CITY OF SCOTTSDALE

WESTWORLD SPORTS FIELDS

DRAINAGE REPORT
(Revised DR Submittal)

Plan #

Case # 36-DR-2021

Q-S # 35-51

X Approved

Corrections

Richard M. Anderson 10/11/2021

Reviewed By Date

Project No.: PG09

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

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 Westworld Sports Fields. The proposed complex will consist of five lighted multi-use fields, curbed parking lots, a restroom and office building with potable water and sewer connections, sidewalks, offsite street improvements and a raw (CAP Canal) water connection for sports field and landscape irrigation. The improvements are situated on a 40-acre site on the east end of Westworld which is located within the Bureau of Reclamation (BOR) floodwater reservoir behind Dike 4 of the CAP Canal dikes. The sports complex will be designed to meet the drainage requirements set forth by the BOR for development within their floodwater impoundment area as well as the design requirements 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 southeast corner of 98th Street and McDowell Mountain Ranch Road. It is situated at the east end of Westworld and bound by Thompson Peak Parkway on the south and McDowell Mountain Ranch on the north and Reata Wash on the west. Refer to Figure 1 below for the vicinity map.

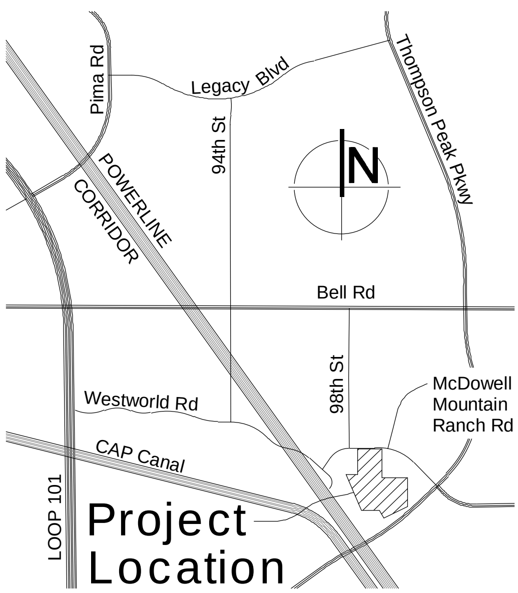


Figure 1: Vicinity Map

2.0 STORM WATER RETENTION

2.1 APPROACH

Since the project lies within the 100-year flood pool behind Dike 4 of the CAP Canal, the volume associated with the new subsurface drainage systems in the sports complex will provide the required storm water retention volume. The subsurface drainage systems consist of new storm drains, culverts and the field drainage pipes, most of which are below the Dike 4 100-year water surface elevation (WSEL) of 1527.8 ft and therefore increase the storage volume within the flood pool. In addition to the volume provided in the pipes and culverts, the void space within the 4" thick gravel layer that underlies the sand-based sports fields also increases the volume within the flood pool. The calculations of the storage volume associated with these subsurface drainage features is documented in the in Appendix A and are shown on the Retention Volume Drainage Area Map, also in Appendix A.

2.2 REQUIRED RETENTION VOLUME

The project site includes areas within the Environmentally Sensitive Lands Ordinance (ESLO) as well as non-ESLO lands. In addition, there are both undisturbed and previously developed areas within the non-ESLA lands. Since the required retention volumes vary for each of these areas, the total required storm water retention volume was calculated as a combination of the following:

1. **Undisturbed Areas (Non-ESLO BOR Property) –** The full 100-year, 2-hour runoff volume was added to the site's retention requirement for the undisturbed desert areas. This is the undisturbed desert within the BOR property which is not under the ESLO jurisdiction.
2. **Existing Westworld Parking Areas and Drives (BOR Property) –** The increase in runoff volume (pre vs. post) associated with the previously disturbed existing gravel parking lot and driveways was also added to the site's required retention volume.
3. **ESLO Parcel (City of Scottsdale Property) -** The increase in runoff volume (pre vs. post) was added for the (ESLO) parcel. This is the undisturbed parcel of land on the east side of the project site that the city recently purchased from the State Land Department. It lies outside of the BOR's property boundary and is included within the ESLO area and therefore is only required to provide retention for the pre versus post increase in runoff volume.

As can be seen on the Drainage Area Map in Appendix A, the project site was separated into four separate areas to calculate the required retention volume. The full 100-year, 2-hour runoff volume was calculated for Retention Areas #1 and #3, both of which are undisturbed desert areas located on BOR property. Retention Area #2 is also located on BOR property, but is currently being used as a parking lot and therefore, due to the pre-existing conditions, only the increase in the runoff (pre vs. post) volume was calculated for it. Retention Area #4 is the City of Scottsdale parcel located on the east side of the project area which lies within the ESLO and therefore, like Retention Area #2, only the increase in runoff volume was calculated. Refer to Appendix A for the Drainage Area Map and the 100-year, 2-hour and pre vs. post runoff volume calculations.

2.3 SUMMARY OF REQUIRED RETENTION

The required 100-year, 2-hour retention volume from the Retention Areas #1 and #3 was calculated to be 19,452 and 18,975 cu.ft. respectively. The required retention volume from the previously disturbed areas within Retention Area #2 was calculated as a net decrease of 27,023 cu. ft. This decrease is due to the large grass turf areas which will significantly reduce runoff compared to the existing gravel parking lot. Finally, the increase in retention volume from development of the ESLO parcel (Retention Area #4) was found to be 8,025 cu.ft. The total required retention volume for the Westworld Sports Fields is 19,429 cu.ft.

2.4 VOLUME PROVIDED

The required storage volume is provided with the sports complex’s subsurface drainage systems. This includes the drainage pipes and the void space within the proposed 4-inch gravel layer under the sports fields. This subsurface storage volume is more than enough to provide the required retention volume. The parking lot storm drain, culverts and field drains provide 17,490 cu.ft. of storage volume. In addition, the open void space in the 4-inch gravel layer under the sports fields is 57,616 cu.ft., which is based on a porosity 35%. The combined subsurface storage volume is 75,106 cu. ft. Refer to Appendix A for the volume calculations associated with the subsurface drainage systems.

2.5 FIRST FLUSH RETENTION VOLUME

The first flush runoff from the project site will be retained within the dead storage pool behind Dike 4. The dead storage pool is the bottom portion of the reservoir that lies below the outlet structure. The reservoir outlet is at elevation 1510.5 feet whereas the bottom of the reservoir at the downstream end of Reata Wash is at an elevation of about 1500.0 ft. Therefore, floodwater runoff must pool to a depth of 10.5 feet before it can escape out of the outlet structure. The volume associated with the dead storage pool is about 450 acre-feet, which far exceeds the first flush runoff from the proposed sports complex. Moreover, the runoff that accumulates in the dead storage pool is pumped to the Water Campus where it is treated for reuse in the City's reclaimed water distribution system.

3.0 HYDROLOGIC ANALYSIS

3.1 APPROACH

The hydrologic analysis for the new Westworld Sports Fields 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).*

Two separate hydrologic models were used to determine design peak discharges for the project site. The first 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. It was used to determine the offsite flows that impact the site upstream of the Old Verde Canal. The second is a new HEC-1 model that was developed for the 100-year, 6- and 24-hour storm events utilizing the District's DDMSW software. The HEC-1 model was used to determine additional offsite flows from the Thompson Peak Parkway storm drain and the area downstream of the Old Verde Canal as well as developed condition peak discharges generated within the project site.

3.2 OFFSITE FLO-2D ANALYSIS

There is a significant drainage area that concentrates along the Old Verde Canal immediately upstream of the project site. As can be seen on the Watershed Map in Appendix B, the offsite watershed is roughly bound by Thompson Peak Parkway to the east, 98th Street to the west and extends upstream into the McDowell Mountain Preserve northeast of the intersection of Bell Road and Thompson Peak Parkway. There are six inflow locations into the Old Verde Canal. Four of the inflows enter the Canal north of McDowell Mountain Ranch Road, while the two others enter south of McDowell Mountain Ranch Road through existing culverts. The contributing drainage areas to these 6 inflows range from 4-acres to 143 acres. Refer to the Offsite Watershed FLO-2D Exhibit in Appendix B for the major inflow locations and the associated drainage area boundaries.

To analyze the upstream watershed, the PPS ADMS 100-year, 6- and 24-hour FLO-2D models were reviewed and modified to better represent existing flow conditions. Upstream of the project site, within the contributing drainage area to the Old Verde Canal, the modifications consisted of adjusting grid elevations and adding hydraulic structures to represent significant storm drain drains to 1) prevent flows from breaking out of the 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 the erroneous flow splits and diversions that can easily occur in FLO-2D models associated with large regional master drainage studies.

The 100-year, 6-hour and 24-hour peak discharges that impact the site are shown on the Offsite Watershed FLO-2D Exhibit in Appendix B. Due to the relatively large offsite watershed areas, the governing storm event for 5 out of the 6 inflow locations is the 100-year, 24-hour storm event. Refer to Appendix B for both the FLO-2D drainage area map and the inflow hydrographs for the 100-year 6- and 24-hour storm events. The digital data for the two FLO-2D models can be found in Appendix F.

While the FLO-2D model adequately represents the inflows into the Old Verde Canal, the 30’x30’ grids lack the detail required to properly represent the storage effects and drainage patterns along the Canal. Therefore, to determine the hydraulic impact of the Old Verde Canal and better define the flows that enter the project site, a two-dimensional US Army Corps of Engineers HEC-RAS model was developed that covers the Canal between 98th Street and Thompson Peak Parkway as well as the entire sports complex site. Refer to Section 4.0 for a more detailed discussion of the HEC-RAS model.

3.3 DESIGN CONDITIONS HEC-1 ANALYSIS

The design conditions HEC-1 hydrologic model was developed to determine the existing and proposed conditions runoff from the offsite and onsite areas downstream of the Old Verde Canal. The HEC-1 model includes existing conditions runoff from the offsite area that lies between the east side of the site and the Old Verde Canal. It also includes the offsite runoff from the existing Golf Course maintenance facility as well as the Thompson Peak Parkway 30-inch storm drain that both discharge to the South Wash and the runoff from the Westworld Equestrian Trailhead that drains to the North Wash. The onsite runoff is also incorporated into the HEC-1 model including the runoff from the proposed parking lot. The results of the onsite runoff analysis were used to design the parking lot drainage system and convey the flows to either the North or South Wash. Refer to Appendix C for the HEC-1 Schematic and Drainage Area Map showing the extents of the HEC-1 model and associated sub-basin drainage area boundaries.

To match the offsite FLO-2D inflow hydrographs, the design conditions HEC-1 model was developed for both the 100-year, 6- and 24-hour storm events. Due to the relatively small sub-basin drainage areas associated with the HEC-1 model, the 100-year, 6-hour storm event governs with slightly higher peak discharges as compared to the 100-year, 24-hour storm event. The design conditions HEC-1 model reflects the grading of the proposed parking lot which was done to create seven shallow sump locations where new grated catch basins will intercept the entire 100-year, 6-hour design peak discharge. Four of the parking lot catch basins will be connected to a new storm drain that discharges into the North Wash, while three other catch basins will be constructed with connector pipes that drain directly to either the North or South Wash. Refer to Appendix D for the location of the proposed parking lot catch basins.

As stated previously, the HEC-1 model includes the offsite inflows from the upstream undeveloped parcels as well as the Thompson Peak Parkway storm drain and the Golf Course maintenance yard. To collect the shallow flows from the upstream parcels, small ditches will be graded within the landscaped area behind the curb for the parking lot. These ditches will drain to one of the four proposed catch basins that were designed to intercept the offsite flows and convey them through the parking lot storm drain to the North Wash. Refer to the Exhibit in Appendix C for a summary of the HEC-1 results as well as the two design conditions HEC-1 models. Appendix F includes digital copies of the HEC-1 models.

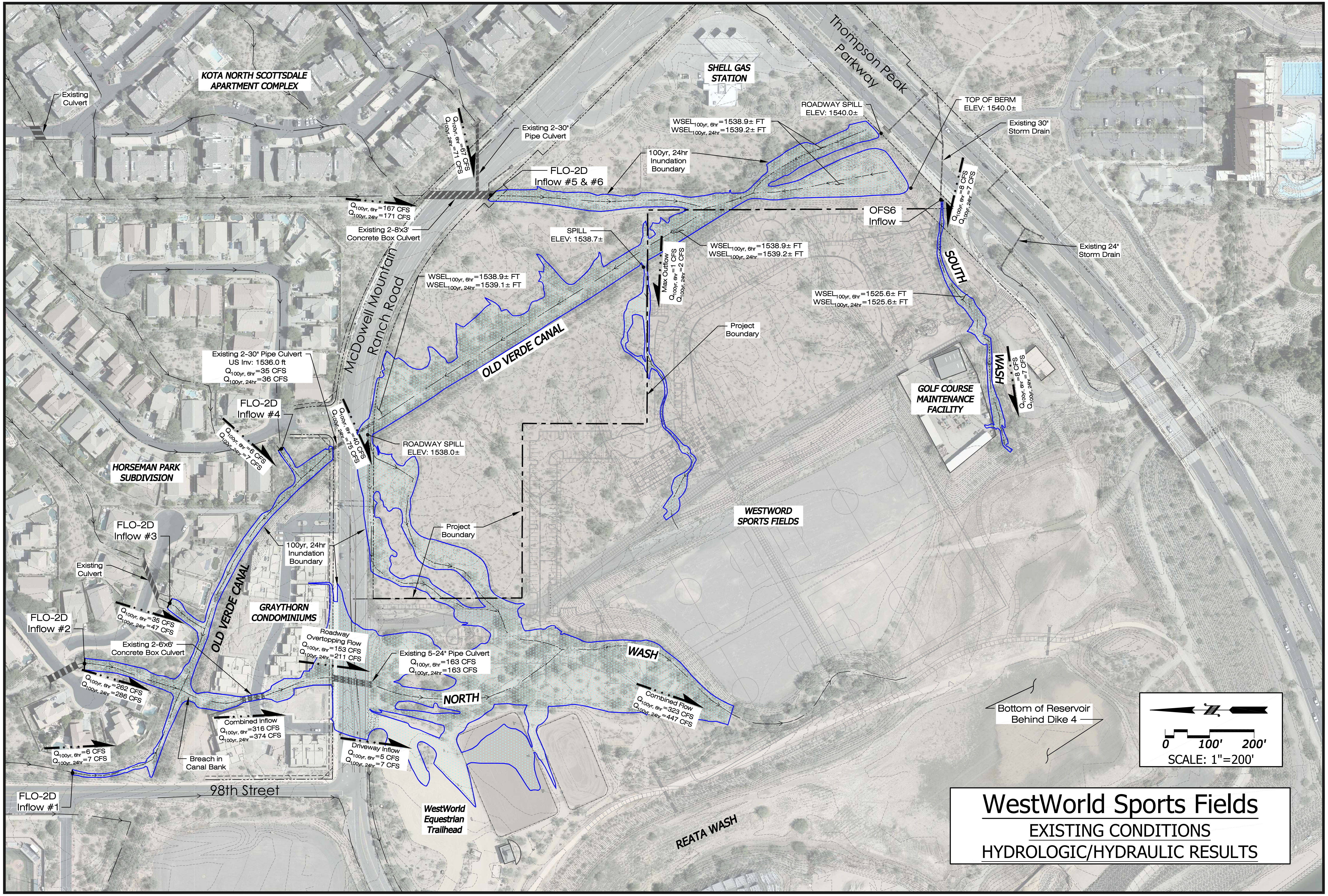
4.0 HEC-RAS MODEL DEVELOPMENT

The Old Verde Canal which is located upstream of the project site has a significant impact on the offsite flows. The Canal has a relatively flat longitudinal slope that intercepts runoff and diverts it in a northwesterly direction to a breach in the Canal bank just east of 98th Street. To model the hydraulic impact of the Canal, a fine grid (2’x2’ grid) HEC-RAS model was developed with recent, detailed topographic mapping of the project site and supplemented with 1-foot City of Scottsdale contour mapping for the area north of McDowell Mountain Ranch Road. As-built plans were used to update the 1-foot contour mapping by adding the drainage features of the recently completed Graythorn Condominiums on the northeast corner of 98th Street and McDowell Mountain Ranch Road.

4.1 EXISTING CONDITIONS HEC-RAS MODEL

The existing conditions HEC-RAS model was developed to determine the impact of the Old Verde Canal and the resulting flow patterns through the project site. The offsite flow that enters the Canal between McDowell Mountain Ranch Road and Thompson Peak Parkway is stored in the Canal and routed northwesterly through the existing dual 30-inch pipe culverts under McDowell Mountain Ranch Road. However, the capacity of the 30-inch culverts is too small to convey the entire 100-year flow. Flow that exceeds the capacity of the culverts will spill out of the Canal at McDowell Mountain Ranch Road and flow into the natural washes that run through the undeveloped parcel south of the roadway. A small amount of flow also spills out of the Canal south of McDowell Mountain Ranch Road through a low spot in the Canal bank. Both of these overflows from the Canal impact the project site. Refer to Figure 2 for the existing condition flows that spill out of the Old Verde Canal.

The offsite flows that enter the Old Verde Canal north of McDowell Mountain Ranch Road concentrate at the breach in the Canal bank located within the Graythorn Condominium complex approximately 200 feet east of 98th Street. The Old Verde Canal was preserved through the condominium complex between McDowell Mountain Ranch Road and 98th Street and a channel was graded to convey the flow from the breach in the Canal bank to the 5-24” pipe culverts underneath McDowell Mountain Ranch Road. The flow from these culverts enters the North Wash on the project site. The development of the condominium complex included a dual 6’x 6’ concrete box culvert under the interior driveway to convey flow from the Canal breach. This box culvert was not included in the HEC-RAS model with the assumption that it was properly sized to convey the 100-year peak discharges from the Old Verde Canal. Refer to Figure 2 for the location of the offsite flows that enter the Old Verde Canal north of McDowell Mountain Ranch Road.



WestWorld Sports Fields

EXISTING CONDITIONS HYDROLOGIC/HYDRAULIC RESULTS

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FIGURE 2: *Existing Conditions Hydrologic/ Hydraulic Results*

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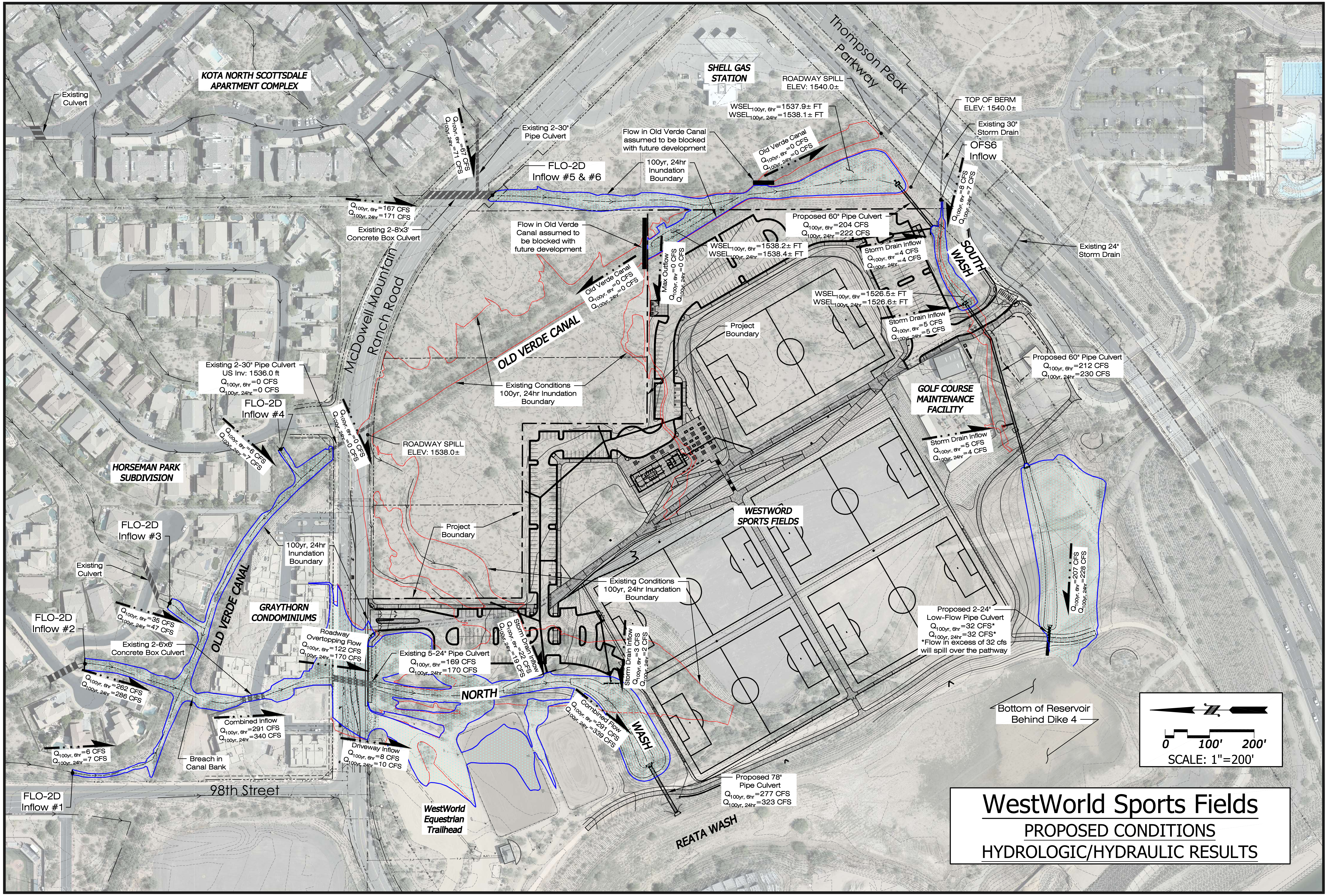
The six FLO-2D inflows to the Old Verde Canal were used as direct hydrograph inputs into the HEC-RAS model to determine the hydraulic impact of the Canal. The routing of the Canal inflows and incorporation of the existing pipe culverts underneath McDowell Mountain Ranch Road were used to determine the flows that impact the site. As can be seen in Figure 2, the HEC-RAS results indicate that there are three flows that impact the site. One is the 75cfs that spills out at McDowell Mountain Ranch Road. The second is the small 2cfs flow that spills out at the low spot in the Canal bank south of McDowell Mountain Ranch Road. The third is the 374cfs that concentrates in the condominium complex north of McDowell Mountain Ranch Road. This flow is conveyed under McDowell Mountain Ranch Road in five 24-inch culverts. But they only have enough capacity for about 163cfs during the 100-year, 24-hour storm event. The remaining 211cfs spills over the roadway. Refer to the Digital Data in Appendix F for the existing conditions HEC-RAS model

4.2 DESIGN CONDITIONS HEC-RAS MODEL

The design conditions HEC-RAS model was developed by incorporating the proposed drainage features for the Westworld Sports Fields into the existing conditions HEC-RAS model. These features include several new culverts within the project area, realignment of the North Wash, and revising the roadway spillover geometry where flow in the North Wash spills over McDowell Mountain Ranch Road. The spillover geometry had to be revised to account for the new sidewalk that will be installed with the project. The design conditions model was run for both the 100-year, 6- and 24-hour storms to analyze the proposed drainage infrastructure for the worst-case scenario.

