Drainage Reports

CITY OF SCOTTSDALE

MULTI-USE SPORTS FIELDS

NWC of Bell Road & 94th Street

DRAFT DESIGN DATA REPORT

Project No. PA75200538

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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 (3) shallow retention basins as well as one 3-foot deep detention basin. All three of the retention basins are located adjacent to the new 94th Street parking lot. Each of the 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 four 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 three 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.

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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 2.0 feet above the basin bottom. This basin outlet configuration allows the basin to function as a retention basin up to a depth of 2.0 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 2.0 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 2.0 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 10-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 10-inch sand layer, the water is already naturally filtered and no first flush storage is necessary. The 12-inch subgrade drainpipe is discharged

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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.

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 19.5 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 6.9 hours. Therefore, the total basin drain time for the large detention basin is approximately 26.4 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

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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-

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1 hydrologic model. The Digital Data in Appendix F 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 an 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. 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

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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 F 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 10-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 10-sand layer has a storage capacity of 2.0 inches (10 in x 0.20 = 2.0 in) over the surface area of field. However, this 2.0 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 1.0 inches was applied to represent the available storage associated with the 20% capillary porosity within the 10-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- and 12-inch pipes as well as the large

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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 basin outfall storm drain.

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.

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Concurrently to the design of these Multi-Use Sports Fields, the City of Scottsdale commissioned Gavan & Barker to investigate the hydrologic and hydraulic impact of the three Reata Wash levee penetration culverts between Bell Road and Hualapai Drive. The analysis and subsequent drainage memorandum had a twofold purpose: 1) determine the amount of flow diverted at each of the three existing culverts and the impact of those flows on the downstream watershed as well as 2) recommended modifications to reduce the amount of flow diverted at each levee penetration culvert to mitigate flooding and sediment deposition in the washes downstream of the diversions. As can be seen in the *Reata Wash Levee – Flow Diversion Culverts* drainage memorandum in Appendix E, the diversion capacity of both culverts that feed the main (west) wash fork and the secondary (east) wash fork are recommended to be significantly reduced. The dual 8'x2' culvert that feeds the west wash fork is recommended to be reduced from 16 feet to 3 feet, while the 10'x2' culvert that feeds the east wash fork is recommended to be reduced from 10 feet to 2 feet. These recommendations have been incorporated into the design FLO-2D model and the design flows for the Multi-Use Sports Fields incorporate the reductions in the Reata Wash levee penetration culverts.

As can be seen in the drainage area map in Appendix B, there is approximately 75 cfs that enters the west fork of the 94th Street Wash through the modified northern dual 8'x2' concrete box culvert from Reata Wash. Just downstream of the culvert, from field inspection it was found that the entire reduced flow that penetrates through Reata Wash stays within the main wash flowing south. 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 265 cfs. For the east fork of the 94th Street Wash there is approximately 55 cfs that enters through the modified 10'x2' concrete box culvert from Reata Wash. This flow increases with the addition of the runoff generated from the contributing watershed area to 150 cfs before it 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.

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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 F.

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 8'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 420 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

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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.

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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 420 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 F 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 420 cfs

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with no overtopping or spilling into the adjacent fields or into the new parking lot. In order to achieve this, a triple barrel 8'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 24-foot culvert width will span the exiting 22-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 1.7 feet from 1578.5 feet to 1580.2 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 have no 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 F 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 F 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

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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 F for the HEC-RAS two-dimensional hydraulics model.

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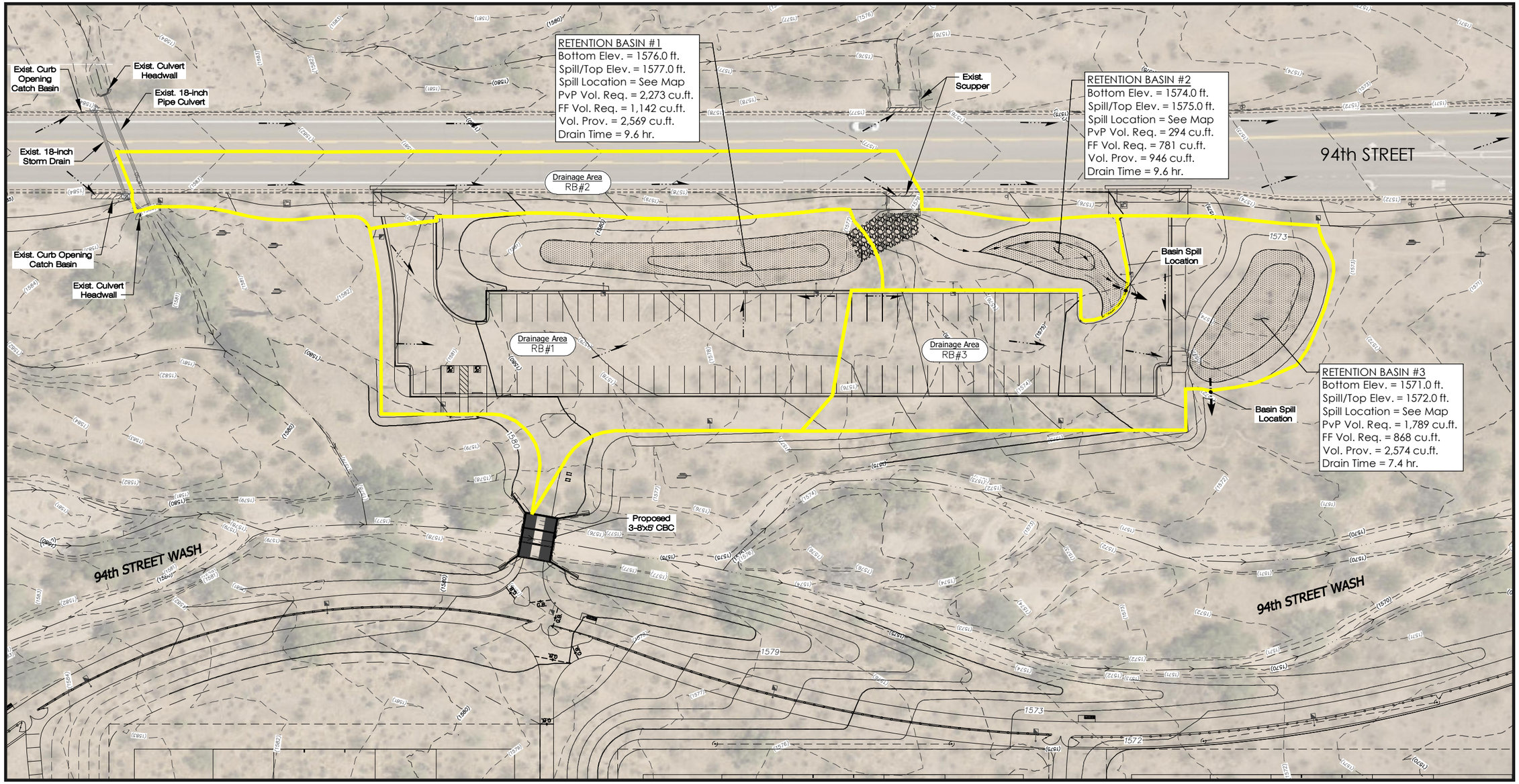
Appendix A: Retention Basin Design Calculations

38-DR-2020
11/25/2020

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	35,100	3,093	5,366	2,273	1,142
RB#2	24,580	3,377	3,671	294	781
RB#3	26,010	2,292	4,081	1,789	868

LEGEND

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

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:	11/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

94th Street Parking Lot Drainage Area Map

Sheet Number:

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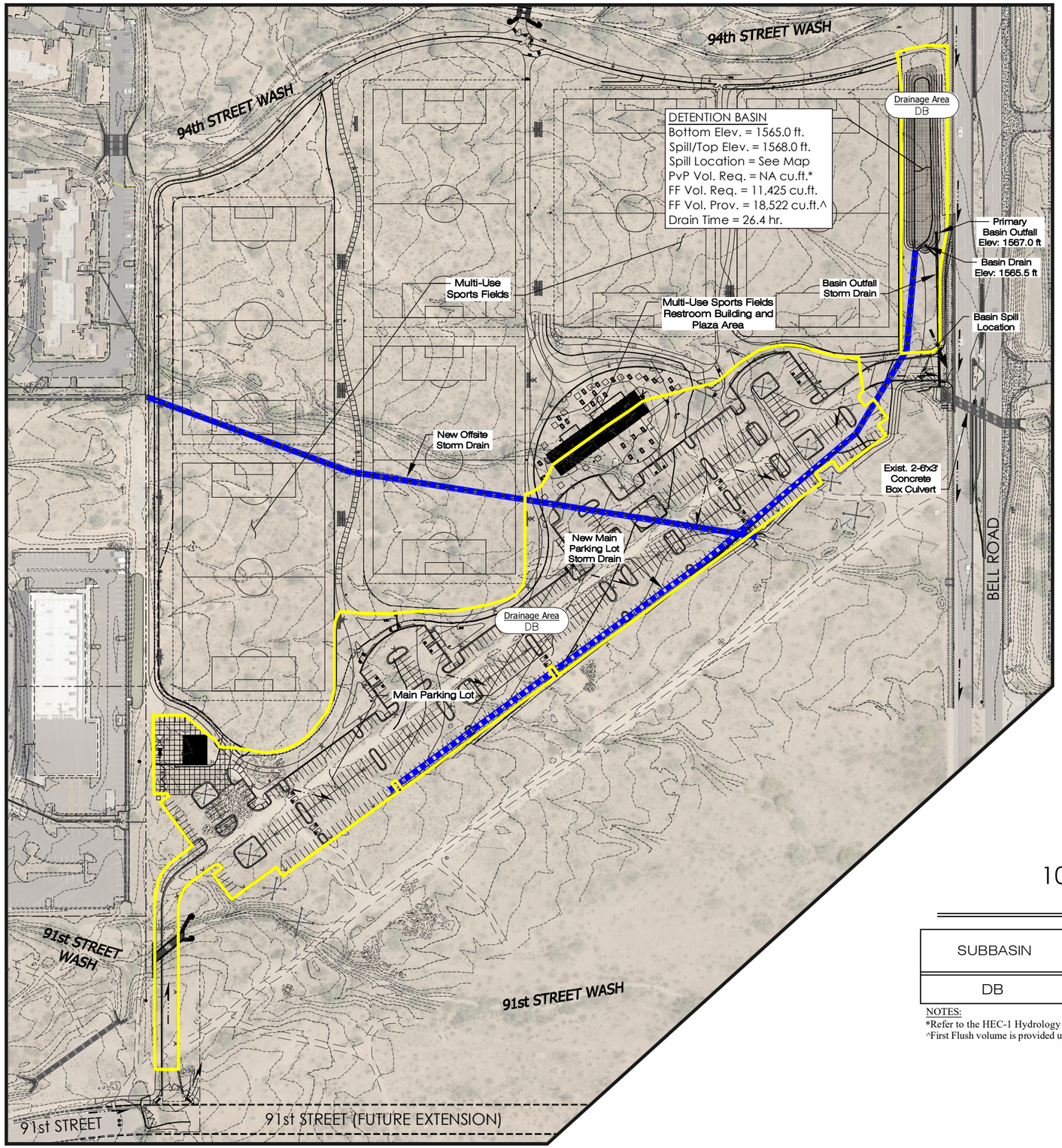

MAIN PARKING LOT

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.)^
DB	333,230	NA*	NA*	NA*	11,425

NOTES:
*Refer to the HEC-1 Hydrology Model in Appendix B for the Pre vs. Post Runoff Analysis
^First Flush volume is provided up to the primary outfall catch basin lip elevation of 1567.00 ft.

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Sheet Title :

Main Parking Lot Drainage Area Map

Sheet Number:

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First Flush Volume Calculation

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11/25/2020

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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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	11,890	0.45	5,350.5	0.5	222.9
Total Contributing Drainage Area:	35,100			**Total First Flush Runoff Volume:**	**1,142**

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)	15,370	0.95	14,601.5	0.5	608.4
Desert Landscaping	9,210	0.45	4,144.5	0.5	172.7
Total Contributing Drainage Area:	24,580			**Total First Flush Runoff Volume:**	**781**

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)	18,270	0.95	17,356.5	0.5	723.2
Desert Landscaping	7,740	0.45	3,483.0	0.5	145.1
Total Contributing Drainage Area:	26,010			**Total First Flush Runoff Volume:**	**868**

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

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	248,510	0.95	236,084.5	0.5	9,836.9
Desert Landscaping	84,720	0.45	38,124.0	0.5	1,588.5
Total Contributing Drainage Area:	333,230			**Total First Flush Runoff Volume:**	**11,425**

^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

38-DR-2020
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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

Gavan & Barker Inc.
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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	35,100	0.45	15,795.0	2.35	3,093.2
Total Contributing Drainage Area:	35,100			**Total Pre Development Runoff Volume**	**3,093**

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	11,890	0.45	5,350.5	2.35	1,047.8
Total Contributing Drainage Area:	35,100			**Total Post Development Runoff Volume**	**5,366**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**2,273**
Total First Flush Volume Required:	**1,142**
Retention Basin #1 Provided Retention Volume:	2,569

^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.
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)	12,370	0.95	11,751.5	2.35	2,301.3
Undeveloped Desert	12,210	0.45	5,494.5	2.35	1,076.0
Total Contributing Drainage Area:	24,580			**Total Pre Development Runoff Volume**	**3,377**

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)	15,370	0.95	14,601.5	2.35	2,859.5
Desert Landscaping	9,210	0.45	4,144.5	2.35	811.6
Total Contributing Drainage Area:	24,100			**Total Post Development Runoff Volume**	**3,671**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**294**
Total First Flush Volume Required:	**781**
Retention Basin #2 Provided Retention Volume:	946

^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	26,010	0.45	11,704.5	2.35	2,292.1
Total Contributing Drainage Area:	26,010			**Total Pre Development Runoff Volume**	**2,292**

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)	18,270	0.95	17,356.5	2.35	3,399.0
Desert Landscaping	7,740	0.45	3,483.0	2.35	682.1
Total Contributing Drainage Area:	26,010			**Total Post Development Runoff Volume**	**4,081**

Total Pre vs. Post Runoff Volume Increase (Required Retention Volume):	**1,789**
Total First Flush Volume Required:	**868**
Retention Basin #3 Provided Retention Volume:	2,574

^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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Civil Engineering & Landscape Architecture

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: 333,230

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: 333,230

Total Post Development Runoff Volume **0**

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

Total First Flush Volume Required: **11,425**

Retention Basin #5 Provided Retention Volume~: 18,522

~The provided detention basin volume is calculated to the primary basin outlet spill elevation of 1567.00 ft.

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Retention Basin Drain Time Calculation

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Basin Drain Times

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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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
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 19.5 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 26.4 hours.

Appendix B: Hydrologic Analysis

38-DR-2020
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Existing Conditions: HEC-1 Schematic w/100-year, 6-hour Model

LEGEND & HEC-1 SYMBOLOGY

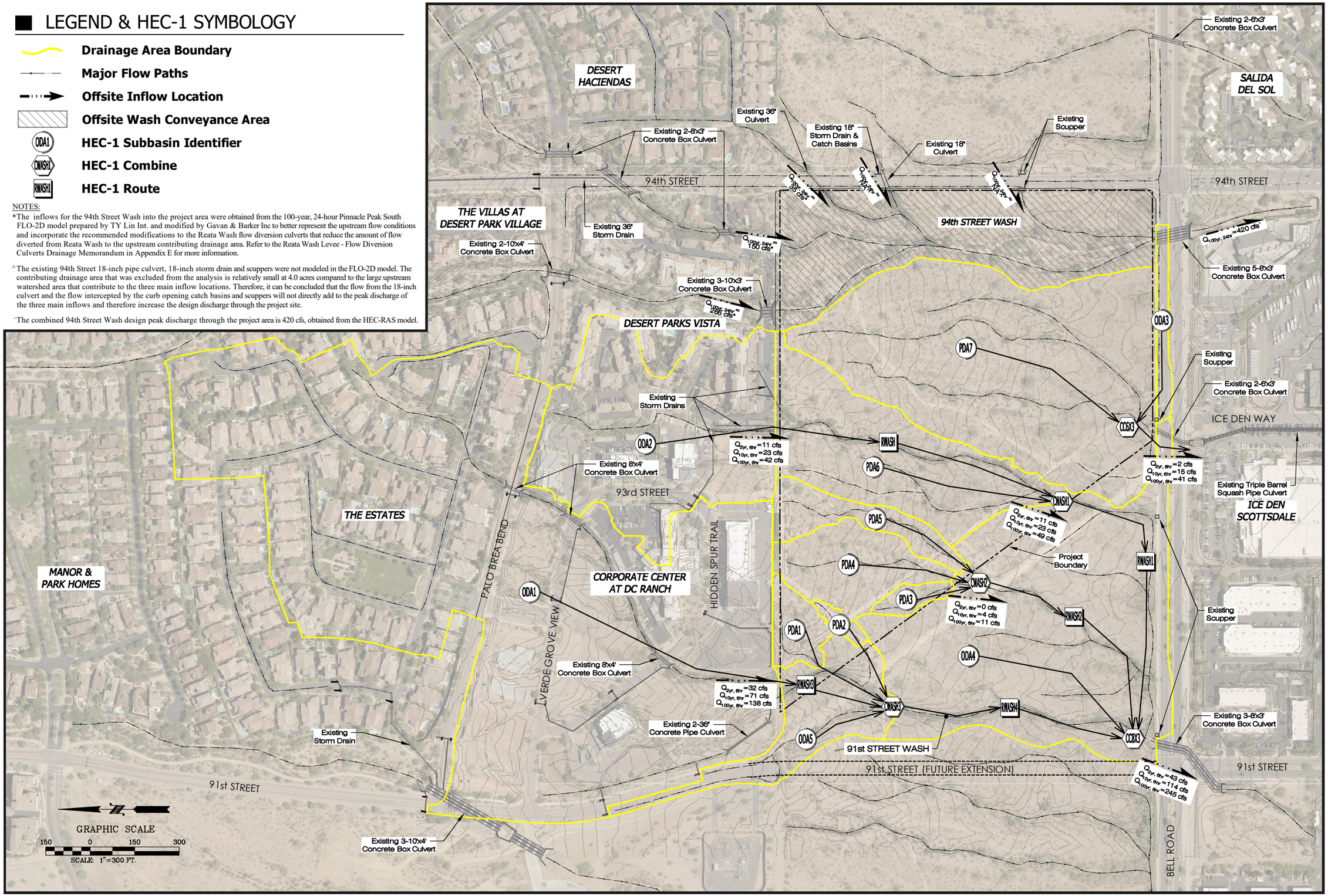
- **Drainage Area Boundary**
- **Major Flow Paths**
- **Offsite Inflow Location**
- **Offsite Wash Conveyance Area**
- **HEC-1 Subbasin Identifier** (ODA1)
- **HEC-1 Combine** (CWASH1)
- **HEC-1 Route** (RWASH1)

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 and incorporate the recommended modifications to the Reata Wash flow diversion culverts that reduce the amount of flow diverted from Reata Wash to the upstream contributing drainage area. Refer to the Reata Wash Levee - Flow Diversion Culverts Drainage Memorandum in Appendix E for more information.

^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 420 cfs, obtained from the HEC-RAS model.



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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

38-DR-2020
11/25/2020

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 *            VERSION 4.1                *
 *                                       *
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*                                       *
*    U.S. ARMY CORPS OF ENGINEERS       *
*    HYDROLOGIC ENGINEERING CENTER      *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*           (916) 756-1104              *
*                                       *
*****************************************
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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

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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
                    *DIAGRAM
            8       IT       2  1JAN99       0     360
            9       IO       5
           10       IN      15
                    *
                    *

           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

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          39          KK   RWASH   ROUTE
          40          RS       1    FLOW
          41          RC   0.045   0.030   0.045     920  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
```

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```
 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
             *
```



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

118          KK    ODA5   BASIN
119          BA   0.006
120          LG    0.29    0.35    4.03    0.42      34
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=ODA5   B=BASIN  C=FLOW  F=CALC
```

```
                    *

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

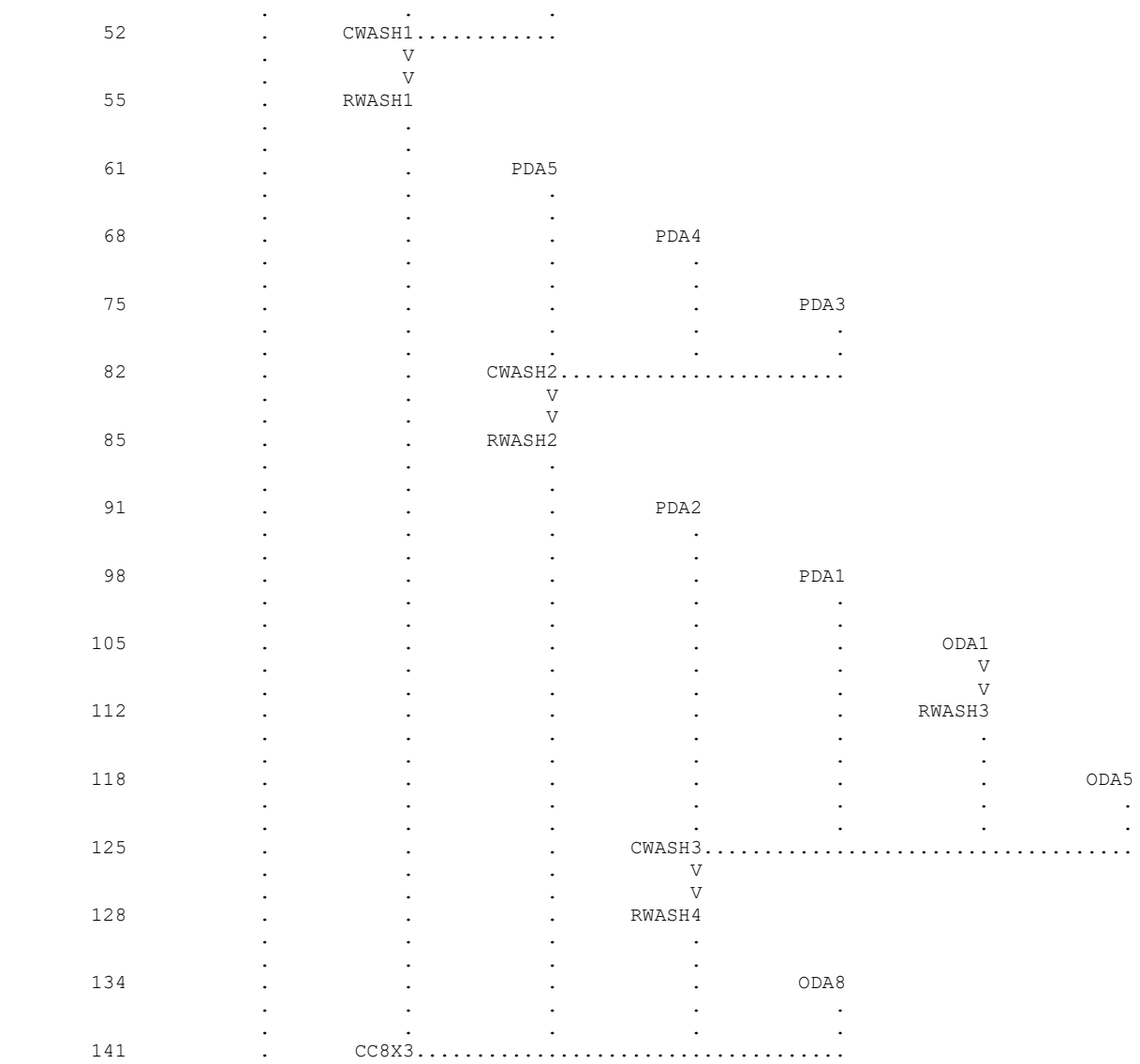
          128       KK  RWASH4   ROUTE
          129       RS       1    FLOW
          130       RC   0.045   0.030   0.045     870  0.0127    3.00
          131       RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
          132       RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    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
                 .           .           .
```



(***) RUNOFF ALSO COMPUTED AT THIS LOCATION

1**********

* *

* FLOOD HYDROGRAPH PACKAGE (HEC-1) *

* JUN 1998 *

****

* *

* U.S. ARMY CORPS OF ENGINEERS *

* HYDROLOGIC ENGINEERING CENTER *

