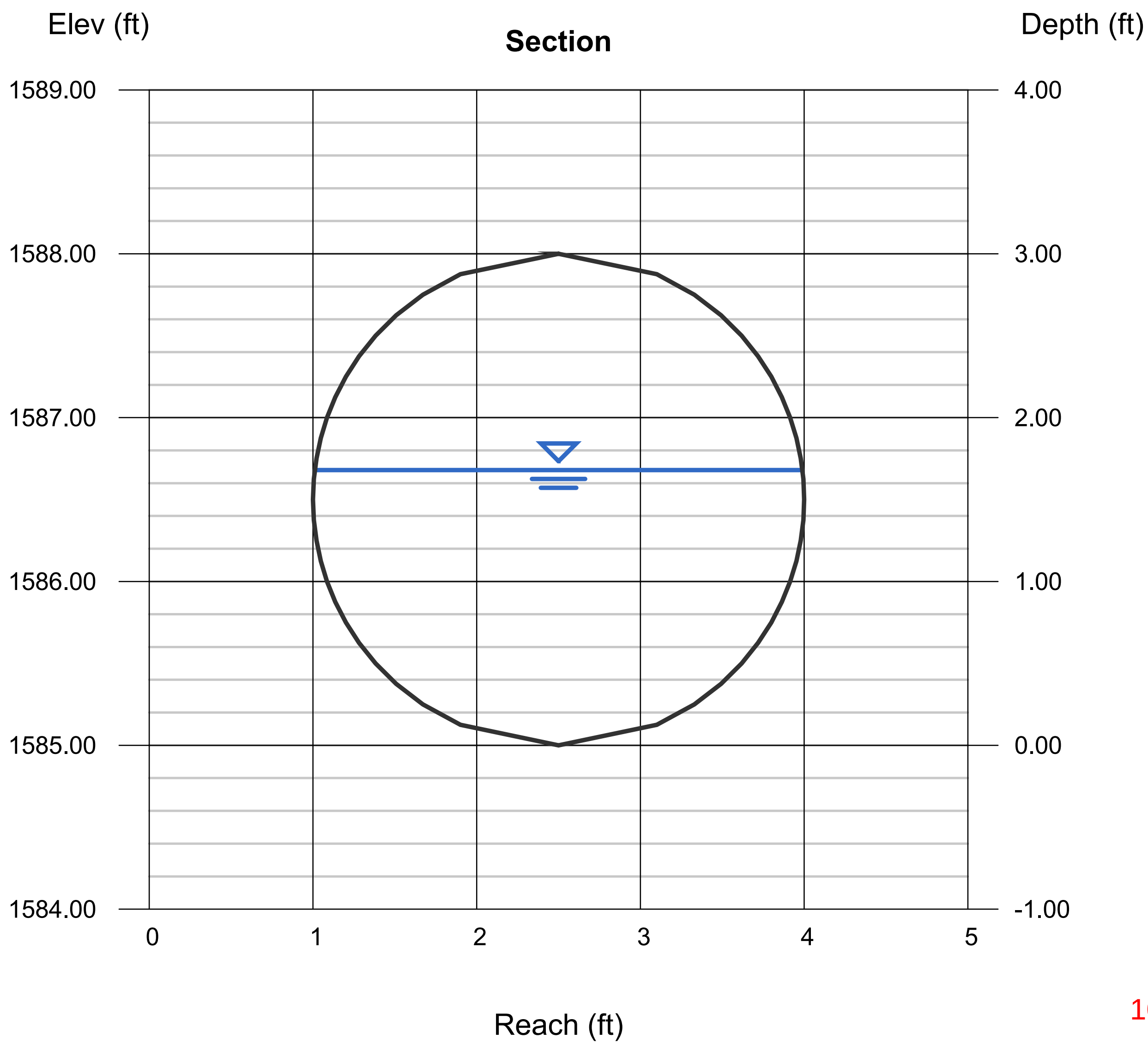

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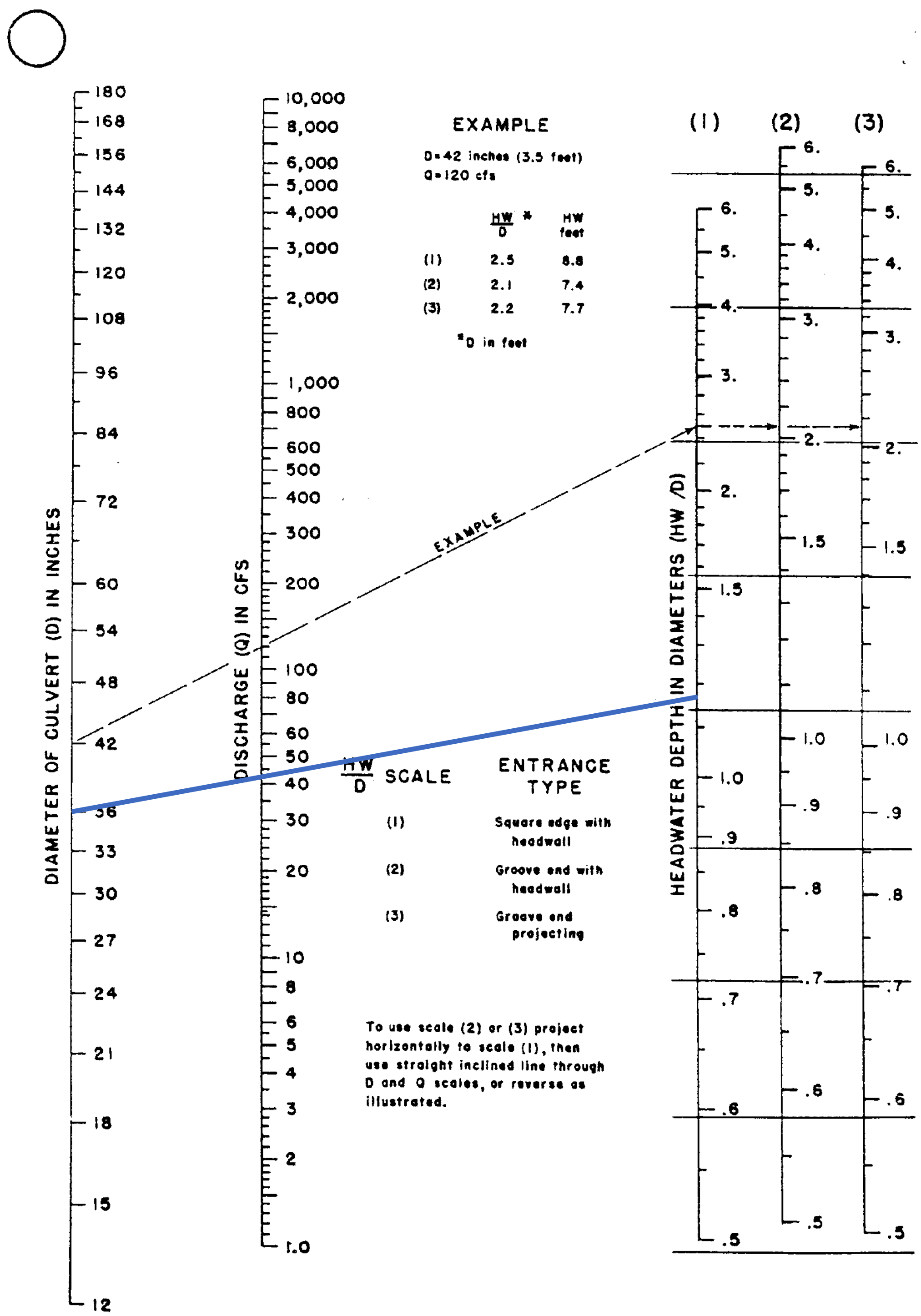
FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)