The new culverts include a 60-inch pipe that diverts the 100-year flow from the Old Verde Canal into the South Wash. This revision also included the addition of artificial levees in the HEC-RAS model to block flow from entering the Old Verde Canal, thereby removing the effect of Canal storage. This resulted in a higher design flow for the 60-inch diversion pipe. The flow was blocked to allow the upstream property owners to fill in the Old Verde Canal, if they choose to do so with future development of their property. Another 60-inch pipe was also added to the design conditions model that conveys flow in the South Wash, under the southern driveway entrance and out to Reata Wash. In addition, a 78-inch pipe culvert was added that conveys the North Wash under the multi-use pathway into Reata Wash. Refer to Figure 3 for the location of the proposed pipe culverts and the location of the artificial levees used to prevent flow from entering the Old Verde Canal.



WestWorld Sports Fields

PROPOSED CONDITIONS HYDROLOGIC/HYDRAULIC RESULTS

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FIGURE 3
Proposed Conditions Hydrologic/ Hydraulic Results

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The design conditions HEC-RAS model also includes the HEC-1 hydrographs for the contributing area outside of the FLO-2D boundary. These include inflow hydrographs for the new parking lot storm drains and the existing storm drain in Thompson Peak Parkway. These storm drain flows discharge to the North and South Washes. See Figure 3 for the Proposed Conditions Hydrologic/Hydraulic Results and refer to the Digital Data in Appendix F for the HEC-RAS model. Section 6.0 provides a more complete discussion of the proposed drainage infrastructure.

5.0 STORM DRAIN DESIGN AND ANALYSIS

New storm drains were designed to collect and convey onsite flows from the proposed parking lot. These storm drains also capture small offsite flows from the adjacent properties. A new storm drain is also proposed that captures runoff the Golf Course maintenance yard and the filled in portion of the South Wash. The storm drains include seven new grated catch basins located in shallow sumps within the new parking lot intercepting flows from the new office/restroom hardscaped areas as well as the paved parking lot. Four of the grated catch basins are connected to the new parking lot storm drain that runs westerly through the northern portion of the parking lot. The three other catch basins drain directly into either the North or South Wash through single connector pipes.

Four new grated catch basins were designed to intercept the offsite flows from the adjacent properties. To limit the number of offsite catch basins, shallow collection ditches were graded within the landscaped area behind the parking lot curb to capture the offsite flows and convey them to the nearest offsite catch basin which are also positioned behind the parking lot curbs. Since the offsite flows originate from undeveloped desert lands, they can be expected to carry significant debris. Therefore, they were designed with raised grates that are 4-inches above the top of the catch basin wall. This provides a 4" high opening around the perimeter of the grate that that is less susceptible to clogging. A fifth catch basin was designed to intercept the offsite flows from the existing golf course maintenance yard as well as surface runoff from the filled in portion of the South Wash. The South Wash will be filled downstream of the driveway entrance. The new catch basin is in a sump to prevent flows from spilling over the Reata Wash embankment and eroding the new multi-use pathway. Refer to the Storm Drain and Culvert Design Location Exhibit in Appendix D for the location of the proposed offsite catch basins.

The storm drains were designed to intercept the governing 100-year, 6-hour peak discharges from the parking lot, hardscape areas and the adjacent, undeveloped offsite parcels. The grading plan includes shallow sumps in the parking lot at the catch basin locations as well behind the curb where the offsite catch basins are situated. These sumps allow the entire the 100-year, 6-hour runoff to be captured without overtopping. This approach ensures that all the runoff generated in the both the parking lot and the offsite watersheds will be intercepted and routed to either the North Wash through the new storm drain or to the South Wash through the 18-inch connector pipes. Refer to Appendix D for the catch basin inlet design calculations as well as the storm drain hydraulic grade line calculations.

6.0 CULVERT DESIGN & WASH HYDRUALIC ANALYSIS

6.1 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 design conditions HEC-RAS model that was documented in Section 4.0 was used to analyze the proposed culverts.

The two main drainageways that impact the site include the North and South Wash. The North Wash enters the north side of the project site at McDowell Mountain Ranch Road approximately 200 feet east of 98th Street. The South Wash runs along the south side of the project site and provides the outfall for the existing 30-inch storm drain in Thompson Peak Parkway. Both washes discharge to the existing floodwater retention area at the downstream end of Reata Wash. Refer to Figure 2 for the location of the North and South Wash and the existing conditions hydraulic results.

6.2 NORTH WASH HYDRAULIC DESIGN

The existing condition hydraulic analysis found that the peak discharge in the North Wash is 374 cfs for the governing 100-year, 24-hour storm event. As can be seen in Figure 2, the existing five 24-inch pipe culverts underneath McDowell Mountain Rach Road do not have enough capacity to convey the entire flow. Of the 374cfs, 163cfs flows through the culverts and the remaining 211 cfs overtops the roadway. Under existing conditions, the roadway has a one-way crown with a cross slope of 2.0% and no curb on the south edge of pavement. The water surface profile in Figure

4 shows that flow from the North Wash spills over the roadway with a maximum depth of 4-inches. Most of the flow that spills over roadway reenters the North Wash just downstream of McDowell Mountain Ranch Road. However, due to the slight longitudinal slope of the roadway toward the west, there is about 7cfs that enters the eastern driveway of the Westworld Equestrian Trailhead, flowing through the parking lot and horse arena before flowing back into the North Wash.

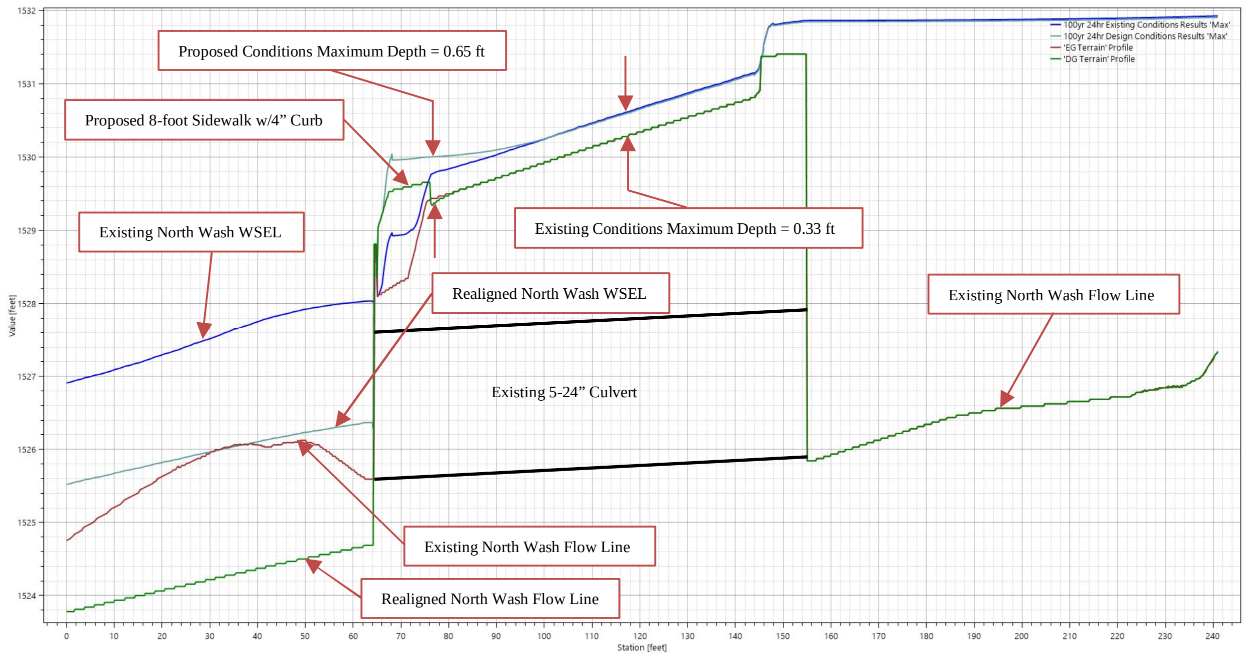


Figure 4: McDowell Mountain Ranch Road Water Surface Elevation Profile

The proposed conditions will include new curb, gutter, and sidewalk along the south side of McDowell Mountain Ranch Road. To keep the maximum depth in the roadway below the allowable 8-inches, a modified 4-inch curb will be used instead of the standard 6-inch curb. With the addition of the 4-inch curb and sidewalk, the maximum water surface elevation over the roadway is raised from 0.33 feet to 0.65 feet, which is slightly less than the maximum allowable depth of 8-inches. Refer to Figure 4 for the comparison of water surface elevations across McDowell Mountain Ranch Road for the existing and proposed conditions. Construction of the curb and sidewalk on the south side of McDowell Mountain Ranch Road will result in slightly more runoff being diverted into the Equestrian Trailhead parking lot. Under proposed conditions, approximately 10cfs enters the Trailhead driveway compared to 7cfs under existing conditions.

Downstream of McDowell Mountain Ranch Road, the North Wash is realigned to provide space for the northern most sports fields. As can be seen in Figure 3, the wash turns to the west before discharging into Reata Wash through a new 78-inch CMP pipe culvert. The pipe culvert was designed to discharge the entire upstream peak discharge of 339cfs underneath the multi-use pathway without flooding the sports fields or the parking lot. Refer to the Culvert Location Exhibit in Appendix D for the location of the pipe arch culvert and realigned North Wash as well as the Digital Data in Appendix F for the design conditions HEC-RAS model.

6.3 SOUTH WASH HYDRAULIC DESIGN

Currently, the only flow contributing to the South Wash is the 30-inch storm drain in Thompson Peak Parkway, but the proposed drainage plan is to route the entire flow from the Old Verde Canal that concentrates south of McDowell Mountain Ranch Road. Under existing conditions, during the governing 100-year, 24-hour storm event, the Canal overtops in two locations upstream of the project site. The main overtopping location is at McDowell Mountain Ranch Road where the existing dual 30-inch pipe culverts are exceeded which causes 75cfs to spill out of the Canal. This flow is routed through two natural washes downstream of the Canal and impacts the project site about 300 feet south of McDowell Mountain Ranch Road. The other overtopping location is where an unpaved access roadway crosses the Canal about halfway between McDowell Mountain Ranch Road and Thompson Peak Parkway. The flow associated with this overtopping is only 2cfs. Refer to Figure 2 for the existing conditions hydraulic results and the digital data in Appendix F for the existing conditions HEC-RAS model.

Allowing the offsite flows to continue to overtop the Old Verde Canal at these two locations would present significant challenges in terms of intercepting the flow at the project boundary. In addition, the parcels upstream of the project boundary would also face significant drainage design challenges in the future when they are developed.

To remedy the problem of overtopping Canal flows, the proposed offsite drainage plan is to divert the entire flow that concentrates within the Old Verde Canal between McDowell Mountain Ranch Road and Thompson Peak Parkway. Since the adjacent property owner's retention basin is hydraulically connected to the Old Verde Canal, the plan is to capture the flow in their retention basin with a large drop inlet structure and convey it in a 60-inch pipe culvert to the South Wash. The 60-inch pipe culvert was designed to convey the governing 100-year, 24-hour peak discharge of 222cfs. The sizing of the drop inlet structure and pipe culvert took into account future development that is assumed to block the flow in the Old Verde Canal, thereby eliminating the attenuation effect of the Canal storage on the design flow for the 60-inch diversion pipe. Refer to Figure 3 for the location proposed 60-inch pipe culvert as well as the location of the assumed future blockages of the Old Verde Canal.

The inlet structure and pipe culvert were designed to lower the water surface elevation in the remaining portion of the Canal and the retention basin to be below the existing conditions water surface elevations. Refer to Figure 3 for the proposed conditions hydraulic results of the Old Verde Canal, the Culvert Location Exhibit in Appendix D for the location of the proposed 60-inch pipe culvert and the digital data in Appendix F for the design conditions HEC-RAS model.

By diverting the flows from the Old Verde Canal to the South Wash, the peak discharge in the Wash will be increased from 7cfs under existing conditions to 230cfs for design conditions. As can be seen in Figure 3, this increased flow is contained in the existing wash and there is no existing drainage infrastructure downstream of the Old Verde Canal that will be impacted by the diverted flow. Moreover, the new culverts in the South Wash associated with the development of the sports complex were designed to accommodate the diverted flow. Therefore, the flow will not have a detrimental impact on any drainage conditions downstream of the Canal.

The diversion will not only improve the drainage conditions for the undeveloped land downstream of the Old Verde Canal, but it will also provide benefit to the properties located on the upstream side of the Old Verde Canal by allowing them to fill their portion of the Old Verde Canal and reclaim it as developable land.

A second drop inlet structure and 60-inch pipe culvert was also designed to pipe the flow in the South Wash from the upstream side of the south driveway entrance to Reata Wash. The benefit of conveying the South Wash in a culvert directly to Reata Wash is threefold. One benefit is that the pipe can be lower which will avoid conflict with the existing shallow 24-inch sewer. Secondly, the pipe will allow the South Wash to be filled in which will provide usable space for maintenance activities and other purposes. Finally, discharging directly to Reata Wash will convey the large flows in the South Wass underneath the new multi-use pathway that runs along the bank of Reata

Wash. Refer to Figure 3 for the extend hydraulic results, the Culvert Location Exhibit in Appendix D for the location of the proposed culvert and the Digital Data in Appendix F for the Design Conditions HEC-RAS model.

7.0 FEMA FLOOD ZONE / LOWEST FLOOR ELEVATION

The site is located within FEMA Flood Zone A (FEMA Map No. 04013C1340L, dated Oct. 16, 2013). The Zone A Floodplain does not include a Base Flood Elevation (BFE), but the BOR established a 100-year water surface elevation (WSEL) of 1526.00 ft (NGVD29) for the flood pool behind Dike 4. This is a very conservative estimate of the BFE because it includes a 100-year runoff volume of 2320 ac-ft plus a long-term sediment accumulation of 1080 ac-ft. With the level of development at Westworld, it seems very unlikely that 1080 ac-ft of sediment would be allowed to accumulate. If the site did experience such sediment loads, the City would be forced to remove the sediment, or it would cover much of the developed area within Westworld.

Since the site design is based on City of Scottsdale vertical datum (NAVD88), we converted the BOR's WSEL to NAVD88 using the National Geodetic Survey's VERTCON program. The conversion obtained from VERTCON is NGVD29 + 1.75 ft = NAVD88. Therefore, the WSEL for the flood pool behind Dike 4 is 1527.75 ft based on City of Scottsdale's vertical datum.

The finished floor of the site's Restroom/Office Building will be set at elevation 1528.75 or higher to be at least one foot above the BFE.

8.0 PRESERVATION OF BOR RESERVOIR VOLUME

Since the project site is located within the BOR's Dike 4 floodwater reservoir that protects the CAP Canal, the storage volume of the flood pool must be preserved. For purposes of calculating flood storage, the BOR distinguishes between LIVE storage and DEAD storage. LIVE storage is the reservoir volume that lies above the invert elevation of the outlet works whereas the DEAD storage is the reservoir volume that lies below the outlet works. Preserving the volume of LIVE storage is paramount, but it is acceptable to the BOR to move soil into the DEAD storage pool and reduce its volume, just so it is clean fill free of vegetation and deleterious materials.

8.1 LIVE VS DEAD STORAGE

As stated above, LIVE storage is the volume above the reservoir's outlet works and DEAD storage is the volume below. Therefore, the invert elevation of the outlet pipes must be known to calculate the LIVE and DEAD storage volumes.

Figure 5: Dike 4 Outlet Works Photograph

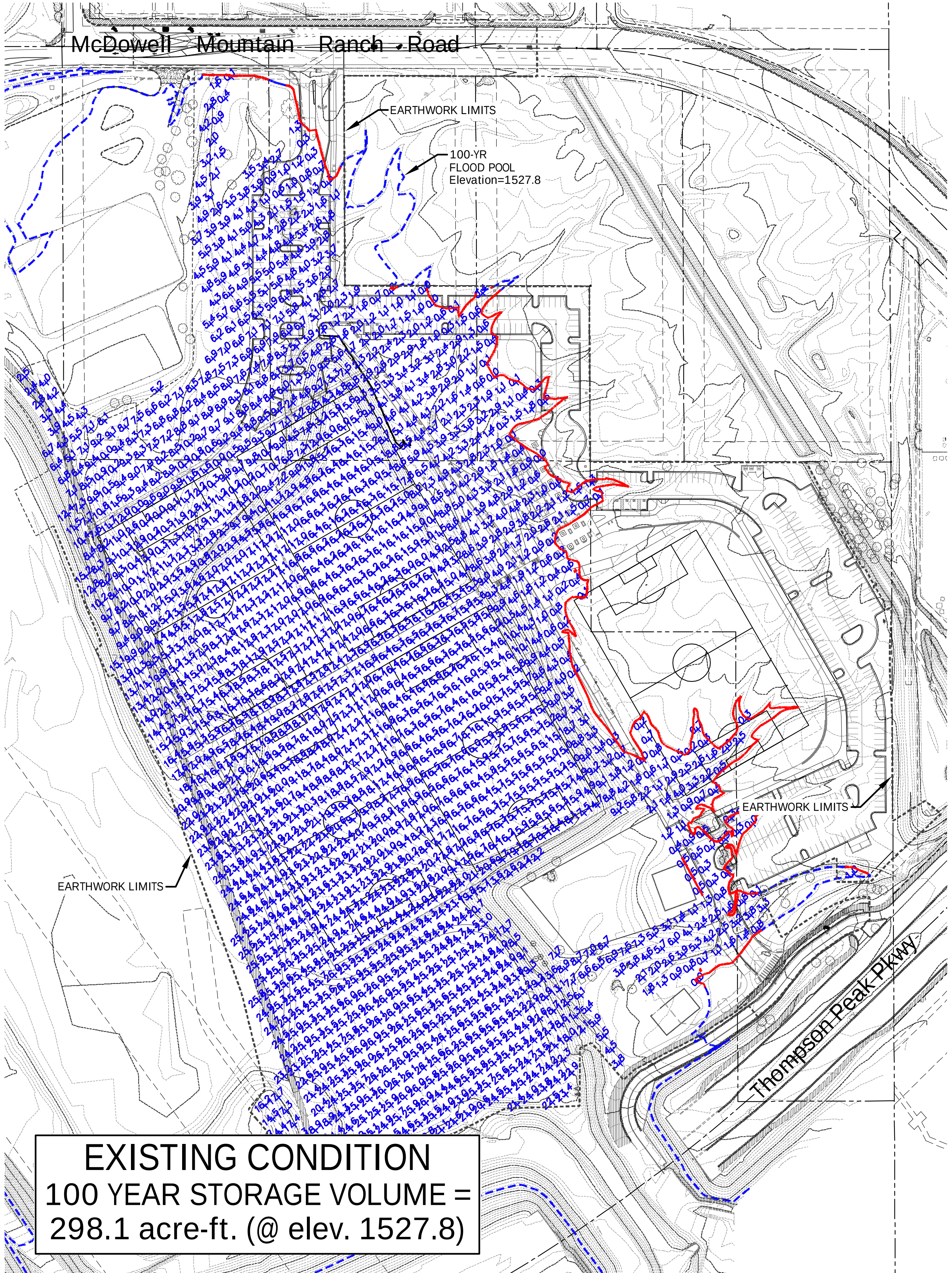
The outlet works for Dike 4 is located about 1,000 feet east of Thompson Peak Pkwy. It consists of 2-72" diameter pipes that discharge to the CAP Canal. We surveyed the level concrete apron in front of the outlet pipes and found it to be at an elevation of 1509.76. The as-built plans indicate that the invert elevation of the 72" pipes is 9 inches above the concrete apron which means the 72" invert is at an elevation of 1510.51. For purposes of the storage calculations, we rounded this elevation to 1510.5. Refer to Figure 5 for a photograph of the Dike 4 Outlet Works.