```
*            VERSION 4.1               *                                        *          609 SECOND STREET          *
*                                      *                                        *       DAVIS, CALIFORNIA 95616       *
*  RUN DATE   16NOV20  TIME  15:45:23  *                                        *           (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
               FLOW                  CUBIC FEET PER SECOND
               STORAGE VOLUME        ACRE-FEET
               SURFACE AREA          ACRES
               TEMPERATURE           DEGREES FAHRENHEIT
   -----DSS---ZOPEN:  Existing File Opened,  File: EC100.DSS
                      Unit:  71;  DSS Version: 6-JG
-----DSS---ZWRITE Unit  71; Vers.    6:  /ODA3/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /ODA3/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /PDA7/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /PDA7/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /CC6X3/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /CC6X3/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /ODA2/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    6:  /ODA2/BASIN/FLOW/01JAN1999/2MIN/CALC/
```

```
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH/ROUTE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH/ROUTE/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA6/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA6/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CWASH1/COMBINE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CWASH1/COMBINE/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH1/ROUTE/FLOW/31DEC1998/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH1/ROUTE/FLOW/01JAN1999/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA5/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA5/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA4/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA4/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA3/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA3/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CWASH2/COMBINE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CWASH2/COMBINE/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH2/ROUTE/FLOW/31DEC1998/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH2/ROUTE/FLOW/01JAN1999/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA2/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA2/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA1/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /PDA1/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /ODA1/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /ODA1/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH3/ROUTE/FLOW/31DEC1998/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /RWASH3/ROUTE/FLOW/01JAN1999/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /ODA5/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /ODA5/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CWASH3/COMBINE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CWASH3/COMBINE/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    5:  /RWASH4/ROUTE/FLOW/31DEC1998/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    5:  /RWASH4/ROUTE/FLOW/01JAN1999/2MIN/CAL/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /ODA8/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /ODA8/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CC8X3/COMBINE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    6:  /CC8X3/COMBINE/FLOW/01JAN1999/2MIN/CALC/
1
                                                  RUNOFF SUMMARY
                                              FLOW IN CUBIC FEET PER SECOND
                                            TIME IN HOURS,  AREA IN SQUARE MILES

                                    PEAK    TIME OF     AVERAGE FLOW FOR MAXIMUM PERIOD      BASIN     MAXIMUM     TIME OF
          OPERATION       STATION   FLOW     PEAK                                           AREA      STAGE     MAX STAGE
+                                                        6-HOUR     24-HOUR     72-HOUR

          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.	4.03	3.	2.	2.	.02	
+							.91	4.03
	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							
+	ODA5	12.	4.10	1.	1.	1.	.01	
	4 COMBINED AT							
+	CWASH3	153.	4.10	15.	8.	8.	.08	

```
          ROUTED TO
+                         RWASH4       149.    4.13          15.          8.          8.        .08
+                                                                                                 1.70        4.13

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

          4 COMBINED AT
+                         CC8X3       245.    4.13          24.         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

38-DR-2020
11/25/2020

LEGEND & HEC-1 SYMBOLOGY

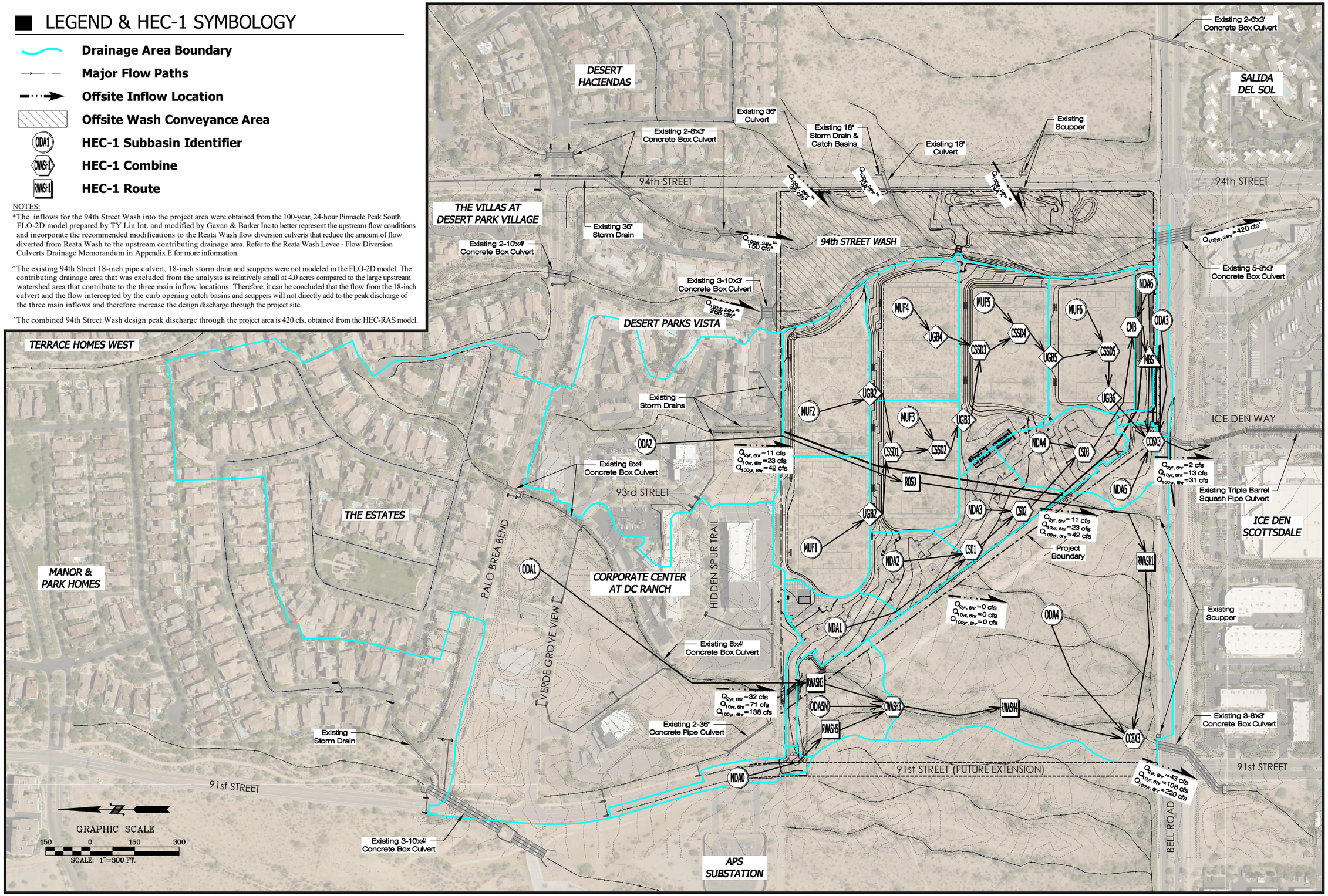
- **Drainage Area Boundary**
- **Major Flow Paths**
- **Offsite Inflow Location**
- **Offsite Wash Conveyance Area**
- **HEC-1 Subbasin Identifier** (ODA1)
- **HEC-1 Combine** (CWASH1)
- **HEC-1 Route** (RWASH1)

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 and incorporate the recommended modifications to the Reata Wash flow diversion culverts that reduce the amount of flow diverted from Reata Wash to the upstream contributing drainage area. Refer to the Reata Wash Levee - Flow Diversion Culverts Drainage Memorandum in Appendix E for more information.

^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 area.

`The combined 94th Street Wash design peak discharge through the project area is 420 cfs, obtained from the HEC-RAS model.



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:	11/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :

Proposed Condition HEC-1 Drainage Area Map

Sheet Number:

1

38-DR-2020
11/25/2020

```
1*****************************************
 *                                       *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
 *               JUN   1998              *
 *            VERSION 4.1                *
 *                                       *
 *  RUN DATE   16NOV20  TIME  15:45:59   *
 *                                       *
 *****************************************
```

```
*****************************************
*                                       *
*    U.S. ARMY CORPS OF ENGINEERS       *
*    HYDROLOGIC ENGINEERING CENTER      *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*           (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


```
          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        06/03/2020
                         *DIAGRAM
             8           IT       2  1JAN99       0    1080
             9           IO       5
            10           IN      15
                         *
                         *

            11           KK    NDA1   BASIN
```

```
          12          BA   0.003
          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    0.10    0.32    4.03    0.39      67
          18          UC   0.171   0.280
          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=NDA1   B=RUNOFF  C=FLOW  F=CALC
                      *

          22          KK    NDA2   BASIN
          23          BA   0.002
          24          LG    0.10    0.32    4.03    0.39      67
          25          UC   0.112   0.149
          26          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
          27          UA     100
          28          ZW   A=NDA2   B=RUNOFF  C=FLOW  F=CALC
                      *

          29          KK    CSD1 COMBINE
          30          HC       2
          31          ZW   A=CSD1   B=COMBINE  C=FLOW  F=CALC
                      *

          32          KK    NDA3   BASIN
          33          BA   0.003
          34          LG    0.09    0.32    4.03    0.39      70
          35          UC   0.108   0.102
          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=NDA3   B=RUNOFF  C=FLOW  F=CALC
                      *
1                                        HEC-1 INPUT                                          PAGE  2

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

          39          KK    CSD2 COMBINE
          40          HC       2
          41          ZW   A=CSD2   B=COMBINE  C=FLOW  F=CALC
                      *

          42          KK    NDA4   BASIN
          43          BA   0.003
          44          LG    0.08    0.33    4.03    0.39      73
          45          UC   0.112   0.106
          46          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
          47          UA     100
          48          ZW   A=NDA4   B=RUNOFF  C=FLOW  F=CALC
                      *
```

```
 49          KK    CSD3 COMBINE
 50          HC       2
 51          ZW   A=CSD3   B=COMBINE  C=FLOW  F=CALC
             *

 52          KK    NDA6   BASIN
 53          BA   0.001
 54          LG    0.20    0.25    4.03    0.48       0
 55          UC   0.080   0.050
 56          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 57          UA     100
 58          ZW   A=NDA6   B=RUNOFF  C=FLOW  F=CALC
             *

 59          KK     CMB COMBINE
 60          HC       2
 61          ZW   A=CMB   B=COMBINE  C=FLOW  F=CALC
             *

 62          KK     MBS STORAGE
 63          RS       1    STOR
 64          SV            0.08    0.08    0.18    0.30    0.43    0.49    0.52    0.55    0.58
 65          SV    0.61    0.64    0.67    0.70    0.73    0.90
 66          SQ                    0.38    0.42    0.45    0.48    0.49    1.41    3.08    5.24
 67          SQ    7.81   10.38   11.76   12.99   14.11    75.0
 68          SE  1565.0  1565.5 1565.51  1566.0  1566.5  1567.0  1567.2  1567.3  1567.4  1567.5
 69          SE  1567.6  1567.7  1567.8  1567.9  1568.0  1568.5
 70          ZW   A=MBS   B=BASIN  C=FLOW  F=CALC
 71          ZW   B=BASIN   C=STOR
 72          ZW   B=BASIN   C=STAGE
             *

 73          KK    MUF1   BASIN
 74          BA   0.005
 75          LG    0.99    0.03    4.03    0.39       4
 76          UC   0.276   0.288
 77          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 78          UA     100
 79          ZW   A=MUF1   B=RUNOFF  C=FLOW  F=CALC
             *
1                                    HEC-1 INPUT                                          PAGE  3

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


 80          KK    UGB1 STORAGE
 81          RS       1    STOR
 82          SV           0.036   0.338
 83          SQ            0.35    0.37
 84          SE  1581.5 1583.67 1584.00
 85          ZW   A=UGB1   B=BASIN  C=FLOW  F=CALC
 86          ZW   B=BASIN   C=STOR
 87          ZW   B=BASIN   C=STAGE
```

```
                    *

     88             KK    MUF2   BASIN
     89             BA   0.005
     90             LG    0.98    0.02    4.03    0.39       4
     91             UC   0.273   0.284
     92             UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
     93             UA     100
     94             ZW   A=MUF2   B=RUNOFF  C=FLOW  F=CALC
                    *

     95             KK    UGB2 STORAGE
     96             RS       1    STOR
     97             SV           0.036   0.334
     98             SQ            0.34    0.37
     99             SE  1581.6 1583.67 1584.00
    100             ZW   A=UGB2   B=BASIN  C=FLOW  F=CALC
    101             ZW   B=BASIN   C=STOR
    102             ZW   B=BASIN   C=STAGE
                    *

    103             KK   CSSD1 COMBINE
    104             HC       2
    105             ZW   A=CSSD1   B=COMBINE  C=FLOW  F=CALC
                    *

    106             KK    MUF3   BASIN
    107             BA   0.005
    108             LG    0.98    0.01    4.03    0.39       4
    109             UC   0.270   0.281
    110             UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
    111             UA     100
    112             ZW   A=MUF3   B=RUNOFF  C=FLOW  F=CALC
                    *

    113             KK   CSSD2 COMBINE
    114             HC       2
    115             ZW   A=CSSD2   B=COMBINE  C=FLOW  F=CALC
                    *

    116             KK    UGB3 STORAGE
    117             RS       1    STOR
    118             SV           0.036   0.331
    119             SQ            1.32    1.43
    120             SE  1575.6 1577.67 1578.00
    121             ZW   A=UGB3   B=BASIN  C=FLOW  F=CALC
1                                                HEC-1 INPUT                                                PAGE  4

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

    122             ZW   B=BASIN   C=STOR
    123             ZW   B=BASIN   C=STAGE
                    *
```

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```
 124          KK    MUF4   BASIN
 125          BA   0.005
 126          LG    0.98    0.01    4.03    0.39       4
 127          UC   0.270   0.281
 128          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 129          UA     100
 130          ZW   A=MUF4   B=RUNOFF  C=FLOW  F=CALC
              *

 131          KK    UGB4 STORAGE
 132          RS       1    STOR
 133          SV           0.035   0.319
 134          SQ            0.34    0.37
 135          SE  1575.6 1577.67 1578.00
 136          ZW   A=UGB4   B=BASIN  C=FLOW  F=CALC
 137          ZW   B=BASIN   C=STOR
 138          ZW   B=BASIN   C=STAGE
              *

 139          KK   CSSD3 COMBINE
 140          HC       2
 141          ZW   A=CSSD3   B=COMBINE  C=FLOW  F=CALC
              *

 142          KK    MUF5   BASIN
 143          BA   0.007
 144          LG    0.73    0.07    4.03    0.39      14
 145          UC   0.260   0.223
 146          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 147          UA     100
 148          ZW   A=MUF5   B=RUNOFF  C=FLOW  F=CALC
              *

 149          KK   CSSD4 COMBINE
 150          HC       2
 151          ZW   A=CSSD4   B=COMBINE  C=FLOW  F=CALC
              *

 152          KK    UGB5 STORAGE
 153          RS       1    STOR
 154          SV           0.034   0.307
 155          SQ            4.55    4.81
 156          SE  1568.4 1571.67 1572.00
 157          ZW   A=UGB6   B=BASIN  C=FLOW  F=CALC
 158          ZW   B=BASIN   C=STOR
 159          ZW   B=BASIN   C=STAGE
              *
```



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

38-DR-2020
11/25/2020

```
160           KK    MUF6   BASIN
161           BA   0.005
162           LG    0.98    0.04    4.03    0.39       8
163           UC   0.275   0.287
164           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
165           UA     100
166           ZW   A=MUF6   B=RUNOFF  C=FLOW  F=CALC
              *

167           KK   CSSD5 COMBINE
168           HC       2
169           ZW   A=CSSD5   B=COMBINE  C=FLOW  F=CALC
              *

170           KK    UGB6 STORAGE
171           RS       1    STOR
172           SV           0.035   0.299
173           SQ            4.88    5.37
174           SE  1567.6 1569.67 1570.00
175           ZW   A=UGB6   B=BASIN  C=FLOW  F=CALC
176           ZW   B=BASIN   C=STOR
177           ZW   B=BASIN   C=STAGE
              *

178           KK    NDA5   BASIN
179           BA   0.002
180           LG    0.35    0.35    4.03    0.43       0
181           UC   0.165   0.157
182           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
183           UA     100
184           ZW   A=NDA5   B=RUNOFF  C=FLOW  F=CALC
              *

185           KK    ODA3   BASIN
186           BA   0.001
187           LG    0.05    0.35    4.03    0.39      95
188           UC   0.185   0.499
189           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
190           UA     100
191           ZW   A=ODA3   B=RUNOFF  C=FLOW  F=CALC
              *

192           KK   CC6X3 COMBINE
193           HC       4
194           ZW   A=CC6X3   B=COMBINE  C=FLOW  F=CALC
              *

195           KK    ODA2   BASIN
196           BA   0.016
197           LG    0.19    0.25    4.03    0.59      51
198           UC   0.142   0.098
199           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
200           UA     100
```

```
 201          ZW   A=ODA2   B=BASIN  C=FLOW  F=CALC
              *
1                                        HEC-1 INPUT                                          PAGE  6

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


 202          KK    ROSD   ROUTE
 203          RK     985  0.0050   0.012            CIRC   3.000
 204          ZW   A=ROSD   B=ROUTE  C=FLOW  F=CALC
              *

 205          KK  RWASH1   ROUTE
 206          RS       1    FLOW
 207          RC   0.045   0.030   0.045    1050  0.0047    3.00
 208          RX    0.00   10.00   20.00   30.00   40.00   50.00   60.00   70.00
 209          RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 210          ZW   A=RWASH1   B=ROUTE  C=FLOW  F=CAL
              *

 211          KK    ODA1   BASIN
 212          BA   0.069
 213          LG    0.20    0.25    4.03    0.59      48
 214          UC   0.247   0.200
 215          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 216          UA     100
 217          ZW   A=ODA1   B=BASIN  C=FLOW  F=CALC
              *

 218          KK  RWASH3   ROUTE
 219          RS       1    FLOW
 220          RC   0.045   0.030   0.045     450  0.0138    3.00
 221          RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
 222          RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 223          ZW   A=RWASH3   B=ROUTE  C=FLOW  F=CAL
              *

 224          KK    NDA0   BASIN
 225          BA   0.002
 226          LG    0.05    0.35    4.03    0.39      95
 227          UC   0.110   0.147
 228          UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 229          UA     100
 230          ZW   A=NDA0   B=BASIN  C=FLOW  F=CALC
              *

 231          KK  RWASH5   ROUTE
 232          RS       1    FLOW
 233          RC   0.045   0.030   0.045     390  0.0166    3.00
 234          RX    0.00   10.00   20.00   25.00   27.00   32.00   42.00   52.00
 235          RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
 236          ZW   A=RWASH5   B=ROUTE  C=FLOW  F=CAL
              *
```

```
          237          KK   ODA5N   BASIN
          238          BA   0.004
          239          LG    0.20    0.25    4.03    0.48       0
          240          UC   0.134   0.122
          241          UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
          242          UA     100
          243          ZW   A=ODA5N   B=BASIN  C=FLOW  F=CALC
                       *
1                                                HEC-1 INPUT                                                PAGE  7

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


          244          KK  CWASH3 COMBINE
          245          HC       3
          246          ZW   A=CWASH3   B=COMBINE  C=FLOW  F=CALC
                       *

          247          KK  RWASH3   ROUTE
          248          RS       1    FLOW
          249          RC   0.045   0.030   0.045     870  0.0127    3.00
          250          RX    0.00   10.00   20.00   25.00   30.00   35.00   45.00   55.00
          251          RY    3.00    2.00    1.00    0.00    0.00    1.00    2.00    3.00
          252          ZW   A=RWASH3   B=ROUTE  C=FLOW  F=CAL
                       *

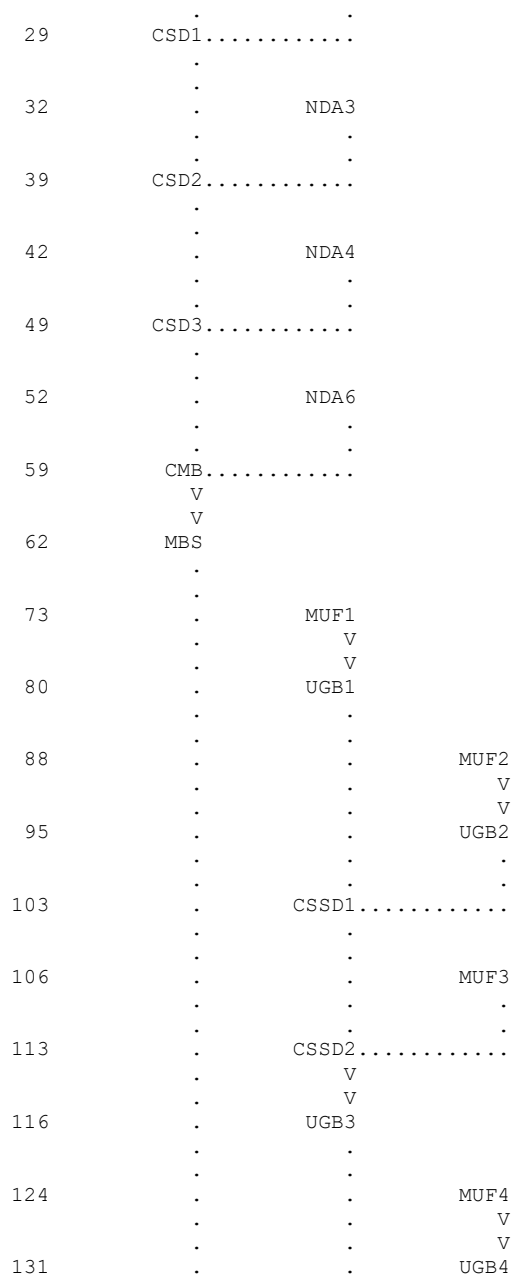
          253          KK    ODA4   BASIN
          254          BA   0.024
          255          LG    0.33    0.35    4.03    0.43       7
          256          UC   0.275   0.187
          257          UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
          258          UA     100
          259          ZW   A=ODA8   B=BASIN  C=FLOW  F=CALC
                       *

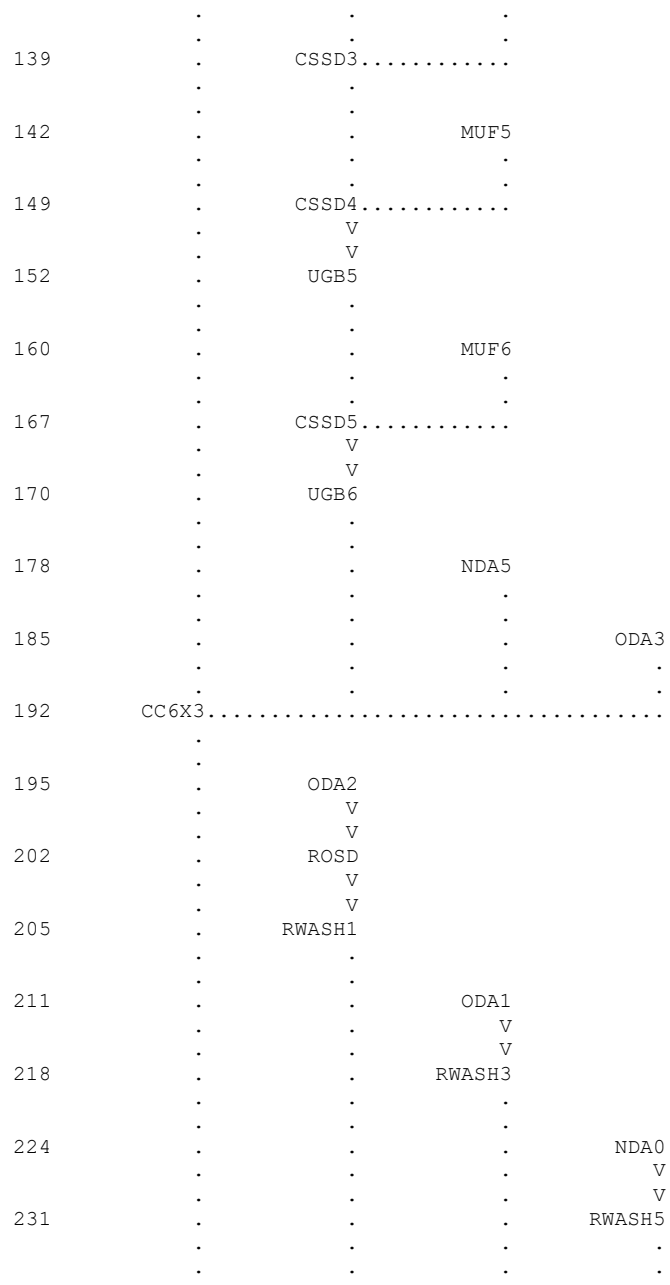
          260          KK   CC8X3 COMBINE
          261          HC       3
          262          ZW   A=CC8X3   B=COMBINE  C=FLOW  F=CALC
                       *
          263          ZZ
1
               SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