36-in Storm Drain
Q=42 cfs
D=3.0 ft
HW/D=1.16
Hence;
HW=3.5 ft

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Main Parking Lot Storm Drain Design Calculations

Main Parking Lot Storm Drain Hydraulic Grade Line (HGL) Summary Table

Location	Type of Headloss	Headloss (ft)	HGL Elevation (ft)
CB#4	Junction Loss (Entrance Headloss)^	0.04	1576.00
CB#3 to CB#4	Storm Drain Friction Headloss*	0.92	1575.29
CB#3	Junction Loss (Straight-Through Catch Basin)	0.01	1573.61
CB#2 to CB#3	Storm Drain Friction Headloss	3.03	1573.60
CB#2	Junction Loss (Straight-Through Catch Basin)	0.03	1570.57
MH#4 to CB#2	Storm Drain Friction Headloss	1.74	1570.54
MH#4	Junction Loss (Combined Junction Loss)	0.22	1568.80
MH#3 to MH#4	Storm Drain Friction Headloss	0.18	1568.58
MH#3	Junction Loss (Bend Headloss)	0.02	1568.40
MH#2 to MH#3	Storm Drain Friction Headloss	0.18	1568.38
MH#2	Junction Loss (Bend Headloss)	0.02	1568.20
MH#1 to MH#2	Storm Drain Friction Headloss	0.19	1568.18
MH#1	Junction Loss (Bend Headloss)	0.05	1567.99
Outlet Headwall to MH#1	Storm Drain Friction Headloss	0.02	1567.94
Outlet Headwall	Junction Loss (Exit Headloss)	0.19	1567.92
		Tailwater Elevation @ Outlet Headwall =	1567.73

* The pipe segment between CB#3 and CB#4 is in open channel flow conditions with the hydraulic grade line governed by the proposed 18" Storm Drain normal depth capacity. Refer to the *Channel Reports* at the end of these calculations for the proposed storm drain normal depth analysis.

^ The hydraulic grade line elevation at Catch Basin #4 is governed by the inlet control interception capacity of the 18" Storm Drain. Refer to the Inlet Control Nomograph calculation at the end of these calculations for the proposed storm drain interception capacity calculation.

Storm Drain - Hydraulic Grade Line Calculation

The procedures outlined in Chapter 4 of the Hydraulics Drainage Design Manual for Maricopa County were used in order to compute the Hydraulic Grade Line (HGL) for the Offsite Storm Drain. The equations and figures used henceforth were also taken from the manual.

The new storm drain was designed to intercept the 100-year 6-hour peak discharge from the main parking lot and plaza areas and convey it to the proposed detention basin north of Bell Road, just east of the entrance drive. The main parking lot was graded to drain to 4 shallow lot spots where new combination curb opening and grate catch basins will be designed to intercept peak discharge from the upstream contributing drainage area. The proposed storm drain is designed to convey the 100-year, 6-hour peak discharge of 6 cfs from the most upstream Catch Basin #1 (HEC-1 Sub-basin ID: NDA1). Further downstream, the storm drain is designed to convey a combined 11 cfs from the Catch Basin #2 (HEC-1 Combine: CSD1), a combined 19 cfs from Catch Basin #3 (HEC-1 Combine: CSD2) and finally 25 cfs from Manhole #4 (HEC-1 Combine: CSD3). No offsite flows are added to the main parking lot storm drain. Refer to Appendix B for the HEC-1 Hydrologic Model Results.

Determine Tailwater Elevation:

The new storm drain discharges into the proposed new detention basin. The starting tailwater elevation for the design of the strom drain was taken as the stage in the new detention basin at the time the 100-year, 6-hour peak discharge enters the detention basin through the storm drain. From the Proposed Conditions HEC-1 Model it was found that at a time of 4:02 hr a peak flow of 25 cfs enters the basin with the water level in the detention basin at 1567.73 ft. Therefore, the starting tailwater elevation for the proposed storm drain is 1567.73 ft.

$$\boxed{\begin{array}{c}\textbf{Tailwater Elevation @ } \boldsymbol{Outlet\ Headwall} = \textbf{1567.73 ft} \\ \left(Water\ Level\ in\ Detention\ Basin\ @\ Peak\ Inflow\right)\end{array}}$$

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Compute the Storm Drain Outlet Headloss at Outlet Headwall

Exit Loss:

$$h_o = 1.0\frac{V^2}{2g} \quad (Equation\ 4.16)$$

where;

$h_o = Outlet\ Headloss\ at\ Manhole$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25.0}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_o = 1.0\frac{V^2}{2g}$$

$$h_o = 1.0\frac{3.54^2}{2 \times 32.2}$$

$$h_o = 0.19\ ft$$

$$\boxed{\mathbf{h_o = 0.19\ \mathit{ft}\ @\ Outlet\ \mathit{Headwall}}}$$

Gavan & Barker Inc.

Compute the Friction Headloss - Proposed 36" Storm Drain (Outlet Headwall to MH#1)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 16\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{3.54^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$$h_f = S_f L$$

$$h_f = 0.0014 \times 16$$

$$h_f = 0.02\ ft$$

$$\boxed{\mathbf{h_f = 0.02\ ft\ \ (Outlet\ Headwall\ to\ MH\#1)}}$$

Gavan & Barker Inc.

Compute the Bend Headloss at Manhole (MH#1)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 39\,°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.24 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.24\ \frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.05\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.05\ ft\ @\ MH\ \#1}}$$

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Compute the Friction Headloss - Proposed 36" Storm Drain (MH#1 to MH#2)

$h_f = S_f L \quad (Equation\ 4.6)$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 139\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25.0\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{3.54^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$h_f = S_f L$

$h_f = 0.0014 \times 139$

$h_f = 0.19\ ft$

$$\boxed{\mathbf{h_f = 0.19\ ft\ \ (MH\#1\ to\ MH\#2)}}$$

Compute the Bend Headloss at Manhole (MH#2)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 25°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.12 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.12\ \frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.02\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.02\ ft\ @\ MH\ \#2}}$$

Compute the Friction Headloss - Proposed 36" Storm Drain (MH#2 to MH#3)

$h_f = S_f L \quad (Equation\ 4.6)$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 127\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{42}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{5.94^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$h_f = S_f L$

$h_f = 0.0014 \times 127$

$h_f = 0.18\ ft$

$\boxed{\mathbf{h_f = 0.18\ ft\ \ (MH\#2\ to\ MH\#3)}}$

Compute the Bend Headloss at Manhole (MH#3)

Bend Loss (no laterals):

$$h_{mh} = k_b \frac{V^2}{2g} \quad (Equation\ 4.12)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$\gamma = Deflection\ Angle$

$\gamma = 28\,°$

$k_b = Bend\ loss\ coefficient$

$k_b = 0.12 \quad (Figure\ 4.10)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = k_b \frac{V^2}{2g}$$

$$h_{mh} = 0.12\ \frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.02\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.02\ ft\ @\ MH\ \#3}}$$

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Compute the Friction Headloss - Proposed 36" Storm Drain (MH#3 to MH#4)

$h_f = S_f L \quad (Equation\ 4.6)$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 126\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{3.0}{4} = 0.750\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{3.54^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0014$$

$h_f = S_f L$

$h_f = 0.0014 \times 126$

$h_f = 0.18\ ft$

$$\boxed{\mathbf{h_f = 0.18\ ft\ \ (MH\#3\ to\ MH\#4)}}$$

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Compute the Combined Headloss at Manhole (MH#4)

At this junction, compute the headloss associated with the Straight-Through Manhole and the lateral inflow at the manhole. The combined headloss is the total headloss at this Manhole (MH#4).