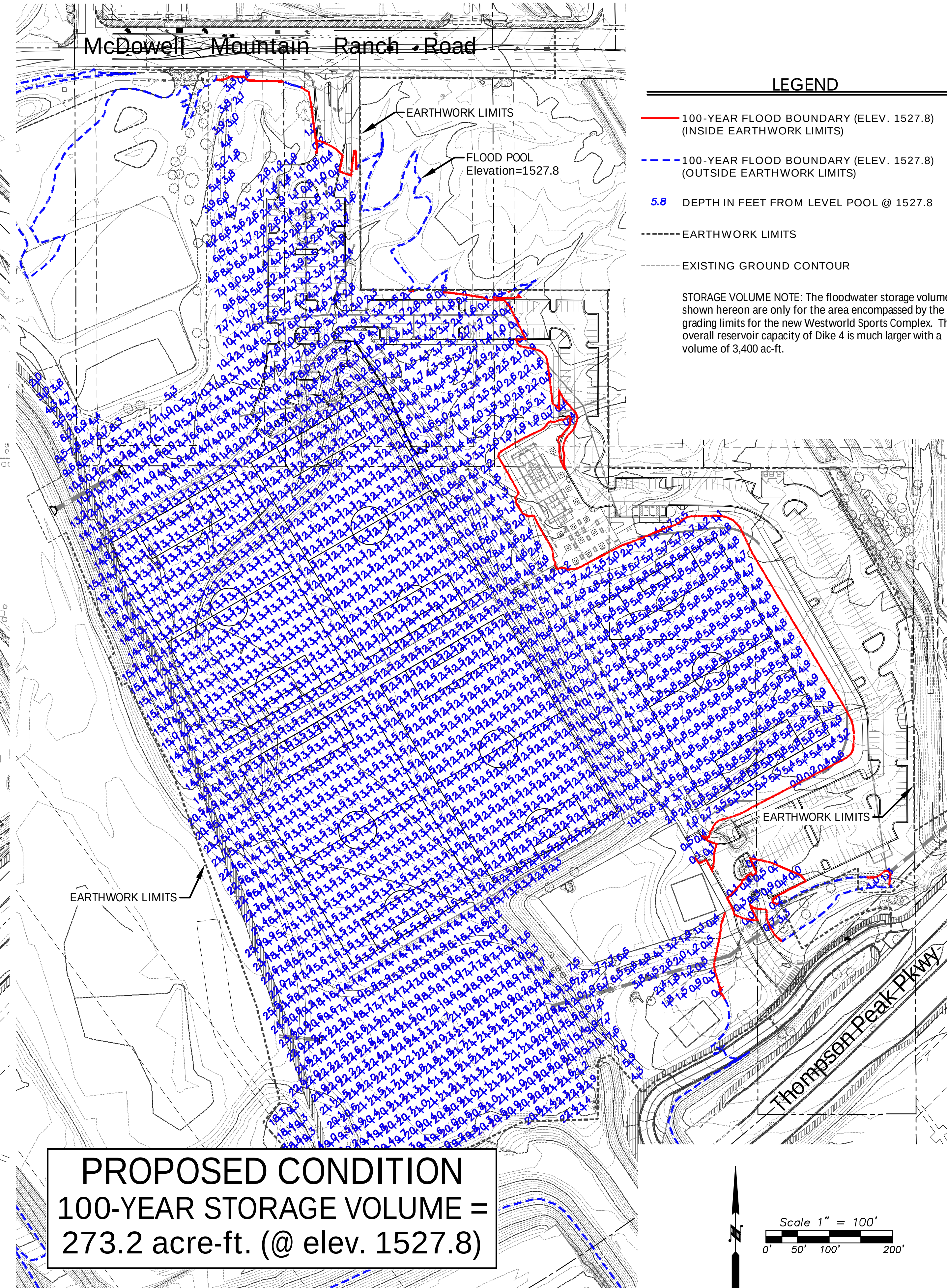
Since the invert of the outlet pipes are at elevation 1510.5, the DEAD storage within the reservoir is the volume below elevation 1510.5 and the LIVE storage is the volume above. We could not measure the invert elevation of the outlet pipes because they are enclosed behind a locked steel gate, but David Johnson with the BOR verified that the pipes are 9 inches above the apron.

8.2 100-YEAR STORAGE VOLUMES

We calculated the 100-YEAR storage volumes for both existing and proposed conditions. These calculations were only done for the area inside the grading limits of the proposed sports complex, no attempt was made to calculate the 100-YEAR storage volume for the rest of the reservoir.

Figure 6 shows the results of the analysis that was done to compute the 100-YEAR storage volumes. The exhibit shows the depth of water (below the 100-year WSEL) on a 25-foot grid for both the existing and proposed conditions. The results indicate that the existing volume is 298.1 acre-feet whereas the proposed volume is 273.2 acre-feet which is a reduction of 24.9 ac-ft, but the reduction occurs within the DEAD storage pool. The LIVE storage volume within the 100-year flood pool is preserved (see Section 8.4).





LEGEND

- 100-YEAR FLOOD BOUNDARY (ELEV. 1527.8) (INSIDE EARTHWORK LIMITS)
- 100-YEAR FLOOD BOUNDARY (ELEV. 1527.8) (OUTSIDE EARTHWORK LIMITS)
- 5.8 DEPTH IN FEET FROM LEVEL POOL @ 1527.8
- EARTHWORK LIMITS
- EXISTING GROUND CONTOUR

STORAGE VOLUME NOTE: The floodwater storage volumes shown hereon are only for the area encompassed by the grading limits for the new Westworld Sports Complex. The overall reservoir capacity of Dike 4 is much larger with a volume of 3,400 ac-ft.

Scale 1" = 100'

0' 50' 100' 200'

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G&B No.	2101
Issue Date:	09/21
Drawn By:	OK
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FIGURE 6
100-Year Reservoir Storage Volume Exhibit

Page Number:

The reduction in the 100-YEAR storage volume is primarily caused by importing building materials to construct the sports complex, including sand and gravel for the playing fields as well asphalt, aggregate base course and concrete to build the parking lots and walkways. These imported materials will equal about 32,000 cubic yards, or 19.8 ac-ft which is about 80% of the total 24.9 ac-ft reduction. The remaining 5.1 ac-ft of volume reduction is due to the material that is being excavated from above the 100-year flood level and moved down to the DEAD storage area. The crest of Dike 4 is 16 feet above the 100-year WSEL. This movement of material within the reservoir will increase the portion of the LIVE storage that lies above the 100-year flood level.

8.3 DEAD STORAGE VOLUMES

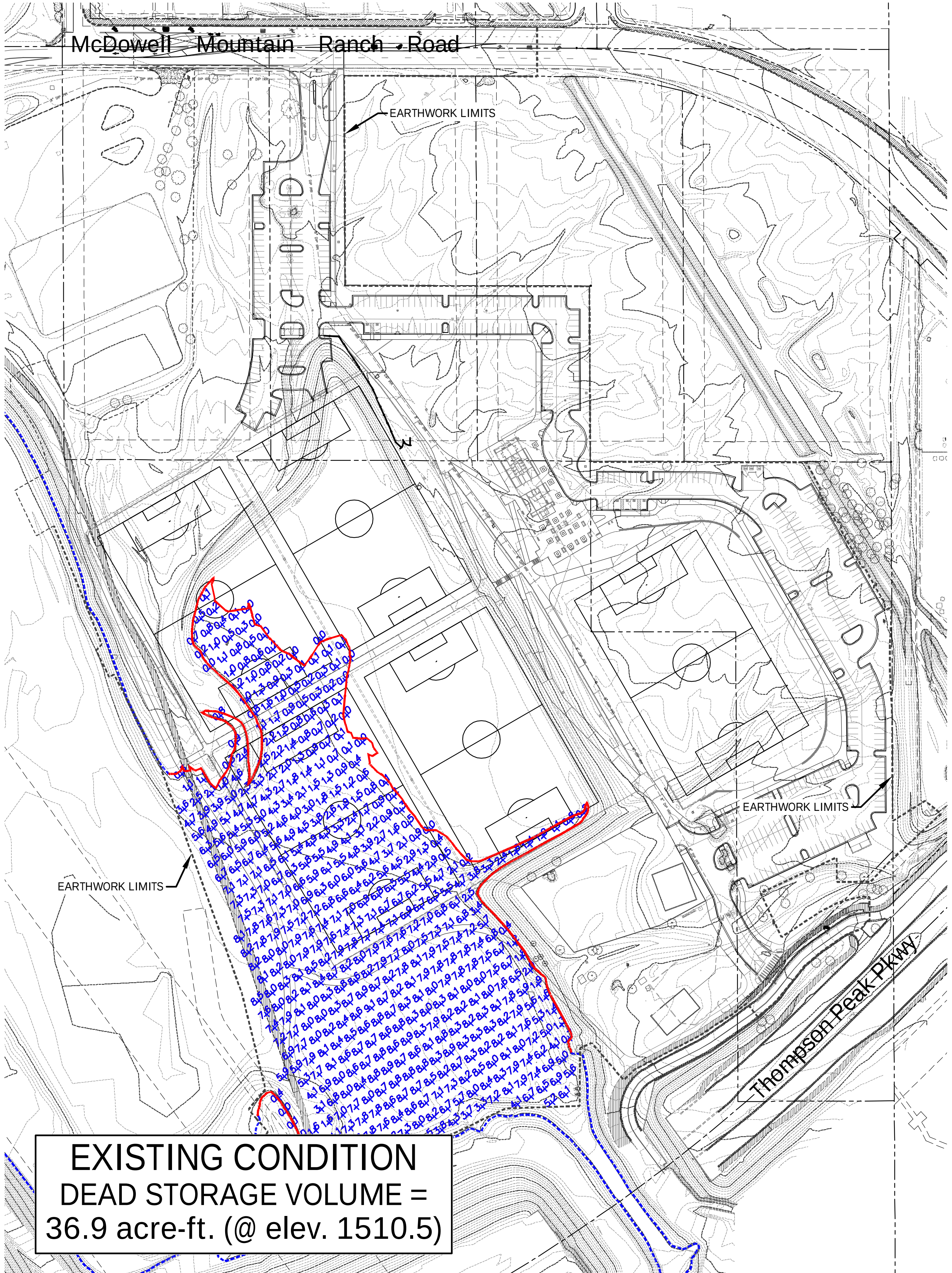
We also calculated the DEAD storage volumes for both existing and proposed conditions. Like the calculations for the 100-year storage volumes, they were only done for the grading limits of the proposed sports complex, no attempt was made to calculate the DEAD storage volume for the rest of the reservoir. Figure 7 shows the results which indicate that the existing DEAD storage volume is 36.9 acre-feet compared to the proposed volume of 11.1 acre-feet, a reduction of 25.8 ac-ft. As stated previously, the BOR allows the DEAD storage pool to be reduced. It's only the LIVE storage that must be preserved.

8.4 LIVE STORAGE PRESERVATION

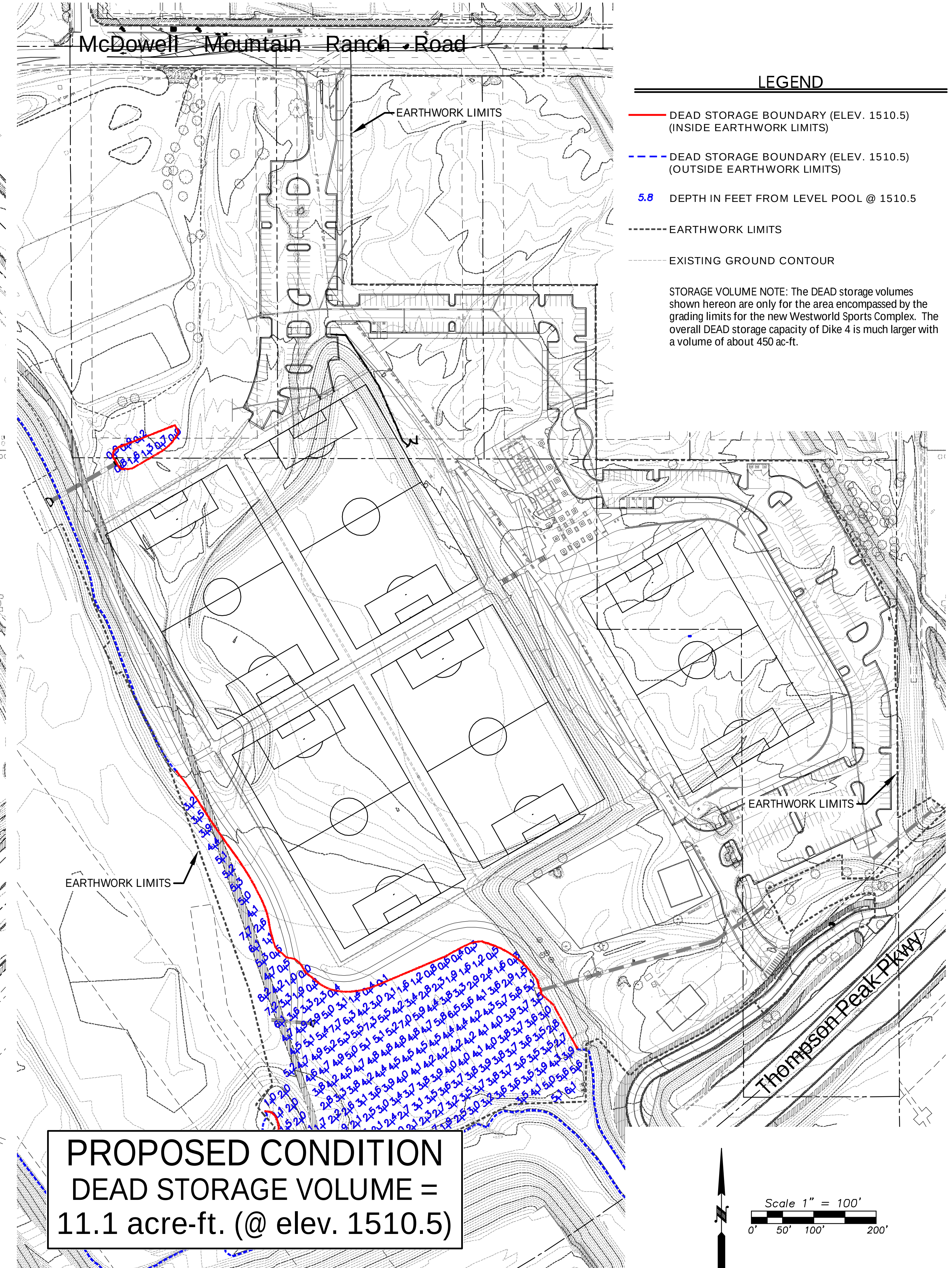
The LIVE storage volumes were determined by subtracting the DEAD storage volume from the 100-YEAR volume.

- **Existing** LIVE Storage = 298.1 ac-ft (100-YEAR Vol.) – 36.9 ac-ft (DEAD Storage Vol) = **261.2 ac-ft**
- **Proposed** LIVE Storage = 273.2 ac-ft (100-YEAR Vol.) – 11.1 ac-ft (DEAD Storage Vol) = **262.1 ac-ft**

Based on these calculations, the LIVE storage will be preserved. In fact, it will increase by 0.9 ac-ft within the 100-year flood pool. Moreover, as described in Section 8.2, the LIVE storage above the 100-year flood pool will also increase by 5.1 ac-ft. This is due to the excavated material that currently lies above the 100-year flood level which will be moved down to the DEAD storage area.



EXISTING CONDITION
DEAD STORAGE VOLUME =
36.9 acre-ft. (@ elev. 1510.5)



PROPOSED CONDITION
DEAD STORAGE VOLUME =
11.1 acre-ft. (@ elev. 1510.5)

LEGEND

- DEAD STORAGE BOUNDARY (ELEV. 1510.5) (INSIDE EARTHWORK LIMITS)
- DEAD STORAGE BOUNDARY (ELEV. 1510.5) (OUTSIDE EARTHWORK LIMITS)
- 5.8 DEPTH IN FEET FROM LEVEL POOL @ 1510.5
- EARTHWORK LIMITS
- EXISTING GROUND CONTOUR

STORAGE VOLUME NOTE: The DEAD storage volumes shown hereon are only for the area encompassed by the grading limits for the new Westworld Sports Complex. The overall DEAD storage capacity of Dike 4 is much larger with a volume of about 450 ac-ft.

N

Scale 1" = 100'

0' 50' 100' 200'

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FIGURE 7
Dead Storage Volume Exhibit

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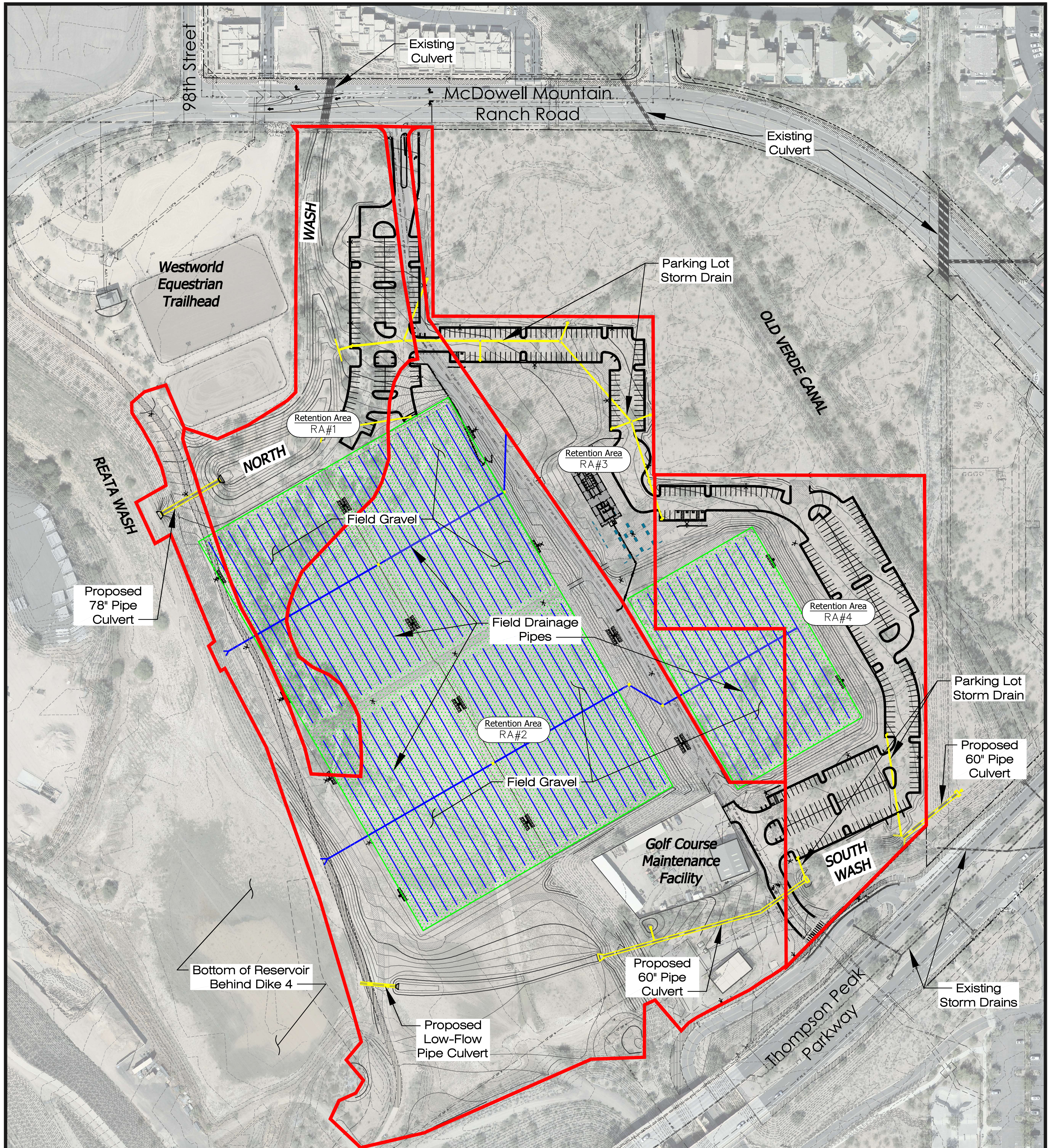
Appendix A: Stormwater Retention Calculations

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



LEGEND

Retention Area Boundary

100-yr, 2-hr RUNOFF VOLUME SUMMARY TABLE

RETENTION AREA	Contributing Drainage Area (sq/ft)	100-yr, 2-hr Runoff Volume (cu.ft.)
RA#1	203,650	19,452
RB#3	175,740	18,975

PRE. vs. POST RUNOFF VOLUME SUMMARY TABLE

RETENTION AREA	Contributing Drainage Area (sq/ft)	Pre Development Runoff Volume (cu.ft.)	Post Development Runoff Volume (cu.ft.)	Increase in Runoff Volume (cu.ft.)
RA#2	818,800	102,621	75,598	-27,023
RB#4	249,030	21,946	29,971	8,025

NOTES:

1) All required retention storage for the Westworld Sports Fields site is provided within the subsurface drainage system of the new sports fields.
2) Under existing conditions, the project site consists of undeveloped desert and previously developed gravel parking areas and access roads. The retention requirements are as follows:
   A. Retention Areas #1 and #3 consists of undeveloped desert. Therefore, the full 100-year, 2-hour runoff was included in the required retention volume.
   B. Retention Area #2 of the project site has been previously developed and therefore only the increase runoff was included in the required retention volume.
   C. Retention Area #4 consists of undeveloped desert. However, since it is located within the ESL Ordinance, only the increase in runoff volume was added to the retention requirement.