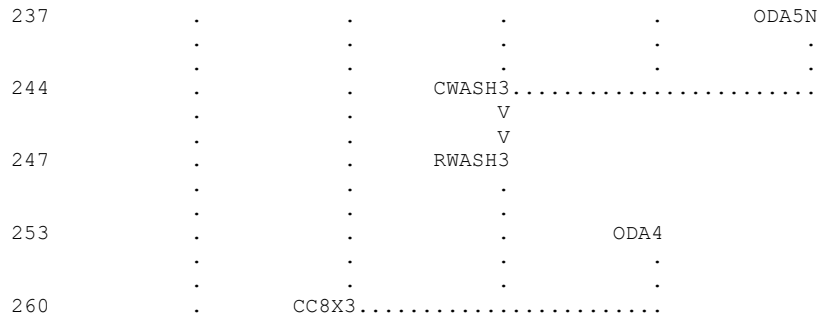
   NO.      (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

    11        NDA1
                .
                .
    22          .        NDA2
                .           .
```

38-DR-2020
11/25/2020


29 CSD1
32 NDA3
39 CSD2
42 NDA4
49 CSD3
52 NDA6
59 CMB
62 MBS
73 MUF1
80 UGB1
88 MUF2
95 UGB2
103 CSSD1
106 MUF3
113 CSSD2
116 UGB3
124 MUF4
131 UGB4


139 CSSD3
142 MUF5
149 CSSD4
152 UGB5
160 MUF6
167 CSSD5
170 UGB6
178 NDA5
185 ODA3
192 CC6X3
195 ODA2
202 ROSD
205 RWASH1
211 ODA1
218 RWASH3
224 NDA0
231 RWASH5



(***) RUNOFF ALSO COMPUTED AT THIS LOCATION

```
1*****************************************
*                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *
*               JUN   1998              *
*            VERSION 4.1                *
*                                       *
*  RUN DATE   16NOV20  TIME  15:45:59   *
*                                       *
*****************************************
```

```
*****************************************
*                                       *
*    U.S. ARMY CORPS OF ENGINEERS       *
*    HYDROLOGIC ENGINEERING CENTER      *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*           (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
06/03/2020

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	1080	NUMBER OF HYDROGRAPH ORDINATES
NDDATE	2JAN99	ENDING DATE
NDTIME	1158	ENDING TIME
ICENT	19	CENTURY MARK

COMPUTATION INTERVAL .03 HOURS
TOTAL TIME BASE 35.97 HOURS

```
          ENGLISH UNITS
               DRAINAGE AREA         SQUARE MILES
               PRECIPITATION DEPTH   INCHES
               LENGTH, ELEVATION     FEET
               FLOW                  CUBIC FEET PER SECOND
               STORAGE VOLUME        ACRE-FEET
               SURFACE AREA          ACRES
               TEMPERATURE           DEGREES FAHRENHEIT
   -----DSS---ZOPEN:  Existing File Opened,  File: PC100.DSS
                      Unit:  71;  DSS Version: 6-JG
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA1/RUNOFF/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA1/RUNOFF/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /NDA1/RUNOFF/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA2/RUNOFF/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA2/RUNOFF/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /NDA2/RUNOFF/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CSD1/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CSD1/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /CSD1/COMBINE/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA3/RUNOFF/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA3/RUNOFF/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /NDA3/RUNOFF/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CSD2/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CSD2/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /CSD2/COMBINE/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA4/RUNOFF/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA4/RUNOFF/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /NDA4/RUNOFF/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CSD3/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CSD3/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /CSD3/COMBINE/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /NDA6/RUNOFF/FLOW/31DEC1998/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   10:  /NDA6/RUNOFF/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /CMB/COMBINE/FLOW/31DEC1998/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   10:  /MBS/BASIN/STOR/02JAN1999/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   30:  /MBS/BASIN/STAGE/01JAN1999/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   28:  /UGB1/BASIN/FLOW/31DEC1998/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   10:  /UGB1/BASIN/FLOW/02JAN1999/2MIN/CALC/
```