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ due\ to\ bend\ at\ manhole$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 25\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{25}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.54\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{3.54^2}{2 \times 32.2}$$

$$h_{mh} = 0.01\ ft$$

Junction Loss (Lateral Inflow):

$$h_j = \frac{2\left(Q_2 V_2 - Q_1 V_1 - Q_3 V_3 \cos\theta\right)}{\left(A_1 + A_2\right) g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad (Equation\ 4.10b)$$

where;

$h_j = Headloss\ at\ Junction\ with\ lateral\ inflow$

$A_1 = Upstream\ Flow\ Area$

$$A_1 = \pi\frac{d^2}{4} = \pi\frac{2.0^2}{4} = 3.14\ ft^2$$

$A_2 = Downstream\ Flow\ Area$

$$A_2 = \pi\frac{d^2}{4} = \pi\frac{3.0^2}{4} = 7.07\ ft^2$$

$Q_1 = Upstream\ Flow\ Rate$

$Q_1 = 19\ cfs$

$Q_2 = Downstream\ Flow\ Rate$

$Q_2 = 25\ cfs$

$Q_3 = Q_2 - Q_1 = Lateral\ Flow\ Rate$

$Q_3 = 6\ cfs$

NOTE: The lateral flow rate represents the flow rate in the 18-inch Catch Basin #1 connector pipe.

$V_1 = Upstream\ Flow\ Velocity$

$$V_1 = \frac{Q_1}{A_1} = \frac{19}{3.14} = 6.05\ \frac{ft}{s}$$

$V_2 = Downstream\ Flow\ Velocity$

$$V_2 = \frac{Q_2}{A_2} = \frac{25}{7.07} = 3.54\ \frac{ft}{s}$$

$V_3 = Lateral\ Flow\ Velocity$

$$V_3 = \frac{Q_3}{A_3} = \frac{6}{\pi \times \left(\frac{1.50^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$\theta = Angle\ between\ lateral\ and\ main\ storm\ drain\ line$

$\theta = 90\,^\circ\ \left(Figure\ 4.7\right)$

$$h_j = \frac{2\left(25 \times 3.54 - 19 \times 6.05 - 6 \times 3.40 \times \cos(90)\right)}{(3.14 + 7.07)\,32.2} + \frac{6.05^2}{2 \times 32.2} - \frac{3.54^2}{2 \times 32.2}$$

$h_j = 0.21\ ft$

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Total Combined Headloss at Manhole (MH#4)

$$h_{mh_{TOTAL}} = h_{mh} + h_j$$

$$h_{mh_{TOTAL}} = 0.01 + 0.21$$

$$h_{mh_{TOTAL}} = 0.22\,ft$$

$$\boxed{\mathbf{h_{mh} = 0.22\,\mathit{ft}\ @\ MH\#4}}$$

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Compute the Friction Headloss - Proposed 24" Storm Drain (MH#4 to CB#2)

$h_f = S_f L \quad (Equation\ 4.6)$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 249\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 19\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 2.0\ ft$

$S_f = Friction\ Slope$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{19}{\pi \times \left(\frac{2.0^2}{4}\right)} = 6.05\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{2.0}{4} = 0.500\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\frac{6.05^2}{2 \times 32.2 \times 0.500^{\frac{4}{3}}} = 0.0070$$

$h_f = S_f L$

$h_f = 0.0070 \times 248$

$h_f = 1.74\ ft$

$$\boxed{\mathbf{h_f = 1.74\ ft\ \ (MH\#4\ to\ CB\#2)}}$$

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Compute the Headloss at Catch Basin (CB #2)

Straight-Through Catch Basin Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ at\ catch\ basin$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 11\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{11}{\pi \times \left(\frac{1.5^2}{4}\right)} = 6.22\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{6.22^2}{2 \times 32.2}$$

$$h_{mh} = 0.03\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.03\ ft\ @\ CB\#2}}$$

Compute the Friction Headloss - Proposed 18" Storm Drain (CB#2 to CB#3)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 278\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 11\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{11}{\pi \times \left(\frac{1.5^2}{4}\right)} = 6.22\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{1.5}{4} = 0.375\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{6.22^2}{2 \times 32.2 \times 0.375^{\frac{4}{3}}} = 0.0109$$

$$h_f = S_f L$$

$$h_f = 0.0109 \times 278$$

$$h_f = 3.03\ ft$$

$$\boxed{\mathbf{h_f = 3.03\ ft\ \ (CB\#2\ to\ CB\#3)}}$$

Compute the Headloss at Catch Basin (CB #3)

Straight-Through Catch Basin Loss:

$$h_{mh} = 0.05\frac{V^2}{2g} \quad (Equation\ 4.11)$$

where;

$h_{mh} = Headloss\ at\ catch\ basin$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge$

$Q = 6\ cfs$

$D = Upstream\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{6}{\pi \times \left(\frac{1.5^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$$h_{mh} = 0.05\frac{V^2}{2g}$$

$$h_{mh} = 0.05\frac{3.40^2}{2 \times 32.2}$$

$$h_{mh} = 0.01\ ft$$

$$\boxed{\mathbf{h_{mh} = 0.01\ ft\ @\ CB\#3}}$$

Compute the Friction Headloss - Proposed 18" Storm Drain (CB#3 to CB#4)

$$h_f = S_f L \quad (Equation\ 4.6)$$

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 278\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 6\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter$

$D = 1.5\ ft$

$S_f = Friction\ Slope$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} \quad (Equation\ 4.4)$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{6}{\pi \times \left(\frac{1.5^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

$$n = 0.013 \quad (Table\ 4.1 - Smooth\ Plastic\ Pipe)$$

$K = Pipe\ Roughness\ Coefficient$

$$K = \frac{2gn^2}{2.21} = \frac{2 \times 32.2 \times 0.013^2}{2.21} = 0.0049$$

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{1.5}{4} = 0.375\ ft$$

$$S_f = K \frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049 \frac{3.40^2}{2 \times 32.2 \times 0.375^{\frac{4}{3}}} = 0.0033$$

$$h_f = S_f L$$

$$h_f = 0.0033 \times 278$$

$$h_f = 0.92\ ft$$

$$\boxed{\mathbf{h_f = 0.92\ ft\ \ (CB\#3\ to\ CB\#4)}}$$

Gavan & Barker Inc.