REQUIRED RUNOFF VOLUME

TOTAL RUNOFF VOLUME = RA#1 + RA#2 + RA#3 + RA#4

TOTAL RUNOFF VOLUME = 19,452 - 27,023 + 18,975 + 8,025

TOTAL RUNOFF VOLUME = 19,429 cu.ft

PROVIDED STORAGE VOLUME

SUBSURFACE STORAGE VOLUME = Field Pipes + Field Gravel + Storm Drain + Culverts

SUBSURFACE STORAGE VOLUME = 3,170 + 57,616 + 2,720 + 11,600

SUBSURFACE STORAGE VOLUME = 75,106 cu.ft

N

0 100' 200'

SCALE: 1"=200'

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Sheet Title :

Retention Volume Drainage Area Map

Sheet Number:

1

1 of 1

100-year, 2-hour Volume Calculation

100-year, 2-hour Runoff Volume Calculations

WestWorld
Multi-Use Sports Fields
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Retention Area#1: 100-yr 2-hr Runoff Volume

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	35,540	0.95	33,763.0	2.35	6,611.9
Desert Landscaping	100,880	0.45	45,396.0	2.35	8,890.1
Grass Areas (Turf Fields)	67,230	0.30	20,169.0	2.35	3,949.8
Total Contributing Drainage Area:	203,650			Total 100-year, 2-hour Runoff Volume	19,452

Retention Area#3: 100-yr 2-hr Runoff Volume

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	61,850	0.95	58,757.5	2.35	11,506.7
Desert Landscaping	26,450	0.45	11,902.5	2.35	2,330.9
Grass Areas (Turf Fields)	87,440	0.30	26,232.0	2.35	5,137.1
Total Contributing Drainage Area:	175,740			Total 100-year, 2-hour Runoff Volume	18,975

^The 100-year, 2-hour rainfall depth was obtained from Appendix 4-1D 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.*

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

Retention Area #2: Pre vs Post 100-year, 2-hour Runoff Volume Calculations

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Retention Area #2: Pre Development 100-yr 2-hr Runoff Volume

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Maintenance Yard & Pathway)	49,600	0.95	47,120.0	2.35	9,227.7
Gravel Access Road & Parking Area	293,080	0.82	240,325.6	2.35	47,063.8
Desert Landscaping	525,720	0.45	236,574.0	2.35	46,329.1
Total Contributing Drainage Area:	818,800			Total Pre Development Runoff Volume	102,621

Retention Area #2: Post Development 100-yr 2-hr Runoff Volume

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	132,140	0.95	125,533.0	2.35	24,583.5
Gravel Access Road & Parking Area	15,300	0.82	12,546.0	2.35	2,456.9
Desert Landscaping	310,290	0.45	139,630.5	2.35	27,344.3
Grass Areas (Turf Fields)	361,070	0.30	108,321.0	2.35	21,212.9
Total Contributing Drainage Area:	818,800			Total Post Development Runoff Volume	75,598
				Total Pre vs. Post Runoff Volume Increase :	-27,023

^The 100-year, 2-hour rainfall depth was obtained from Appendix 4-1D 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.*

Retention Area #4: Pre vs Post 100-year, 2-hour Runoff Volume Calculations

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Retention Area #4: Pre Development 100-yr 2-hr Runoff Volume

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Undeveloped Desert	249,030	0.45	112,063.5	2.35	21,945.8
Total Contributing Drainage Area:	249,030			Total Pre Development Runoff Volume	21,946

Retention Area #4: Post Development 100-yr 2-hr Runoff Volume

Cover Type	Area (A) (sq.ft)	Runoff Coefficient* (C)	Area x C	Rainfall Depth^ (inches)	Runoff Volume (cu.ft)
Impermeable (Sidewalk, Parking, etc)	104,200	0.95	98,990.0	2.35	19,385.5
Desert Landscaping	36,930	0.45	16,618.5	2.35	3,254.5
Grass Areas (Turf Fields)	74,130	0.30	22,239.0	2.35	4,355.1
Undeveloped Desert (NAOS)	33,770	0.45	15,196.5	2.35	2,976.0
Total Contributing Drainage Area:	249,030			Total Post Development Runoff Volume	29,971
				Total Pre vs. Post Runoff Volume Increase :	8,025

^The 100-year, 2-hour rainfall depth was obtained from Appendix 4-1D 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.*

Subsurface Storage Volume Calculation

Subsurface Storage Volume Calculations

WestWorld
Multi-Use Sports Fields
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Project No.: PG09

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Parking Lot Storm Drain Pipes

Storm Drain Pipe Diameters	Total Pipe Length (ft)	Pipe Cross-Sectional Area (sq.ft)	Total Pipe Volume (cu.ft)
15-inch	660	1.23	809.9
18-inch	370	1.77	653.8
24-inch	400	3.14	1,256.6
		Total Storm Drain Pipe Volume:	2,720

Culverts

Culvert Diameter and Type	Total Culvert Length (ft)	Culvert Cross-Sectional Area (sq.ft)	Total Culvert Volume (cu.ft)
60-inch Pipe Culvert^	388	19.63	7,618.4
78-inch Pipe Culvert	120	33.18307237	3,982.0
		Total Culvert Volume:	11,600

^Does not include the portion of the 60" Culvert that is above the Dike 4 Flood Pool Elevation of 1527.80 ft

Sand-Based Multi-Use Field Drain Pipes

Field Drain Pipe Diameters	Total Pipe Length (ft)	Pipe Cross-Sectional Area (sq.ft)	Total Pipe Volume (cu.ft)
4-inch	22,200	0.0871	1,933.4
12-inch	1,575	0.7854	1,237.0
		Total Field Drain Pipe Volume:	3,170

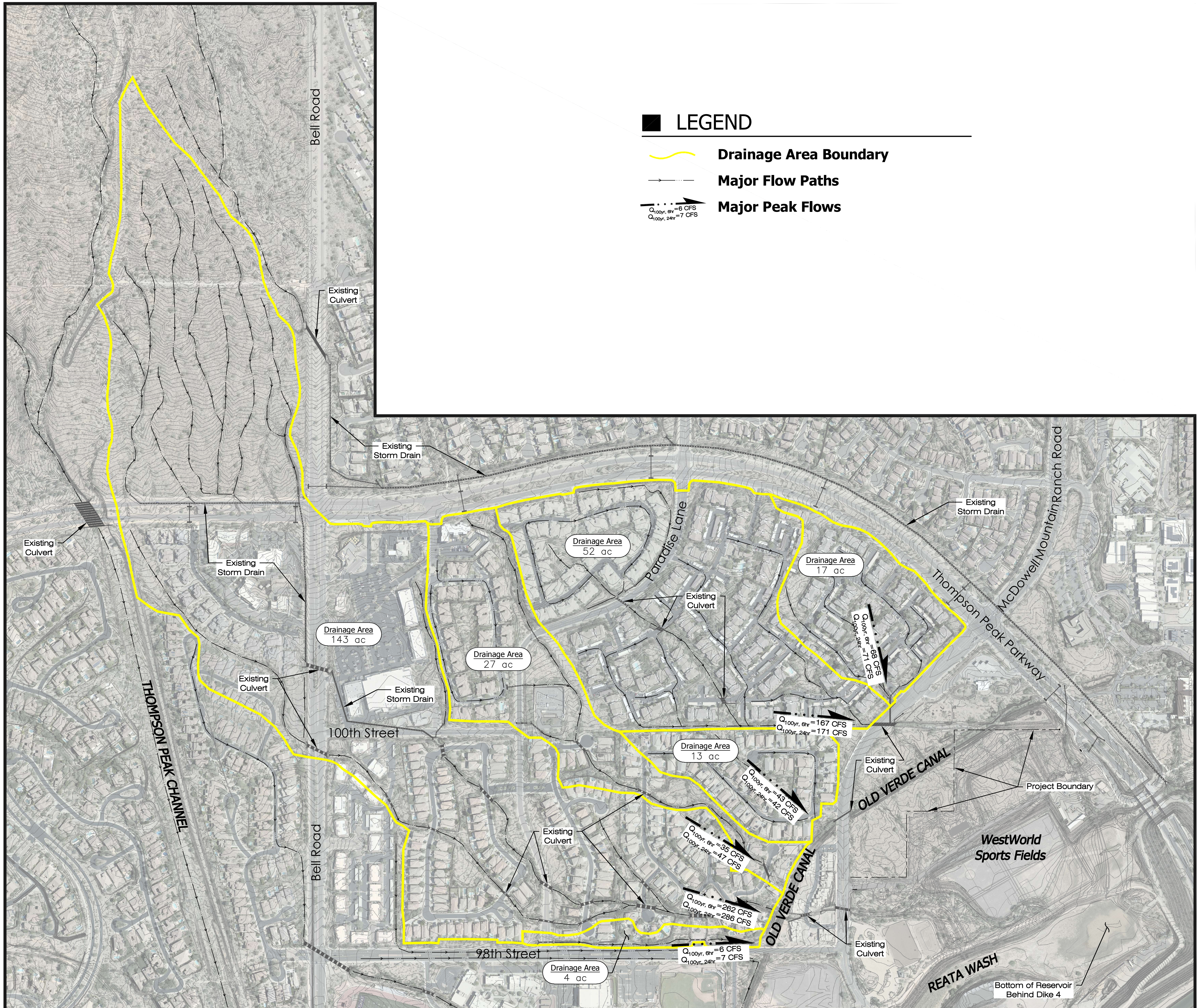
Sand-Based Multi-Use Field Gravel Layer

Gravel Layer	Total Gravel Surface Area (sq.ft)	Gravel Porosity* (%)	Total Open Void Space (cu.ft)
4-inch Thick Gravel Layer	498,840	35%	57,616.0
		Total Field Gravel Layer Volume:	57,616

*A porosity of 35% was used to calculate the total open void space in the field gravel layer.

Appendix B: Offsite Hydrologic Analysis

Offsite FLO-2D Model Watershed Map



0 300' 600'

SCALE: 1"=600'

WestWorld Sports Fields

OFFSITE WATERSHED FLO-2D EXHIBIT

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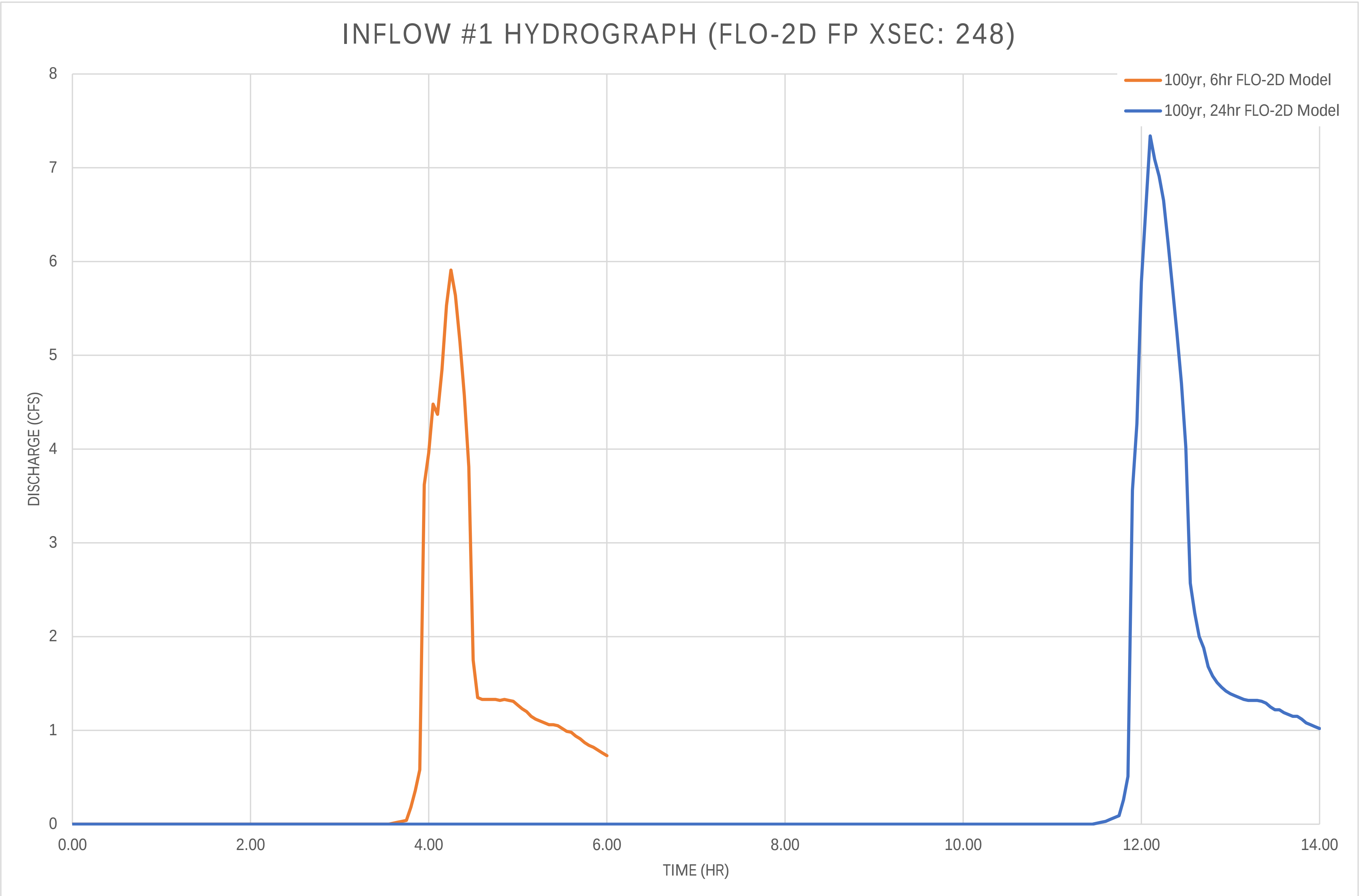
Offsite Watershed FLO-2D Exhibit

Sheet Number:

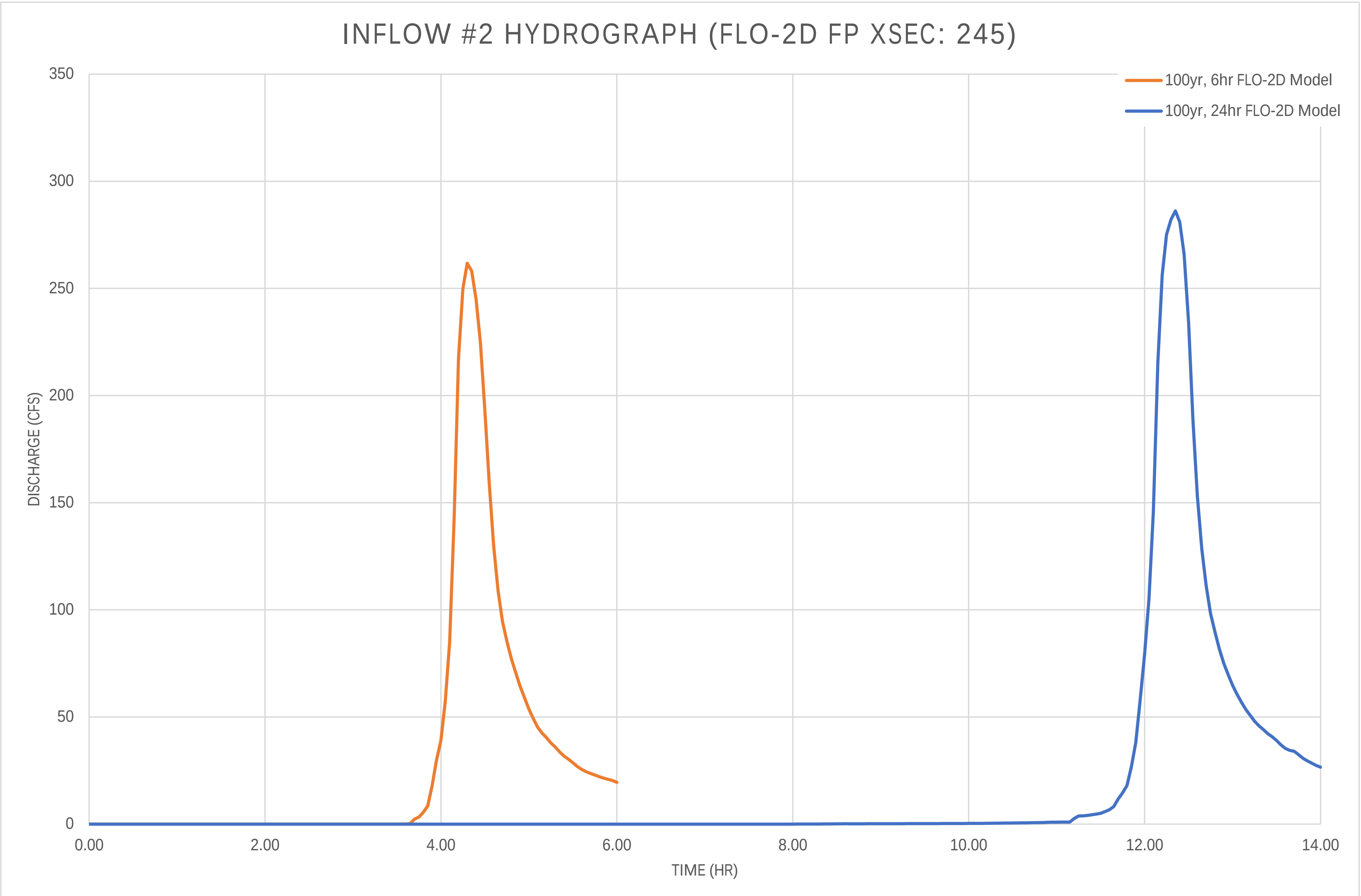
1

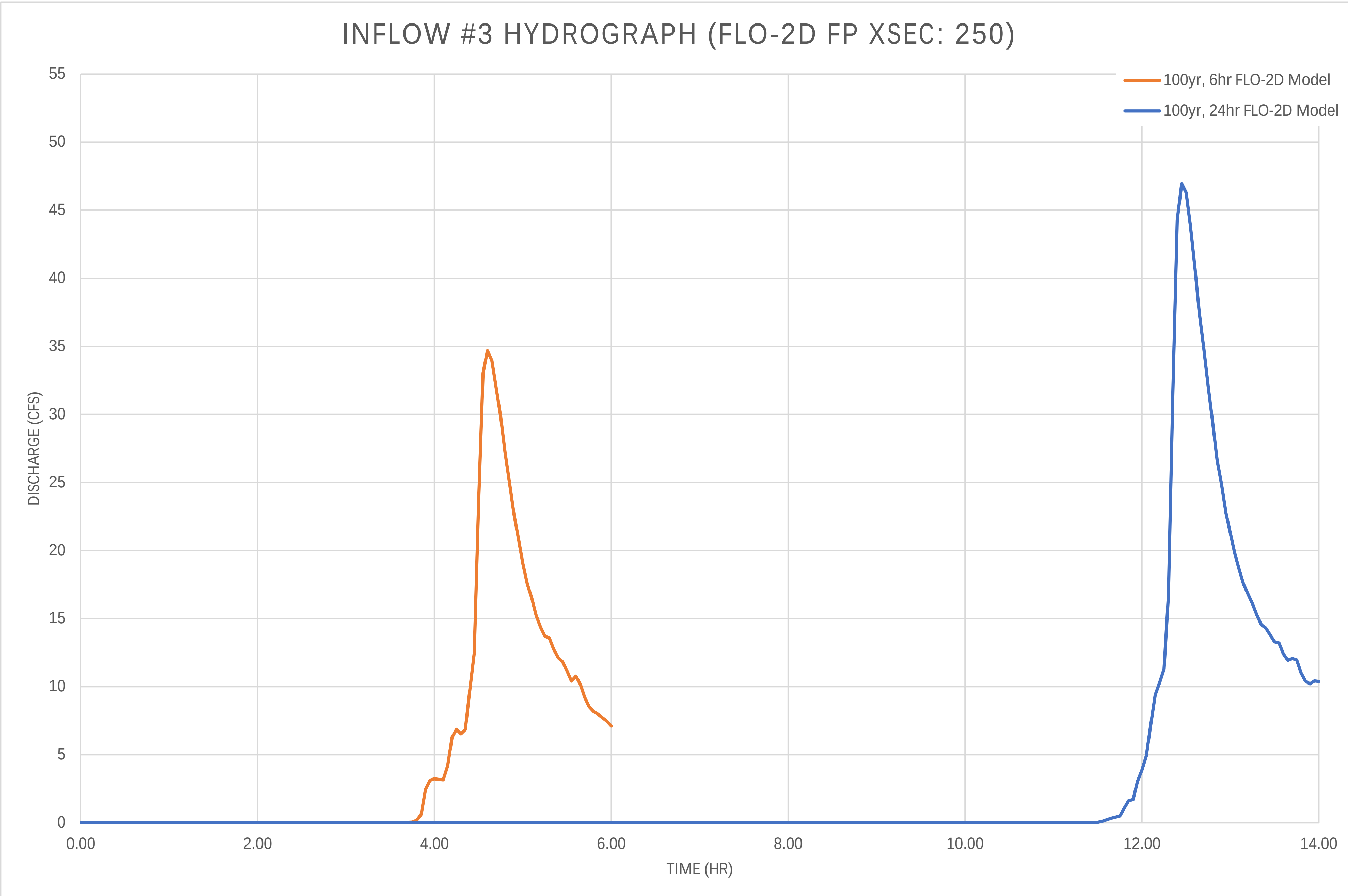
1 of 1

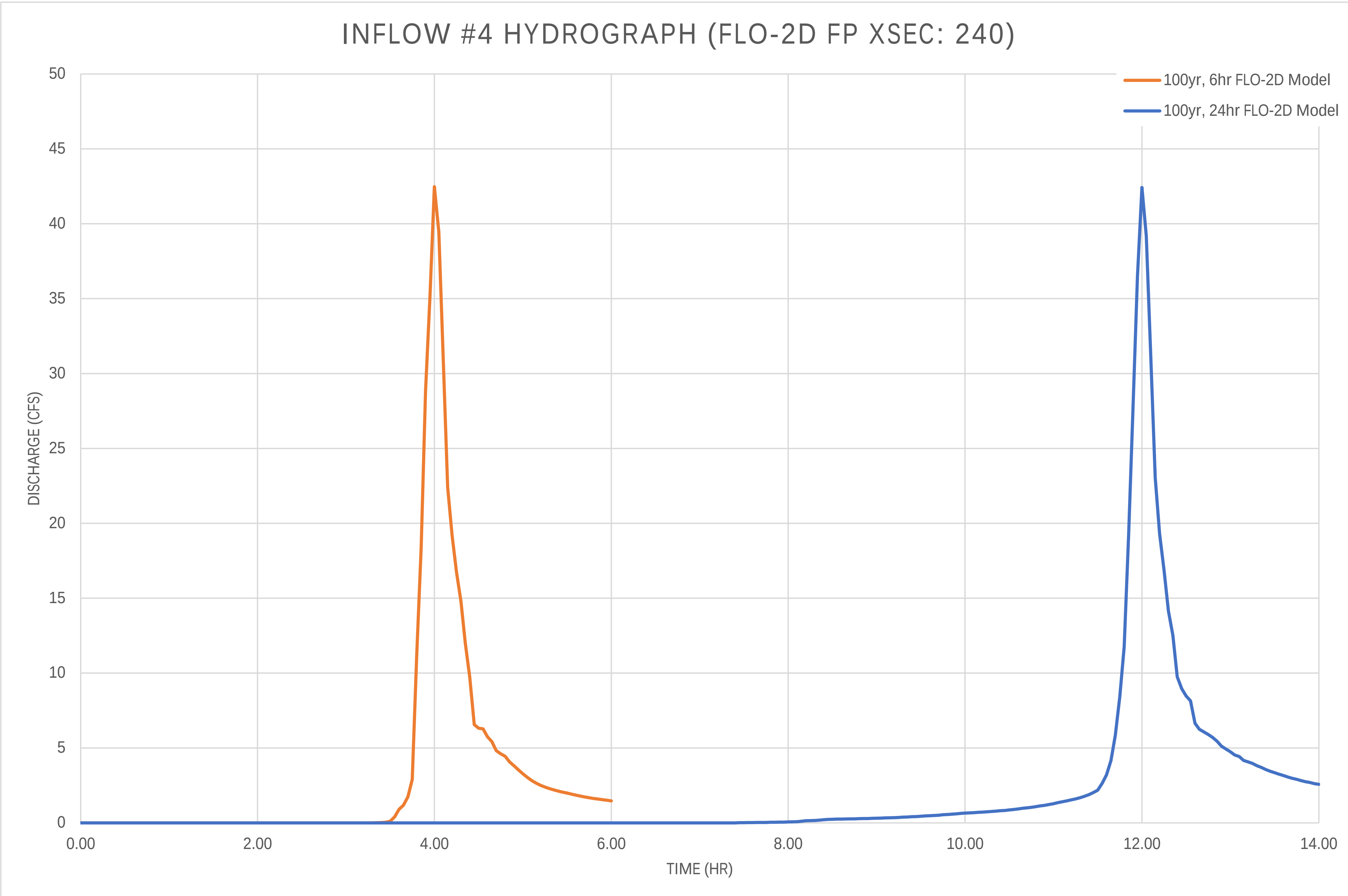
Offsite FLO-2D Model Inflow Hydrographs

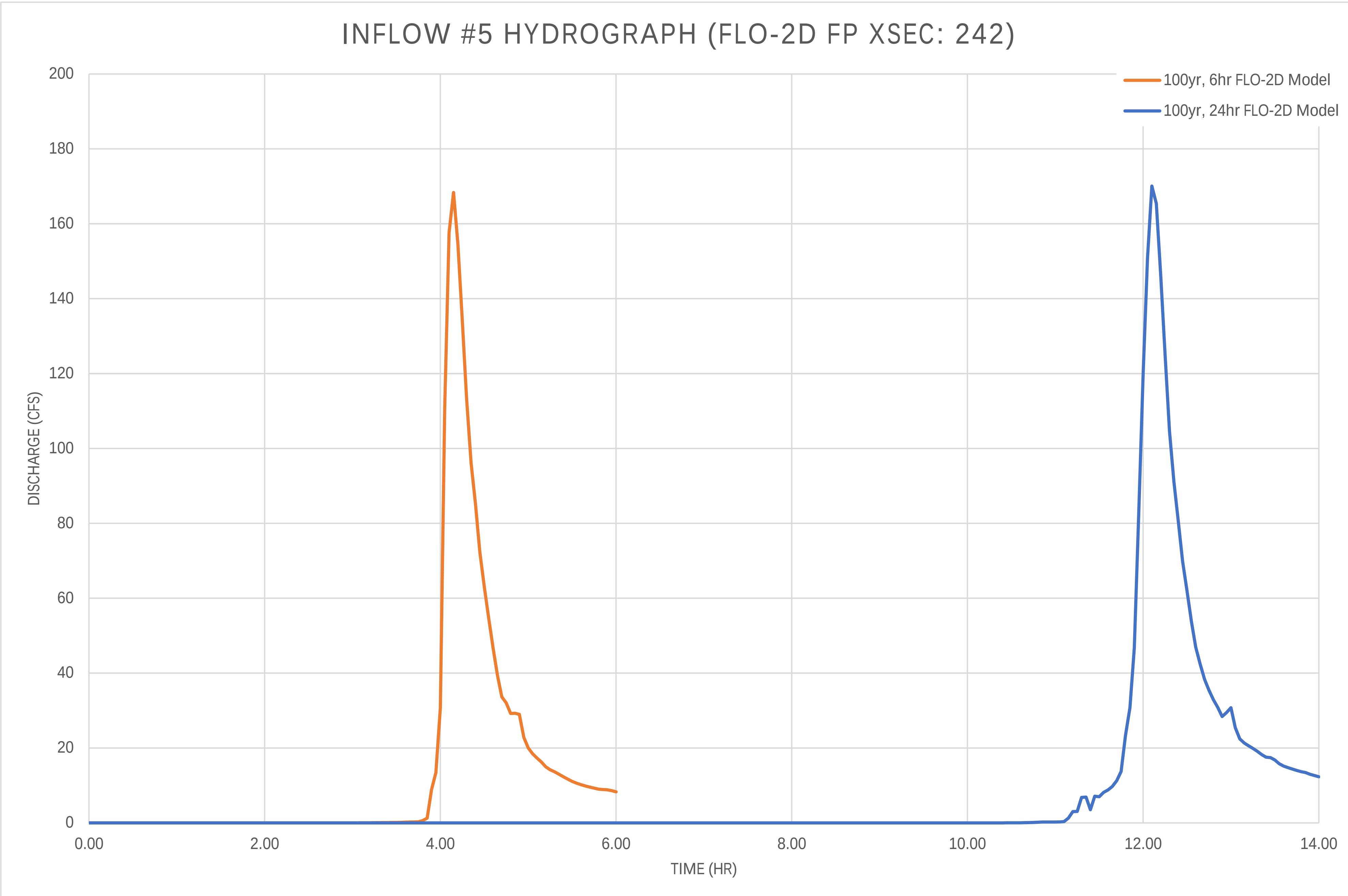


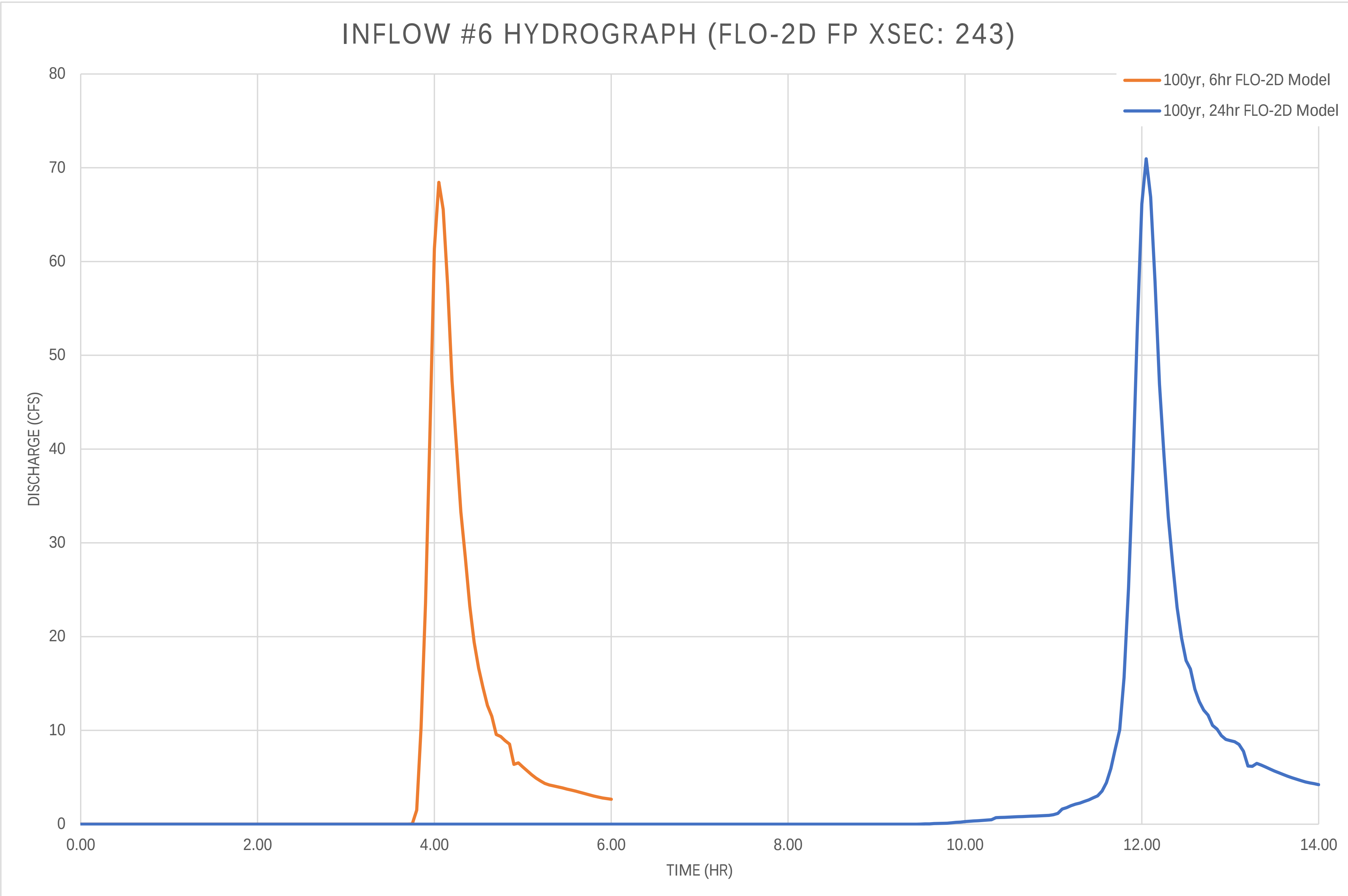
INFLOW #1 HYDROGRAPH (FLO-2D FP XSEC: 248)
100yr, 6hr FLO-2D Model
100yr, 24hr FLO-2D Model
DISCHARGE (CFS)
TIME (HR)
0
1
2
3
4
5
6
7
8
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00


INFLOW #2 HYDROGRAPH (FLO-2D FP XSEC: 245)
100yr, 6hr FLO-2D Model
100yr, 24hr FLO-2D Model
DISCHARGE (CFS)
350
300
250
200
150
100
50
0
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
TIME (HR)


INFLOW #3 HYDROGRAPH (FLO-2D FP XSEC: 250)
100yr, 6hr FLO-2D Model
100yr, 24hr FLO-2D Model
DISCHARGE (CFS)
TIME (HR)
0
5
10
15
20
25
30
35
40
45
50
55
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00

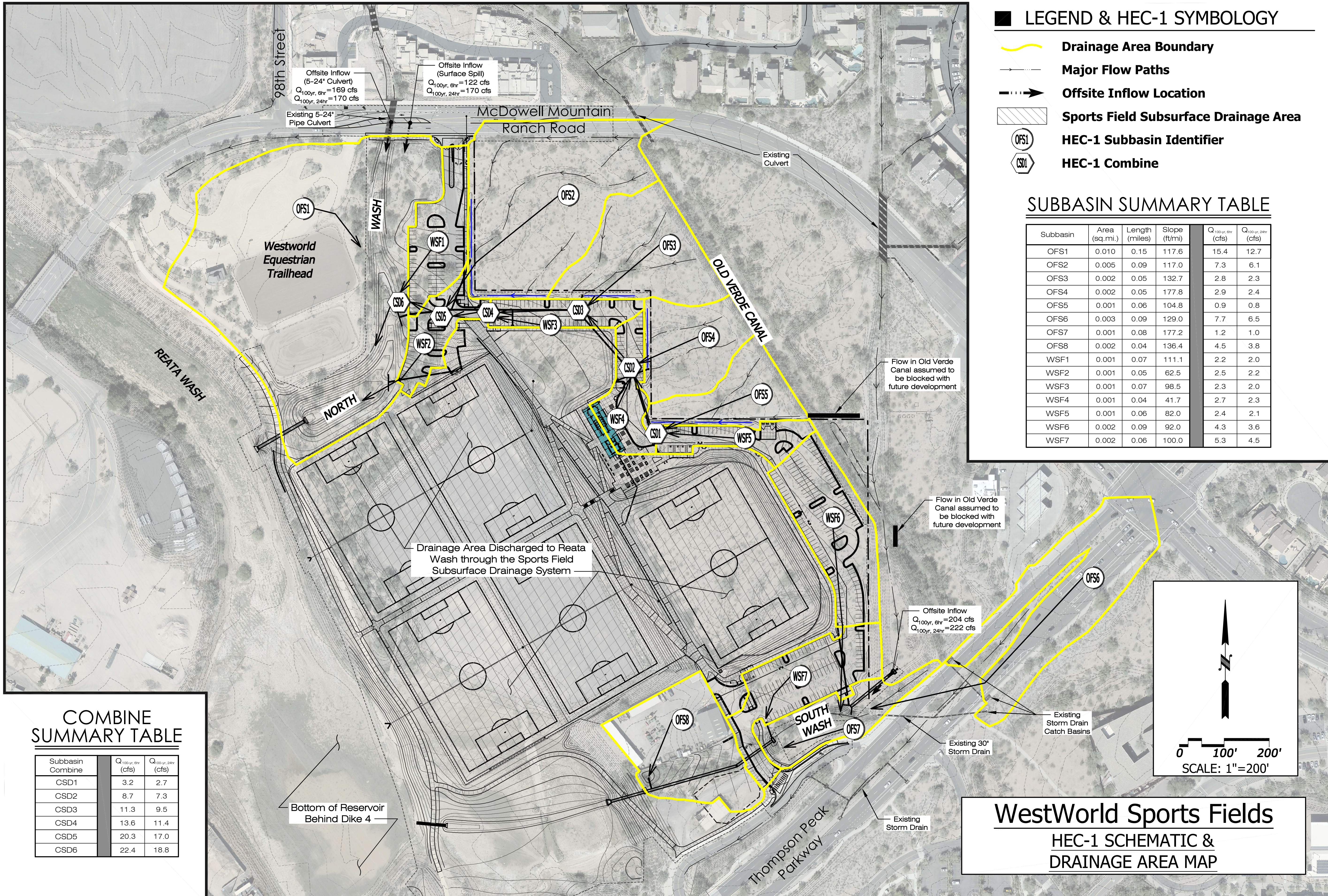

INFLOW #4 HYDROGRAPH (FLO-2D FP XSEC: 240)
100yr, 6hr FLO-2D Model
100yr, 24hr FLO-2D Model
DISCHARGE (CFS)
50
45
40
35
30
25
20
15
10
5
0
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
TIME (HR)


INFLOW #5 HYDROGRAPH (FLO-2D FP XSEC: 242)
100yr, 6hr FLO-2D Model
100yr, 24hr FLO-2D Model
DISCHARGE (CFS)
0
20
40
60
80
100
120
140
160
180
200
TIME (HR)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00


INFLOW #6 HYDROGRAPH (FLO-2D FP XSEC: 243)
100yr, 6hr FLO-2D Model
100yr, 24hr FLO-2D Model
DISCHARGE (CFS)
TIME (HR)
0
10
20
30
40
50
60
70
80
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00

Appendix C: Design Hydrologic Analysis

Design HEC-1 Schematic and Drainage Area Map



LEGEND & HEC-1 SYMBOLOGY

- Drainage Area Boundary
- Major Flow Paths
- Offsite Inflow Location
- Sports Field Subsurface Drainage Area
- HEC-1 Subbasin Identifier
- HEC-1 Combine

SUBBASIN SUMMARY TABLE

Subbasin	Area (sq.mi.)	Length (miles)	Slope (ft/mi)	$Q_{100-yr, 6hr}$ (cfs)	$Q_{100-yr, 24hr}$ (cfs)
OFS1	0.010	0.15	117.6	15.4	12.7
OFS2	0.005	0.09	117.0	7.3	6.1
OFS3	0.002	0.05	132.7	2.8	2.3
OFS4	0.002	0.05	177.8	2.9	2.4
OFS5	0.001	0.06	104.8	0.9	0.8
OFS6	0.003	0.09	129.0	7.7	6.5
OFS7	0.001	0.08	177.2	1.2	1.0
OFS8	0.002	0.04	136.4	4.5	3.8
WSF1	0.001	0.07	111.1	2.2	2.0
WSF2	0.001	0.05	62.5	2.5	2.2
WSF3	0.001	0.07	98.5	2.3	2.0
WSF4	0.001	0.04	41.7	2.7	2.3
WSF5	0.001	0.06	82.0	2.4	2.1
WSF6	0.002	0.09	92.0	4.3	3.6
WSF7	0.002	0.06	100.0	5.3	4.5

COMBINE SUMMARY TABLE

Subbasin Combine	$Q_{100-yr, 6hr}$ (cfs)	$Q_{100-yr, 24hr}$ (cfs)
CSD1	3.2	2.7
CSD2	8.7	7.3
CSD3	11.3	9.5
CSD4	13.6	11.4
CSD5	20.3	17.0
CSD6	22.4	18.8

WestWorld Sports Fields

HEC-1 SCHEMATIC & DRAINAGE AREA MAP

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Issue Date:	09/21
Drawn By:	OK
Checked By:	MTG

Sheet Title :

HEC-1 SCHEMATIC & DRAINAGE AREA MAP

Sheet Number:

1

1 of 1

100-year, 6-hour HEC-1 Model

```
1*****************************************                                                                  ***************************************
 *                                       *                                                                  *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                                  *   U.S. ARMY CORPS OF ENGINEERS      *
 *              JUN   1998               *                                                                  *   HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.1                 *                                                                  *         609 SECOND STREET           *
 *                                       *                                                                  *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   17SEP21  TIME  12:32:14   *                                                                  *          (916) 756-1104             *
 *                                       *                                                                  *                                     *
 *****************************************                                                                  ***************************************


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


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

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

1                                                        HEC-1 INPUT                                                   PAGE  1

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

              1          ID        City of Scottsdale
              2          ID        WESTWORLD MUSF - WestWorld Multi-Use Sports Fields
              3          ID        100 YEAR
              4          ID        6 Hour  Storm
              5          ID        Unit Hydrograph: Clark
              6          ID        05/21/2021
                         *DIAGRAM
              7          IT       2  1JAN99       0     361
              8          IO       5
              9          IN      15
                         *

             10          KK    OFS5   BASIN
             11          BA   0.001
             12          PB   2.755  0.0001
```

```
      13           PC   0.000   0.008   0.016   0.025   0.033   0.041   0.050   0.058   0.066   0.074
      14           PC   0.087   0.099   0.118   0.138   0.216   0.377   0.834   0.911   0.931   0.950
      15           PC   0.962   0.972   0.983   0.991   1.000
      16           LG    0.35    0.35    2.75    1.09       0
      17           UC   0.186   0.309
      18           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      19           UA     100
      20           ZW   A=OFS5   B=BASIN  C=FLOW  F=CALC
                   *

      21           KK    WSF5   BASIN
      22           BA   0.001
      23           LG    0.07    0.34    2.75    0.93      81
      24           UC   0.104   0.162
      25           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
      26           UA     100
      27           ZW   A=WSF5   B=BASIN  C=FLOW  F=CALC
                   *

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

      31           KK    OFS4   BASIN
      32           BA   0.002
      33           LG    0.35    0.35    2.75    1.09       0
      34           UC   0.141   0.132
      35           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      36           UA     100
      37           ZW   A=OFS4   B=BASIN  C=FLOW  F=CALC
                   *

      38           KK    WSF4   BASIN
      39           BA   0.001
      40           LG    0.07    0.34    2.75    0.93      84
      41           UC   0.104   0.117
      42           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
      43           UA     100
      44           ZW   A=WSF4   B=BASIN  C=FLOW  F=CALC
                   *
1                                             HEC-1 INPUT                                          PAGE  2

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


      45           KK    CSD2 COMBINE
      46           HC       3
      47           ZW   A=CSD2  B=COMBINE  C=FLOW  F=CALC
                   *

      48           KK    OFS3   BASIN
      49           BA   0.002
```

```
   50           LG    0.35    0.35    2.75    1.09       0
   51           UC   0.154   0.146
   52           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
   53           UA     100
   54           ZW   A=OFS3   B=BASIN  C=FLOW  F=CALC
                *

   55           KK    CSD3 COMBINE
   56           HC       2
   57           ZW   A=CSD3  B=COMBINE  C=FLOW  F=CALC
                *

   58           KK    WSF3   BASIN
   59           BA   0.001
   60           LG    0.07    0.34    2.75    0.93      81
   61           UC   0.106   0.188
   62           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
   63           UA     100
   64           ZW   A=WSF3   B=BASIN  C=FLOW  F=CALC
                *

   65           KK    CSD4 COMBINE
   66           HC       2
   67           ZW   A=CSD4  B=COMBINE  C=FLOW  F=CALC
                *

   68           KK    OFS2   BASIN
   69           BA   0.005
   70           LG    0.32    0.35    2.75    1.06      11
   71           UC   0.189   0.173
   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=OFS2   B=BASIN  C=FLOW  F=CALC
                *

   75           KK    CSD5 COMBINE
   76           HC       2
   77           ZW   A=CSD5  B=COMBINE  C=FLOW  F=CALC
                *

   78           KK    WSF1   BASIN
   79           BA   0.001
   80           LG    0.08    0.34    2.75    0.93      76
   81           UC   0.121   0.217
   82           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
   83           UA     100
   84           ZW   A=WSF1   B=BASIN  C=FLOW  F=CALC
                *
```