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-----DSS---ZWRITE Unit  71; Vers.   28:  /UGB1/BASIN/STOR/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   28:  /UGB1/BASIN/STOR/01JAN1999/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   10:  /UGB1/BASIN/STAGE/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /MUF2/RUNOFF/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   30:  /MUF2/RUNOFF/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /MUF2/RUNOFF/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   27:  /UGB2/BASIN/FLOW/31DEC1998/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   30:  /CSSD2/COMBINE/FLOW/31DEC1998/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   10:  /CSSD2/COMBINE/FLOW/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   26:  /UGB3/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   26:  /UGB3/BASIN/FLOW/01JAN1999/2MIN/CALC/
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-----DSS---ZWRITE Unit  71; Vers.   26:  /UGB3/BASIN/STOR/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /UGB3/BASIN/STOR/02JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   26:  /UGB3/BASIN/STAGE/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   26:  /UGB3/BASIN/STAGE/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.   10:  /UGB3/BASIN/STAGE/02JAN1999/2MIN/CALC/
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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	NDA1	6.	4.07	1.	0.	0.	.00		
+	HYDROGRAPH AT	NDA2	5.	4.03	0.	0.	0.	.00		
+	2 COMBINED AT	CSD1	11.	4.03	1.	0.	0.	.00		
+	HYDROGRAPH AT	NDA3	8.	4.00	1.	0.	0.	.00		
+	2 COMBINED AT	CSD2	19.	4.03	2.	0.	0.	.01		
+	HYDROGRAPH AT	NDA4	8.	4.00	1.	0.	0.	.00		
+	2 COMBINED AT	CSD3	28.	4.03	3.	1.	0.	.01		
+	HYDROGRAPH AT	NDA6	3.	4.00	0.	0.	0.	.00		

```
          2 COMBINED AT
+                         CMB        30.    4.00          3.          1.          0.        .01
          ROUTED TO
+                         MBS        21.    4.10          2.          1.          0.        .01
+                                                                                        1568.06        4.10
          HYDROGRAPH AT
+                        MUF1         7.    4.10          1.          0.          0.        .00
          ROUTED TO
+                        UGB1         0.    4.67          0.          0.          0.        .00
+                                                                                        1583.94        5.03
          HYDROGRAPH AT
+                        MUF2         8.    4.10          1.          0.          0.        .00
          ROUTED TO
+                        UGB2         0.    4.83          0.          0.          0.        .00
+                                                                                        1583.96        5.07
          2 COMBINED AT
+                       CSSD1         1.    4.80          1.          0.          0.        .01
          HYDROGRAPH AT
+                        MUF3         8.    4.10          1.          0.          0.        .00
          2 COMBINED AT
+                       CSSD2         9.    4.10          1.          1.          0.        .01
          ROUTED TO
+                        UGB3         1.    4.77          1.          1.          0.        .01
+                                                                                        1577.94        4.90
          HYDROGRAPH AT
+                        MUF4         8.    4.10          1.          0.          0.        .00
          ROUTED TO
+                        UGB4         0.    4.90          0.          0.          0.        .00
+                                                                                        1577.99        5.07
          2 COMBINED AT
+                       CSSD3         2.    4.87          2.          1.          0.        .02
          HYDROGRAPH AT
+                        MUF5        13.    4.07          1.          0.          0.        .01
          2 COMBINED AT
+                       CSSD4        14.    4.07          3.          1.          1.        .03
          ROUTED TO
+                        UGB5         5.    4.47          3.          1.          1.        .03
+                                                                                        1571.98        4.50
```

```
         HYDROGRAPH AT
+                         MUF6         7.    4.10          1.          0.          0.        .00

         2 COMBINED AT
+                        CSSD5        12.    4.10          3.          1.          1.        .03

         ROUTED TO
+                         UGB6         5.    4.87          3.          1.          1.        .03
+                                                                                          1569.98        4.93

         HYDROGRAPH AT
+                         NDA5         4.    4.03          0.          0.          0.        .00

         HYDROGRAPH AT
+                         ODA3         2.    4.07          0.          0.          0.        .00

         4 COMBINED AT
+                        CC6X3        31.    4.10          6.          2.          1.        .05

         HYDROGRAPH AT
+                         ODA2        42.    4.03          3.          1.          1.        .02

         ROUTED TO
+                         ROSD        42.    4.03          3.          1.          1.        .02

         ROUTED TO
+                       RWASH1        35.    4.10          3.          1.          1.        .02
+                                                                                              .83        4.10

         HYDROGRAPH AT
+                         ODA1       138.    4.07         14.          3.          2.        .07

         ROUTED TO
+                       RWASH3       136.    4.10         14.          3.          2.        .07
+                                                                                             1.61        4.10

         HYDROGRAPH AT
+                         NDA0         5.    4.03          1.          0.          0.        .00

         ROUTED TO
+                       RWASH5         5.    4.03          1.          0.          0.        .00
+                                                                                              .44        4.03

         HYDROGRAPH AT
+                        ODA5N         9.    4.07          0.          0.          0.        .00

         3 COMBINED AT
+                       CWASH3       149.    4.10         15.          4.          3.        .07

         ROUTED TO
+                       RWASH3       145.    4.10         15.          4.          3.        .07
+                                                                                             1.68        4.10
```

```
          HYDROGRAPH AT
+                            ODA4        42.     4.17           3.          1.          0.        .02

          3 COMBINED AT
+                           CC8X3       220.     4.13          21.          5.          4.        .11
1
                                        SUMMARY OF KINEMATIC WAVE - MUSKINGUM-CUNGE ROUTING
                                            (FLOW IS DIRECT RUNOFF WITHOUT BASE FLOW)
                                                                                 INTERPOLATED TO
                                                                                COMPUTATION INTERVAL
            ISTAQ    ELEMENT      DT        PEAK    TIME TO     VOLUME       DT       PEAK    TIME TO     VOLUME
                                                      PEAK                                     PEAK

                                  (MIN)      (CFS)      (MIN)      (IN)      (MIN)      (CFS)      (MIN)        (IN)

               ROSD  MANE           .57      41.79    242.61       1.94       2.00      41.71    242.00         1.93


 CONTINUITY SUMMARY (AC-FT) - INFLOW= .1650E+01 EXCESS= .0000E+00 OUTFLOW= .1652E+01 BASIN STORAGE= .7605E-12 PERCENT ERROR=   -.1



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


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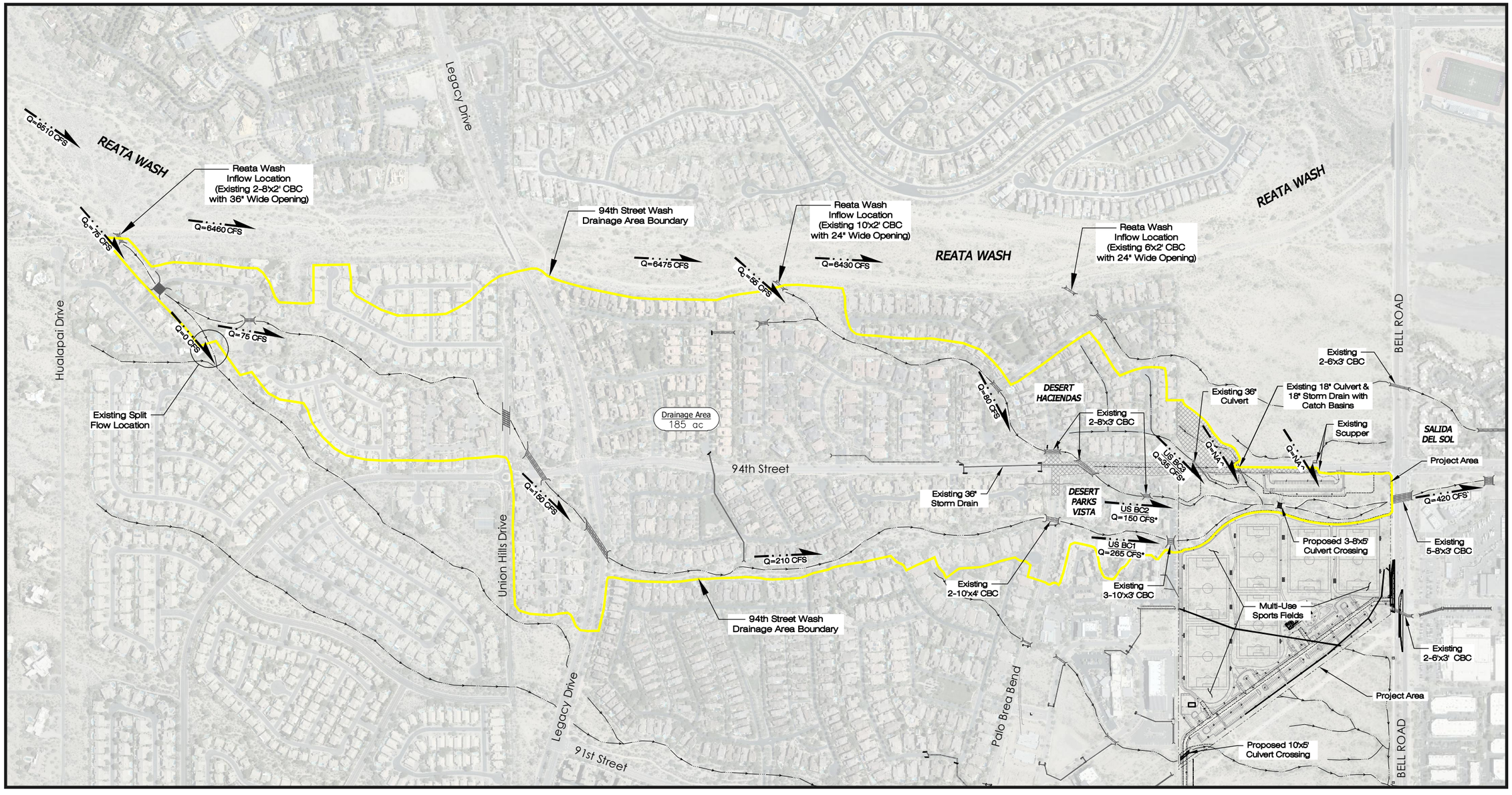
94th Street Wash Drainage Area Map & Inflow Hydrographs

38-DR-2020
11/25/2020



LEGEND

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

0 300' 600'

SCALE: 1" = 600'

94th STREET WASH

DRAINAGE AREA MAP

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 and incorporate teh recommended modifications to the Reata Wash flow diversion culverts that reduce the amount of flow diverted from Reata Wash to the upstream contributing drainage area. 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 F. Refer to the Reata Wash Levee - Flow Diversion Culverts Drainage Memorandum in Appendix E for a detailed analysis of the Reata Wash levee penetration culverts.

^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 420 cfs, obtained from the HEC-RAS model.

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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:	11/20
Drawn By:	JDH/OK
Checked By:	MTG

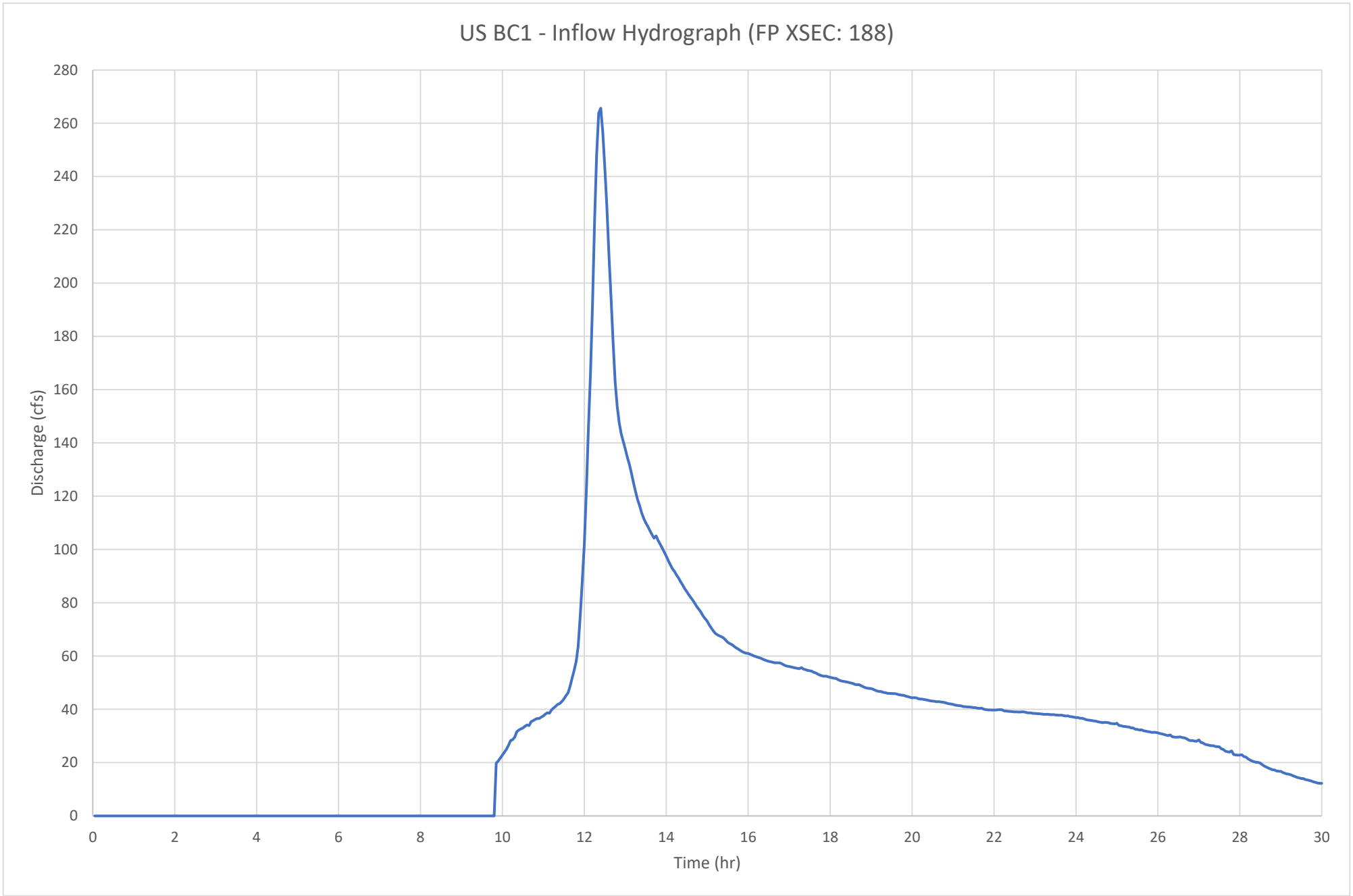
Sheet Title :

94th Street Wash Drainage Area Map

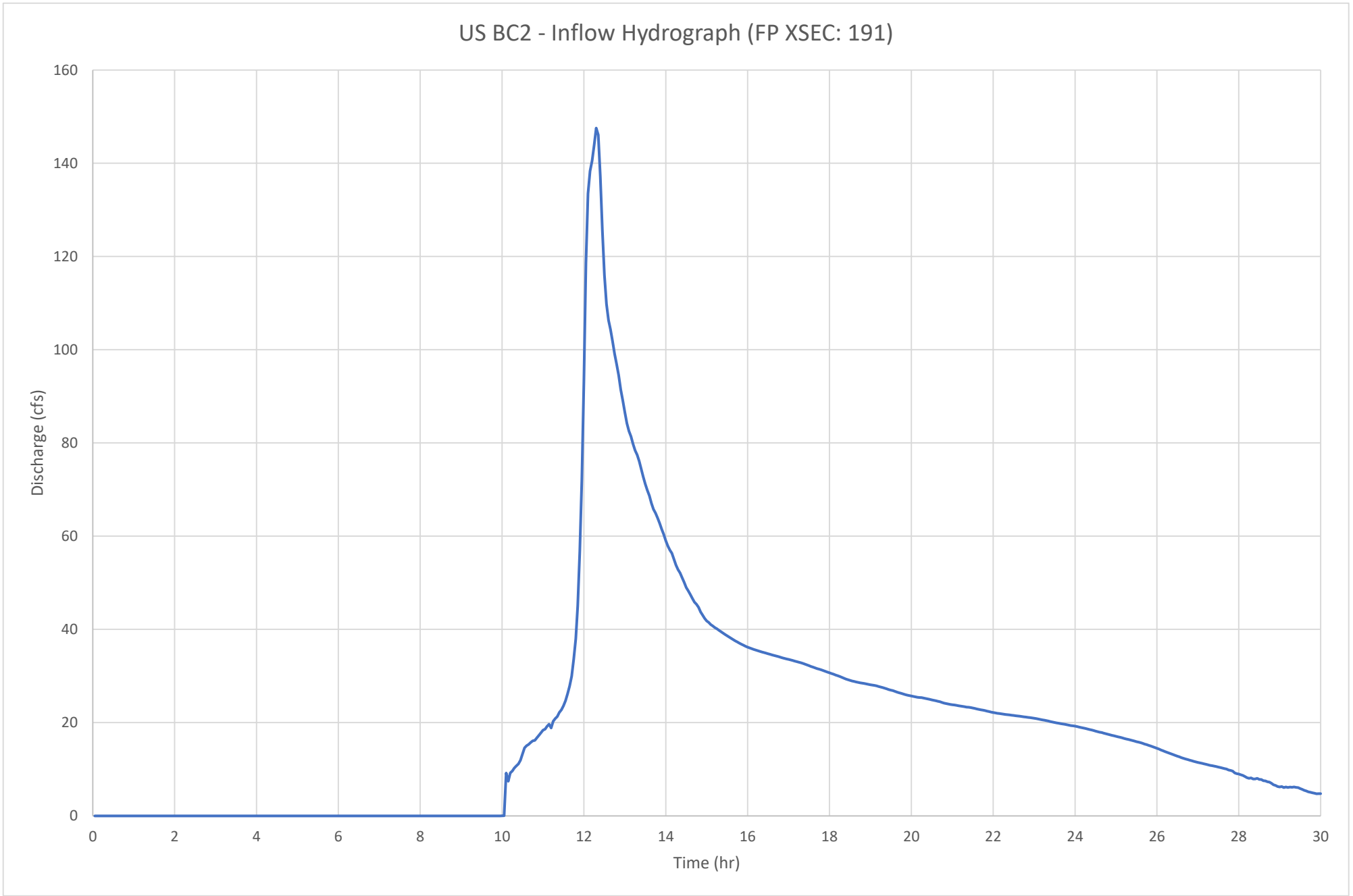
Sheet Number:

1

38-DR-2020
11/25/2020



38-DR-2020
11/25/2020



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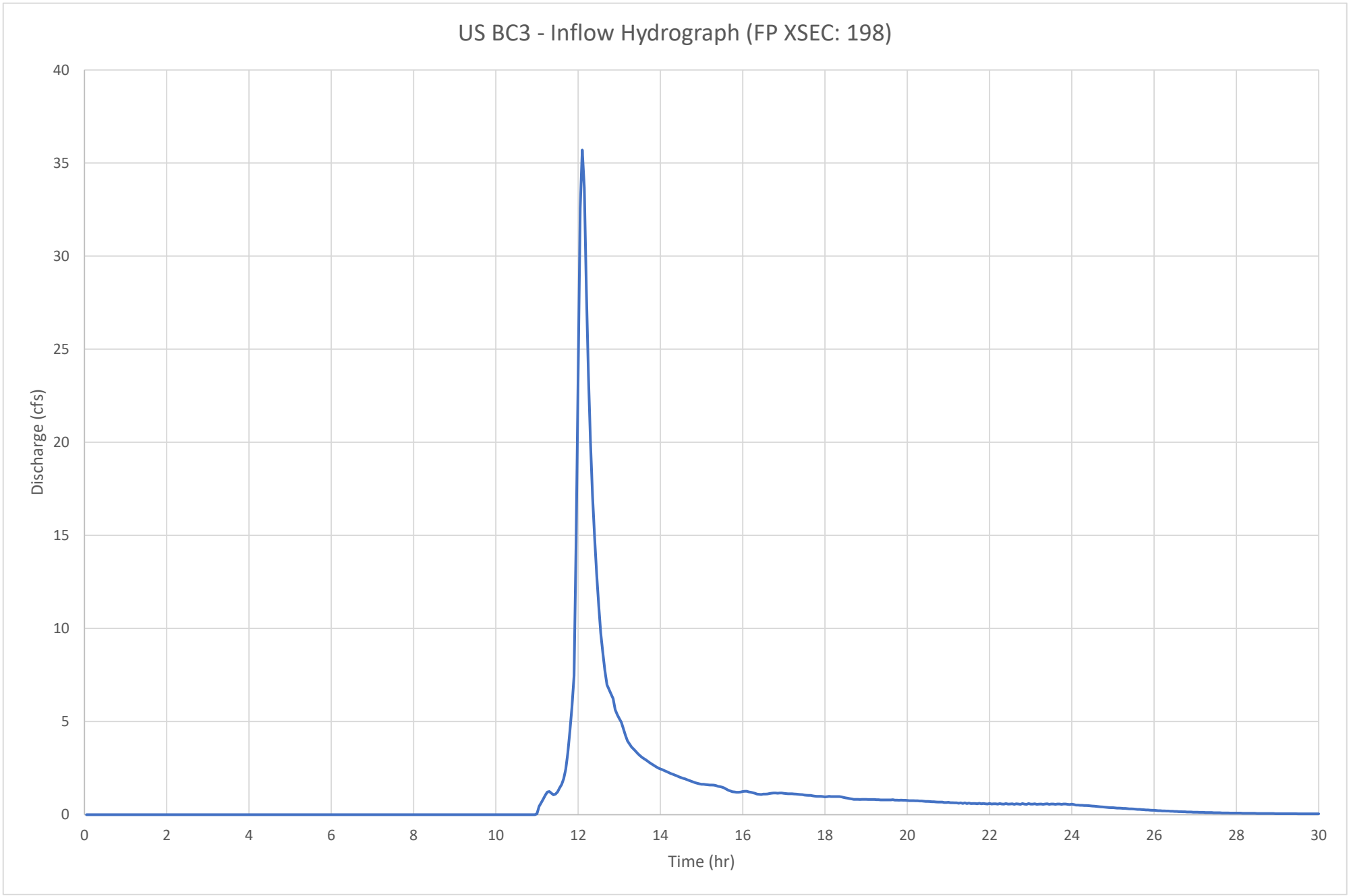
11/25/2020



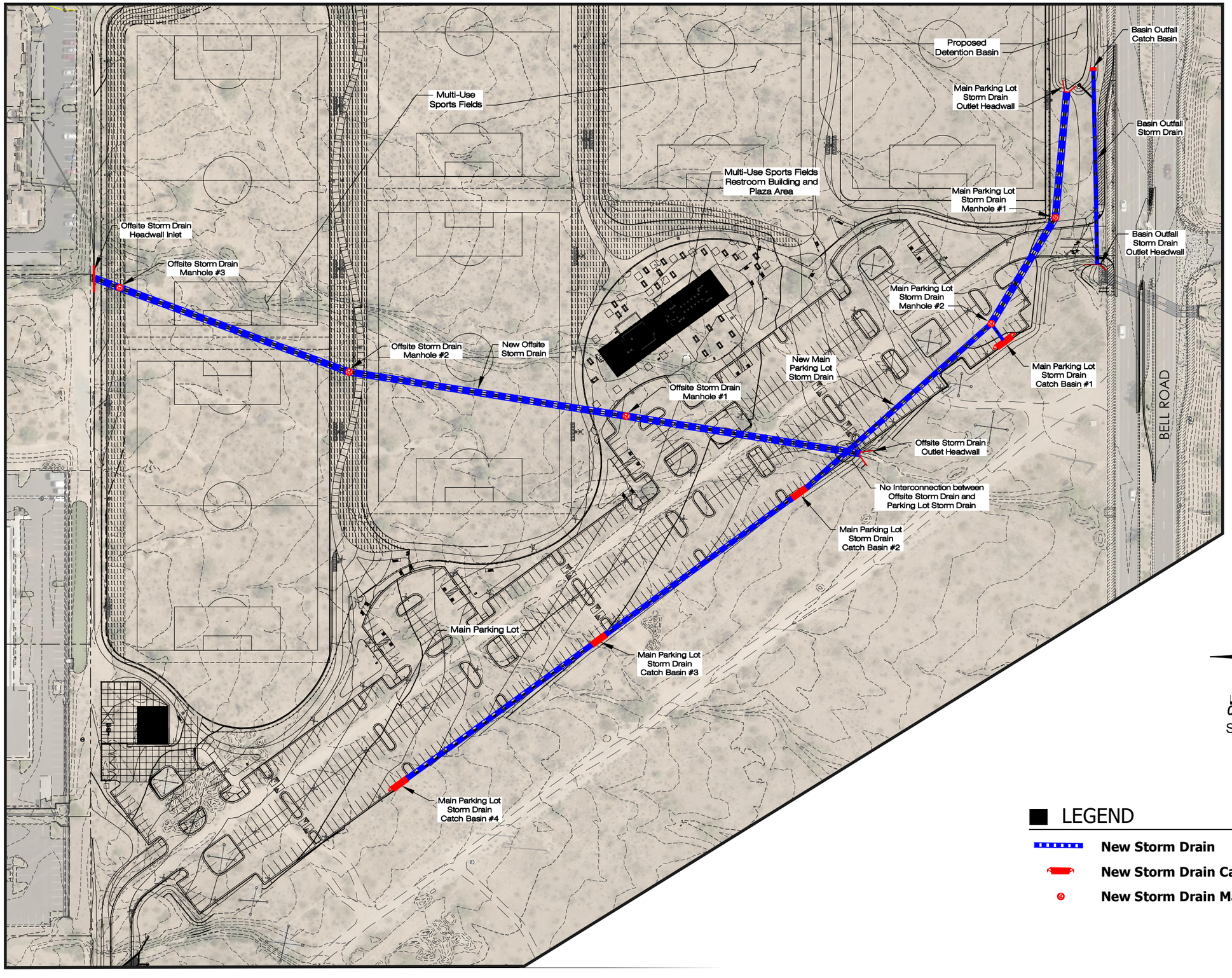
38-DR-2020
11/25/2020

Appendix C: Storm Drain Design Calculations

38-DR-2020
11/25/2020

Storm Drain Design Location Map

38-DR-2020
11/25/2020



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Checked By:	MTG

Sheet Title :

Storm Drain Design Location Map

Sheet Number:

1

38-DR-2020
11/25/2020

Offsite Storm Drain Design Calculations

38-DR-2020
11/25/2020

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*	0.13	1585.09
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	1580.42
Offsite MH#2	Junction Loss (Bend Headloss)^	0.04	1576.23
Offsite MH#1 to Offsite MH#2	Storm Drain Friction Headloss	1.46	1575.59
Offsite MH#1	Junction Loss (Straight-Through Headloss)	0.03	1574.13
Outlet HW to Offsite MH#1	Storm Drain Friction Headloss	1.05	1574.10
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 offsite storm drain 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 storm 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 elevation 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;

$h_o = Outlet\ Headloss\ at\ Manhole$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42.0\ cfs$

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

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

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

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

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

$$h_o = 0.55\ ft$$

$h_o = 0.55\ ft$ @ Outlet *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;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 270\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

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

$D = 3.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

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

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

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

$h_f = S_f L$

$h_f = 0.0039 \times 270$

$h_f = 1.05\ ft$

$$\mathbf{h_f = 1.05\ ft\ (Outlet\ Headwall\ to\ Offsite\ MH\#1)}$$

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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 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*

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

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

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

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

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

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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;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 375\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42.0\ cfs$

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

$D = 3.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

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

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

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

$h_f = S_f L$

$h_f = 0.0039 \times 375$

$h_f = 1.46\ ft$

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

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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;

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

$\gamma = Deflection\ Angle$

$\gamma = 11°$

$k_b = Bend\ loss\ coefficient$

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

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

$Q = 42\ cfs$

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

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

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

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

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

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

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

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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;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 304\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

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

$D = 3.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

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

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

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

$h_f = S_f L$

$h_f = 0.0039 \times 304$

$h_f = 1.19\ ft$

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

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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 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*