Compute the Entrance Headloss at Catch Basin (CB#4)

Entrance Loss:

$$h_i = k_{en}\frac{V^2}{2g} \quad (Equation\ 4.15)$$

where;

$h_i = Headloss\ at\ Pipe\ Entrance$

$k_{en} = Entrance\ Loss\ Coefficient$

$k_{en} = 0.20 \quad (Table\ 5.1)$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{6}{\pi \times \left(\frac{1.5^2}{4}\right)} = 3.40\ \frac{ft}{s}$$

$$h_i = k_{en}\frac{V^2}{2g}$$

$$h_i = 0.20\,\frac{3.40^2}{2 \times 32.2}$$

$$h_i = 0.04\ ft$$

$$\boxed{\mathbf{h}_i = \mathbf{0.04}\ \boldsymbol{ft}\ @\ \mathbf{CB\#4}}$$

Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Monday, Sep 7 2020

18" Storm Drain Normal Depth Analysis (CB#3 to CB#4)

Circular		**Highlighted**	
Diameter (ft)	= 1.50	Depth (ft)	= 0.79
		Q (cfs)	= 6.000
		Area (sqft)	= 0.94
Invert Elev (ft)	= 1575.00	Velocity (ft/s)	= 6.35
Slope (%)	= 1.10	Wetted Perim (ft)	= 2.44
N-Value	= 0.013	Crit Depth, Yc (ft)	= 0.95
		Top Width (ft)	= 1.50
Calculations		EGL (ft)	= 1.42
Compute by:	Known Q		
Known Q (cfs)	= 6.00		

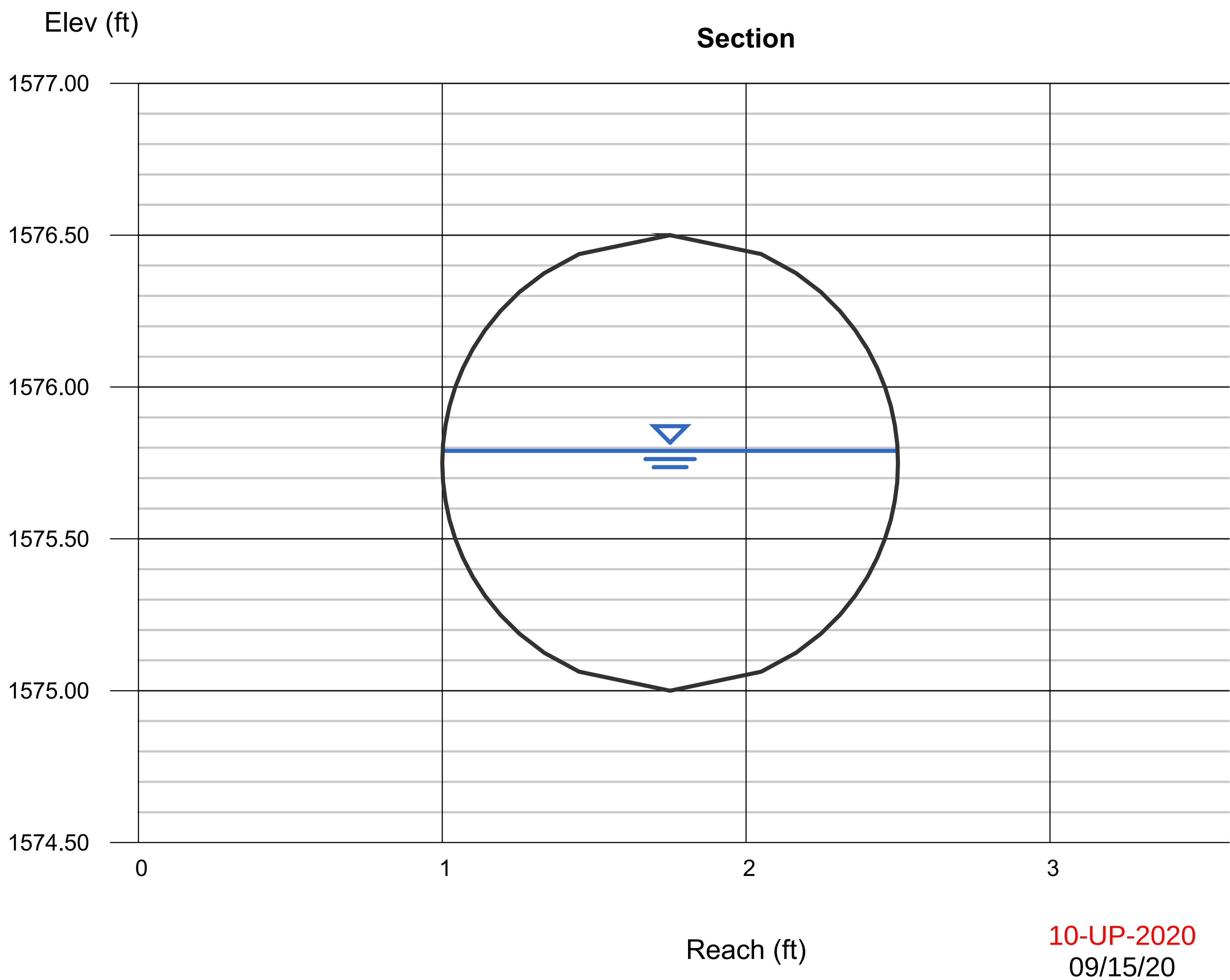

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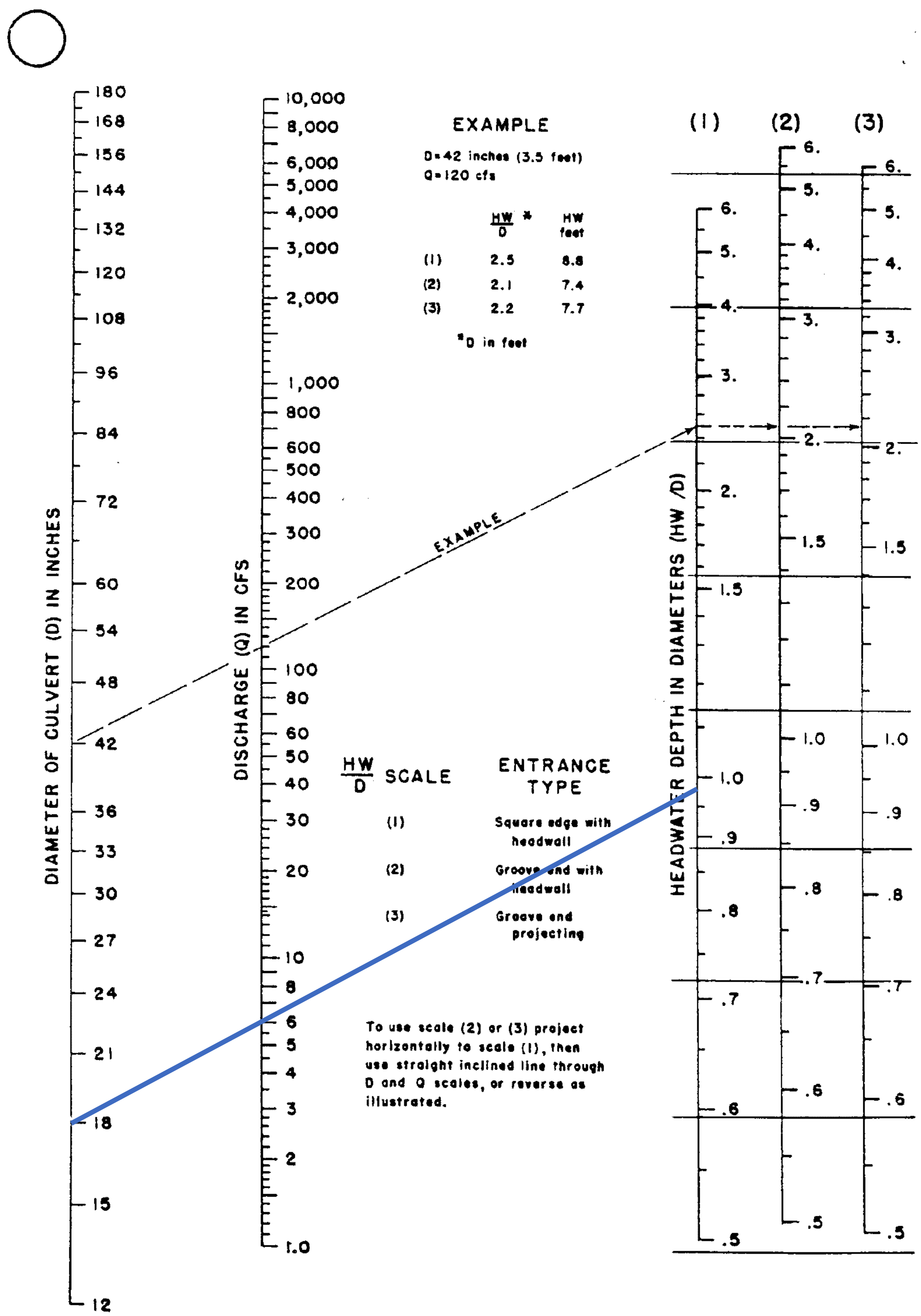
FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)