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

```
  85           KK    CSD6 COMBINE
  86           HC       2
  87           ZW   A=CSD6  B=COMBINE  C=FLOW  F=CALC
               *

  88           KK    OFS1   BASIN
  89           BA   0.010
  90           LG    0.16    0.31    2.75    1.01       3
  91           UC   0.173   0.160
  92           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  93           UA     100
  94           ZW   A=OFS1   B=BASIN  C=FLOW  F=CALC
               *

  95           KK    WSF2   BASIN
  96           BA   0.001
  97           LG    0.07    0.34    2.75    0.93      84
  98           UC   0.120   0.165
  99           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 100           UA     100
 101           ZW   A=WSF2   B=BASIN  C=FLOW  F=CALC
               *

 102           KK    WSF6   BASIN
 103           BA   0.002
 104           LG    0.12    0.35    2.75    0.93      71
 105           UC   0.135   0.202
 106           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 107           UA     100
 108           ZW   A=WSF6   B=BASIN  C=FLOW  F=CALC
               *

 109           KK    WSF7   BASIN
 110           BA   0.002
 111           LG    0.10    0.35    2.75    0.93      76
 112           UC   0.104   0.109
 113           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 114           UA     100
 115           ZW   A=WSF7   B=BASIN  C=FLOW  F=CALC
               *

 116           KK    OFS6   BASIN
 117           BA   0.003
 118           LG    0.08    0.34    2.87    0.85      76
 119           UC   0.108   0.124
 120           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
 121           UA     100
 122           ZW   A=OFS6   B=BASIN  C=FLOW  F=CALC
               *
1                                          HEC-1 INPUT                                          PAGE  4

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

```
          123           KK    OFS7   BASIN
          124           BA   0.001
          125           LG    0.35    0.35    3.86    0.51       0
          126           UC   0.163   0.335
          127           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
          128           UA     100
          129           ZW   A=OFS7   B=BASIN  C=FLOW  F=CALC
                        *

          130           KK    OFS8   BASIN
          131           BA   0.002
          132           LG    0.22    0.35    2.75    0.92      46
          133           UC   0.114   0.119
          134           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
          135           UA     100
          136           ZW   A=OFS8   B=BASIN  C=FLOW  F=CALC
                        *
          137           ZZ
1
                    SCHEMATIC DIAGRAM OF STREAM NETWORK
 INPUT
  LINE      (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

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

    10        OFS5
                 .
                 .
    21           .        WSF5
                 .           .
                 .           .
    28        CSD1............
                 .
                 .
    31           .        OFS4
                 .           .
                 .           .
    38           .           .        WSF4
                 .           .           .
                 .           .           .
    45        CSD2........................
                 .
                 .
    48           .        OFS3
                 .           .
                 .           .
    55        CSD3............
                 .
                 .
    58           .        WSF3
                 .           .
                 .           .
```

```
  65        CSD4............
               .
               .
  68           .        OFS2
               .           .
               .           .
  75        CSD5............
               .
               .
  78           .        WSF1
               .           .
               .           .
  85        CSD6............
               .
               .
  88           .        OFS1
               .           .
               .           .
  95           .           .        WSF2
               .           .           .
               .           .           .
 102           .           .           .        WSF6
               .           .           .           .
               .           .           .           .
 109           .           .           .           .        WSF7
               .           .           .           .           .
               .           .           .           .           .
 116           .           .           .           .           .        OFS6
               .           .           .           .           .           .
               .           .           .           .           .           .
 123           .           .           .           .           .           .        OFS7
               .           .           .           .           .           .           .
               .           .           .           .           .           .           .
 130           .           .           .           .           .           .           .        OFS8

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                                                  ***************************************
 *                                       *                                                  *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                                                  *    U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998               *                                                  *    HYDROLOGIC ENGINEERING CENTER    *
 *           VERSION 4.1                 *                                                  *          609 SECOND STREET          *
 *                                       *                                                  *       DAVIS, CALIFORNIA 95616       *
 *  RUN DATE   17SEP21  TIME  12:32:14   *                                                  *           (916) 756-1104            *
 *                                       *                                                  *                                     *
 *****************************************                                                  ***************************************
```

City of Scottsdale
WESTWORLD MUSF - WestWorld Multi-Use Sports Fields
100 YEAR

```
                              6 Hour  Storm
                              Unit Hydrograph: Clark
                              05/21/2021

   8 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         361  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      1JAN99  ENDING DATE
                      NDTIME        1200  ENDING TIME
                      ICENT           19  CENTURY MARK

                   COMPUTATION INTERVAL     .03 HOURS
                        TOTAL TIME BASE   12.00 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: 100YR 6HR DESIGN MODEL.DSS
                      Unit:  71;  DSS Version: 6-JG
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS5/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS5/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /WSF5/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /WSF5/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD1/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD1/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS4/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS4/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /WSF4/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /WSF4/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD2/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD2/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS3/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS3/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD3/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD3/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /WSF3/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /WSF3/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD4/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /CSD4/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS2/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    3:  /OFS2/BASIN/FLOW/01JAN1999/2MIN/CALC/
```

```
 -----DSS---ZWRITE Unit  71; Vers.    3:  /CSD5/COMBINE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /CSD5/COMBINE/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF1/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF1/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /CSD6/COMBINE/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /CSD6/COMBINE/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /OFS1/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /OFS1/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF2/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF2/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF6/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF6/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF7/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /WSF7/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /OFS6/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /OFS6/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /OFS7/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    3:  /OFS7/BASIN/FLOW/01JAN1999/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    2:  /OFS8/BASIN/FLOW/31DEC1998/2MIN/CALC/
 -----DSS---ZWRITE Unit  71; Vers.    2:  /OFS8/BASIN/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
+                             OFS5        1.     4.10            0.          0.          0.          .00

          HYDROGRAPH AT
+                             WSF5        2.     4.03            0.          0.          0.          .00

          2 COMBINED AT
+                             CSD1        3.     4.03            0.          0.          0.          .00

          HYDROGRAPH AT
+                             OFS4        3.     4.07            0.          0.          0.          .00

          HYDROGRAPH AT
+                             WSF4        3.     4.00            0.          0.          0.          .00

          3 COMBINED AT
+                             CSD2        9.     4.03            1.          0.          0.          .00

          HYDROGRAPH AT
+                             OFS3        3.     4.07            0.          0.          0.          .00

          2 COMBINED AT
+                             CSD3       11.     4.03            1.          0.          0.          .01
```

```
         HYDROGRAPH AT
+                          WSF3        2.    4.03          0.          0.          0.        .00

         2 COMBINED AT
+                          CSD4       14.    4.03          1.          1.          1.        .01

         HYDROGRAPH AT
+                          OFS2        7.    4.10          1.          0.          0.        .00

         2 COMBINED AT
+                          CSD5       20.    4.07          2.          1.          1.        .01

         HYDROGRAPH AT
+                          WSF1        2.    4.03          0.          0.          0.        .00

         2 COMBINED AT
+                          CSD6       22.    4.07          2.          1.          1.        .01

         HYDROGRAPH AT
+                          OFS1       15.    4.10          1.          0.          0.        .01

         HYDROGRAPH AT
+                          WSF2        2.    4.03          0.          0.          0.        .00

         HYDROGRAPH AT
+                          WSF6        4.    4.03          0.          0.          0.        .00

         HYDROGRAPH AT
+                          WSF7        5.    4.00          0.          0.          0.        .00

         HYDROGRAPH AT
+                          OFS6        8.    4.03          1.          0.          0.        .00

         HYDROGRAPH AT
+                          OFS7        1.    4.10          0.          0.          0.        .00

         HYDROGRAPH AT
+                          OFS8        4.    4.03          0.          0.          0.        .00


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


   -----DSS---ZCLOSE Unit:  71,   File: 100YR 6HR DESIGN MODEL.DSS
              Pointer Utilization:   .26
              Number of Records:     42
              File Size:    157.7  Kbytes
              Percent Inactive:    .0
```

100-year, 24-hour HEC-1 Model

```
1****************************************                                                             ****************************************
 *                                      *                                                             *                                      *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)  *                                                             *    U.S. ARMY CORPS OF ENGINEERS      *
 *              JUN   1998              *                                                             *    HYDROLOGIC ENGINEERING CENTER     *
 *           VERSION 4.1                *                                                             *          609 SECOND STREET           *
 *                                      *                                                             *       DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   17SEP21  TIME  12:32:34  *                                                             *           (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



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            DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
            KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM


1                                                      HEC-1 INPUT                                                   PAGE  1

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

              1           ID        City of Scottsdale
              2           ID        WESTWORLD MUSF - WestWorld Multi-Use Sports Fields
              3           ID        100 YEAR
              4           ID        24 Hour Storm
              5           ID        Unit Hydrograph: Clark
              6           ID        05/21/2021
                          *DIAGRAM
              7           IT       2  1JAN99       0     721
              8           IO       5
              9           IN      15
                          *

             10           KK    OFS5   BASIN
             11           BA   0.001
             12           PB   3.842  0.0001
             13           PC   0.000   0.002   0.005   0.008   0.011   0.014   0.017   0.020   0.023   0.026
```

```
      14            PC   0.029   0.032   0.035   0.038   0.041   0.044   0.048   0.052   0.056   0.060
      15            PC   0.064   0.068   0.072   0.076   0.080   0.085   0.090   0.095   0.100   0.105
      16            PC   0.110   0.115   0.120   0.126   0.133   0.140   0.147   0.155   0.163   0.172
      17            PC   0.181   0.191   0.203   0.218   0.236   0.257   0.283   0.387   0.663   0.707
      18            PC   0.735   0.758   0.776   0.791   0.804   0.815   0.825   0.834   0.842   0.849
      19            PC   0.856   0.863   0.869   0.875   0.881   0.887   0.893   0.898   0.903   0.908
      20            PC   0.913   0.918   0.922   0.926   0.930   0.934   0.938   0.942   0.946   0.950
      21            PC   0.953   0.956   0.959   0.962   0.965   0.968   0.971   0.974   0.977   0.980
      22            PC   0.983   0.986   0.989   0.992   0.995   0.998   1.000
      23            LG    0.35    0.35    2.75    1.09       0
      24            UC   0.186   0.309
      25            UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      26            UA     100
      27            ZW   A=OFS5   B=BASIN  C=FLOW  F=CALC
                    *

      28            KK    WSF5   BASIN
      29            BA   0.001
      30            LG    0.07    0.34    2.75    0.93      81
      31            UC   0.104   0.162
      32            UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
      33            UA     100
      34            ZW   A=WSF5   B=BASIN  C=FLOW  F=CALC
                    *

      35            KK    CSD1 COMBINE
      36            HC       2
      37            ZW   A=CSD1  B=COMBINE  C=FLOW  F=CALC
                    *

      38            KK    OFS4   BASIN
      39            BA   0.002
      40            LG    0.35    0.35    2.75    1.09       0
      41            UC   0.141   0.132
      42            UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
      43            UA     100
      44            ZW   A=OFS4   B=BASIN  C=FLOW  F=CALC
                    *
1                                         HEC-1 INPUT                                              PAGE  2

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


      45            KK    WSF4   BASIN
      46            BA   0.001
      47            LG    0.07    0.34    2.75    0.93      84
      48            UC   0.104   0.117
      49            UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
      50            UA     100
      51            ZW   A=WSF4   B=BASIN  C=FLOW  F=CALC
                    *

      52            KK    CSD2 COMBINE
      53            HC       3
```

```
  54           ZW   A=CSD2  B=COMBINE  C=FLOW  F=CALC
               *

  55           KK    OFS3   BASIN
  56           BA   0.002
  57           LG    0.35    0.35    2.75    1.09       0
  58           UC   0.154   0.146
  59           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  60           UA     100
  61           ZW   A=OFS3   B=BASIN  C=FLOW  F=CALC
               *

  62           KK    CSD3 COMBINE
  63           HC       2
  64           ZW   A=CSD3  B=COMBINE  C=FLOW  F=CALC
               *

  65           KK    WSF3   BASIN
  66           BA   0.001
  67           LG    0.07    0.34    2.75    0.93      81
  68           UC   0.106   0.188
  69           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
  70           UA     100
  71           ZW   A=WSF3   B=BASIN  C=FLOW  F=CALC
               *

  72           KK    CSD4 COMBINE
  73           HC       2
  74           ZW   A=CSD4  B=COMBINE  C=FLOW  F=CALC
               *

  75           KK    OFS2   BASIN
  76           BA   0.005
  77           LG    0.32    0.35    2.75    1.06      11
  78           UC   0.189   0.173
  79           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
  80           UA     100
  81           ZW   A=OFS2   B=BASIN  C=FLOW  F=CALC
               *
```



```
  82           KK    CSD5 COMBINE
  83           HC       2
  84           ZW   A=CSD5  B=COMBINE  C=FLOW  F=CALC
               *

  85           KK    WSF1   BASIN
  86           BA   0.001
  87           LG    0.08    0.34    2.75    0.93      76
  88           UC   0.101   0.178
  89           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
```

```
 90           UA     100
 91           ZW   A=WSF1   B=BASIN  C=FLOW  F=CALC
              *

 92           KK    CSD6 COMBINE
 93           HC       2
 94           ZW   A=CSD6  B=COMBINE  C=FLOW  F=CALC
              *

 95           KK    OFS1   BASIN
 96           BA   0.010
 97           LG    0.16    0.31    2.75    1.01       3
 98           UC   0.173   0.160
 99           UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
100           UA     100
101           ZW   A=OFS1   B=BASIN  C=FLOW  F=CALC
              *

102           KK    WSF2   BASIN
103           BA   0.001
104           LG    0.07    0.34    2.75    0.93      84
105           UC   0.101   0.136
106           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
107           UA     100
108           ZW   A=WSF2   B=BASIN  C=FLOW  F=CALC
              *

109           KK    WSF6   BASIN
110           BA   0.002
111           LG    0.12    0.35    2.75    0.93      71
112           UC   0.135   0.202
113           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
114           UA     100
115           ZW   A=WSF6   B=BASIN  C=FLOW  F=CALC
              *

116           KK    WSF7   BASIN
117           BA   0.002
118           LG    0.10    0.35    2.75    0.93      76
119           UC   0.104   0.109
120           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
121           UA     100
122           ZW   A=WSF7   B=BASIN  C=FLOW  F=CALC
              *
```


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

123           KK    OFS6   BASIN
124           BA   0.003
125           LG    0.08    0.34    2.87    0.85      76
126           UC   0.108   0.124
127           UA       0     5.0    16.0    30.0    65.0    77.0    84.0    90.0    94.0    97.0
```

```
           128            UA     100
           129            ZW   A=OFS6   B=BASIN  C=FLOW  F=CALC
                          *

           130            KK    OFS7   BASIN
           131            BA   0.001
           132            LG    0.35    0.35    3.86    0.51       0
           133            UC   0.163   0.335
           134            UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
           135            UA     100
           136            ZW   A=OFS7   B=BASIN  C=FLOW  F=CALC
                          *

           137            KK    OFS8   BASIN
           138            BA   0.002
           139            LG    0.22    0.35    2.75    0.92      46
           140            UC   0.114   0.119
           141            UA       0     3.0     5.0     8.0    12.0    20.0    43.0    75.0    90.0    96.0
           142            UA     100
           143            ZW   A=OFS8   B=BASIN  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

    10        OFS5
                 .
                 .
    28           .        WSF5
                 .           .
                 .           .
    35        CSD1............
                 .
                 .
    38           .        OFS4
                 .           .
                 .           .
    45           .           .        WSF4
                 .           .           .
                 .           .           .
    52        CSD2........................
                 .
                 .
    55           .        OFS3
                 .           .
                 .           .
    62        CSD3............
                 .
                 .
    65           .        WSF3
```

```
                   .           .
                   .           .
   72         CSD4............
                   .
                   .
   75              .        OFS2
                   .           .
                   .           .
   82         CSD5............
                   .
                   .
   85              .        WSF1
                   .           .
                   .           .
   92         CSD6............
                   .
                   .
   95              .        OFS1
                   .           .
                   .           .
  102              .           .        WSF2
                   .           .           .
                   .           .           .
  109              .           .           .        WSF6
                   .           .           .           .
                   .           .           .           .
  116              .           .           .           .        WSF7
                   .           .           .           .           .
                   .           .           .           .           .
  123              .           .           .           .           .        OFS6
                   .           .           .           .           .           .
                   .           .           .           .           .           .
  130              .           .           .           .           .           .        OFS7
                   .           .           .           .           .           .           .
                   .           .           .           .           .           .           .
  137              .           .           .           .           .           .           .        OFS8

 (***) RUNOFF ALSO COMPUTED AT THIS LOCATION
1*****************************************                                                    ***************************************
 *                                        *                                                   *                                     *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)    *                                                   *    U.S. ARMY CORPS OF ENGINEERS     *
 *              JUN   1998                *                                                   *    HYDROLOGIC ENGINEERING CENTER    *
 *           VERSION 4.1                  *                                                   *         609 SECOND STREET           *
 *                                        *                                                   *      DAVIS, CALIFORNIA 95616        *
 *  RUN DATE   17SEP21  TIME  12:32:34    *                                                   *          (916) 756-1104             *
 *                                        *                                                   *                                     *
 *****************************************                                                    ***************************************
```

City of Scottsdale
WESTWORLD MUSF - WestWorld Multi-Use Sports Fields

```
                              100 YEAR
                              24 Hour Storm
                              Unit Hydrograph: Clark
                              05/21/2021

   8 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         721  NUMBER OF HYDROGRAPH ORDINATES
                      NDDATE      2JAN99  ENDING DATE
                      NDTIME        0000  ENDING TIME
                      ICENT           19  CENTURY MARK

                   COMPUTATION INTERVAL      .03 HOURS
                        TOTAL TIME BASE    24.00 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: 100YR 24HR DESIGN MODEL.DSS
                      Unit:  71;  DSS Version: 6-JG
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS5/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS5/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF5/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF5/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD1/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD1/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS4/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS4/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF4/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF4/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD2/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD2/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS3/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS3/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD3/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD3/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF3/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF3/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD4/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD4/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS2/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS2/BASIN/FLOW/01JAN1999/2MIN/CALC/
```

```
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD5/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD5/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF1/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF1/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD6/COMBINE/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /CSD6/COMBINE/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS1/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS1/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF2/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF2/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF6/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF6/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF7/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /WSF7/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS6/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS6/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS7/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    2:  /OFS7/BASIN/FLOW/01JAN1999/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    1:  /OFS8/BASIN/FLOW/31DEC1998/2MIN/CALC/
-----DSS---ZWRITE Unit  71; Vers.    1:  /OFS8/BASIN/FLOW/01JAN1999/2MIN/CALC/
1
```

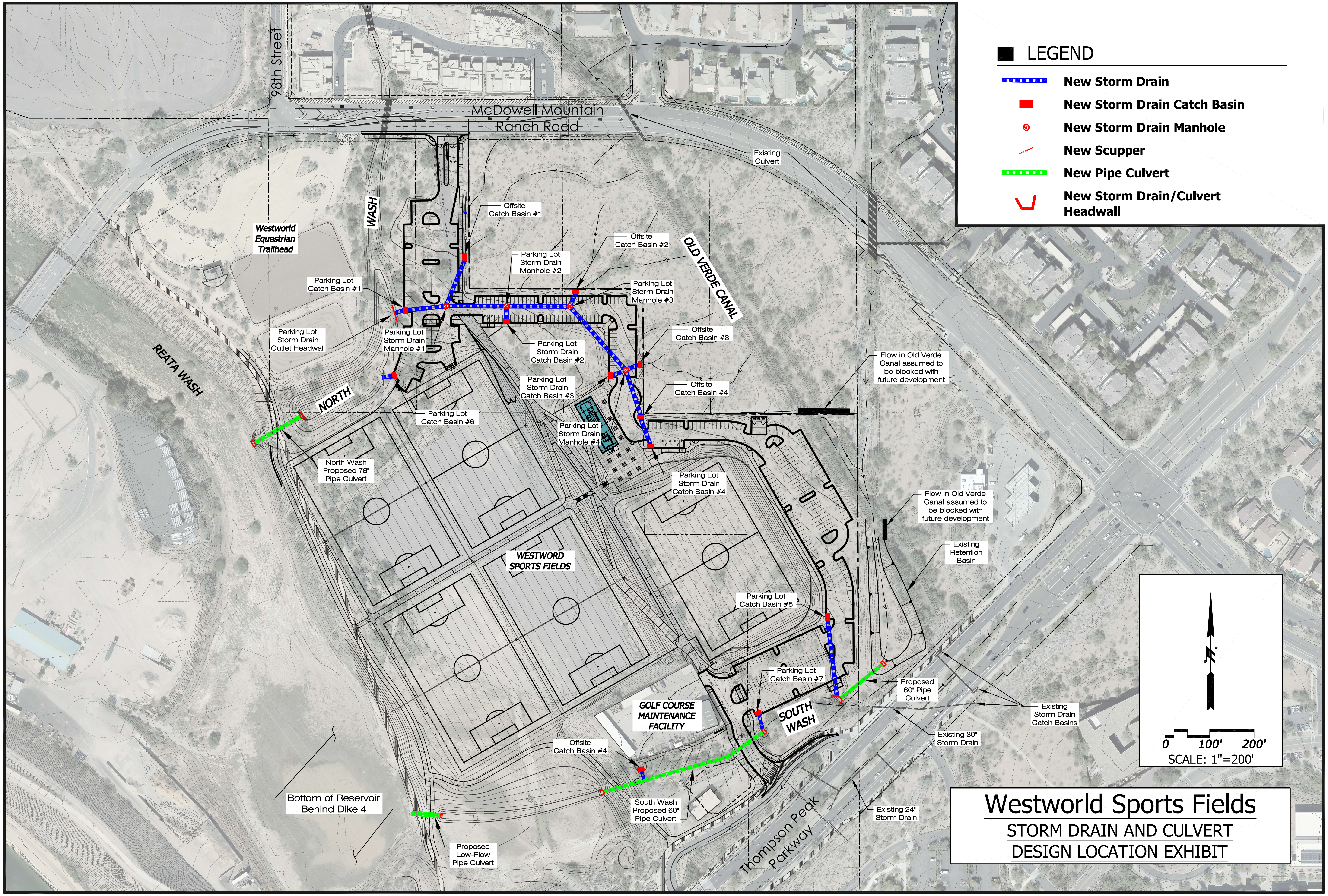
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

	OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
+	HYDROGRAPH AT	OFS5	1.	12.10	0.	0.	0.	.00		
+	HYDROGRAPH AT	WSF5	2.	12.03	0.	0.	0.	.00		
+	2 COMBINED AT	CSD1	3.	12.03	0.	0.	0.	.00		
+	HYDROGRAPH AT	OFS4	2.	12.07	0.	0.	0.	.00		
+	HYDROGRAPH AT	WSF4	2.	12.00	0.	0.	0.	.00		
+	3 COMBINED AT	CSD2	7.	12.03	1.	0.	0.	.00		
+	HYDROGRAPH AT	OFS3	2.	12.07	0.	0.	0.	.00		
+	2 COMBINED AT	CSD3	9.	12.03	1.	0.	0.	.01		
	HYDROGRAPH AT									

```
+                             WSF3         2.   12.03           0.          0.          0.         .00

          2 COMBINED AT
+                             CSD4        11.   12.03           1.          0.          0.         .01

          HYDROGRAPH AT
+                             OFS2         6.   12.10           0.          0.          0.         .00

          2 COMBINED AT
+                             CSD5        17.   12.07           2.          0.          0.         .01

          HYDROGRAPH AT
+                             WSF1         2.   12.03           0.          0.          0.         .00

          2 COMBINED AT
+                             CSD6        19.   12.07           2.          1.          1.         .01

          HYDROGRAPH AT
+                             OFS1        13.   12.10           1.          0.          0.         .01

          HYDROGRAPH AT
+                             WSF2         2.   12.00           0.          0.          0.         .00

          HYDROGRAPH AT
+                             WSF6         4.   12.03           0.          0.          0.         .00

          HYDROGRAPH AT
+                             WSF7         4.   12.00           0.          0.          0.         .00

          HYDROGRAPH AT
+                             OFS6         7.   12.00           1.          0.          0.         .00

          HYDROGRAPH AT
+                             OFS7         1.   12.10           0.          0.          0.         .00

          HYDROGRAPH AT
+                             OFS8         4.   12.03           0.          0.          0.         .00


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


    -----DSS---ZCLOSE Unit:  71,   File: 100YR 24HR DESIGN MODEL.DSS
               Pointer Utilization:   .26
               Number of Records:     42
               File Size:    157.7  Kbytes
               Percent Inactive:    .0
```

Appendix D: Storm Drain and Culvert Design Hydraulic Analysis

Storm Drain and Culvert Layout Exhibit



Westworld Sports Fields

STORM DRAIN AND CULVERT DESIGN LOCATION EXHIBIT

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 :

WESTWORLD SPORTS FIELDS
CITY OF SCOTTSDALE
PROJECT NUMBER: PG09

Submittal :

G&B No.	2101
Issue Date:	09/21
Drawn By:	OK
Checked By:	MTG

Sheet Title :
STORM DRAIN & CULVERT DESIGN LOCATION EXHIBIT

Sheet Number:
1
1 of 1

Parking Lot Inlet Design Calculations

Gavan & Barker Inc.