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

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

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

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

$h_{mh} = 0.03\ ft$ @ *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;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 34\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 42\ cfs$

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

$D = 3.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

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

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

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

$h_f = S_f L$

$h_f = 0.0039 \times 34$

$h_f = 0.13\ ft$

$h_f = 0.13\ ft$ (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;

$h_i = Headloss\ at\ Pipe\ Entrance$

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

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

$V = Velocity\ of\ Flow$

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

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

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

$$h_i = 0.11\ ft$$

$h_i = 0.11\ ft$ @ Headwall *Inlet*

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Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Nov 18 2020

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

Circular

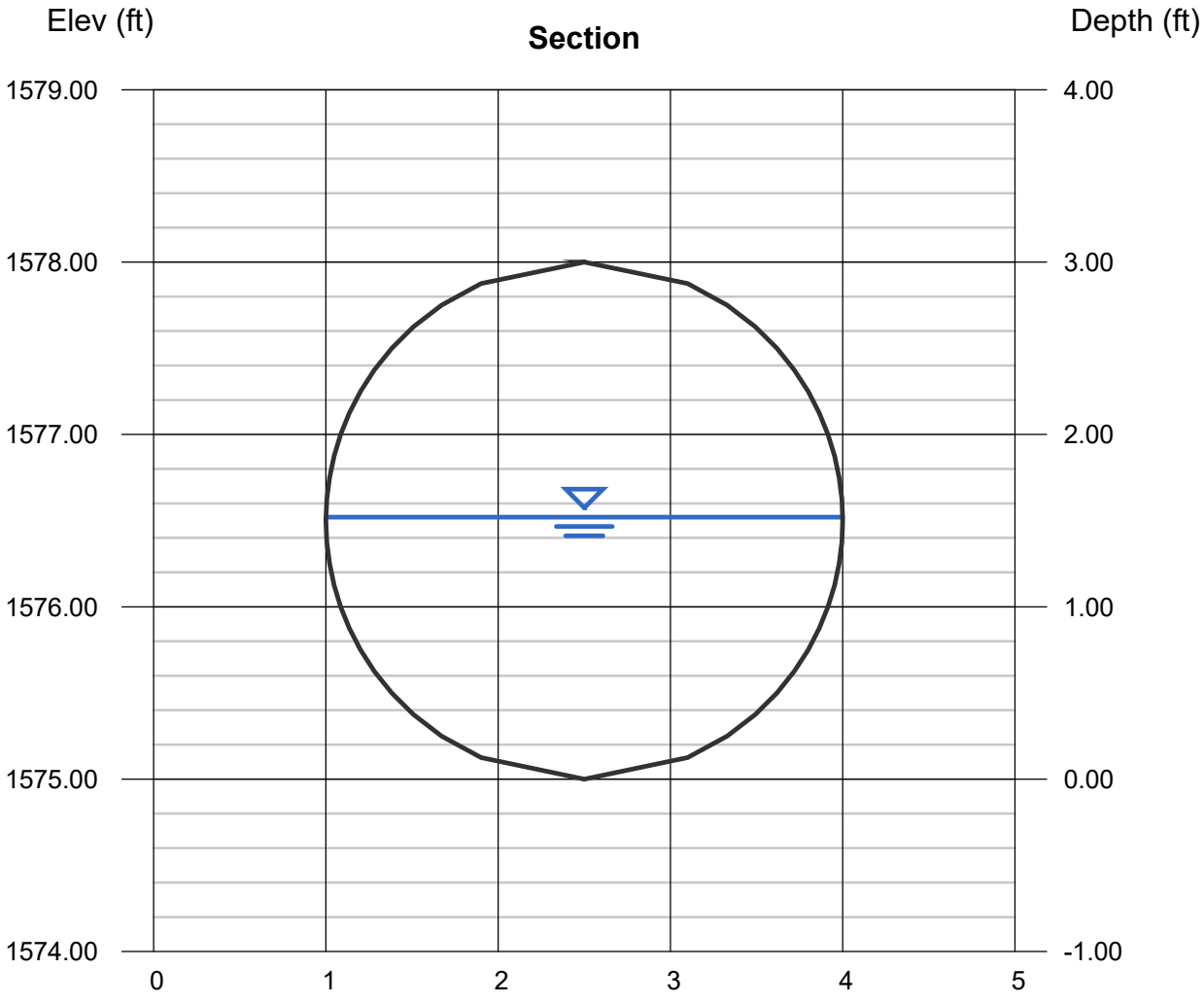
Diameter (ft)	= 3.00
Invert Elev (ft)	= 1575.00
Slope (%)	= 1.53
N-Value	= 0.013

Calculations

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

Highlighted

Depth (ft)	= 1.52
Q (cfs)	= 42.00
Area (sqft)	= 3.60
Velocity (ft/s)	= 11.67
Wetted Perim (ft)	= 4.75
Crit Depth, Yc (ft)	= 2.12
Top Width (ft)	= 3.00
EGL (ft)	= 3.64



Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Wednesday, Nov 18 2020

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

Circular

Diameter (ft)	= 3.00
Invert Elev (ft)	= 1575.00
Slope (%)	= 11.08
N-Value	= 0.013

Calculations

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

Highlighted

Depth (ft)	= 0.89
Q (cfs)	= 42.00
Area (sqft)	= 1.77
Velocity (ft/s)	= 23.76
Wetted Perim (ft)	= 3.46
Crit Depth, Yc (ft)	= 2.12
Top Width (ft)	= 2.74
EGL (ft)	= 9.67

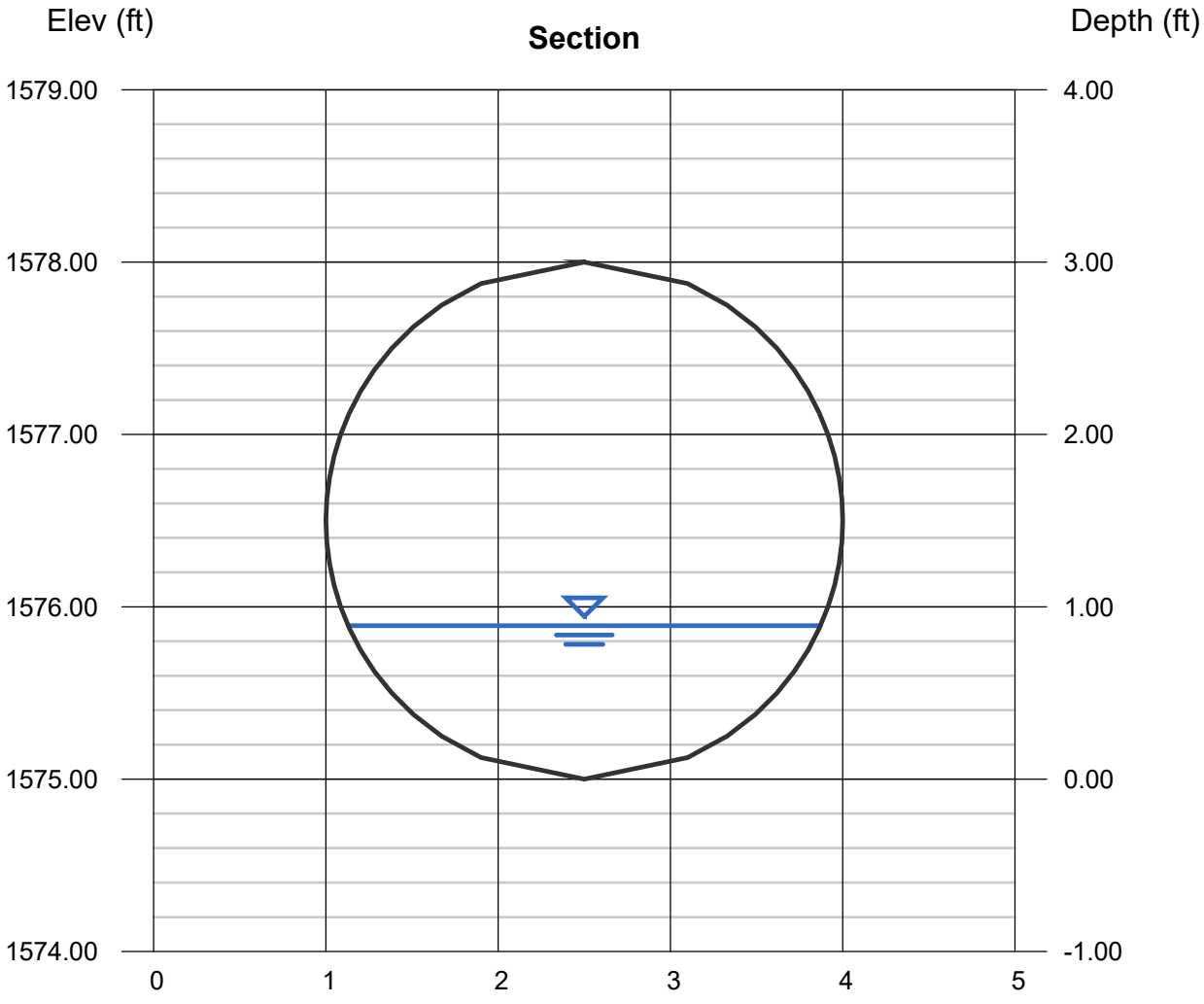


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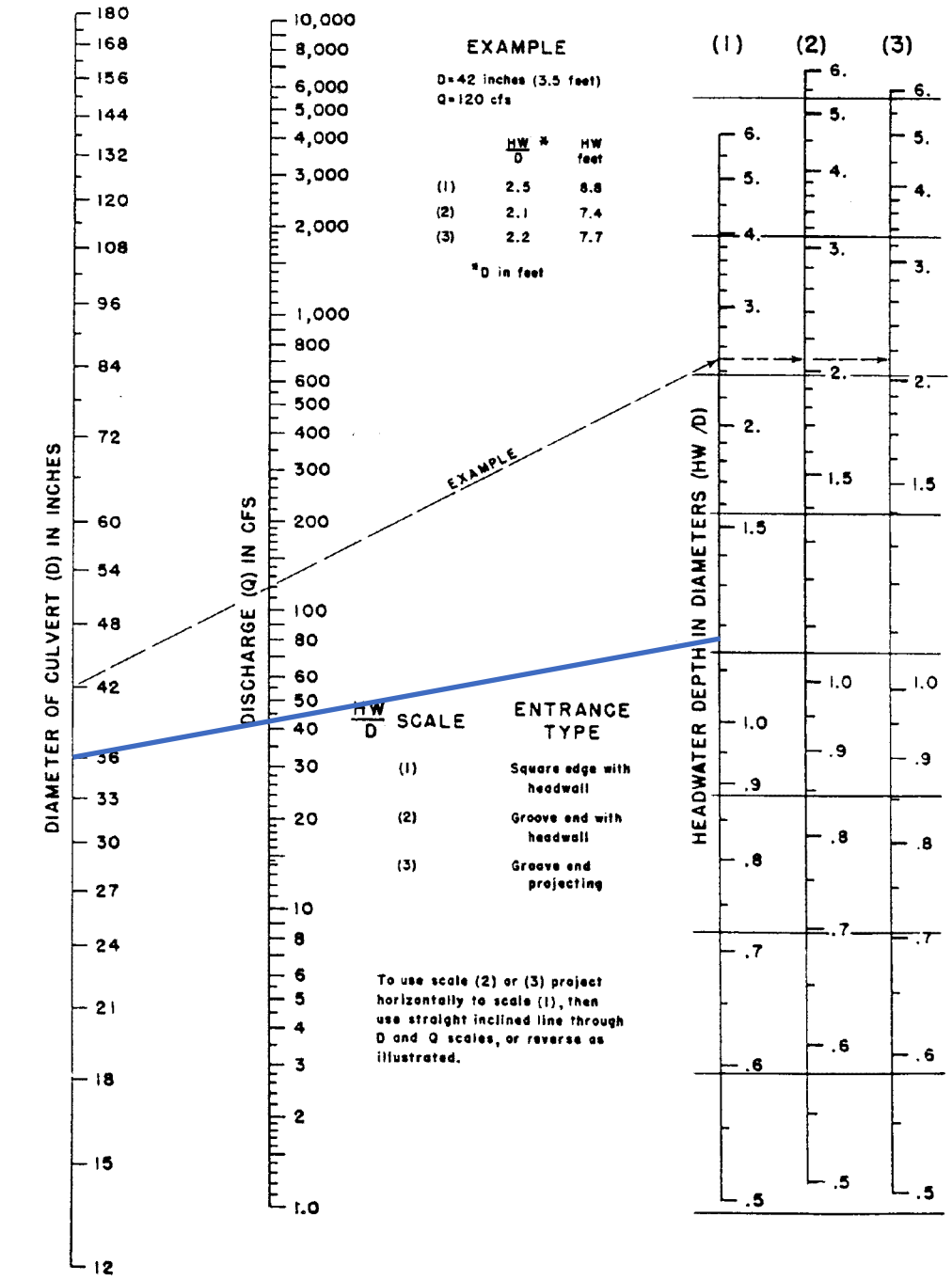
FIGURE 5.20
INLET CONTROL HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS
(USDOT, FHWA, HDS-5, 1985)



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

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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)	0.07	1576.96
CB#3 to CB#4	Storm Drain Friction Headloss	2.55	1576.89
CB#3	Junction Loss (Straight-Through Catch Basin)	0.01	1574.34
CB#2 to CB#3	Storm Drain Friction Headloss	3.15	1574.33
CB#2	Junction Loss (Straight-Through Catch Basin)	0.03	1571.18
MH#2 to CB#2	Storm Drain Friction Headloss	2.24	1571.15
MH#2	Junction Loss (Combined Junction Loss)	0.31	1568.91
MH#1 to MH#2	Storm Drain Friction Headloss	0.28	1568.60
MH#1	Junction Loss (Bend Headloss)	0.03	1568.32
Outlet Headwall to MH#1	Storm Drain Friction Headloss	0.28	1568.29
Outlet Headwall	Junction Loss (Exit Headloss)	0.24	1568.01

Tailwater Elevation @ Outlet Headwall = 1567.77

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 28 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 28 cfs enters the basin with the water level in the detention basin at 1567.77 ft. Therefore, the starting tailwater elevation for the proposed storm drain is 1567.77 ft.

Tailwater Elevation @ *Outlet Headwall* = 1567.77 ft

(Water Level in Detention Basin @ Peak Inflow)

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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;

$h_o = Outlet\ Headloss\ at\ Headwall$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 28.0\ cfs$

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

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{28.0}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.96\ \frac{ft}{s}$$

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

$$h_o = 1.0\frac{3.96^2}{2 \times 32.2}$$

$$h_o = 0.24\ ft$$

$h_o = 0.24\ ft$ @ Outlet *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;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 157\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 28\ cfs$

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

$D = 3.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{28}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.96\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{3.96^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0018$$

$h_f = S_f L$

$h_f = 0.0018 \times 157$

$h_f = 0.28\ ft$

$h_f = 0.28\ ft$ (Outlet *Headwall to MH*#1)

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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;

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

$\gamma = Deflection\ Angle$

$\gamma = 26°$

$k_b = Bend\ loss\ coefficient$

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

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

$Q = 28\ cfs$

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

$D = 3.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{28}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.96\ \frac{ft}{s}$$

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

$$h_{mh} = 0.12 \frac{3.96^2}{2 \times 32.2}$$

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

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

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 154\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 28.0\ cfs$

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

$D = 3.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{28}{\pi \times \left(\frac{3.0^2}{4}\right)} = 3.96\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{3.96^2}{2 \times 32.2 \times 0.750^{\frac{4}{3}}} = 0.0018$$

$h_f = S_f L$

$h_f = 0.0018 \times 154$

$h_f = 0.28\ ft$

$$\boxed{\mathbf{h_f = 0.28\ ft\ \ (MH\#1\ to\ MH\#2)}}$$

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Compute the Combined Headloss at Manhole (MH#2)

At this junction, compute the headloss associated with the Bend Loss at the Manhole and the lateral inflow at the manhole. The combined headloss is the total headloss at this Manhole (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;

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

$\gamma = Deflection\ Angle$

$\gamma = 16°$

$k_b = Bend\ loss\ coefficient$

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

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

$Q = 19\ cfs$

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

$D = 2.0\ ft$

$V = Velocity\ of\ Flow$

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

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

$$h_{mh} = 0.08 \frac{6.05^2}{2 \times 32.2}$$

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

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Junction Loss (Lateral Inflow):

$$h_j = \frac{2\left(Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta\right)}{\left(A_1 + A_2\right)g} + \frac{V_1^2}{2g} - \frac{V_2^2}{2g} \qquad \left(Equation\ 4.10b\right)$$

where;

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

$A_1 = Upstream\ Flow\ Area$

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

$A_2 = Downstream\ Flow\ Area$

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

$Q_1 = Upstream\ Flow\ Rate$

$$Q_1 = 19\ cfs$$

$Q_2 = Downstream\ Flow\ Rate$

$$Q_2 = 28\ cfs$$

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

$$Q_3 = 8\ cfs$$

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

$V_1 = Upstream\ Flow\ Velocity$

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

$V_2 = Downstream\ Flow\ Velocity$

$$V_2 = \frac{Q_2}{A_2} = \frac{28}{7.07} = 3.96\ \frac{ft}{s}$$

$V_3 = Lateral\ Flow\ Velocity$

$$V_3 = \frac{Q_3}{A_3} = \frac{8}{\pi \times \left(\frac{2.00^2}{4}\right)} = 2.55\ \frac{ft}{s}$$

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

$$\theta = 67^\circ\ \left(Figure\ 4.7\right)$$

$$h_j = \frac{2\left(28 \times 3.96 - 19 \times 6.05 - 8 \times 2.55 \times \cos\left(67\right)\right)}{\left(3.14 + 7.07\right)32.2} + \frac{6.05^2}{2 \times 32.2} - \frac{3.96^2}{2 \times 32.2}$$

$$h_j = 0.25\ ft$$

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Total Combined Headloss at Manhole (MH#2)

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

$$h_{mh_{TOTAL}} = 0.05 + 0.25$$

$$h_{mh_{TOTAL}} = 0.31\ ft$$

$$\mathbf{h_{mh} = 0.31\ ft\ @\ MH\#2}$$

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

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

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 320\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 19\ cfs$

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

$D = 2.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

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

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

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

$h_f = S_f L$

$h_f = 0.0070 \times 320$

$h_f = 2.24\ ft$

h_f = 2.24 *ft* (*MH#2 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*

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

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

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

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

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

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

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

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 289\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 11\ cfs$

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

$D = 1.5\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

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

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

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

$h_f = S_f L$

$h_f = 0.0109 \times 289$

$h_f = 3.15\ ft$

$$\boxed{\mathbf{h_f = 3.15\ ft\ \ (CB\#2\ to\ CB\#3)}}$$

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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*

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

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

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

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

$h_{mh} = 0.01\ ft$ @ CB#3

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Compute the Friction Headloss - Proposed 15" Storm Drain (CB#3 to CB#4)

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

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 296\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 6\ cfs$

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

$D = 1.25\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{6}{\pi \times \left(\frac{1.25^2}{4}\right)} = 4.89\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{1.25}{4} = 0.313\ ft$$

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{4.89^2}{2 \times 32.2 \times 0.313^{\frac{4}{3}}} = 0.0086$$

$h_f = S_f L$

$h_f = 0.0086 \times 296$

$h_f = 2.55\ ft$

$$\mathbf{h_f = 2.55\ ft\ \ (CB\#3\ to\ CB\#4)}$$

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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;

$h_i = Headloss\ at\ Pipe\ Entrance$

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

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{6}{\pi \times \left(\frac{1.25^2}{4}\right)} = 4.89\ \frac{ft}{s}$$

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

$$h_i = 0.20\,\frac{4.89^2}{2 \times 32.2}$$

$$h_i = 0.07\ ft$$

$h_i = 0.07\ ft$ @ CB#4

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Main Parking Lot Storm Drain Inlet Sizing Calculations

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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}=8 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

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Catch Basin Design @ CB#1

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;

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

$Spill\ Elev = Low\ Point\ in\ the\ Curb\ Elevation\ at\ Catch\ Basin = 1570.00\ ft$

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

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.

$Top\ of\ Curb\ Elevation\ at\ Catch\ Basin = 1570.00\ ft$

$Weir\ Elev = 1570.00\ ft - 4.75\ in = 1569.60\ ft$

$d = Spill\ Elev - Lip\ Elev$

$d = 1570.00 - 1569.60$

$\boxed{d = 0.40\ ft}$

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

$h = 5\ in$

$\boxed{h = 0.42\ ft}$

$d < h$

$0.40\ ft < 0.42\ ft$

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;

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

$C_w = Weir\ Flow\ Coefficient = 2.3$

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

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

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

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

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

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

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

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

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

$Q_i = 9.16$

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

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 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.

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Connector Pipe Sizing:

Determine the total (friction + inlet) headloss:

$h_f = S_f L$ (Equation 4.6 - Drainage Design Manual for Maricopa County, Hydraulics)

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

Try a 24 inch (d = 2.00 ft) Pipe:

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

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