18-in Storm Drain
Q=6 cfs
D=1.5 ft
HW/D=0.98
Hence;
HW=1.5 ft

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Main Parking Lot Storm Drain Inlet Sizing Calculations

Main Parking Lot – Catch Basin Design Calculations

The main parking at the Multi-Use Fields NWC of Bell Road & 94th Street sports complex is a ¼ mile long, linear parking lot located underneath the powerline corridor. The parking long is graded to drain to four shallow sumps that are drained by four catch basins connected to the proposed parking lot storm drain. The storm drain is discharged to the new detention basin located north of Bell Road, east of the entrance drive. The four sumps and corresponding catch basins are located approximately equidistant to each other, each designed to intercept the 100-year, 6-hour peak discharge from the parking lot and adjacent plaza area.

The four proposed inlets are located along the western edge of the main parking lot and are identified as Catch Basin #1 (CB#1) as the southernmost inlet, with Catch Basin #4 (CB#4) being the northernmost inlet. The remaining two inlets, Catch Basin #2 (CB#2) and Catch Basin #3 (CB#3) located in the middle of the parking lot. The 100-year, 6-hour design peak discharges and the corresponding HEC-1 Sub-Basin IDs for each inlet are as follows:

- Catch Basin #1 (CB#1) – Q_{100}=6 cfs (NDA4)
- Catch Basin #2 (CB#2) – Q_{100}=8 cfs (NDA3)
- Catch Basin #3 (CB#3) – Q_{100}=5 cfs (NDA2)
- Catch Basin #4 (CB#4) – Q_{100}=6 cfs (NDA1)

Refer to the Storm Drain Design Location Map at the beginning of these calculations for the exact location of the proposed catch basins as well as Appendix B for the HEC-1 Hydrologic Model.

It is recommended to install a City of Phoenix Type “Q” (Triple) combination catch basin (Std. Det. No P1572) with a total curb opening and grate length of 10 feet at each of the four sump locations. As can be seen in the following catch basin design calculations, the proposed catch basins have the capacity to intercept the entire 100-year, 6-hour design peak discharge. With the proposed catch basins located on the west curb line of the proposed parking lot, the additional benefit of using the Type “Q” catch basins is that it has the maintenance basin underneath the gutter, which will not protrude behind the back of curb into the fill slope

Gavan & Barker Inc.

Catch Basin Design @ CB#1

Determine if Catch Basin operates as a Weir or as an Orifice:

$$d \le h \qquad \Rightarrow \textit{Weir Flow}$$
$$h \ge d \ge 1.4h \qquad \Rightarrow \textit{Transitional Flow}$$
$$d \ge 1.4h \qquad \Rightarrow \textit{Orifice Flow}$$

where;

$d = \textit{Depth of Sump at Proposed Catch Basin Inlet} = \textit{Spill Elev} - \textit{Weir Elev}$

$\textit{Spill Elev} = \textit{Low Point in the Curb Elevation at Catch Basin} = 1570.10\ ft$

$\textit{Weir Elev} = \textit{Catch Basin Weir Elevation} = \textit{Top of Curb Elevation at Catch Basin} - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$\textit{Top of Curb Elevation at Catch Basin} = 1570.10\ ft$

$\textit{Weir Elev} = 1570.10\ ft - 4.75\ in = 1569.70\ ft$

$d = \textit{Spill Elev} - \textit{Lip Elev}$

$d = 1570.10 - 1569.70$

$\left[d = 0.40\ ft\right]$

$h = \textit{Height of Curb Opening portion of the Catch Basin}$

$h = 5\ in$

$\left[h = 0.42\ ft\right]$

$d < h$

$0.40\ ft < 0.42\ ft$

$\boxed{\textbf{Weir Flow}}$

Gavan & Barker Inc.

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w \left(L_f + 1.8W\right) d^{1.5} \quad \text{(Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics)}$$

where;

$Q_i = Combination\ Basin\ Flow\ Interception\ Capacity$

$C_w = Weir\ Flow\ Coefficient = 2.3$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin = 0.40$

$L = Length\ of\ Proposed\ Combination\ Catch\ Basin = 10.0\ ft$

$L_f = Effective\ Length\ of\ Catch\ Basin = C_f \times L$

$C_f = Clogging\ Factor = 0.80$ (Table 6.8 - FCDMC Drainage Policies and Standards Manual)

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$[L_f = 8.0\ ft]$

$W = Width\ of\ Depressed\ Gutter = 4.3\ ft\ (Modified\ per\ COP\ Std\ Det\ P1572)$

$$Q_i = C_w \left(L_f + 1.8W\right) d^{1.5}$$

$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.40^{1.5}$$

$$Q_i = 9.16$$

$$\boxed{\boldsymbol{Q_i = 9\ cfs}}$$

The proposed City of Phoenix Type "Q" combination catch basin (Std. Det. P1572) at CB #1 has an interception capacity of 9 cfs, which is greater than the 100-year, 6-hour design peak discharge of 6 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Connector Pipe Sizing:

Determine the total (friction + inlet) headloss:

$$h_f = S_f L \quad \text{(Equation 4.6 - Drainage Design Manual for Maricopa County, Hydraulics)}$$

$$h_i = (1 + k_{en})\left(\frac{V^2}{2g}\right)$$

Try a 18 inch (d = 1.50 ft) Pipe:

$$S_f = Pipe\ Friction\ Slope = K\frac{V^2}{2gR^{\frac{4}{3}}} \quad \text{(Equation 4.4 - Design Manual for Maricopa County, Hydraulics)}$$

$$K = Roughness\ Coefficient = \frac{2gn^2}{2.21} \quad \text{(Equation 4.5 - Design Manual for Maricopa County, Hydraulics)}$$

$$n = Manning's\ roughness\ coefficient = 0.013\ \text{(Typ)}$$

$$g = Acceleration\ due\ to\ Gravity = 32.2\ \frac{ft}{s^2}$$

$$V = Velocity\ in\ Pipe = \frac{Q}{A}$$

$$Q = Flow\ Rate\ in\ Pipe = 6\ cfs$$

$$A = Cross - Section\ area\ of\ Pipe = \pi\frac{d^2}{4} = \pi\frac{1.50^2}{4} = 1.77\ ft^2$$

$$R_h = Hydraulic\ Radius = \frac{A}{W_p} = \frac{\pi\frac{d^2}{4}}{\pi d} = \frac{d}{4} = \frac{1.50}{4} = 0.375\ ft$$

$$L = Length\ of\ Pipe = 28\ ft$$

$$k_{en} = Entrance\ Loss\ Coefficient = 0.2\ \text{(Table 5.1 - Design Manual for Maricopa County, Hydraulics)}$$

$$K = \frac{2gn^2}{2.21} = \frac{2(32.2)(0.013)^2}{2.21}$$

$$[K = 0.0049]$$

$$V = \frac{Q}{A} = \frac{6}{1.77}$$

$$\left[V = 3.39\ \frac{ft}{s}\right]$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = (0.0049)\frac{3.39^2}{2(32.2)(0.375)^{\frac{4}{3}}}$$

$$[S_f = 0.0032]$$

Friction Headloss:

$$h_f = S_f L$$

$$h_f = (0.0032)(28)$$

$$[h_f = 0.09\,ft]$$

Inlet and Manhole Headloss:

$$h_i = (1 + k_{en})\left(\frac{V^2}{2g}\right)$$

$$h_i = (1 + 0.2)\left(\frac{3.39^2}{2(32.2)}\right)$$

$$[h_i = 0.21\,ft]$$

Total Headloss:

$$h_t = h_f + h_i$$

$$h_t = 0.09 + 0.21$$

$$h_t = 0.30$$

$$\boxed{h_t = 0.3\,ft}$$

Available Head: h_a

Upstream HW Elevation: 1569.10 *ft* (Six inches below the proposed catch basin gutter elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" below the gutter elevation at the inlet. The gutter elevation of Catch Basin #1 is 1569.60 ft.

Downstream HW Elevation: 1568.80 *ft* (Storm Drain HGL at Connection Manhole (MH#4))

$$h_a = Upstream\ HW - Downstream\ HW$$

$$h_a = 1569.10 - 1568.80$$

$$h_a = 0.30$$

$$\boxed{h_a = 0.3\,ft}$$

The available head is equal to the total catch basin connector pipe headloss, therefore:

The 18-inch connector pipes have sufficient capacity to convey the intercepted flow

Gavan & Barker Inc.

Catch Basin Design @ CB#2

Determine if Catch Basin operates as a Weir or as an Orifice:

$$d \le h \qquad \Rightarrow \text{Weir Flow}$$
$$h \ge d \ge 1.4h \qquad \Rightarrow \text{Transitional Flow}$$
$$d \ge 1.4h \qquad \Rightarrow \text{Orifice Flow}$$

where;

$d = \text{Depth of Sump at Proposed Catch Basin Inlet} = \text{Spill Elev} - \text{Weir Elev}$

$\text{Spill Elev} = \text{Low Point in the Curb Elevation at Catch Basin} = 1573.50\ ft$

$\text{Weir Elev} = \text{Catch Basin Weir Elevation} = \text{Top of Curb Elevation at Catch Basin} - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$\text{Top of Curb Elevation at Catch Basin} = 1573.50\ ft$

$\text{Weir Elev} = 1573.50\ ft - 4.75\ in = 1573.10\ ft$

$d = \text{Spill Elev} - \text{Lip Elev}$

$d = 1573.50 - 1573.10$

$[d = 0.40\ ft]$

$h = \text{Height of Curb Opening portion of the Catch Basin}$

$h = 5\ in$

$[h = 0.42\ ft]$

$d < h$

$0.40\ ft < 0.42\ ft$

Weir Flow

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w\left(L_f + 1.8W\right)d^{1.5} \quad \left(\text{Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics}\right)$$

where;

$Q_i = Combination\ Basin\ Flow\ Interception\ Capacity$

$C_w = Weir\ Flow\ Coefficient = 2.3$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin = 0.40$

$L = Length\ of\ Proposed\ Combination\ Catch\ Basin = 10.0\ ft$

$L_f = Effective\ Length\ of\ Catch\ Basin = C_f \times L$

$C_f = Clogging\ Factor = 0.80\ \left(\text{Table 6.8 - FCDMC Drainage Policies and Standards Manual}\right)$

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = Width\ of\ Depressed\ Gutter = 4.3\ ft\ \left(Modified\ per\ COP\ Std\ Det\ P1572\right)$

$$Q_i = C_w\left(L_f + 1.8W\right)d^{1.5}$$

$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.40^{1.5}$$

$$Q_i = 9.16$$

$$\boxed{\boldsymbol{Q_i = 9\ cfs}}$$

The proposed City of Phoenix Type "Q" combination catch basin (Std. Det. P1572) at CB #2 has an interception capacity of 9 cfs, which is greater than the 100-year, 6-hour design peak discharge of 8 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Catch Basin Design @ CB#3

Determine if Catch Basin operates as a Weir or as an Orifice:

$d \le h \quad \Rightarrow Weir\ Flow$

$h \ge d \ge 1.4h \quad \Rightarrow Transitional\ Flow$

$d \ge 1.4h \quad \Rightarrow Orifice\ Flow$

where;

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin\ Inlet = Spill\ Elev - Weir\ Elev$

$Spill\ Elev = Low\ Point\ in\ the\ sump,\ gutter\ elevation\ at\ landscaped\ island = 1575.40\ ft$

$Weir\ Elev = Catch\ Basin\ Weir\ Elevation = Top\ of\ Curb\ Elevation\ at\ Catch\ Basin - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$Top\ of\ Curb\ Elevation\ at\ Catch\ Basin = 1575.50\ ft$

$Weir\ Elev = 1575.50\ ft - 4.75\ in = 1575.10\ ft$

$d = Spill\ Elev - Lip\ Elev$

$d = 1575.40 - 1575.10$

$\left[d = 0.30\ ft\right]$

$h = Height\ of\ Curb\ Opening\ portion\ of\ the\ Catch\ Basin$

$h = 5\ in$

$\left[h = 0.42\ ft\right]$

$d < h$

$0.30\ ft < 0.42\ ft$

Weir Flow

Gavan & Barker Inc.

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w \left(L_f + 1.8W\right) d^{1.5} \quad \left(\text{Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics}\right)$$

where;

$Q_i = Combination\ Basin\ Flow\ Interception\ Capacity$

$C_w = Weir\ Flow\ Coefficient = 2.3$

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin = 0.30$

$L = Length\ of\ Proposed\ Combination\ Catch\ Basin = 10.0\ ft$

$L_f = Effective\ Length\ of\ Catch\ Basin = C_f \times L$

$C_f = Clogging\ Factor = 0.80 \left(\text{Table 6.8 - FCDMC Drainage Policies and Standards Manual}\right)$

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = Width\ of\ Depressed\ Gutter = 4.3\ ft \left(Modified\ per\ COP\ Std\ Det\ P1572\right)$

$$Q_i = C_w \left(L_f + 1.8W\right) d^{1.5}$$

$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.30^{1.5}$$

$$Q_i = 5.95$$

$$\boxed{\mathbf{Q_i = 6\ cfs}}$$

The proposed City of Phoenix Type “Q” combination catch basin (Std. Det. P1572) at CB #3 has an interception capacity of 6 cfs, which is greater than the 100-year, 6-hour design peak discharge of 5 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Gavan & Barker Inc.