Parking Lot – Catch Basin Design Calculations

The majority of the new Westworld Sports Fields parking lot is graded to drain to either the realigned North Wash that separates the sports fields from the equestrian trailhead or the South Wash that will be partially piped with a 60-inch pipe culvert from the southern entrance drive to Reata Wash. There are also four locations where offsite flows impact the parking lot. The new parking lot is graded to drain to four shallow sumps that will be drained by four catch basins connected to a proposed parking lot storm drain. This storm drain, which is located north of the restroom/office building will also include four catch basins to intercept the offsite flows and convey them to the North Wash. There are single catch basins and connector pipes located west and southeast of the restroom/office building and will drain the remaining portion of the parking lot to either the North or South Wash. Refer to Storm Drain and Culvert Design Location Exhibit at the beginning of these calculations for the locations of the proposed storm drain and parking lot catch basins .

From the hydrologic analysis it was found that the 100-year, 6-hour storm event produces higher peak discharges for the design watershed located downstream of the Old Verde Canal. Therefore, the seven proposed catch basins were designed to intercept the entire 100-year, 6-hour peak discharges without any bypass. 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) – $Q_{100yr,\ 6hr}$=2.2 cfs (WSF1)
- Catch Basin #2 (CB#2) – $Q_{100yr,\ 6hr}$=2.3 cfs (WSF3)
- Catch Basin #3 (CB#3) – $Q_{100yr,\ 6hr}$=2.7 cfs (WSF4)
- Catch Basin #4 (CB#4) – $Q_{100yr,\ 6hr}$=2.4 cfs (WSF5)
- Catch Basin #5 (CB#5) – $Q_{100yr,\ 6hr}$=4.3 cfs (WSF6)
- Catch Basin #6 (CB#6) – $Q_{100yr,\ 6hr}$=2.5 cfs (WSF2)
- Catch Basin #7 (CB#7) – $Q_{100yr,\ 6hr}$=5.3 cfs (WSF7)

Refer to Appendix C for the Design HEC-1 Hydrologic Model showing the contributing drainage areas to each catch basin as well as the governing 100-year, 6-hour HEC-1 model.

It is recommended to install a MAG Type “G” double grate catch basin (Std. Det. No.: 537) with a City of Scottsdale grate per Std. Det. No.: 2535 at each one of the seven parking lot 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.

Catch Basin Design @ CB#1:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w \, P d^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ "G" Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std. Det.\ 2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1521.25 - 1520.90$

$[d = 0.35\ ft]$

$Q_i = C_w \, P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.35^{1.5}$

$Q_i = 2.98$

$[Q_i = 3.0\ cfs]$

The interception capacity of the proposed MAG Type "G" Double Grated Catch Basin is 3.0 cfs, which is greater than the 100-year, 6-hour peak discharge of 2.2 cfs.

Gavan & Barker Inc.

Catch Basin Design @ CB#2:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w\,Pd^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ ''G'' Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std.Det.\ 2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1523.42 - 1522.70$

$[d = 0.80\ ft]$

$Q_i = C_w\,P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.72^{1.5}$

$Q_i = 8.80$

$[Q_i = 8.8\ cfs]$

The interception capacity of the proposed MAG Type "G" Double Grated Catch Basin is 8.8 cfs, which is greater than the 100-year, 6-hour peak discharge of 2.3 cfs.

Gavan & Barker Inc.

Catch Basin Connector Pipe Design for Catch Basin #2:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f L$

($Equation$ 4.6 – $Drainage$ $Design$ $Manual$ for $Maricopa$ $County$, $Hydraulics$)

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

($Equation$ 4.15 – $Drainage$ $Design$ $Manual$ for $Maricopa$ $County$, $Hydraulics$)

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$(Equation 4.5)

$n = Manning's\ Roughness$

$n = 0.013$ ($Table$ 4.1 – $Smooth$ $Plastic$ $Pipe$)

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 2.3\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area$ (15" $Pipe$)

$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$

$V = Velocity\ of\ Flow$

$V = \frac{Q}{A} = \frac{2.3}{1.23} = 1.87\frac{ft}{s}$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 33\ ft$

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

$k_{en} = 0.20$

($Table$ 5.1 – $Drainage$ $Design$ $Manual$ for $Maricopa$ $County$, $Hydraulics$)

Friction Headloss:

$h_f = S_f\,L$

$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\,L$

$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\,L$

$h_f = \frac{2 * 32.2 * 0.013^2}{2.21}\left(\frac{1.87^2}{2 * 32.2 * 0.313^{\frac{4}{3}}}\right) * 33$

$[h_f = 0.04\,ft]$

Entrance Headloss:

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

$h_i = (1 + 0.2)\left(\frac{1.87^2}{2 * 32.2}\right)$

$[h_i = 0.08\,ft]$

Total Headloss:

$h_{Total} = h_f + h_{en}$
$h_{Total} = 0.04 + 0.08$
$h_{Total} = 0.12$

$[h_{Total} = 0.1\,ft]$

Available Head: h_a

Upstream HW Elevation: 1522.20 ft (6-inches below Catch Basin Grate Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6” blow the grate elevation at the catch basin. The grate elevation at Catch Basin is 1522.70 ft.

Downstream HW Elevation: 1521.67 ft (Storm Drain Hydraulic Grade Line (HGL) at Pipe Junction #1)

h_a = Upstream HW – Downstream HW = 1522.20 – 1521.67 = 0.53

$\boxed{h_a = 0.5\,ft}$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Gavan & Barker Inc.

Catch Basin Design @ CB#3:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w \, P d^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ ''G'' Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std. Det.\ 2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1527.75 - 1527.15$

$[d = 0.60\ ft]$

$Q_i = C_w \, P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.60^{1.5}$

$Q_i = 6.69$

$$[Q_i = 6.7\ cfs]$$

The interception capacity of the proposed MAG Type “G” Double Grated Catch Basin is 6.7 cfs, which is greater than the 100-year, 6-hour peak discharge of 2.7 cfs.

Gavan & Barker Inc.

Catch Basin Connector Pipe Design for Catch Basin #3:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 2.7\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{2.7}{1.23} = 2.20\frac{ft}{s}$$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 36\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\ L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2*32.2*0.013^2}{2.21}\left(\frac{2.20^2}{2*32.2*0.313^{\frac{4}{3}}}\right)*36$$

$$\left[h_f = 0.06\ ft\right]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{2.20^2}{2*32.2}\right)$$

$$[h_i = 0.09\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.06 + 0.09$$
$$h_{Total} = 0.15$$

$$[h_{Total} = 0.2\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1526.65 ft (6-inches below Catch Basin Grate Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" blow the grate elevation at the catch basin. The grate elevation at Catch Basin is 1527.15 ft.

Downstream HW Elevation: 1526.00 ft (Storm Drain Hydraulic Grade Line (HGL) at Manhole #4)

h_a = Upstream HW – Downstream HW = 1526.65 – 1526.00 = 0.65

$$\boxed{h_a = 0.7\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Gavan & Barker Inc.

Catch Basin Design @ CB#4:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w \, P d^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ ''G'' Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std. Det.\ 2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1528.34 - 1527.90$

$[d = 0.44\ ft]$

$Q_i = C_w \, P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.44^{1.5}$

$Q_i = 4.20$

$[Q_i = 4.2\ cfs]$

The interception capacity of the proposed MAG Type "G" Double Grated Catch Basin is 4.2 cfs, which is greater than the 100-year, 6-hour peak discharge of 2.4 cfs.

Catch Basin Connector Pipe Design for Catch Basin #4:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f\,L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 2.4\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$

$V = Velocity\ of\ Flow$

$V = \frac{Q}{A} = \frac{2.4}{1.23} = 1.95\frac{ft}{s}$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 69\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2 * 32.2 * 0.013^2}{2.21}\left(\frac{1.95^2}{2 * 32.2 * 0.313^{\frac{4}{3}}}\right) * 69$$

$$[h_f = 0.09\ ft]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{1.95^2}{2 * 32.2}\right)$$

$$[h_i = 0.07\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.09 + 0.07$$
$$h_{Total} = 0.16$$

$$[h_{Total} = 0.2\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1527.40 ft (6-inches below Catch Basin Grate Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" blow the grate elevation at the catch basin. The grate elevation at Catch Basin is 1527.90 ft.

Downstream HW Elevation: 1526.40 ft (Storm Drain Hydraulic Grade Line (HGL) at Offsite CB#4)

h_a = Upstream HW – Downstream HW = 1527.40 – 1526.40 = 1.00

$$\boxed{h_a = 1.0\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Catch Basin Design @ CB#5:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w \, Pd^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ ''G'' Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std. Det.\ 2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1530.20 - 1529.74$

$[d = 0.46\ ft]$

$Q_i = C_w \, P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.46^{1.5}$

$Q_i = 4.49$

$[Q_i = 4.5\ cfs]$

The interception capacity of the proposed MAG Type "G" Double Grated Catch Basin is 4.5 cfs, which is greater than the 100-year, 6-hour peak discharge of 4.3 cfs.

Catch Basin Connector Pipe Design for Catch Basin #5:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 18-inch (d=1.5 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 4.3\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (18"\ Pipe)$

$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.50^2}{4}\right) = 1.77\ ft^2$

$V = Velocity\ of\ Flow$

$V = \frac{Q}{A} = \frac{4.3}{1.77} = 2.43\frac{ft}{s}$

$R = Hydraulic\ Radius$

$R = \frac{D}{4} = \frac{1.50}{4} = 0.375\ ft$

$L = Connector\ Pipe\ Length$

$L = 178\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2*32.2*0.013^2}{2.21}\left(\frac{2.43^2}{2*32.2*0.375^{\frac{4}{3}}}\right)*178$$

$$[h_f = 0.30\, ft]$$

Entrance Headloss:

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

$$h_i = (1+0.2)\left(\frac{2.43^2}{2*32.2}\right)$$

$$[h_i = 0.11\, ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.30 + 0.11$$
$$h_{Total} = 0.41$$

$$[h_{Total} = 0.4\, ft]$$

Available Head: h_a

Upstream HW Elevation: 1529.24 ft (6-inches below Catch Basin Grate Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6” blow the grate elevation at the catch basin. The grate elevation at Catch Basin is 1529.74 ft.

Downstream HW Elevation: 1528.75 ft (Water Surface Elevation in South Wash at Outlet Headwall)

h_a = Upstream HW – Downstream HW = 1529.24 – 1528.75 = 0.49

$$\boxed{h_a = 0.5\, ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 18-inch connector pipe has a sufficient capacity to convey the intercepted flow

Catch Basin Design @ CB#6:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w\, P d^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ ''G'' Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std.Det.\,2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1515.75 - 1515.40$

$[d = 0.35\ ft]$

$Q_i = C_w\, P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.35^{1.5}$

$Q_i = 2.98$

$[Q_i = 3.0\ cfs]$

The interception capacity of the proposed MAG Type "G" Double Grated Catch Basin is 3.0 cfs, which is greater than the 100-year, 6-hour peak discharge of 2.5 cfs.

Gavan & Barker Inc.

Catch Basin Connector Pipe Design for Catch Basin #6:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 2.5\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{2.5}{1.23} = 2.03\frac{ft}{s}$$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 27\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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Friction Headloss:

$$h_f = S_f \, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2*32.2*0.013^2}{2.21}\left(\frac{2.03^2}{2*32.2*0.313^{\frac{4}{3}}}\right)*27$$

$$[h_f = 0.04\ ft]$$

Entrance Headloss:

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

$$h_i = (1+0.2)\left(\frac{2.03^2}{2*32.2}\right)$$

$$[h_i = 0.08\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.04 + 0.08$$
$$h_{Total} = 0.12$$

$$[h_{Total} = 0.1\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1514.90 ft (6-inches below Catch Basin Grate Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" blow the grate elevation at the catch basin. The grate elevation at Catch Basin is 1515.40 ft.

Downstream HW Elevation: 1513.6 ft (Water Surface Elevation in North Wash at Outlet Headwall, at the time corresponding to the peak storm drain inflow.)

h_a = Upstream HW – Downstream HW = 1514.90 – 1513.60 = 1.30

$$\boxed{h_a = 1.3\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Catch Basin Design @ CB#7:

Weir Flow, Sump Condition, Grated Catch Basin:

$Q_i = C_w\,Pd^{1.5}$

$(Equation\ 3.21 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Grated\ Catch\ Basin\ Interception\ Capacity$

$C_w = Weir\ Coefficent$

$C_w = 3.0$

$P = Perimeter\ of\ Grate\ (disregarding\ grate\ bars)$

$P_{Total} = Total\ Perimeter\ of\ MAG\ Type$ ''G'' Double Catch Basin = 12.0 ft

$W_{Bars} = Total\ Width\ of\ COS\ Std. Det.\ 2535\ Bars$ = 2.5 ft

$P = P_{Total} - W_{Bars} = 12.0 - 2.5 = 9.5\ ft$

$[P = 9.5\ ft]$

$C_f = Grate\ Perimeter\ Clogging\ Factor$

$C_f = 0.50$

$(Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$

$P_f = P * C_f = 9.5 * 0.50 = 4.75\ ft$

$[P_f = 4.8\ ft]$

$d = Depth\ of\ Flow\ at\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Grate\ Elevation$

$d = 1527.13 - 1526.53$

$[d = 0.60\ ft]$

$Q_i = C_w\,P_f d^{1.5}$

$Q_i = 3.0 * 4.8 * 0.60^{1.5}$

$Q_i = 6.69$

$$[Q_i = 6.7\ cfs]$$

The interception capacity of the proposed MAG Type “G” Double Grated Catch Basin is 6.7 cfs, which is greater than the 100-year, 6-hour peak discharge of 5.3 cfs.

Catch Basin Connector Pipe Design for Catch Basin #7:

Determine the Total (Friction + Inlet) Headloss:

$$h_f = S_f\,L$$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 5.3\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{5.3}{1.23} = 4.31\frac{ft}{s}$$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 35\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\, L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\, L$$

$$h_f = \frac{2*32.2*0.013^2}{2.21}\left(\frac{4.31^2}{2*32.2*0.313^{\frac{4}{3}}}\right)*35$$

$$[h_f = 0.23\, ft]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{4.31^2}{2*32.2}\right)$$

$$[h_i = 0.29\, ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.23 + 0.29$$
$$h_{Total} = 0.52$$

$$[h_{Total} = 0.5\, ft]$$

Available Head: h_a

Upstream HW Elevation: 1526.03 ft (6-inches below Catch Basin Grate Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6” blow the grate elevation at the catch basin. The grate elevation at Catch Basin is 1526.53 ft.

Downstream HW Elevation: 1519.9 ft (Water Surface Elevation at South Wash Drop Inlet Structure)

h_a = Upstream HW – Downstream HW = 1526.03 – 1519.90 = 6.13

$$\boxed{h_a = 6.1\, ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Offsite Inlet Design Calculations

Offsite – Catch Basin Design Calculations

There are four locations where offsite flows impact the new Westworld Sports Fields. The Weiss and Thomas parcels that are located north and east of the new improvements drain in a southwesterly direction contributing runoff downstream of the Old Verde Canal and McDowell Mountain Ranch Road to the new parking lot. The contributing area is small with the much larger upstream drainage area being diverted by the Old Verde Canal. Refer to Appendix C for an exhibit showing the offsite contributing drainage areas and governing 100-year, 6-hour HEC-1 Model.

To prevent offsite flows from entering the new parking lot, four offsite raised grate catch basins are proposed at the four main inflow locations along with shallow crown ditches within the landscaped median upstream of the parking lot to divert shallow offsite flows to the main inflow locations. The raised grate catch basins are connected to the parking lot storm drain and were designed to intercept the 100-year, 6-hour peak discharge from the existing undeveloped offsite watershed. Since the upstream contributing watershed consist of natural desert with sandy bottom washes, the weir elevation of the raised grate catch basins will be located approximately 6-inches above the wash inverts to prevent sediment from entering the parking lot storm drain. A fifth raised grate catch basin was designed to intercept the runoff from the maintenance yard as well as the filled in portion of the South Wash west of the southern entranced driveway. This first catch basin is connected to the proposed 60-inch pipe culvert. The 100-year, 6-hour design peak discharges and the corresponding HEC-1 Sub-Basin IDs for each offsite catch basin are as follows:

- Offsite Catch Basin #1 (OCB#1) – $Q_{100yr,\ 6hr}$=7.3 cfs (OFS2)
- Offsite Catch Basin #2 (OCB#2) – $Q_{100yr,\ 6hr}$=2.8 cfs (OFS3)
- Offsite Catch Basin #3 (OCB#3) – $Q_{100yr,\ 6hr}$=3.0 cfs (OFS4)
- Offsite Catch Basin #4 (OCB#4) – $Q_{100yr,\ 6hr}$=1.0 cfs (OFS5)
- Offsite Catch Basin #5 (OCB#5) – $Q_{100yr,\ 6hr}$=4.5 cfs (OFS8)

Refer to Storm Drain and Culvert Design Location Exhibit at the beginning of these calculations for the locations of the proposed offsite raised grate catch basins.

It is recommended to install a MAG Type "G" single grate catch basin (Std. Det. No.: 537) with a modified raised grate based on the City of Scottsdale Std. Det. No.: 2535 at each one of the five offsite locations. As can be seen in the following design calculations, the proposed catch basins have the capacity to intercept the entire 100-year, 6-hour offsite design peak discharge.

Offsite Catch Basin Design @ OCB#1:

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

$$d \leq h \qquad \rightarrow Weir\ Flow$$
$$h > d > 1.4h \qquad \rightarrow Transitional\ Flow$$
$$d \geq 1.4h \qquad \rightarrow Orifice\ Flow$$

where,

$d = Depth\ of\ Flow\ at\ Raised\ Grate\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Catch\ Basin\ Weir\ Lip\ Elevation$

$d = 1525.00 - 1524.40$

$[d = 0.60\ ft]$

$h = 4''\ Height\ of\ Riased\ Grate$

$[h = 0.33\ ft]$

$d \geq 1.4h$

$0.60\ ft \geq 1.4 * 0.33$

$0.60\ ft \geq 0.46\ ft$

$\boxed{\textbf{Orifice Flow}}$

<u>Orifice Flow, Sump Condition, Raised Grate Catch Basin:</u>

$Q_i = C_0\, hL\sqrt{2gd_0}$

$(Equation\ 3.14 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Raised\ Grate\ Catch\ Basin\ Flow\ Interception\ Capacity$
$C_0 = Orifice\ Coefficent$
$C_0 = 0.67$
$L = Perimeter\ of\ Raised\ Grate$
$L = 8.0\ ft\ (MAG\ Type\ 'G' Single\ Grate\ Catch\ Basin)$
$C_f = Clogging\ Factor$
$C_f = 0.80\ (Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$
$L_f = Effective\ Perimeter\ of\ Riased\ Grate = C_f * L$
$L_f = C_f * L$
$L_f = 0.80 * 8.0$
$[L_f = 6.4\ ft]$
$g = Gravity = 32.2\frac{ft}{s^2}$
$d_0 = Effective\ Depth\ at\ Center\ of\ Riased\ Grate\ Opening$
$d_0 = d - \frac{h}{2}$
$d_0 = 0.60 - \frac{0.33}{2}$
$[d_0 = 0.43\ ft]$

$Q_i = C_0\, hL\sqrt{2gd_0}$
$Q_i = 0.67 * 0.33 * 6.4\sqrt{2 * 32.2 * 0.43}$
$Q_i = 7.45$

$[Q_i = 7.5\ cfs]$

The interception capacity of the proposed MAG Type “G” Single Raised Grate Catch Basin is 7.5 cfs, which is greater than the 100-year, 6-hour peak discharge of 7.3 cfs.