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

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

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

$Q = Flow\ Rate\ in\ Pipe = 8\ cfs$

$A = Cross-Section\ area\ of\ Pipe = \pi\frac{d^2}{4} = \pi\frac{2.00^2}{4} = 3.14\ ft^2$

$R_h = Hydraulic\ Radius = \frac{A}{W_p} = \frac{\pi\frac{d^2}{4}}{\pi d} = \frac{d}{4} = \frac{2.0}{4} = 0.50\ ft$

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

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

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

$[K = 0.0049]$

$V = \frac{Q}{A} = \frac{8}{3.14}$

$\left[V = 2.55\ \frac{ft}{s}\right]$

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$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = (0.0049)\frac{2.55^2}{2\,(32.2)(0.50)^{\frac{4}{3}}}$$

$$\left[S_f = 0.0012\right]$$

Friction Headloss:

$$h_f = S_f L$$

$$h_f = (0.0012)(28)$$

$$\left[h_f = 0.0\,ft\right]$$

Inlet and Manhole Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{2.55^2}{2\,(32.2)}\right)$$

$$\left[h_i = 0.1\,ft\right]$$

Total Headloss:

$$h_t = h_f + h_i$$

$$h_t = 0.0 + 0.1$$

$$h_t = 0.1$$

$$\boxed{h_t = 0.1\,ft}$$

Available Head: h_a

Upstream HW Elevation: 1569.00 *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 1569.50 ft.

Downstream HW Elevation: 1568.91 *ft* (Storm Drain HGL at Connection Manhole (MH#2))

$$h_a = Upstream\ HW - Downstream\ HW$$

$$h_a = 1569.00 - 1568.91$$

$$h_a = 0.09$$

$$\boxed{h_a = 0.1\,ft}$$

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

The 24-inch connector pipes have sufficient capacity to convey the intercepted flow

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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;

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

$Spill\ Elev = Low\ Point\ in\ the\ Curb\ Elevation\ at\ Catch\ Basin = 1573.37\ ft$

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

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.

$Top\ of\ Curb\ Elevation\ at\ Catch\ Basin = 1573.37\ ft$

$Weir\ Elev = 1573.37\ ft - 4.75\ in = 1572.97\ ft$

$d = Spill\ Elev - Lip\ Elev$

$d = 1573.37 - 1572.97$

$\boxed{d = 0.40\ ft}$

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

$h = 5\ in$

$\boxed{h = 0.42\ ft}$

$d < h$

$0.40\ ft < 0.42\ ft$

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;

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

$C_w = Weir\ Flow\ Coefficient = 2.3$

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

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

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

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

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

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

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

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

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

$$Q_i = 9.16$$

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

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.

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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.

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

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

$d = Spill\ Elev - Lip\ Elev$

$d = 1575.40 - 1575.10$

$\boxed{d = 0.30\ ft}$

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

$h = 5\ in$

$\boxed{h = 0.42\ ft}$

$d < h$

$0.30\ ft < 0.42\ ft$

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)

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

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

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

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

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

$$Q_i = 5.95$$

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

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.

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Catch Basin Design @ CB#4

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;

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

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

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

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.

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

$Weir\ Elev = 1579.00\ ft - 4.75\ in = 1578.60\ ft$

$d = Spill\ Elev - Lip\ Elev$

$d = 1578.90 - 1578.60$

$\boxed{d = 0.30\ ft}$

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

$h = 5\ in$

$\boxed{h = 0.42\ ft}$

$d < h$

$0.30\ ft < 0.42\ ft$

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)

$L_f = 0.80 \times L$

$L_f = 0.80 \times 10.0$

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

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

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

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

$$Q_i = 5.95$$

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

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 equal to 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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Basin Outfall Storm Drain Design Calculations

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Basin Outfall Storm Drain Hydraulic Grade Line (HGL) Summary Table

Location	Type of Headloss	Headloss (ft)	HGL Elevation (ft)
Catch Basin	Junction Loss (Entrance Headloss)	0.08	1565.90
Outlet Headwall to Catch Basin	Storm Drain Friction Headloss	1.22	1565.82
Outlet Headwall	Junction Loss (Exit Headloss)	0.40	1564.60

Tailwater Elevation @ Outlet Headwall = 1564.20

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 detention basin outfall storm drain was designed to convey the 100-year 6-hour peak discharge from the new detention basin located north of Bell Road, just east of the entrance drive to the headwall of the existing dual 6'x3' concrete box culvert just west of the entrance drive. The primary basin outfall is through a raised grate catch basin that has a lip elevation at 1567.00 ft, 2 feet above the basin bottom. Below the lip elevation, the basin is drained through a low flow bleed-off catch basin and pipe to an elevation of 1565.50 feet. The lowest 6 inches of the basin infiltrates through the basin bottom. During the 100-year design storm, the basin fills completely to an elevation of 1568.00 feet. At this point, the peak flow out of the basin, through the outfall storm drain was calculated to be 16 cfs (HEC-1 Basin Route: MBS). Refer to Appendix B for the HEC-1 Hydrologic Model Results.

Determine Tailwater Elevation:

The new basin outfall storm drain discharges at the upstream end of the existing dual 6'x3' concrete box culvert that crosses underneath Bell Road, just west of the entrance drive. With the construction of the proposed Mult-Use Sports fields, the majority of the upstream watershed to the culvert has been cut off and diverted through the detention basin. Therefore, no tailwater is expected to impact the hydraulic performance of the basin outfall storm drain. A conservative starting tailwater elevation, equal to the soffit of the proposed basin outfall storm drain was taken. The invert of the new 24" basin outfall storm drain at the outfall is 1562.20 ft. Therefore, the starting tailwater elevation for the proposed storm drain is 1564.20 ft.

Tailwater Elevation @ *Outlet Headwall* = 1564.20 ft
(Soffit Elevation of Proposed Basin Outfall Storm Drain at Outlet Headwall)

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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;

$h_o = Outlet\ Headloss\ at\ Headwall$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 16.0\ cfs$

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

$D = 2.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi \times \left(\frac{D^2}{4}\right)} = \frac{16.0}{\pi \times \left(\frac{2.0^2}{4}\right)} = 5.09\ \frac{ft}{s}$$

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

$$h_o = 1.0\frac{5.09^2}{2 \times 32.2}$$

$$h_o = 0.40\ ft$$

$h_o = 0.40\ ft$ @ Outlet *Headwall*

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Compute the Friction Headloss - Proposed 24" Basin Outfall Storm Drain (Outlet Headwall to Catch Basin)

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

where;

$h_f = Friction\ Headloss$

$L = Length\ of\ Storm\ Drain$

$L = 243\ ft$

$Q = Storm\ Drain\ Design\ Discharge$

$Q = 16\ cfs$

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

$D = 2.0\ ft$

$S_f = Friction\ Slope$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{16}{\pi \times \left(\frac{2.0^2}{4}\right)} = 5.09\ \frac{ft}{s}$$

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

$$S_f = K\frac{V^2}{2gR^{\frac{4}{3}}} = 0.0049\ \frac{5.09^2}{2 \times 32.2 \times 0.500^{\frac{4}{3}}} = 0.0050$$

$h_f = S_f L$

$h_f = 0.0050 \times 243$

$h_f = 1.22\ ft$

$h_f = 1.22\ ft$ (Outlet *Headwall to* Catch *Basin*)

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Compute the Entrance Headloss at Catch Basin

Entrance Loss:

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

where;

$h_i = Headloss\ at\ Pipe\ Entrance$

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

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{16}{\pi \times \left(\frac{2.00^2}{4}\right)} = 5.09\ \frac{ft}{s}$$

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

$$h_i = 0.20\ \frac{5.09^2}{2 \times 32.2}$$

$$h_i = 0.08\ ft$$

$$\mathbf{h_i = 0.08\ ft\ @\ Catch\ Basin}$$

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Appendix D: Culvert Design & Wash Hydraulic Analysis

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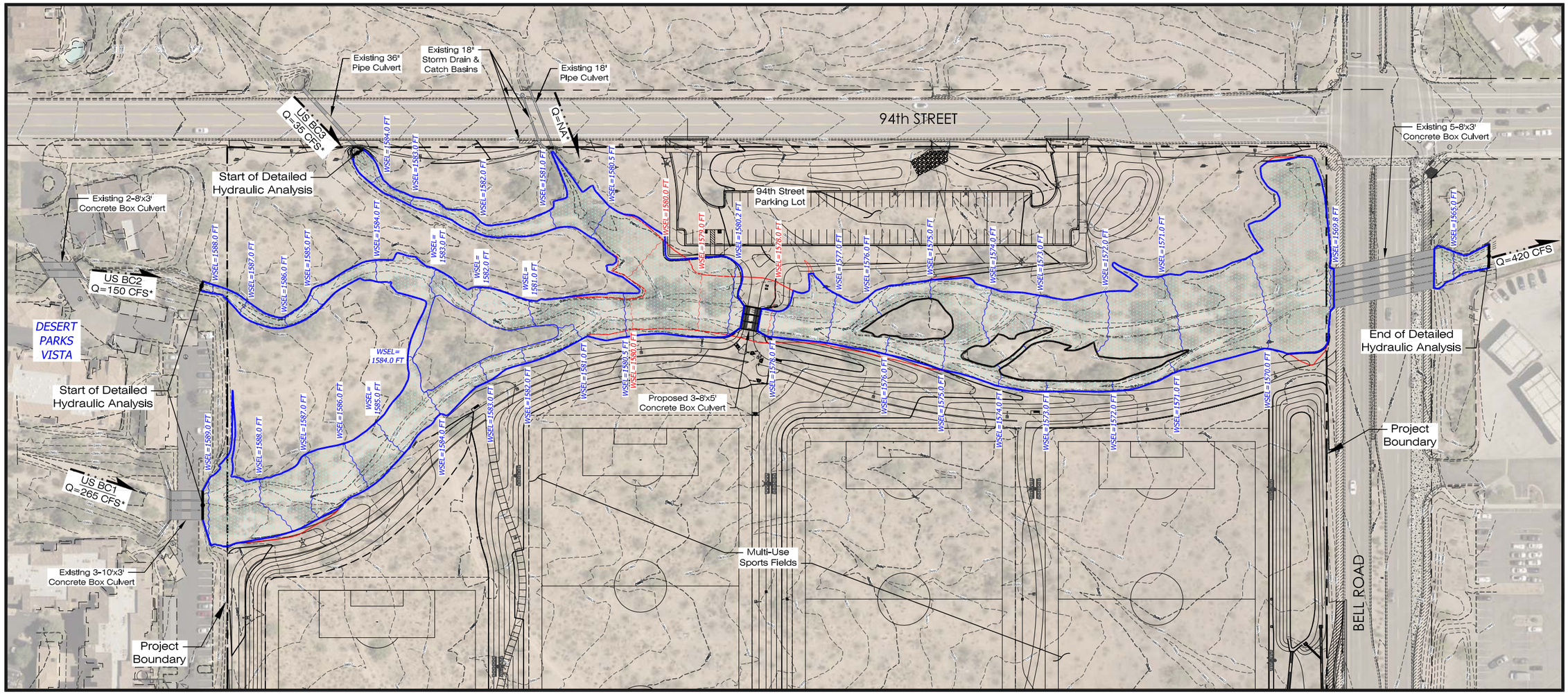

LEGEND

- **Proposed Condition Inundation Boundary**
- **Proposed Condition Water Surface Elevation**
- **Existing Condition Inundation Boundary**
- **Existing Condition Water Surface Elevation**
- Q=265 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 by Gavan & Barker Inc. to better represent the upstream flow conditions and incorporate the recommended modifications to the Reata Wash flow diversion culverts that reduce the amount of flow diverted from Reata Wash to the upstream contributing drainage area. Refer to the Reata Wash Levee - Flow Diversion Culverts Drainage Memorandum in Appendix E for more information as well as the Digital Data in Appendix F 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 265 cfs and the east fork with a peak discharge of 150 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
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:	11/20
Drawn By:	JDH/OK
Checked By:	MTG

Sheet Title :
94th Street Wash Culvert Hydraulic Analysis

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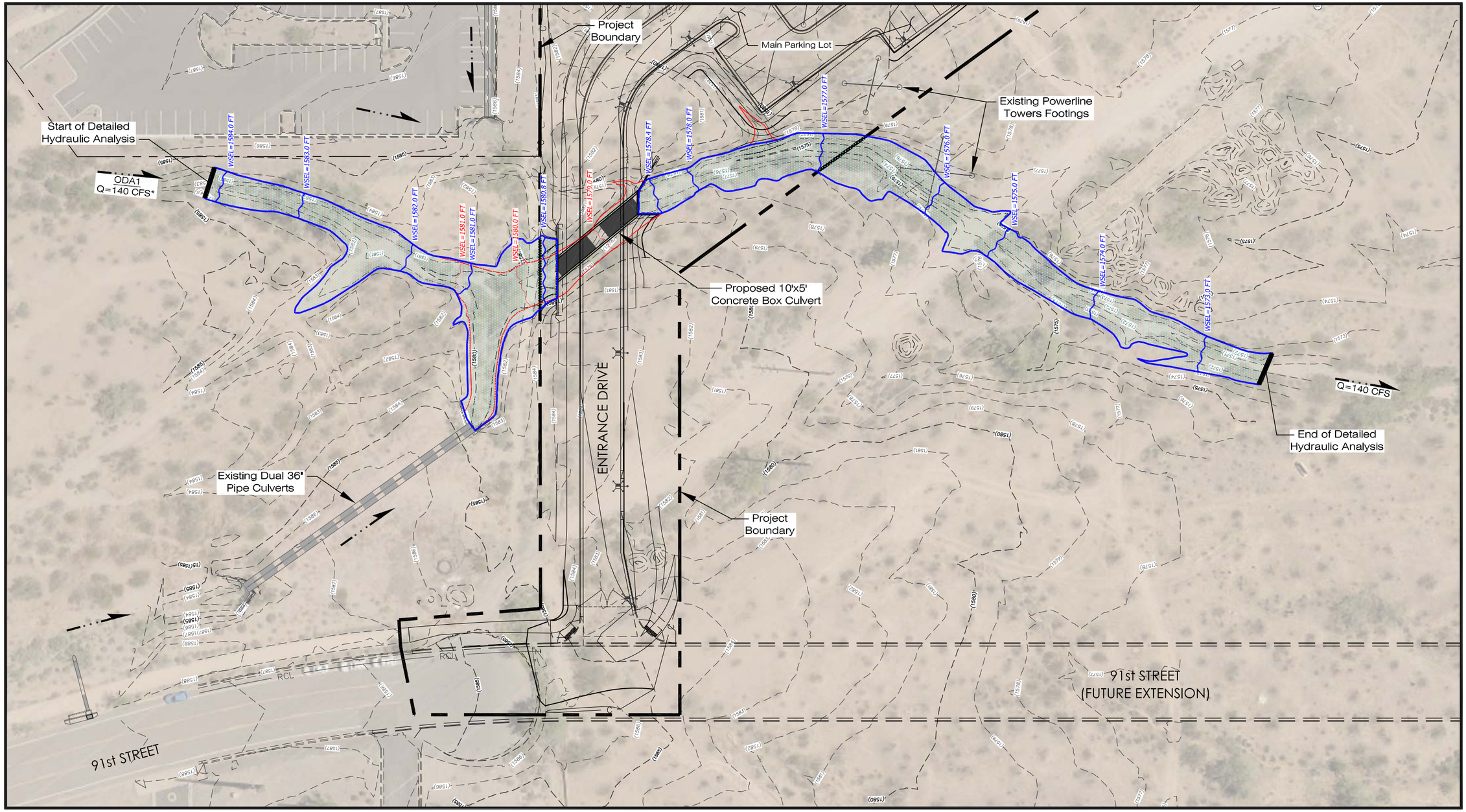

LEGEND

- **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'

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Civil Engineering
Landscape Architecture

Project :

Multi-Use Sports Fields
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Sheet Title :

91st Street Wash Culvert Hydraulic Analysis

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Appendix E: Reata Wash Levee – Flow Diversion Culverts Drainage Memorandum

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Memorandum

To:	**Joe Phillips, PE**
From:	**Mark Gavan, PE and Omer Karovic, PE**
Date:	**October 16, 2020**
Project:	**Multi-Use Sports Fields** **Bell Road and 94th Street**
Subject:	**Reata Wash Levee – Flow Diversion Culverts**

PURPOSE OF MEMORANDUM

The purpose of this memorandum is twofold. One is to document the hydrologic and hydraulic analysis that was performed to determine the amount of flow diverted at each of the three existing culverts that penetrate the Reata Wash levee. The second purpose is to recommend modifications to reduce the amount of flow diverted at each culvert to mitigate flooding and sediment deposition in the washes downstream of the diversions.

APPENDICES

- **Appendix A –** 100-Year Peak Flow Exhibit (Existing vs. Proposed Conditions)
- **Appendix B –** As-Built Plans (Reata Wash Levee and Culverts)
- **Appendix C –** Wash Low Flow Hydraulic Analysis
- **Appendix D –** Existing vs. Proposed 100-Year Flood Hydrographs

INTRODUCTION / PROBLEM DESCRIPTION

The impetus for this analysis is the repeated flooding and sediment deposition problems that occur along the washes downstream of the Reata Wash flow diversion culverts between Hualapai Drive and Bell Road. Each of the three culverts discharge to sandy bottom washes that are susceptible to erosion and sedimentation. This is particularly problematic upstream of Bell Road where sedimentation in the washes has nearly clogged several roadway culverts. In addition to the sediment deposition issues, flooding has also been a problem. In October of 2018, significant flooding occurred along 94th Street, south of Bell Road, which threatened homes within the Salida del Sol apartment complex on the southeast corner of Bell Road and 94th Street.

Refer to the Exhibit in Appendix A for the location of the three culverts that divert flows out of Reata Wash. The North culvert, which lies just south of Hualapai Drive is a dual 8'x2' concrete box culvert. The Center culvert, located approximately 1,400 feet south of Legacy Drive within the Sera Brisa subdivision is a single barrel 10'x2' concrete box culvert and the South culvert is a 6'x2' box culvert which is located within the Desert Haciendas subdivision about 3,000 feet south of Legacy Drive. The invert of the North culvert is about 2 feet lower than the flowline of Reata Wash, whereas the invert of the Center culvert is about equal to the wash bottom and the invert of

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the South culvert is about 2 feet above the wash. The following photographs of the three culverts were taken during a site visit made on August 26, 2020.