Catch Basin Design @ CB#4

Determine if Catch Basin operates as a Weir or as an Orifice:

$d \leq h \quad \Rightarrow Weir\ Flow$

$h \geq d \geq 1.4h \quad \Rightarrow Transitional\ Flow$

$d \geq 1.4h \quad \Rightarrow Orifice\ Flow$

where;

$d = Depth\ of\ Sump\ at\ Proposed\ Catch\ Basin\ Inlet = Spill\ Elev - Weir\ Elev$

$Spill\ Elev = Low\ Point\ in\ the\ sump,\ gutter\ elevation\ at\ landscaped\ island = 1578.90\ ft$

$Weir\ Elev = Catch\ Basin\ Weir\ Elevation = Top\ of\ Curb\ Elevation\ at\ Catch\ Basin - 4.75\ in$

The catch basin weir elevation is located at the lip of the modified gutter pan. Per COP Std. Det. P1572 the gutter pan lip elevation is 4.75" below the catch basin top of curb elevation.

$Top\ of\ Curb\ Elevation\ at\ Catch\ Basin = 1579.00\ ft$

$Weir\ Elev = 1579.00\ ft - 4.75\ in = 1578.60\ ft$

$d = Spill\ Elev - Lip\ Elev$

$d = 1578.90 - 1578.60$

$\left[d = 0.30\ ft\right]$

$h = Height\ of\ Curb\ Opening\ portion\ of\ the\ Catch\ Basin$

$h = 5\ in$

$\left[h = 0.42\ ft\right]$

$d < h$

$0.30\ ft < 0.42\ ft$

$\boxed{\textbf{Weir Flow}}$

Gavan & Barker Inc.

Weir Flow, Sump Condition, Curb Opening Catch Basin Sizing:

$$Q_t = C_w\left(L_f + 1.8W\right)d^{1.5} \quad \left(\text{Equation 3.11 - Drainage Design Manual for Maricopa County, Hydraulics}\right)$$

where;

$Q_i = \textit{Combination Basin Flow Interception Capacity}$

$C_w = \textit{Weir Flow Coefficient} = 2.3$

$d = \textit{Depth of Sump at Proposed Catch Basin} = 0.30$

$L = \textit{Length of Proposed Combination Catch Basin} = 10.0\ ft$

$L_f = \textit{Effective Length of Catch Basin} = C_f \times L$

$C_f = \textit{Clogging Factor} = 0.80\ \left(\text{Table 6.8 - FCDMC Drainage Policies and Standards Manual}\right)$

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

$\left[L_f = 8.0\ ft\right]$

$W = \textit{Width of Depressed Gutter} = 4.3\ ft\ \left(\textit{Modified per COP Std Det P1572}\right)$

$$Q_i = C_w\left(L_f + 1.8W\right)d^{1.5}$$

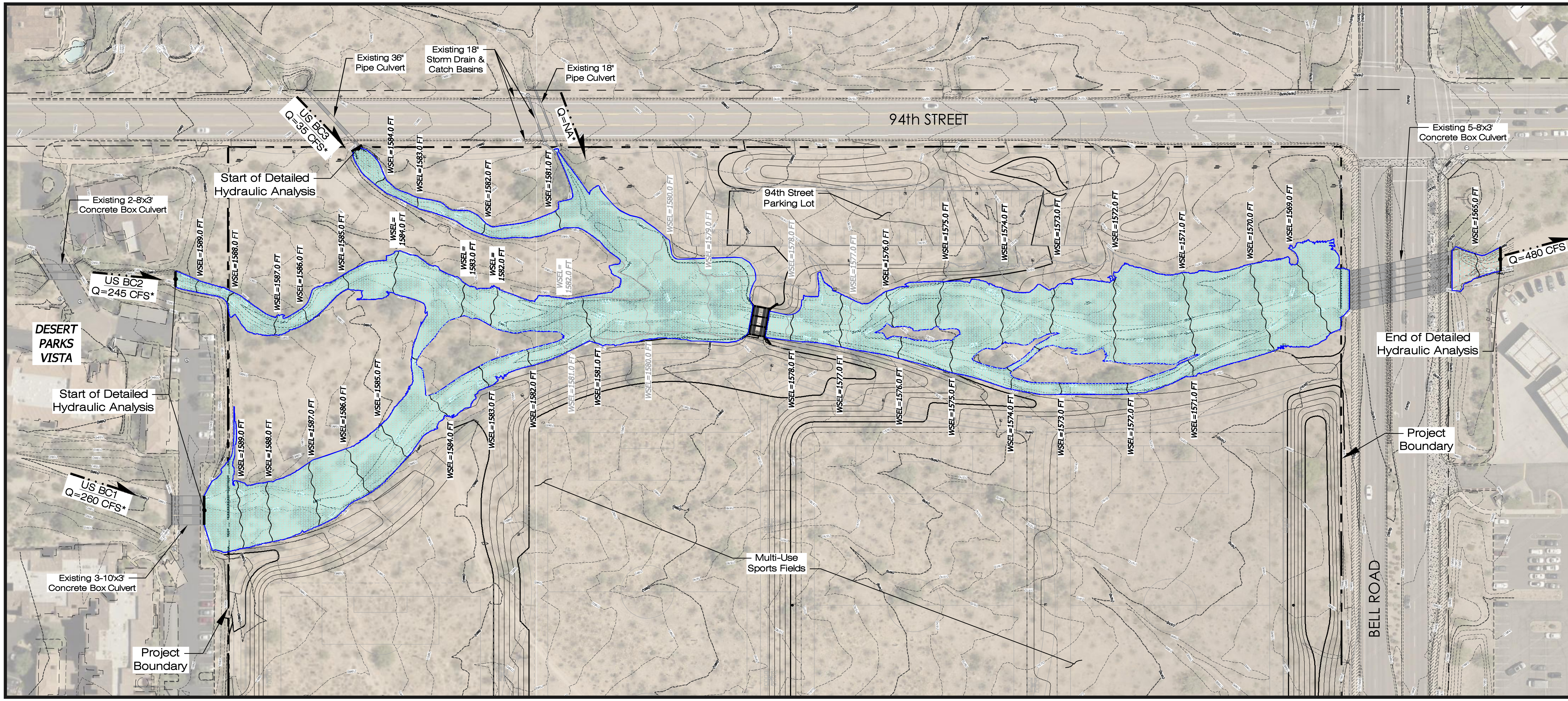
$$Q_i = 2.3\,(8.0 + 1.8 \times 4.3)\,0.30^{1.5}$$

$$Q_i = 5.95$$

$$\boxed{\mathbf{Q_i = 6\ cfs}}$$

The proposed City of Phoenix Type "Q" combination catch basin (Std. Det. P1572) at CB #4 has an interception capacity of 6 cfs, which is greater than the 100-year, 6-hour design peak discharge of 6 cfs. Due to the high clogging potential of grates in sump locations, the interception capacity of the grate portion of the catch basin has been ignored.

Appendix D: Culvert Design & Wash Hydraulic Analysis



LEGEND

Proposed Condition Inundation Boundary

Proposed Condition Water Surface Elevation

Existing Condition Inundation Boundary

Existing Condition Water Surface Elevation

Q=260 CFS **Max Peak Discharge & Flow Direction**

94th STREET WASH

Culvert Hydraulic Analysis

NOTES:

*The 94st Street Wash design discharges upstream of the project area were determined with the 100-year, 24-hour Pinnacle Peak South FLO-2D model prepared by TY Lin International and modified to reflect current flow conditions by Gavan & Barker Inc. as part of this project. Refer to Appendix B for the inflow hydrographs and Appendix E for the modified FLO-2D model.

There are three main wash inflows that enter the project site. The two largest ones enter from the Desert Parks Vista Condominium Complex, just north of the project site and include the west fork that has a peak discharge of 260 cfs and the east fork with a peak discharge of 245 cfs. The third, smaller inflow enters underneath 94th Street through a 36-inch pipe culvert and has a peak discharge of 35 cfs. There is a fourth inflow location across 94th Street that consists of two curb opening catch basins that drain 94th Street and a 18-inch pipe culvert that conveys flows underneath the street. However, the contributing drainage area to that 18-inch culvert and two curb opening catch basins is small compared to the large upstream watershed areas that contributed to the three main inflow locations. Therefore, it can be concluded that the flow from the 18-inch culvert and the flow intercepted by the curb opening catch basins will not directly add to the peak discharge of the three main inflows and therefore increase the design discharge through the project site.

0 60' 120'

SCALE: 1" = 120'

Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
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Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :	
G&B No.	2003
Issue Date:	05/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

94th Street Wash Culvert Hydraulic Analysis

Sheet Number:

1

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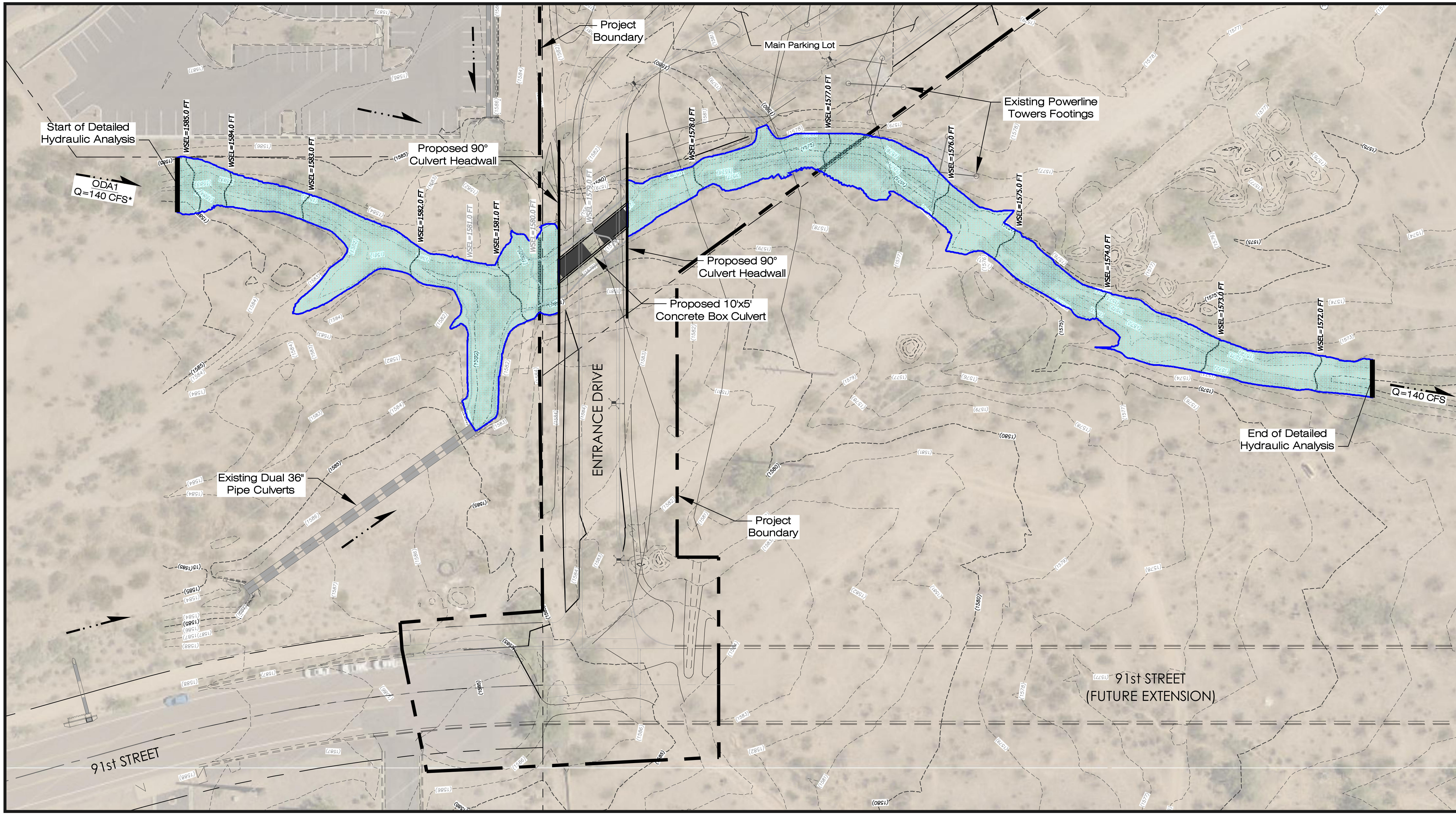

LEGEND

- Proposed Condition Inundation Boundary
- Proposed Condition Water Surface Elevation
- Existing Condition Inundation Boundary
- Existing Condition Water Surface Elevation
- Q=140 CFS — Max Peak Discharge & Flow Direction

91st STREET WASH

Hydraulic Analysis

NOTES:

*The 91st Street Wash 100-year, 6-hour design discharge upstream of the proposed culvert is 140 cfs. For hydraulic modeling purposes, the hydrograph associated with the design discharge was applied to the main 91st Street was, however it includes the flow from the exiting dual 36-inch pipe culvert and the buisiness park northeast of the proposed culvert crossing. Refer to the HEC-1 Hydrologic Modeling in Appendix ZZZ for the hydrologic results as well as a detailed drainage area map showing the contributing drainage area to the proposed culvert.

0 30' 60'

SCALE: 1" = 60'

Gavan & Barker Inc.

Gavan & Barker, Inc.
3030 N. Central Ave.
Suite 700
Phoenix, Arizona 85012
Phone: 602-200-0031
Fax: 602-200-0032

Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
NWC Bell Rd & 94th St
CITY OF SCOTTSDALE
CONTRACT NUMBER: 2020-068-COS

Submittal :

G&B No.	2003
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Checked By:	MTG

Sheet Title :

91st Street Wash Culvert Hydraulic Analysis

Sheet Number:

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Appendix E: Digital Data

If pdf copy, click HERE to download the Digital Data

[Digital Data CD]

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