The Reata Wash levee and the three flow diversion culverts were constructed by the DC Ranch developer to provide flood protection for the community. The drainage report was not found, but from inspection of the as-built plans it appears that the culverts were installed to mimic the pre-existing conditions in locations where flow would split out of the Reata Wash. The flow splits were identified as "404 Washes" on the as-built plans. While we did not obtain a copy of the 404 permit, it seems clear that the levee penetrating culverts were installed to preserve flows in the 404 jurisdictional washes that would otherwise be cut off with the construction of the levee. According to information on the as-built plans, the North double-barrel 8'x2' box culvert was designed to divert 150 cfs during the 100-year flood on Reata Wash which was identified as 13,370 cfs on the as-built plans. Similarly, the Center 10'x2' culvert was designed to divert 140 cfs, while the South 6'x2'culvert was designed to divert 60 cfs. Refer to Appendix B for the as-built plans.

Reata Wash looking downstream at North Culvert: Berm built to direct flow away from culvert entrance

Upstream View of North Culvert

Downstream View of North Culvert

Split flow location downstream of North Culvert, looking downstream:

North Culvert Photographs

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Reata Wash looking Downstream at Center Culvert:
Berm built to direct flow away from culvert entrance

Upstream View of Center Culvert

Downstream View of Center Culvert

View of Downstream Wash
Looking Upstream towards Center Culvert

Center Culvert Photographs

The City of Scottsdale commissioned the *Reata Wash Flood Control Improvement Study* which was prepared by Wood, Patel & Associates and is dated November 2, 2016. It indicates that this segment of Reata Wash has sufficient capacity to convey both the existing conditions and proposed conditions, 100-year flood. According to the study, the existing conditions flow is about 14,000 cfs and the proposed conditions flows is approximately 12,000 cfs. The study report acknowledges the existence of the three diversion culverts, but specifically states that they were ignored in the hydraulic analysis. In other words, there was no reduction made to the Reata Wash peak discharge based on flow being diverted through three culverts. Therefore, the flow through the diversion culverts can be reduced without negatively impacting the hydraulic conditions on Reata Wash.

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View looking East at Reata Wash from atop the Levee at the South Culvert

Upstream View of South Culvert

Downstream View of South Culvert

Dual 36-inch Pipe Culverts Downstream of South Culvert

South Culvert Photographs

HYDROLOGIC AND HYDRAULIC ANALYSIS

A hydrologic analysis was performed to estimate the 100-year flows that pass through the diversion culverts and the accumulation of flows in the washes downstream of the culverts, including peak flows through the proposed multi-use sports fields. The hydrologic analysis was performed using 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 hydrologic and hydraulic analysis was done in accordance with the City of Scottsdale *Drainage Standards and Policies Manual (DSPM)* with procedures outlined in the Flood Control District of Maricopa County *Hydraulics Manual.*

Existing Conditions Analysis

The PPS ADMS 100-year, 24-hour FLO-2D model was reviewed and modified to better represent existing flow conditions. Two primary modifications were made to the existing model. The first

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was to adjust the rating tables and invert elevations of the Reata Wash diversion culverts and the other was to adjust grid elevations in the local watershed area downstream of the culverts. Grid adjustments in the local watershed were made to better represent existing drainage patterns. There were locations in the FLO-2D model where runoff was erroneously breaking out of washes. To better represent the flow in the watershed, grid elevations were adjusted based on inspection of contour mapping, aerial photography, and as-built plans.

In addition to the grid adjustments in the local watershed, the FLO-2D rating tables for the three levee culverts in the FLO-2D model were adjusted (i.e. the Hydraulic Structure rating tables in the HYSTRUCT.DAT file were modified). The original rating tables were found to be acceptable but were slightly different from the ones calculated using the procedures outlined in the District's Hydraulics manual. Since new rating tables for the proposed conditions had to be developed, the existing conditions rating tables were also updated for consistency. Additionally, the upstream and downstream FLO-2D model culvert invert elevations at all three locations were adjusted to match the as-built plans for the culverts. No clogging factors were assigned to the culverts, even though the North culvert is currently about two-thirds full of sediment as evidenced by the photographs. Clogging factors were not assigned because sediment could be scoured out of the culverts during a major flood event on Reata Wash.

As can be seen in Table 1, the existing conditions peak flow through the North, double-barrel 8'x2' culvert is 375 cfs with a discharge volume of 117 acre-feet. The peak flow through the 10'x2' Center culvert is 270 cfs with a discharge volume of 93 acre-feet and the peak flow through the South 6'x2' culvert is 90 cfs with a discharge volume of 9 acre-feet. Compared to the design discharges, the existing condition peak flow rates at each of the three Reata Wash levee culverts are significantly greater. Refer to Appendix A for a Peak Flow exhibit showing the existing conditions peak discharges at pertinent locations and to Appendix D for the existing conditions flood hydrographs at the three culverts.

Reata Wash Levee Penetration	Design Peak Discharge	Peak Discharges		Volumes	
		Ex. Con.	Prop. Con.	Ex. Con.	Prop. Con.
	(cfs)	*(cfs)*	*(cfs)*	*(ac-ft)*	*(ac-ft)*
North Culvert	150	375	75	117	64
Center Culvert	140	270	55	93	36
South Culvert	60	90	30	9	4

Table 1: 100-Year Flow Summary Table – Reata Wash Diversion Culverts

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Proposed Conditions Analysis

Under proposed conditions, the size of the culvert openings is reduced to restrict the amount of flow that is diverted out of Reata Wash. Reducing the culvert openings will help mitigate the sedimentation and flooding problems that are occurring within the downstream washes.

The only difference between the existing and proposed conditions hydrologic analysis are the necessary adjustments to the three culvert rating tables. The existing conditions rating tables were calculated based on the full geometry of the existing culverts, ignoring potential clogging from sediment and debris, whereas the proposed conditions rating tables are based on the size of the reduced culvert opening, also ignoring clogging.

The reduction in the culvert opening is based on the bank full capacity calculations that were done for each one of the three downstream washes, just downstream of the diversion culverts. While this represents a significant reduction in the exisitng conditions flows, it is still enough flow to fill the jurisdictional width of the washes and help maintain the riparian habitat. The low-flow wash analysis is based on a depth of flow of one (1) foot, the slope of the wash flowline and the width of the sandy bottom. This hydraulic analysis found that the low-flow conveyance capacity of the wash downstream of the North culvert is about of 80 cfs, while the washes downstream of the Center and South culverts have a low-flow conveyance capacity of 60 and 50 cfs, respectively. Refer to the exhibit in Appendix A for the location of the cross sections used in the hydraulic analysis and to Appendix C for the normal depth calculations.

Once the low-flow conveyance capacity of the downstream washes was determined, the proposed culvert openings were sized to limit the flow through the culverts to an amount that is near but doesn't exceed the low flow capacity. Through a "trail and success" iterative approach it was found that reducing the North culvert opening from 16 feet wide to 3 feet wide, will provide a culvert flow of 75 cfs during the 100-year, 24-hour storm event. This reduction in width can be achieved in two ways; either by installing two separate 18-inch openings that match the height of the existing culvert or by providing one 36-inch wide opening on one barrel and completely blocking the second barrel. The analysis found that for the Center culvert, the upstream opening should be reduced from 10 feet wide to 2 feet, which will reduce the peak discharge through the culvert to 55 cfs during the 100-year, 24-hour storm event. For the South culvert, the upstream opening should be reduced from 6 feet wide to 2 feet, which will reduce the peak discharge through the culvert to 30 cfs. Refer to Table 1 for the summary of proposed conditions peak discharges and volumes through the three levee penetrating culverts.

These reduced flows, while sufficient to help maintain vegetative growth within the receiving washes, are significantly lower than under existing conditions, reducing the sedimentation problems and flooding potential in the downstream watershed. However, due to the relatively large 220-acre local watershed area, the peak discharge that concentrate at Bell Road is still significant. As can be seen in Table 2, the 100 year, 24-hour peak discharge at the five (5) barrel 8'x3' Bell Road box culvert just west of 94th Street is only reduced by about 25%, from 590 cfs to 425 cfs. The reduction in peak discharge at the dual 6'x3 box culvert just east of 94th Street is more significant, reduced from 100 cfs to 55 cfs. Refer to the results exhibit in Appendix A for the location of the two downstream Bell Road culverts and Appendix D for the existing and proposed conditions flood hydrographs for each culvert.

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Bell Road Culvert	Peak Discharges		Volumes	
	Ex. Con.	Prop. Con.	Ex. Con.	Prop. Con.
	(cfs)	*(cfs)*	*(ac-ft)*	*(ac-ft)*
5-8'x3'	590	425	248	142
2-6'x3'	100	55	16	11

Table 2: 100-Year Flow Summary Table – Bell Road Culverts

In addition to reducing the peak discharges that concentrate at Bell Road, the volume of flow diverted out of Reata Wash is significantly decreased. As can be seen in Table 2, as well as in the flood hydrographs in Appendix D, the volume that concentrates at the 5-8'x3' Bell Road box culvert is reduced from 248 acre-feet to 142 acre-feet. Therefore, even though the peak discharge at the culvert will only be reduced by about 28%, the volume of water at the culvert is reduced by over 40%. The volume that concentrates at the dual 6'x3' Bell Road box culvert east of 90th Street is similarly reduced from 16 acre-feet to 11 acre-feet. Not only is the reduction in peak discharge important, but by reducing the runoff volume and thereby reducing the duration of flooding, the sediment deposition problems will be improved.

CONCLUSIONS/RECOMMENDATIONS

1. **Flooding and Sedimentation Problems –** The wash that flows along the west side of 94th Street, through the new Multi-Use Sports Fields Complex north of Bell Road, is a significant source of flooding and sedimentation problems. The roadway culverts upstream at Legacy Drive and at Union Hills Drive have experienced significant clogging due to sediment deposition and the area downstream of Bell Road along 94th Street was flooded during a storm in October 2018.

2. **Impact of Reata Wash Flow Diversions –** There are three box culverts that divert flow out of Reata Wash that contribute significantly to the flooding and sedimentation problems. Two of them contribute flow to the wash along the west side of 94th Street and the other adds flow to a wash that crosses Bell Road, east of 94th Street which also contributes to the flooding problems downstream of Bell Road.

3. **Existing Condition Flows vs. Design Flows –** The existing culverts that penetrate the Reata Wash levee pass about twice as much flow during the 100-year flood when compared to their design flows. Since the planned flows are being exceeded under existing conditions, it makes sense to modify the culvert openings to reduce the amount of flow being diverted out of Reata Wash.

4. **Future Reata Wash Flood Control Improvements –** The design report for the future Reata Wash Flood Control Project acknowledges the existence of the three diversion

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culverts, but specifically states that they were ignored in the hydraulic analysis of Reata Wash. That is, the 100-year design flow for Reata Wash was not reduced at the diversion culverts. Moreover, the report indicates that this reach of Reata Wash, between Hualapai Drive and Bell Road, can convey the 100-year flood without any further improvements. Therefore, the flow through the diversion culverts can be reduced without negatively impacting the hydraulic conditions on Reata Wash.

5. **Proposed Reduction in Size of Culvert Openings –** It is proposed to reduce the size of the culvert openings to help mitigate flooding and sedimentation problems on the downstream washes. It is envisioned that the size reductions will be done by attaching steel plates to the inlet headwalls that have cutouts for the reduced opening size. Table 3 shows the proposed opening sizes. The reduced culvert openings were sized to limit the flow through the culverts to an amount that is near but does not exceed the bank full capacity of the sandy bed width of the existing, downstream washes.

6. **Downstream Benefits –** The reduced opening size will result in significant reduction in the 100-year peak flow in the downstream washes. Even though the local watershed is about 220 acres, the peak discharge on the 94th Street wash will be reduced by about 50% at Legacy Drive and by about 25% at Bell Road. Moreover, the duration of flooding will be considerably reduced which will help mitigate the erosion and sedimentation problems along the downstream washes.

7. **404 Jurisdictional Washes –** The 404 permit was not reviewed as part of this evaluation. While we believe that the reduced openings provide adequate flows for preserving the riparian habitat along the downstream washes, the legal requirements related to the existing 404 permit were not reviewed.

Reata Wash Levee Penetration	Existing Condition		Proposed Condition	
	Height	Width	Height	Width
	(ft)	*(ft)*	*(ft)*	*(ft)*
North Culvert*	2	16	2	3
Center Culvert	2	10	2	2
South Culvert	2	6	2	2

*The north culvert is a dual 8'x2' concrete box culvert. The proposed reduction can either be achieved by one 36" wide opening or two 18" wide openings

Table 3: Proposed Reduction in Culvert Openings

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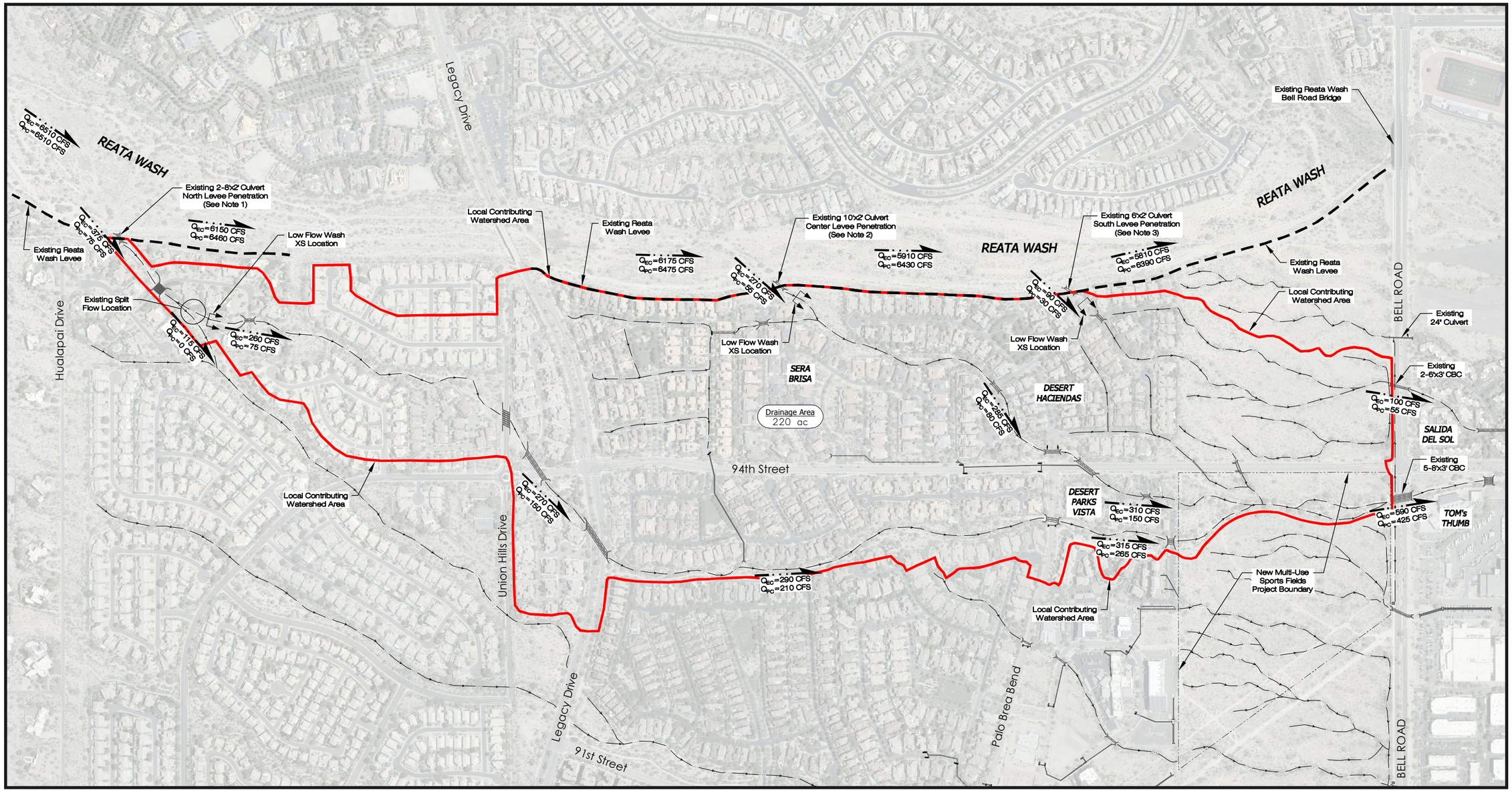
Appendix A: 100-Year Peak Flow Exhibit (Existing vs Proposed Conditions)



LEGEND

- Local Watershed Boundary
- Q_{EC}=260 CFS, Q_{PC}=260 CFS — Existing (Q_{EC}) vs. Proposed (Q_{PC}) Peak Flows
- Flow Paths
- Existing Culvert Crossings
- Low Flow Wash Cross Section Location
- Reata Wash Levee

NOTES:

1) Under existing conditions, the Reata Wash North Levee Penetration Culvert (2-8'x2' CBC) has a diversion capacity of 375 cfs during the 100-year, 24-hour storm event. By reducing the upstream culvert opening from 16 feet wide to 2-18" wide openings (alternatively a single 36" wide opening can also be used), the proposed conditions diversion capacity is lowered to 75 cfs.

2) Under existing conditions, the Reata Wash Center Levee Penetration Culvert (10'x2' CBC) has a diversion capacity of 270 cfs during the 100-year, 24-hour storm event. By reducing the upstream culvert opening from 10 feet to 2 feet wide, the proposed conditions diversion capacity is lowered to 55 cfs.

3) Under existing conditions, the Reata Wash South Levee Penetration Culvert (6'x2' CBC) has a diversion capacity of 90 cfs during the 100-year, 24-hour storm event. By reducing the upstream culvert opening from 6 feet to 2 feet wide, the proposed conditions diversion capacity is lowered to 30 cfs.

4) The 100-year peak flows shown hereon were obtained from a modified version of the Pinnacle Peak South ADMS FLO-2D Model. Refer to the attached memorandum for a description of the modifications that were made to the model.

100-YEAR PEAK FLOW EXHIBIT

0 300' 600'

SCALE: 1" = 600'

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
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Checked By:	MTG

Sheet Title :

100-Year Peak Flow Exhibit

Sheet Number:

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Appendix B: As-Built Plans
(Reata Wash Levee and Culverts)

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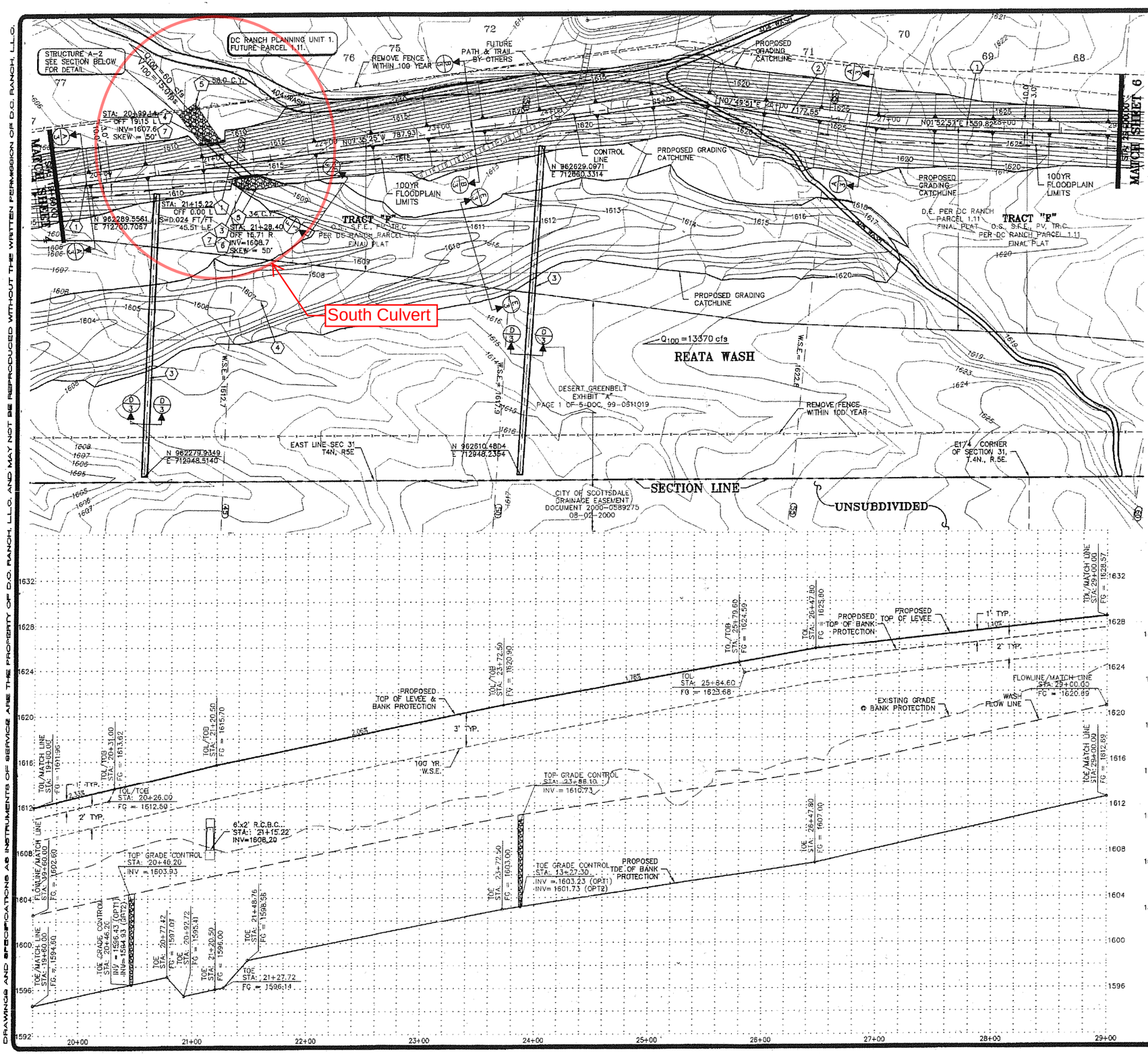

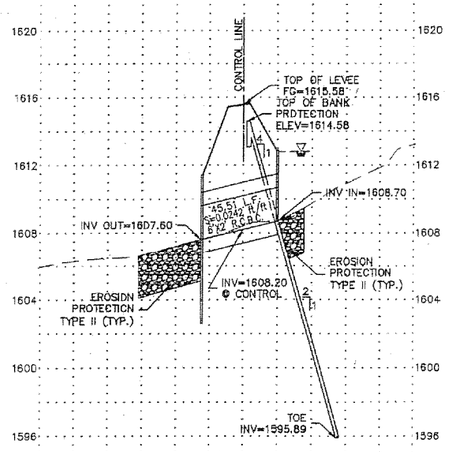

STA 21+15.22
STRUCTURE A-2
LOOKING NORTH
SEE THIS SHEET FOR PLAN VIEW
6'X2' R.C.B.C.
Q_{100} = 60 cfs
V_{100} = 15.0 fps
DHW = 4.1'

40 0 20 40
Horz 1" = 40 ft.
Vert 1" = 4 ft.

GRADING NOTES

1. CONSTRUCT BANK PROTECTION PER SECTION A-A, SHEET 3.
2. CONSTRUCT BANK PROTECTION PER SECTION B-B, SHEET 3.
3. CONSTRUCT GRADE CONTROL STRUCTURE PER SECTION D-D, SHEET 3.
4. CONSTRUCT GRADED CHANNEL PER SECTION E-E, SHEET 3.
5. CONSTRUCT GRADED CHANNEL PER SECTION F-F, SHEET 3.

DRAINAGE NOTES

1. CONSTRUCT SINGLE BARREL S=6'X H=2' REINFORCED CONCRETE BOX CULVERT A.D.O.T. STD. DTL. B-D2.10 FOR 6X3 SINGLE BARREL TABLE 1 W/MODIFIED HEIGHT.
3. CONSTRUCT INLET HEADWALL PER A.D.O.T. STD. DET. B-04.30, MODIFIED FOR H=2' 4:1 SLOPE, SKEW = PLAN VIEW.
4. CONSTRUCT OUTLET HEADWALL PER A.D.O.T. STD. DET. B-04.10, MODIFIED FOR H=2', 4:1 SLOPE, SKEW = PLAN VIEW.
5. INSTALL NATIVE ROCK RIP-RAP EROSION PROTECTION. TYPE II (TYP).
6. INSTALL PIPE ACCESS BARRIER PER MODIFIED ADOT STD. DTL. C-13.75. SEE SHEET 3.
7. CONSTRUCT HANDRAIL PER DETAIL SHEET 2.

DMB
WOOD/PATEL
Civil Engineers
Hydrologists
Land Surveyors
Construction Managers
(602) 335-8500

DC RANCH
SCOTTSDALE
SOUTH FLOOD PROTECTION IMPROVEMENT PLAN
GRADING PLAN

DESIGNER	S.Z.
DRAWN	K.L.
CHECKED	A.P.
DATE	10/08/03
SCALE	1" = 40'
JOB NO.	021690
SHEET	5

51-NP-2003
3642-02-4
54-ZN-85#7

38-DR-2020
11/25/2020

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OC RANCH PLANNING UNIT 1.
FUTURE PARCEL 1.11.

EXISTING FENCE

MATCH SHEET 5

100YR FLOODPLAIN LIMITS

TRACT "E"
O.S. S.F.E. PV 18.6
PER DC RANCH PARCEL 1.11
FINAL PLAT

D.E. PER DC RANCH
PARCEL 1.11
FINAL PLAT

Q_{100} = 13370 cfs

W.S.E. = 1638.8

W.S.E. = 1638.2

DESERT GREENBELT
EXHIBIT "A"
PAGE 1 OF 5 DOC. 99-0611019

REMOVE FENCE
WITHIN 100 YEAR

REMOVE FENCE
WITHIN 100 YEAR

EAST LINE SEC 31
T4N, R5E

NE CORNER OF SE1/4 SE1/4 NE1/4
OF SECTION 31, T.4N., R.5E.

SECTION LINE

CITY OF SCOTTSDALE
DRAINAGE EASEMENT
DOCUMENT 2000-0589275
08-02-2000

REATA WASH

UNSUBDIVIDED

PROPOSED GRADING
CATCHLINE

CONTROL LINE

PROPOSED GRADING
CATCHLINE

100YR
FLOODPLAIN
LIMITS

STRUCTURE A-1
SEE SECTION BELOW
FOR DETAIL

STA: 38+68.11
OFF 0.00 R
S=0.026 FT/FT
65.42 L.F.

W.S.E. = 1644.2

PROPOSED GRADING
CATCHLINE

PROPOSED GRADING
CATCHLINE

OUT PARCEL
ELIZABETH R. SUMMERS
TAX PARCEL 217-12-009H
DOCUMENT 98-1017506 11-12-98

OUT PARCEL
CITY OF SCOTTSDALE
TAX PARCEL 217-12-009F
DOCUMENT 96-0499791 07-12-96

OUT PARCEL
CITY OF SCOTTSDALE
TAX PARCEL 217-12-009J
DOCUMENT 96-0545865 08-02-96

OUT PARCEL
CITY OF SCOTTSDALE
TAX PARCEL 217-12-009F
DOCUMENT 96-0499791 07-12-96

OUT PARCEL
ARIZONA DISTRICT COUNCIL
OF THE ASSEMBLIES OF GOD
TAX PARCEL 217-12-009G
DOCUMENT 02-0364155
04-09-02

MATCH SHEET 4

STA: 38+53.12
OFF 20.37 L
INV=1638.4
SKEW = 50°

STA: 38+88.85
OFF 34.09 R
INV=1640.10
SKEW = 50°

CURVE TABLE					
CURVE	DELTA	RADIUS	ARC	TANGENT	CHORD
C4	27°37'34"	350.00'	168.76'	86.05'	167.13'

40 0 20 40
Horz 1" = 40 ft.
Vert 1" = 4 ft.

Center Culvert

GRADING NOTES

1. CONSTRUCT BANK PROTECTION PER SECTION A-A, SHEET 3.
2. CONSTRUCT BANK PROTECTION PER SECTION B-B, SHEET 3.
4. CONSTRUCT GRADED CHANNEL PER SECTION E-E, SHEET 3.

DRAINAGE NOTES

2. CONSTRUCT SINGLE BARREL S=10'X H=2' REINFORCED CONCRETE BOX CULVERT A.D.O.T. STD. DTL. B-02.10 FOR 10X3 SINGLE BARREL TABLE 1 W/MODIFIED HEIGHT.
3. CONSTRUCT INLET HEADWALL PER A.D.O.T. STD. DET. B-04.30, MODIFIED FOR H=2' 4:1 SLOPE, SKEW = PLAN VIEW.
4. CONSTRUCT OUTLET HEADWALL PER A.D.O.T. STD. DET. B-04.10, MODIFIED FOR H=2', 4:1 SLOPE, SKEW = PLAN VIEW.
5. INSTALL NATIVE ROCK RIP-RAP EROSION PROTECTION. TYPE II (TYP).
6. INSTALL PIPE ACCESS BARRIER PER MODIFIED ADOT STD. DTL. C-13.76. SEE SHEET 3.
7. CONSTRUCT HANDRAIL PER DETAIL SHEET 2.

PROPOSED
TOP OF LEVEE

PROPOSED
TOP OF BANK
PROTECTION

1' TYP.

2' TYP.

2.57%

1.10%

TOL/MATCH LINE
STA: 29+00.00
FG = 1638.57

TOL
STA: 29+57.00
FG = 1629.20

FLOWLINE/MATCH LINE
STA: 29+00.00
FG = 1620.69

TOE/MATCH LINE
STA: 29+00.00
FG = 1612.69

TOE
STA: 29+57.00
FG = 1614.00

TOL
STA: 32+53.50
FG = 1636.80

TOE
STA: 32+53.50
FG = 1621.00

TOL/TOB
STA: 32+61.20
FG = 1636.84

TOB
STA: 32+86.20
FG = 1635.85

EXISTING GRADE
@ BANK PROTECTION

WASH
FLOW LINE

PROPOSED
TOE OF BANK
PROTECTION

TOL/TOB
STA: 34+77.70
FG = 1641.00

TOE
STA: 34+77.70
FG = 1625.00

PROPOSED
TOP OF LEVEE &
BANK PROTECTION

3' TYP.

2.64%

1.87%

100 YR
W.S.E.

TOL/TOB
STA: 37+38.60
FG = 1647.90

TOL/TOB
STA: 37+38.60
FG = 1632.00

TOL/TOB/MATCH LINE
STA: 39+00.00
FG = 1650.44

1.58%

FLOWLINE/MATCH LINE
STA: 39+00.00
FG = 1641.27

10'X2' R.C.B.C.
STA: 38+68.11
INV=1639.05

TOE
STA: 35+31.08
FG = 1632.73

TOE
STA: 38+42.61
FG = 1631.27

TOE
STA: 38+83.61
FG = 1631.59

TOE/MATCH LINE
STA: 39+00.00
FG = 1633.27

1652
1648
1644
1640
1636
1632
1628
1624
1620
1616
1612

29+00 30+00 31+00 32+00 33+00 34+00 35+00 36+00 37+00 38+00 39+00

CONTROL LINE

TOP OF LEVEE &
BANK PROTECTION
ELEV=1649.94

INV IN=1640.1

INV OUT=
1638.40

65.42 L.F.
@ 0.026 ft/ft
10'X2' R.C.B.C.

EROSION
PROTECTION
TYPE II (TYP.)

INV=1639.05
@ CONTROL

EROSION
PROTECTION
TYPE II (TYP.)

TOE
INV=1631.47

STA 38+68.11
STRUCTURE A-1

LOOKING NORTH
SEE THIS SHEET FOR PLAN VIEW
10'X2' R.C.B.C.
Q_{100} = 140 cfs
V_{100} = 18.4 fps
DHW = 6.7'

DMB

WOOD/PATEL
Civil Engineers
Hydrologists
Water Resources
Construction Managers
(602) 335-0500

DC RANCH
SCOTTSDALE
SOUTH FLOOD PROTECTION IMPROVEMENT PLAN
GRADING PLAN

Professional Engineer Certificate No. 32854 SHAWN ZOU Date Signed 12-31-03 ARIZONA, U.S.A.

DESIGNER	S.Z.
DRAWN	K.L.
CHECKED	A.P.
DATE	10/08/03
SCALE	1" = 40'
JOB NO.	021690
SHEET	6

54-ZN-85#7 3642-02-4 51-NP-2003

38-DR-2020
11/25/2020

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North Culvert

LA REGGIA PER DOC No. 2001-0005826 Bk.551 Pg.48

SECTION LINE

MATCH SHEET 6

NAOS & DRAINAGE EASEMENT

STRUCTURE A-1 2-8'x2' R.C.B.C. SEE THIS SHEET FOR PROFILE.

STRUCTURE A-2 2-36" R.G.R.C.P. SEE THIS SHEET FOR PROFILE.

CONTROL LINE PROPOSED TOP OF LEVEE & GRADING BANK PROTECTION CATCHLINE

REATA WASH

EXISTING CITY RIGHT-OF-WAY FOR DRAINAGE PURPOSES DOC. 99-0610109

MATCH VIEW TO BELOW LEFT/THIS SHEET

100YR FLOODPLAIN LIMIT

100YR FLOODPLAIN & FUTURE ENCROACHMENT LIMIT

MATCH VIEW ABOVE RIGHT/THIS SHEET

PROPOSED GRADING CATCHL LINE

100YR FLOODPLAIN LIMIT CONTROL LINE

TOP OF LEVEE & BANK PROTECTION

TRACT "G" N.O.S., D.E., PVT. TR. E, S.F.E.

DC RANCH PARCELS 2.17, 2.18 AND 2.19 PER DOCUMENT 2002-0994689 Bk.607, Pg.7

PROPOSED FUTURE LOTTING DC RANCH PARCEL 2.19

PIMA ACRES ESTATES BOOK 217,PAGE 46, MCR

HUALAPAI DRIVE

EXISTING SCHOOL SITE TEMPORARY SEWER & EASEMENT, C.O.S. No. 96-DR-98-1637-99J

20' SEWER EASEMENT

EXISTING CITY RIGHT-OF-WAY FOR DRAINAGE PURPOSES

404 WASH

PROPOSED TOP OF BANK PROTECTION

2-36" R.C.B.C. INV=1733.35 @ CONTROL

WASH FLOW LINE

EXIST. GRADE

2-8'X2' R.C.B.C. STA: 21+00.00 @ CONTROL LINE

TOP GRADE CONTROL STA 21+70.00 INV= 1725.93

TOE GRADE CONTROL STA 21+70.00 INV= 1719.90 (OPT1) INV= 1716.93 (OPT2)

PROPOSED TOE

100 YR. W.S.E.

EX. 8" SEWER INV=1725.77± @ CONTROL

INV=1727.90

INV=1714.31

INV=1747.47

INV=1735.96

GRADING NOTES

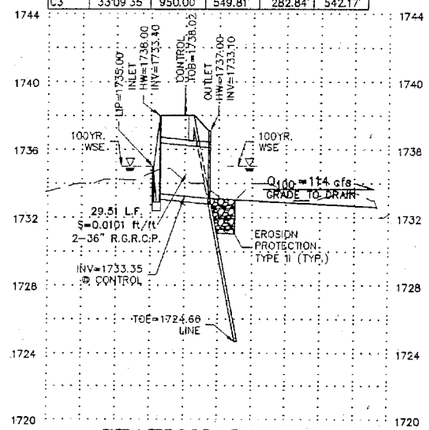
(2) CONSTRUCT GRADE CONTROL STRUCTURE PER SECTION B-B, SHEET 3.
(4) CONSTRUCT LEVEE & BANK PROTECTION PER SECTION A-A SHEET 3
(5) INSTALL TYPE II LOOSE RIP-RAP 3.0' THICK WITH MIRAFI 140 NL FILTER FABRIC OR APPROVED EQUAL. SEE DETAIL ON SHEET 2.

DRAINAGE NOTES

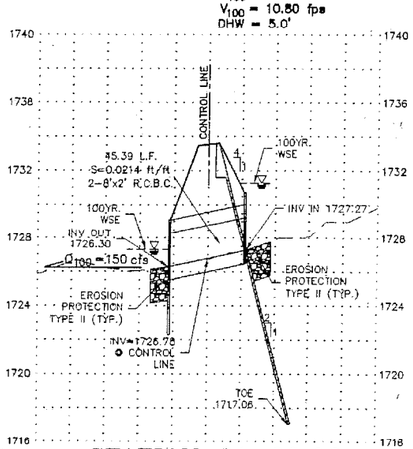
(2) INSTALL OROP INLET STRUCTURE PER DETAIL ON SHEET 2.
(3) CONSTRUCT HANDRAIL PER DETAIL SHEET 2.
(4) CONSTRUCT INLET HEADWALL PER MODIFIED ADOT STD. DET. B-04.30.
(5) INSTALL PIPE ACCESS BARRIER PER MODIFIED ADOT STD. DTL. C-13.75. SEE SHEET 3.
(24) INSTALL 2-8'X2' R.C.B.C MODIFIED FOR DEPTH PER ADOT STD. DTL. B-02.20
(25) CONSTRUCT OUTLET HEADWALL PER MODIFIED AOOT STD. DET. B-04.10.
(27) INSTALL 2-36" R.G.R.C.P. CLASS III
(28) CONSTRUCT OUTLET HEADWALL FOR 2-36" R.G.R.C.P. PER M.A.G. STD. DET. 501-2.

Horz 1" = 40 ft.
Vert 1" = 4 ft.

CURVE TABLE					
CURVE	DELTA	RADIUS	ARC	TANGENT	CHORD
C3	33°09'35"	950.00'	549.81'	282.84'	542.17'



STATION 21+00.00
LOOKING NORTH
SEE THIS SHEET FOR PLAN VIEW
2 - 36" R.C.B.C.
STRUCTURE A-2
Q_{100} = 114cfs
V_{100} = 10.80 fps
DHW = 5.0'



STATION 21+00.00
LOOKING NORTH
SEE THIS SHEET FOR PLAN VIEW
2 - 2'x8' R.C.B.C.
STRUCTURE A-1
Q_{100} = 150cfs
V_{100} = 15.8 fps
DHW = 4.0'

Two working days before you dig CALL FOR THE BLUE STAKES 263-1100 Blue Stake, Inc. CALL COLLECT

DMB
WOOD/PATEL
Civil Engineers
Hydrologists
Land Surveyors
Construction Managers
(602) 336-8800

DC RANCH
SCOTTSDALE
NORTH FLOOD PROTECTION IMPROVEMENT PLANS
GRADING PLAN

Professional Engineering Consultants CORPORATE 32854 SHIMIN ZOU Date Signed 10-8-03 ARIZONA, U.S.A.

DESIGNER	S.Z.
DRAWN	K.L./T.E.S.
CHECKED	A.P.
DATE	10/08/03
SCALE	1" = 40'
JOB NO.	021684
SHEET	7 OF

50 -NP- 2003
3642-02-2
54-ZN-85#7

38-DR-2020
11/25/2020

Appendix C: Wash Low Flow Hydraulic Analysis

Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Tuesday, Aug 25 2020

Low Flow Wash Capacity Downstream of North Culvert

Rectangular

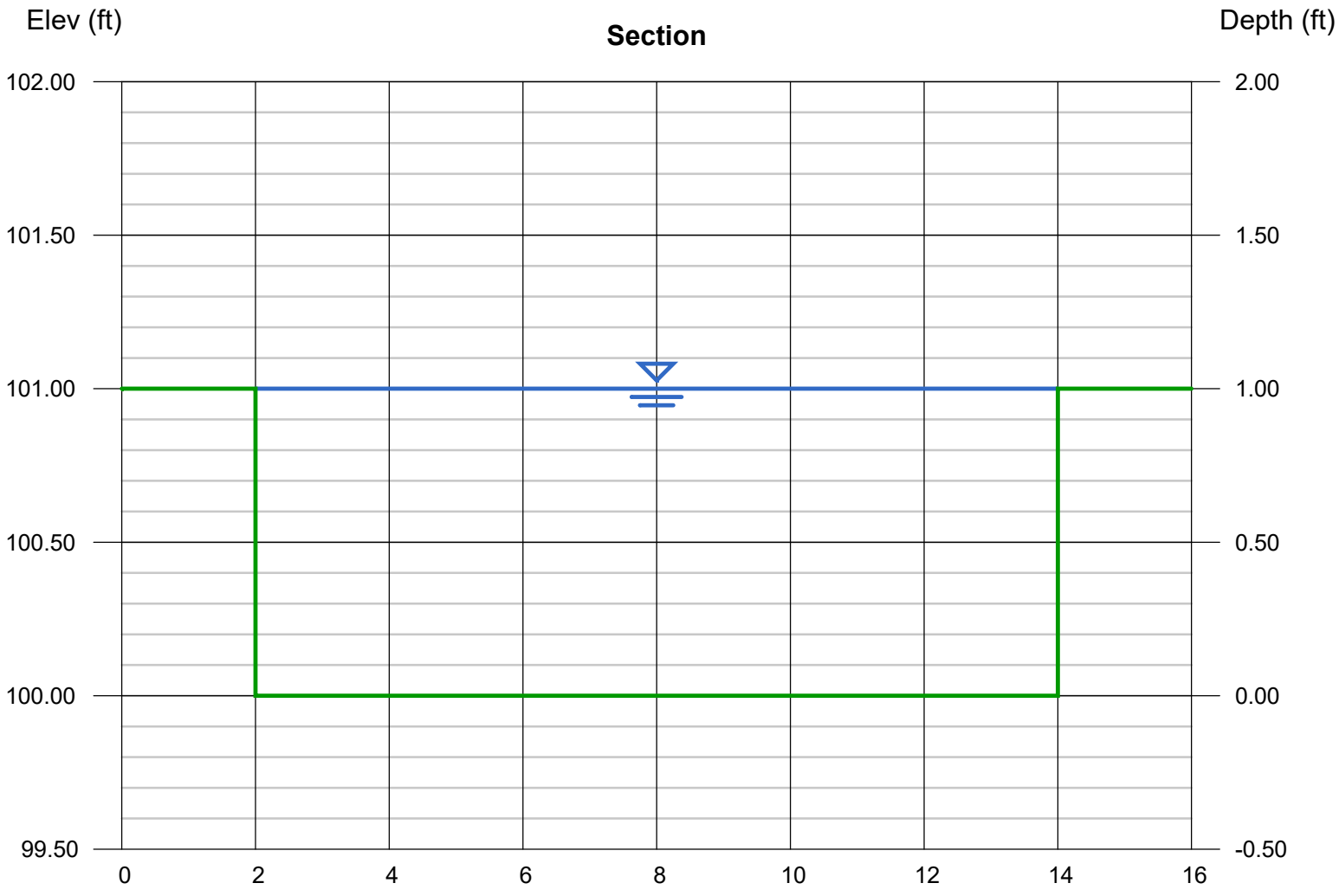
Bottom Width (ft)	= 12.00
Total Depth (ft)	= 1.00
Invert Elev (ft)	= 100.00
Slope (%)	= 2.20
N-Value	= 0.030

Calculations

Compute by:	Known Depth
Known Depth (ft)	= 1.00

Highlighted

Depth (ft)	= 1.00
Q (cfs)	= 79.55
Area (sqft)	= 12.00
Velocity (ft/s)	= 6.63
Wetted Perim (ft)	= 14.00
Crit Depth, Yc (ft)	= 1.00
Top Width (ft)	= 12.00
EGL (ft)	= 1.68



38-DR-2020
11/25/2020

Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Tuesday, Aug 25 2020

Low Flow Wash Capacity Downstream of Center Culvert

Rectangular

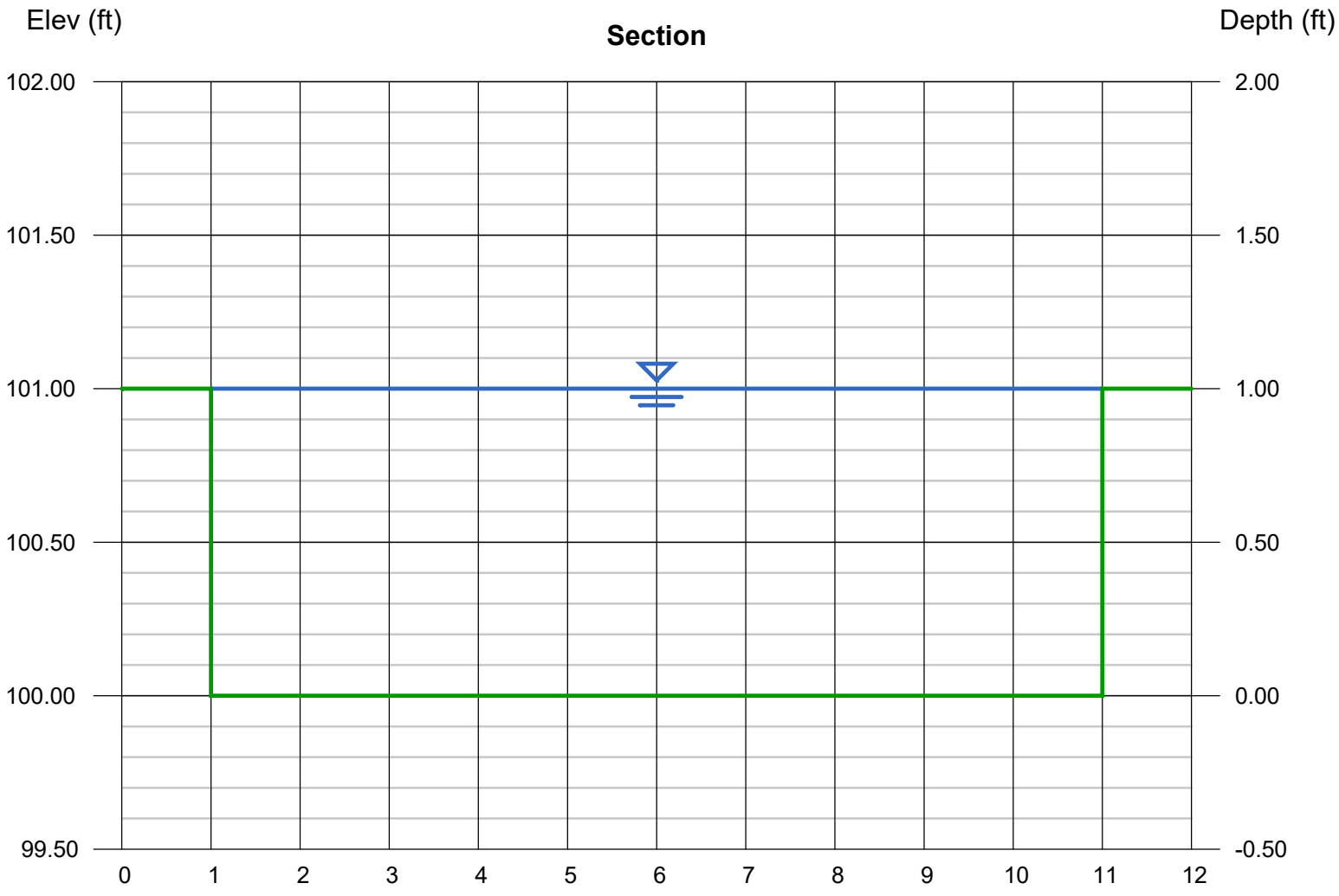
Bottom Width (ft)	= 10.00
Total Depth (ft)	= 1.00
Invert Elev (ft)	= 100.00
Slope (%)	= 1.80
N-Value	= 0.030

Calculations

Compute by:	Known Depth
Known Depth (ft)	= 1.00

Highlighted

Depth (ft)	= 1.00
Q (cfs)	= 58.85
Area (sqft)	= 10.00
Velocity (ft/s)	= 5.88
Wetted Perim (ft)	= 12.00
Crit Depth, Yc (ft)	= 1.00
Top Width (ft)	= 10.00
EGL (ft)	= 1.54



Channel Report

Hydraflow Express Extension for Autodesk® AutoCAD® Civil 3D® by Autodesk, Inc. Tuesday, Aug 25 2020

Low Flow Wash Capacity Downstream of South Culvert

Rectangular

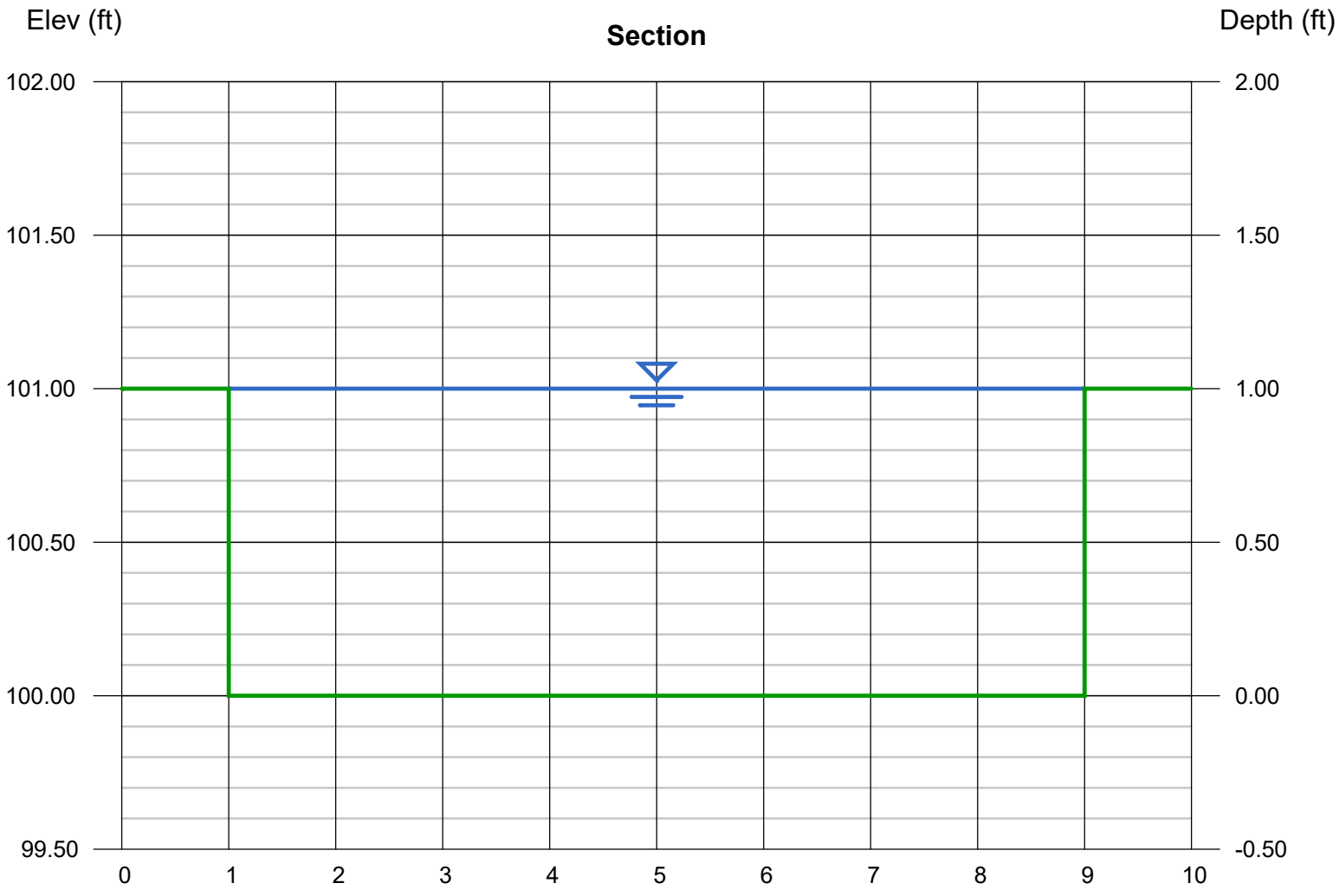
Bottom Width (ft)	= 8.00
Total Depth (ft)	= 1.00
Invert Elev (ft)	= 100.00
Slope (%)	= 2.00
N-Value	= 0.030

Calculations

Compute by:	Known Depth
Known Depth (ft)	= 1.00

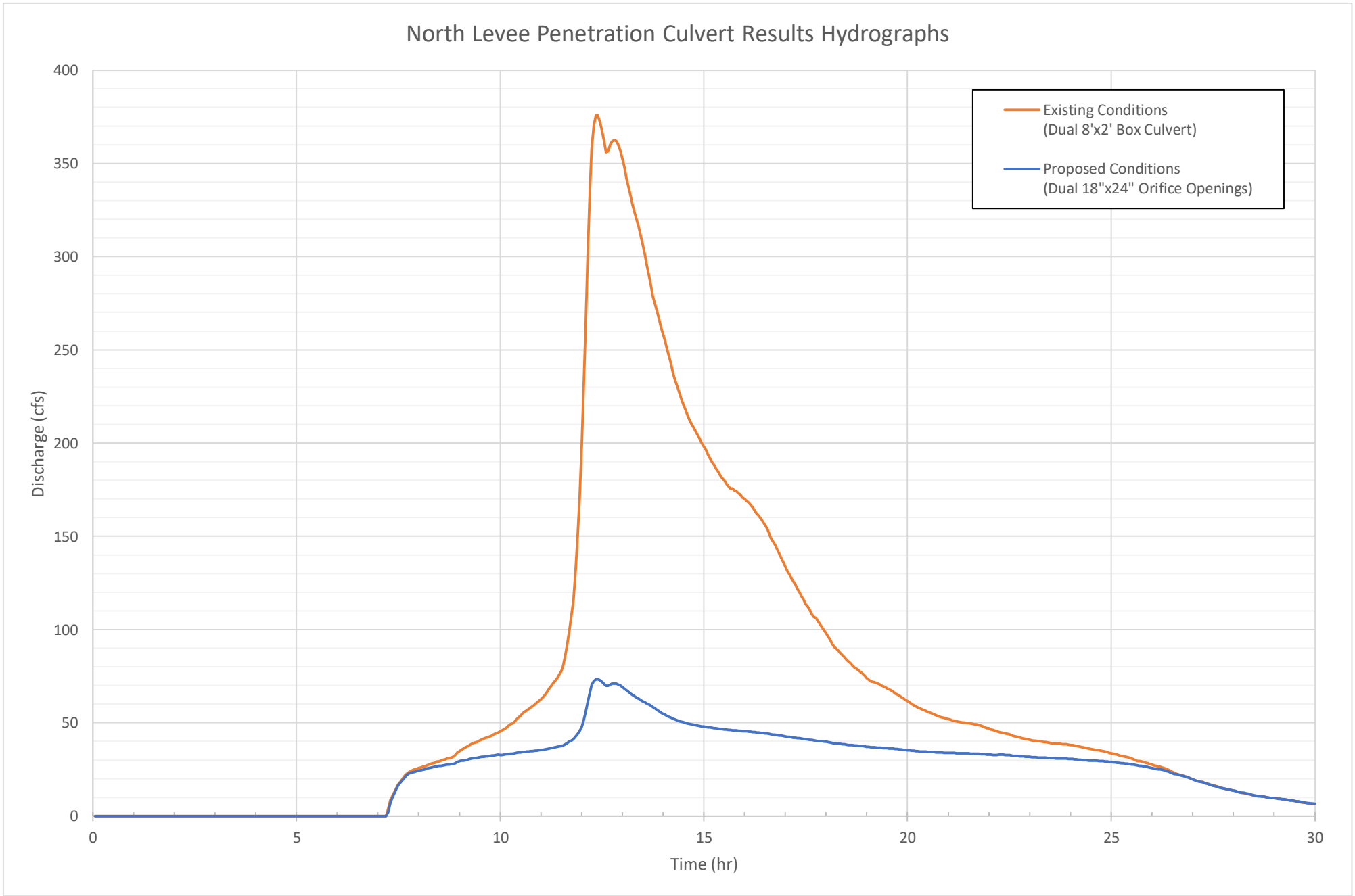
Highlighted

Depth (ft)	= 1.00
Q (cfs)	= 48.29
Area (sqft)	= 8.00
Velocity (ft/s)	= 6.04
Wetted Perim (ft)	= 10.00
Crit Depth, Yc (ft)	= 1.00
Top Width (ft)	= 8.00
EGL (ft)	= 1.57

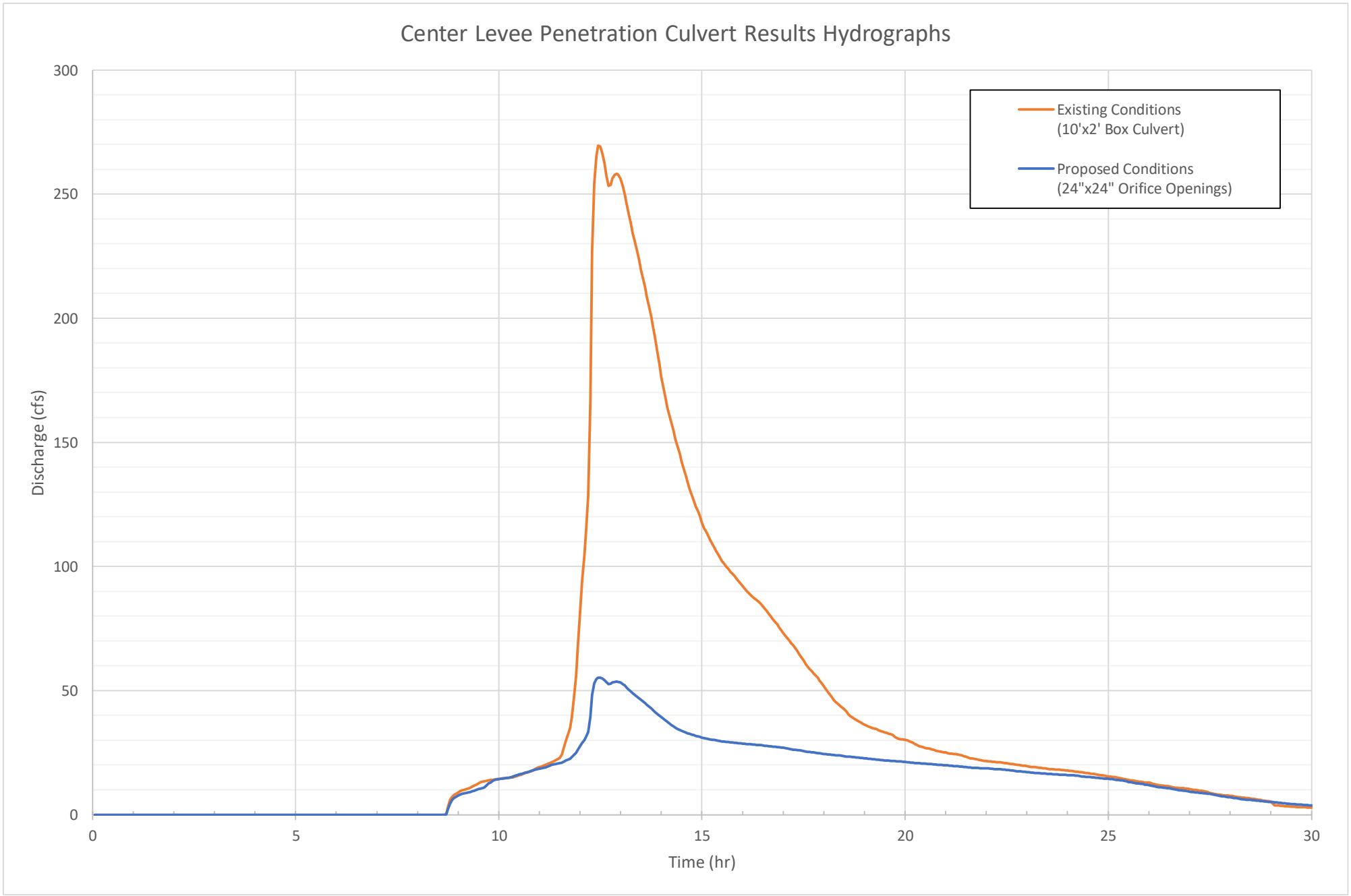


Appendix D: Existing vs. Proposed 100-Year Flood Hydrographs

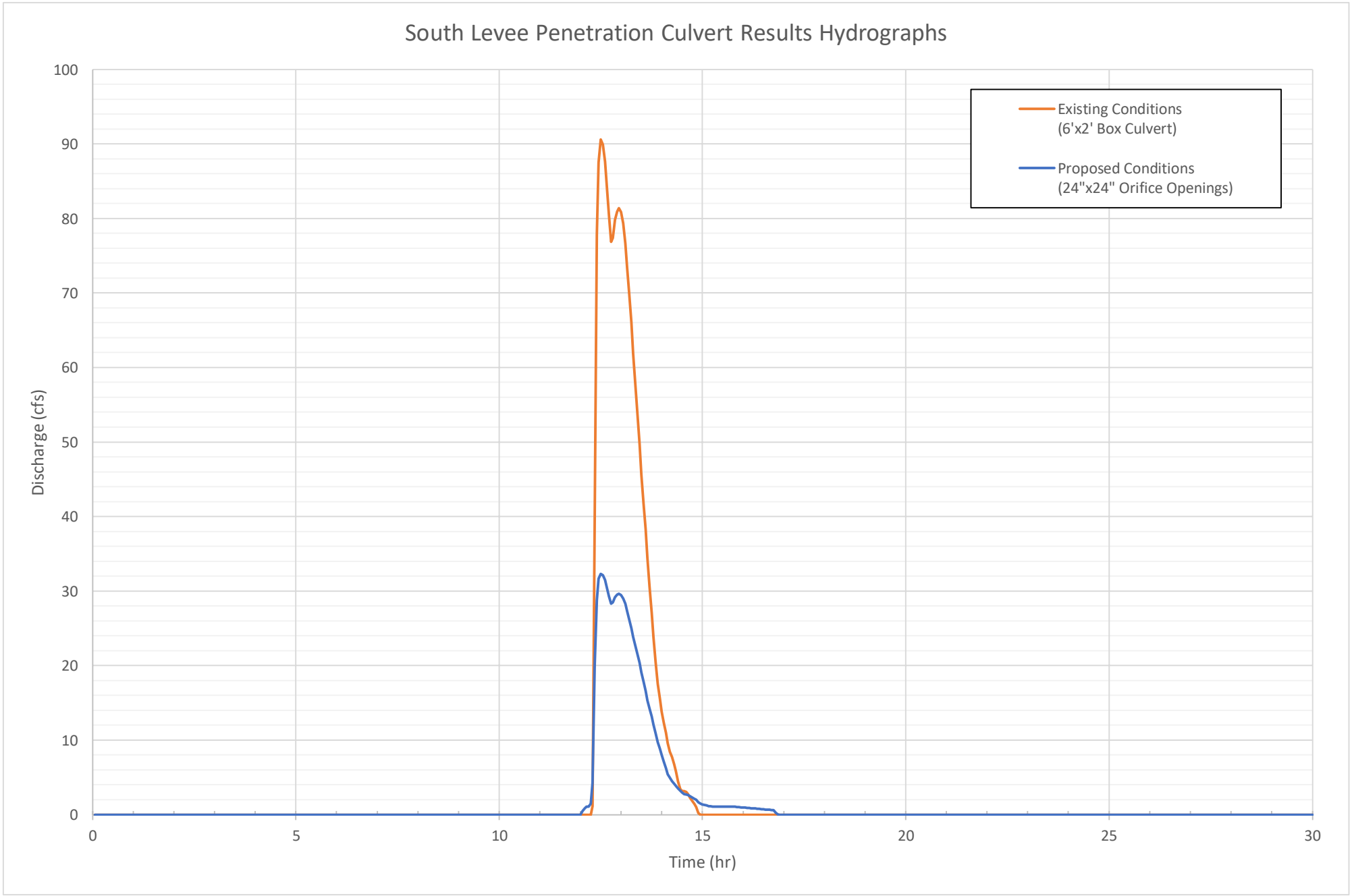
Flood Hydrographs – Reata Wash Diversion Culverts



38-DR-2020
11/25/2020

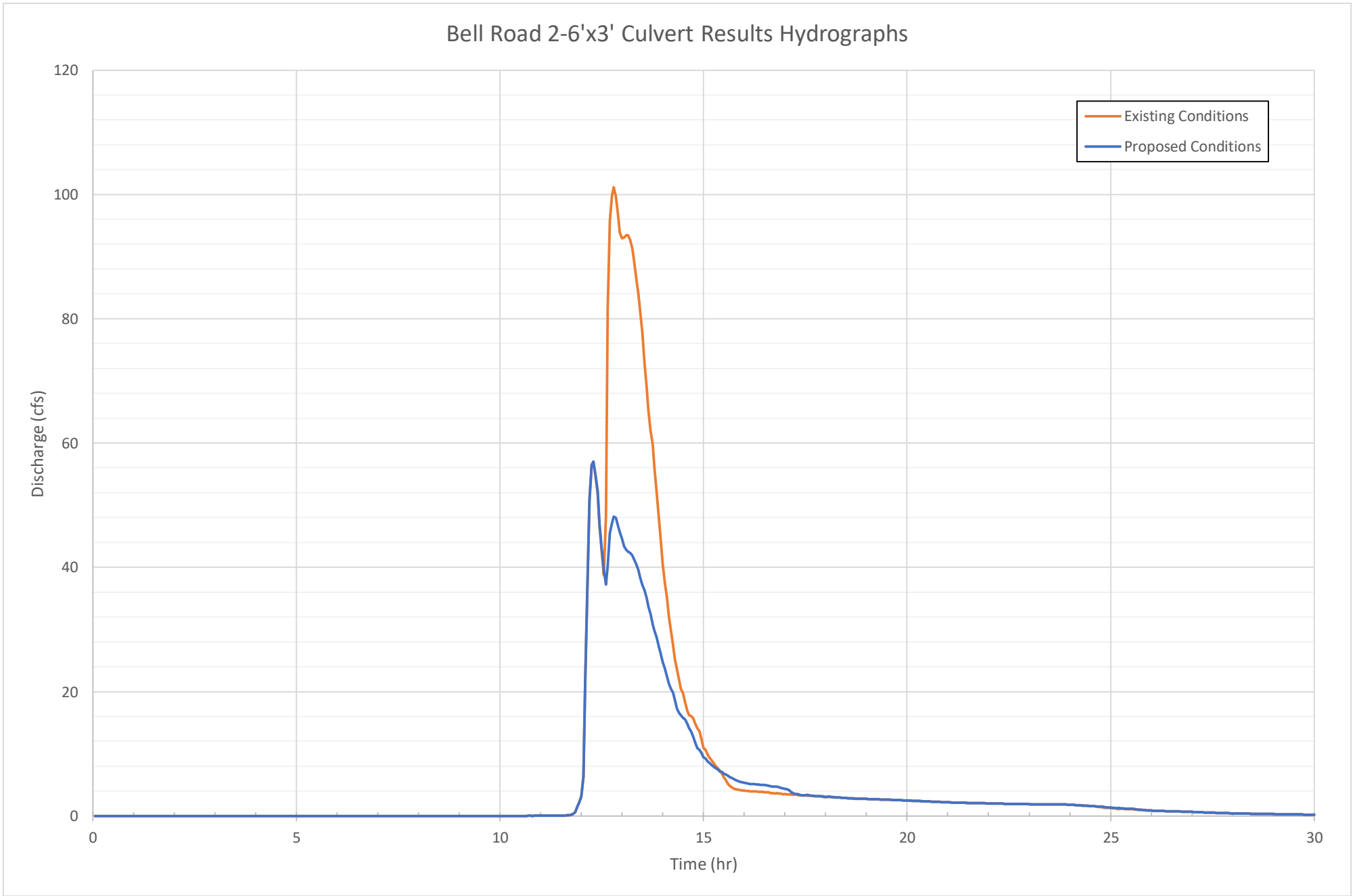


38-DR-2020
11/25/2020

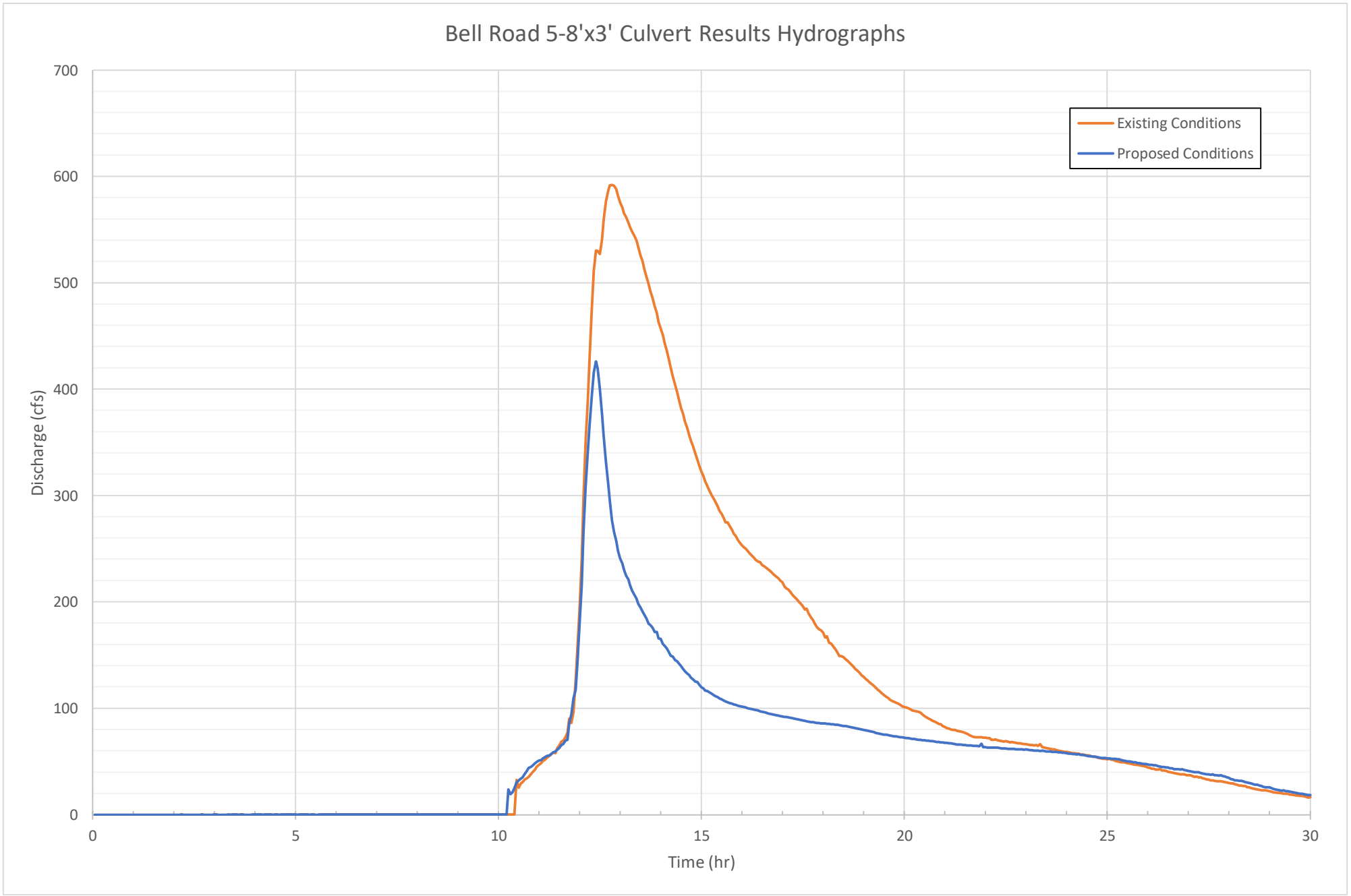


38-DR-2020
11/25/2020

Flood Hydrographs – Bell Road Culverts

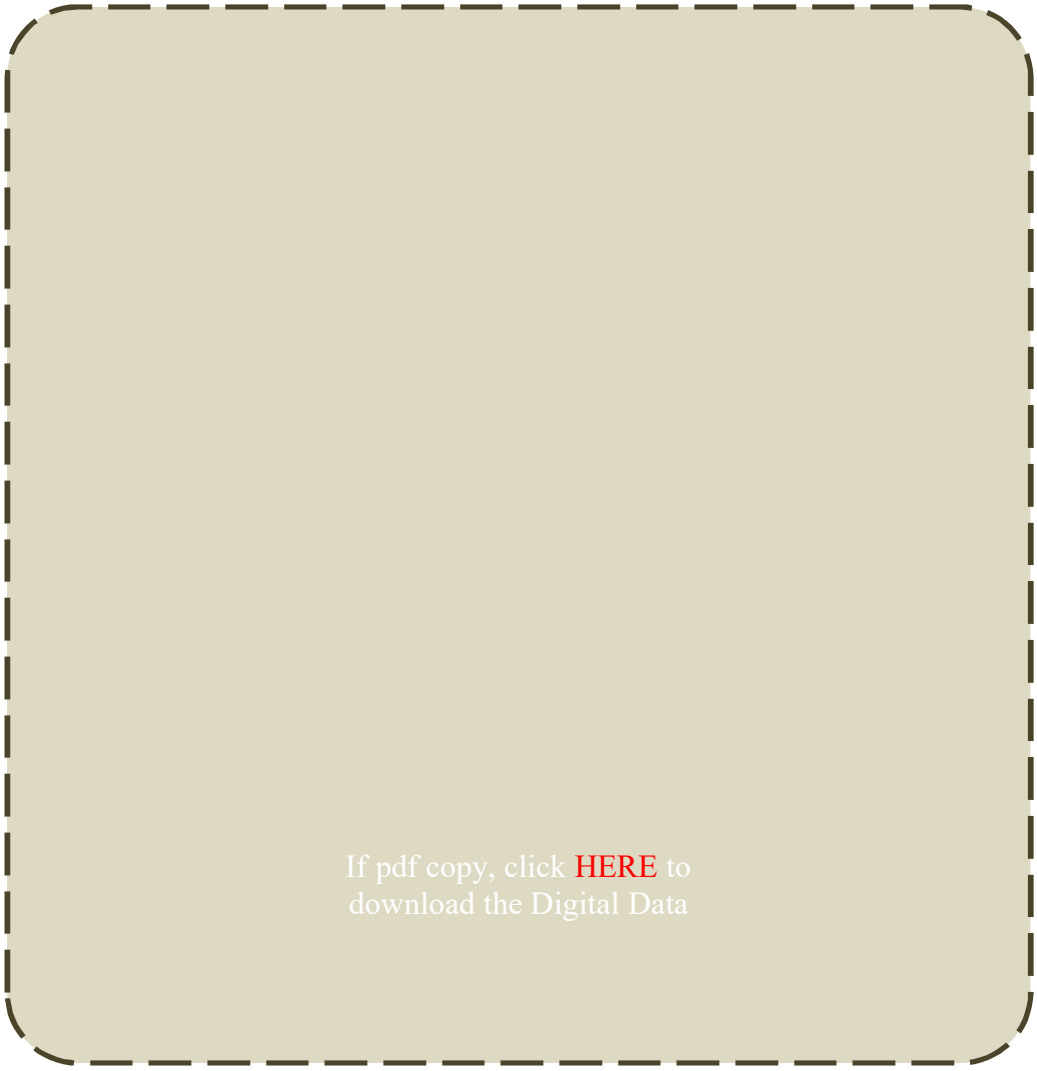


38-DR-2020
11/25/2020



38-DR-2020
11/25/2020

Appendix F: Digital Data



[Digital Data CD]