Gavan & Barker Inc.

Catch Basin Connector Pipe Design for Offsite Catch Basin #1:

Determine the Total (Friction + Inlet) Headloss:

$$h_f = S_f\, L$$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 7.3\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{7.3}{1.23} = 5.93\frac{ft}{s}$$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 116\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2 * 32.2 * 0.013^2}{2.21}\left(\frac{5.93^2}{2 * 32.2 * 0.313^{\frac{4}{3}}}\right) * 116$$

$$\left[h_f = 1.46\ ft\right]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{5.93^2}{2 * 32.2}\right)$$

$$[h_i = 0.66\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 1.46 + 0.66$$
$$h_{Total} = 2.12$$

$$[h_{Total} = 2.1\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1523.90 ft (6-inches below Raised Grate Catch Basin Weir Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6” blow the weir elevation at the catch basin. The weir elevation at the Raised Grate Catch Basin is 1524.40 ft.

Downstream HW Elevation: 1521.04 ft (Storm Drain Hydraulic Grade Line (HGL) at Manhole #1)

h_a = Upstream HW – Downstream HW = 1523.90 – 1521.04 = 2.86

$$\boxed{h_a = 2.9\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Offsite Catch Basin Design @ OCB#2:

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

$d \leq h \quad \rightarrow Weir\ Flow$
$h > d > 1.4h \quad \rightarrow Transitional\ Flow$
$d \geq 1.4h \quad \rightarrow Orifice\ Flow$

where,

$d = Depth\ of\ Flow\ at\ Raised\ Grate\ Catch\ Basin\ (Depth\ of\ Sump)$
$d = Sump\ Spill\ Elevation - Catch\ Basin\ Weir\ Lip\ Elevation$
$d = 1527.80 - 1527.00$
$[d = 0.80\ ft]$
$h = 4''\ Height\ of\ Riased\ Grate$
$[h = 0.33\ ft]$

$d \geq 1.4h$
$0.80\ ft \geq 1.4 * 0.33$
$0.80\ ft \geq 0.46\ ft$

$\boxed{\textbf{Orifice Flow}}$

Orifice Flow, Sump Condition, Raised Grate Catch Basin:

$$Q_i = C_0\, hL\sqrt{2gd_0}$$

$(Equation\ 3.14 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Raised\ Grate\ Catch\ Basin\ Flow\ Interception\ Capacity$

$C_0 = Orifice\ Coefficent$

$C_0 = 0.67$

$L = Perimeter\ of\ Raised\ Grate$

$L = 8.0\ ft\ (MAG\ Type\ 'G' Single\ Grate\ Catch\ Basin)$

$C_f = Clogging\ Factor$

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

$L_f = Effective\ Perimeter\ of\ Riased\ Grate = C_f * L$

$L_f = C_f * L$

$L_f = 0.80 * 8.0$

$[L_f = 6.4\ ft]$

$g = Gravity = 32.2 \dfrac{ft}{s^2}$

$d_0 = Effective\ Depth\ at\ Center\ of\ Riased\ Grate\ Opening$

$d_0 = d - \dfrac{h}{2}$

$d_0 = 0.80 - \dfrac{0.33}{2}$

$[d_0 = 0.63\ ft]$

$$Q_i = C_0\, hL\sqrt{2gd_0}$$
$$Q_i = 0.67 * 0.33 * 6.4\sqrt{2 * 32.2 * 0.63}$$
$$Q_i = 9.01$$

$$[Q_i = 9.0\ cfs]$$

The interception capacity of the proposed MAG Type “G” Single Raised Grate Catch Basin is 9.0 cfs, which is greater than the 100-year, 6-hour peak discharge of 2.8 cfs.

Gavan & Barker Inc.

Catch Basin Connector Pipe Design for Offsite Catch Basin #2:

Determine the Total (Friction + Inlet) Headloss:

$$h_f = S_f\,L$$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ (Equation 4.5)

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 2.8\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{2.8}{1.23} = 2.28\frac{ft}{s}$$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 34\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\ L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2 * 32.2 * 0.013^2}{2.21}\left(\frac{2.28^2}{2 * 32.2 * 0.313^{\frac{4}{3}}}\right) * 34$$

$$[h_f = 0.06\ ft]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{2.28^2}{2 * 32.2}\right)$$

$$[h_i = 0.10\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.06 + 0.10$$
$$h_{Total} = 0.16$$

$$[h_{Total} = 0.2\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1526.50 ft (6-inches below Raised Grate Catch Basin Weir Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" blow the weir elevation at the catch basin. The weir elevation at the Raised Grate Catch Basin is 1527.00 ft.

Downstream HW Elevation: 1522.27 ft (Storm Drain Hydraulic Grade Line (HGL) at Manhole #3)

h_a = Upstream HW – Downstream HW = 1526.50 – 1523.70 = 2.80

$$\boxed{h_a = 2.8\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Offsite Catch Basin Design @ OCB#3:

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

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

where,

$d = Depth\ of\ Flow\ at\ Raised\ Grate\ Catch\ Basin\ (Depth\ of\ Sump)$
$d = Sump\ Spill\ Elevation - Catch\ Basin\ Weir\ Lip\ Elevation$
$d = 1529.30 - 1528.80$
$[d = 0.50\ ft]$
$h = 4''\ Height\ of\ Riased\ Grate$
$[h = 0.33\ ft]$

$d \ge 1.4h$
$0.50\ ft \ge 1.4 * 0.33$
$0.50\ ft \ge 0.46\ ft$

Orifice Flow

Orifice Flow, Sump Condition, Raised Grate Catch Basin:

$$Q_i = C_0\, hL\sqrt{2gd_0}$$

$(Equation\ 3.14 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Raised\ Grate\ Catch\ Basin\ Flow\ Interception\ Capacity$
$C_0 = Orifice\ Coefficent$
$C_0 = 0.67$
$L = Perimeter\ of\ Raised\ Grate$
$L = 8.0\ ft\ (MAG\ Type\ 'G' Single\ Grate\ Catch\ Basin)$
$C_f = Clogging\ Factor$
$C_f = 0.80\ (Table\ 6.8 - FCDMC\ Drainage\ Policies\ and\ Standards\ Manual)$
$L_f = Effective\ Perimeter\ of\ Riased\ Grate = C_f * L$
$L_f = C_f * L$
$L_f = 0.80 * 8.0$
$[L_f = 6.4\ ft]$
$g = Gravity = 32.2\frac{ft}{s^2}$
$d_0 = Effective\ Depth\ at\ Center\ of\ Riased\ Grate\ Opening$
$d_0 = d - \frac{h}{2}$
$d_0 = 0.50 - \frac{0.33}{2}$
$[d_0 = 0.33\ ft]$

$Q_i = C_0\, hL\sqrt{2gd_0}$
$Q_i = 0.67 * 0.33 * 6.4\sqrt{2 * 32.2 * 0.33}$
$Q_i = 6.52$

$$[Q_i = 6.5\ cfs]$$

The interception capacity of the proposed MAG Type "G" Single Raised Grate Catch Basin is 9.0 cfs, which is greater than the 100-year, 6-hour peak discharge of 3.0 cfs.

Catch Basin Connector Pipe Design for Offsite Catch Basin #3:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 2.8\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$

$V = Velocity\ of\ Flow$

$V = \frac{Q}{A} = \frac{3.0}{1.23} = 2.44\frac{ft}{s}$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 35\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\, L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\, L$$

$$h_f = \frac{2 * 32.2 * 0.013^2}{2.21}\left(\frac{2.44^2}{2 * 32.2 * 0.313^{\frac{4}{3}}}\right) * 35$$

$$[h_f = 0.07\ ft]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{2.44^2}{2 * 32.2}\right)$$

$$[h_i = 0.11\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$

$$h_{Total} = 0.07 + 0.11$$

$$h_{Total} = 0.18$$

$$[h_{Total} = 0.2\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1528.30 ft (6-inches below Raised Grate Catch Basin Weir Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6" blow the weir elevation at the catch basin. The weir elevation at the Raised Grate Catch Basin is 1528.80 ft.

Downstream HW Elevation: 1526.00 ft (Storm Drain Hydraulic Grade Line (HGL) at Manhole #4)

h_a = Upstream HW – Downstream HW = 1528.30 – 1526.00 = 2.30

$$\boxed{h_a = 2.3\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Offsite Catch Basin Design @ OCB#4:

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

$d \leq h \qquad \rightarrow Weir\ Flow$
$h > d > 1.4h \qquad \rightarrow Transitional\ Flow$
$d \geq 1.4h \qquad \rightarrow Orifice\ Flow$

where,

$d = Depth\ of\ Flow\ at\ Raised\ Grate\ Catch\ Basin\ (Depth\ of\ Sump)$
$d = Sump\ Spill\ Elevation - Catch\ Basin\ Weir\ Lip\ Elevation$
$d = 1529.70 - 1529.20$
$[d = 0.50\ ft]$
$h = 4''\ Height\ of\ Riased\ Grate$
$[h = 0.33\ ft]$

$d \geq 1.4h$
$0.50\ ft \geq 1.4 * 0.33$
$0.50\ ft \geq 0.46\ ft$

$\boxed{\textbf{Orifice Flow}}$

<u>Orifice Flow, Sump Condition, Raised Grate Catch Basin:</u>

$$Q_i = C_0\, hL\sqrt{2gd_0}$$

$$(Equation\ 3.14 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$$

where,

$Q_i = Raised\ Grate\ Catch\ Basin\ Flow\ Interception\ Capacity$

$C_0 = Orifice\ Coefficent$

$C_0 = 0.67$

$L = Perimeter\ of\ Raised\ Grate$

$L = 8.0\ ft\ (MAG\ Type\ 'G'Single\ Grate\ Catch\ Basin)$

$C_f = Clogging\ Factor$

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

$L_f = Effective\ Perimeter\ of\ Riased\ Grate = C_f * L$

$L_f = C_f * L$

$L_f = 0.80 * 8.0$

$[L_f = 6.4\ ft]$

$g = Gravity = 32.2\dfrac{ft}{s^2}$

$d_0 = Effective\ Depth\ at\ Center\ of\ Riased\ Grate\ Opening$

$d_0 = d - \dfrac{h}{2}$

$d_0 = 0.50 - \dfrac{0.33}{2}$

$[d_0 = 0.33\ ft]$

$$Q_i = C_0\, hL\sqrt{2gd_0}$$
$$Q_i = 0.67 * 0.33 * 6.4\sqrt{2 * 32.2 * 0.33}$$
$$Q_i = 6.52$$

$$[Q_i = 6.5\ cfs]$$

The interception capacity of the proposed MAG Type “G” Single Raised Grate Catch Basin is 9.0 cfs, which is greater than the 100-year, 6-hour peak discharge of 1.0 cfs.

Catch Basin Connector Pipe Design for Offsite Catch Basin #3:

Determine the Total (Friction + Inlet) Headloss:

$$h_f = S_f\ L$$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge\ (inc. Flow\ flow\ from\ Parking\ Lot\ CB\#4)$

$Q = 3.4\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{3.4}{1.23} = 2.76\frac{ft}{s}$$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 110\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

<u>Friction Headloss:</u>

$$h_f = S_f\ L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right)\ L$$

$$h_f = \frac{2*32.2*0.013^2}{2.21}\left(\frac{2.76^2}{2*32.2*0.313^{\frac{4}{3}}}\right)*110$$

$$[h_f = 0.30\ ft]$$

<u>Entrance Headloss:</u>

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

$$h_i = (1 + 0.2)\left(\frac{2.76^2}{2*32.2}\right)$$

$$[h_i = 0.14\ ft]$$

<u>Total Headloss:</u>

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.30 + 0.14$$
$$h_{Total} = 0.44$$

$$[h_{Total} = 0.4\ ft]$$

<u>Available Head: h_a</u>

Upstream HW Elevation: 1528.70 ft (6-inches below Raised Grate Catch Basin Weir Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6” blow the weir elevation at the catch basin. The weir elevation at the Raised Grate Catch Basin is 1529.20 ft.

Downstream HW Elevation: 1526.00 ft (Storm Drain Hydraulic Grade Line (HGL) at Manhole #4)

h_a = Upstream HW – Downstream HW = 1528.70 – 1526.00 = 2.70

$$\boxed{h_a = 2.7\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

<u>Offsite Catch Basin Design @ OCB#5:</u>

<u>Determine if Catch Basin operates as a Weir or as an Orifice:</u>

$d \le h \quad \rightarrow Weir\ Flow$

$h > d > 1.4h \quad \rightarrow Transitional\ Flow$

$d \ge 1.4h \quad \rightarrow Orifice\ Flow$

where,

$d = Depth\ of\ Flow\ at\ Raised\ Grate\ Catch\ Basin\ (Depth\ of\ Sump)$

$d = Sump\ Spill\ Elevation - Catch\ Basin\ Weir\ Lip\ Elevation$

$d = 1521.50 - 1519.50$

$[d = 2.00\ ft]$

$h = 4''Height\ of\ Riased\ Grate$

$[h = 0.33\ ft]$

$d \ge 1.4h$

$2.00\ ft \ge 1.4 * 0.33$

$2.00\ ft \ge 0.46\ ft$

$\boxed{\textbf{Orifice Flow}}$

Orifice Flow, Sump Condition, Raised Grate Catch Basin:

$Q_i = C_0\, hL\sqrt{2gd_0}$

$(Equation\ 3.14 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$Q_i = Raised\ Grate\ Catch\ Basin\ Flow\ Interception\ Capacity$

$C_0 = Orifice\ Coefficent$

$C_0 = 0.67$

$L = Perimeter\ of\ Raised\ Grate$

$L = 8.0\ ft\ (MAG\ Type\ 'G'Single\ Grate\ Catch\ Basin)$

$C_f = Clogging\ Factor$

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

$L_f = Effective\ Perimeter\ of\ Riased\ Grate = C_f * L$

$L_f = C_f * L$

$L_f = 0.80 * 8.0$

$[L_f = 6.4\ ft]$

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

$d_0 = Effective\ Depth\ at\ Center\ of\ Riased\ Grate\ Opening$

$d_0 = d - \frac{h}{2}$

$d_0 = 2.00 - \frac{0.33}{2}$

$[d_0 = 1.83\ ft]$

$Q_i = C_0\, hL\sqrt{2gd_0}$

$Q_i = 0.67 * 0.33 * 6.4\sqrt{2 * 32.2 * 1.83}$

$Q_i = 15.36$

$[Q_i = 15.4\ cfs]$

The interception capacity of the proposed MAG Type “G” Single Raised Grate Catch Basin is 15.4 cfs, which is greater than the 100-year, 6-hour peak discharge of 4.5 cfs.

Catch Basin Connector Pipe Design for Offsite Catch Basin #1:

Determine the Total (Friction + Inlet) Headloss:

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Try a 15-inch (d=1.25 ft) Connector Pipe:

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

$K = Pipe\ Roughness\ Coefficient = \frac{2gn^2}{2.21}$ *(Equation 4.5)*

$n = Manning's\ Roughness$

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

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

$Q = Connector\ Pipe\ Design\ Discharge$

$Q = 4.5\ cfs$

$A = Connector\ Pipe\ Cross - Section\ Area\ (15"\ Pipe)$

$A = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$

$V = Velocity\ of\ Flow$

$V = \frac{Q}{A} = \frac{4.5}{1.23} = 3.66\frac{ft}{s}$

$R = Hydraulic\ Radius$

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

$L = Connector\ Pipe\ Length$

$L = 22\ ft$

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

$k_{en} = 0.20$

$(Table\ 5.1 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

Friction Headloss:

$$h_f = S_f\, L$$

$$h_f = K\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2gn^2}{2.21}\left(\frac{V^2}{2gR^{\frac{4}{3}}}\right) L$$

$$h_f = \frac{2 * 32.2 * 0.013^2}{2.21}\left(\frac{3.66^2}{2 * 32.2 * 0.313^{\frac{4}{3}}}\right) * 22$$

$$[h_f = 0.11\ ft]$$

Entrance Headloss:

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

$$h_i = (1 + 0.2)\left(\frac{3.66^2}{2 * 32.2}\right)$$

$$[h_i = 0.25\ ft]$$

Total Headloss:

$$h_{Total} = h_f + h_{en}$$
$$h_{Total} = 0.11 + 0.25$$
$$h_{Total} = 0.36$$

$$[h_{Total} = 0.4\ ft]$$

Available Head: h_a

Upstream HW Elevation: 1519.00 ft (6-inches below Raised Grate Catch Basin Weir Elevation)

Per the City of Scottsdale Drainage Standards and Policies Manual, the hydraulic grade line (HGL) shall be a minimum of 6” blow the weir elevation at the catch basin. The weir elevation at the Raised Grate Catch Basin is 1519.50 ft.

Downstream HW Elevation: 1512.45 ft (Soffit Elevation of 60” Storm Drain Culvert at Junction Tee)

h_a = Upstream HW – Downstream HW = 1519.00 – 1512.46 = 6.54

$$\boxed{h_a = 6.5\ ft}$$

The available head is greater than the total headloss in the catch basin and connector pipe, therefore:

The 15-inch connector pipe has a sufficient capacity to convey the intercepted flow

Storm Drain Hydraulic Grade Line Calculations

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

Location	Type of Headloss	Headloss (ft)	HGL Elevation (ft)
MH#3*	Inlet Control Headwater Elevation	-	1526.00
	Junction Loss (Entrance Headloss)	0.07	1523.53
MH#2 to MH#3	Storm Drain Normal Depth	0.88 feet	
	Storm Drain Friction Headloss	1.19	1523.46
MH#2	Junction Loss (Combined Junction Loss)	0.24	1522.27
PJ#1 to MH#2	Storm Drain Friction Headloss	0.36	1522.03
PJ#1	Junction Loss (Lateral Inflow)	0.09	1521.67
MH#1 to PJ#1	Storm Drain Friction Headloss	0.49	1521.58
MH#1	Junction Loss (Straight-Through Manhole)	0.01	1521.09
CB#1 to MH#1	Storm Drain Friction Headloss	0.74	1521.08
CB#1	Junction Loss (Bend Headloss)	0.05	1520.34
Outlet Headwall to CB#1	Storm Drain Friction Headloss	0.30	1520.29
Outlet Headwall	Junction Loss (Exit Headloss)	0.79	1519.99
		Tailwater Elevation @ Outlet Headwall =	1519.20

*The inlet control headwater elevation governs the hydraulic grade line elevation at Manhole #1. The governing HGL of 1526.00 feet was calculated with an inlet control headwater depth of 2.0 feet and a proposed 18" storm drain invert elevation of 1524.00 ft. Refer to the HGL Calculations in this Apendix for the Inlet Control Headwater Depth calculation.

Storm Drain – Hydraulic Grade Line Calculation:

The procedures outlined in Chapter 4 of the Hydraulics Drainage Design Manual for Maricopa County were used in order to compute the Hydraulic Grade Line (HGL) for the Westworld Sports Fields north parking lot 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 northern portion of the new parking lot as well as the offsite flows that enter the project site from the two undeveloped parcels to the east. The storm drain discharges convey the intercepted flows to the west through the proposed parking lot discharging to the realigned north wash. The northern portion of the new parking lot was graded to drain to 4 shallow sumps where new grated catch basin will be designed to intercept the from the upstream contributing drainage area. An additionally 4 raised grate catch basins were designed at major offsite inflow locations. The proposed storm drain is designed to convey the following governing 100-year, 6-hour peak discharges:

Manhole #4 to Manhole #3	8.3 cfs
Manhole #2 to Manhole #2	11.3 cfs
Manhole #2 to Manhole #1	13.6 cfs
Manhole #1 to Catch Basin #1	20.4 cfs
Catch Basin #1 Outlet Headwall	22.4 cfs

Refer to Appendix C for the HEC-1 Hydrologic Model Results and Appendix D for the Storm Drain Layout Exhibit, showing the alignment of the north parking lot storm drain and location of the proposed catch basins and manholes.

Determine Tailwater Elevation:

The new storm drain discharges into the realigned north wash. The starting tailwater elevation for the design of the storm drain was taken as either 1) the peak stage within the north wash at the time the 100-year, 6-hour peak discharge from the storm drain enters the wash (1517.45 ft) or 2) the soffit elevation of the storm drain at the outlet headwall (1519.20 ft). The conservative soffit elevation of 1519.20 ft was taken as the starting tailwater elevation for the proposed storm drain.

Tailwater Elevation @ Outlet Headwall = 1519.20 ft
(Storm Drain Soffit Elevation @ Outlet Headwall)

<u>Compute the Storm Drain Outlet Headloss at Outlet Headwall:</u>

<u>Exit Loss</u>

$$h_0 = 1.0\left(\frac{V^2}{2g}\right)$$

$(Equation\ 4.16 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_0 = Outlet\ Headloss\ at\ Headwall$

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

$Q = Storm\ Drain\ Design\ Discharge = 22.4\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter = 2.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{22.4}{\pi * \left(\frac{2.0^2}{4}\right)}$$

$$\left[V = 7.13\frac{ft}{s}\right]$$

$$h_0 = 1.0\left(\frac{V^2}{2g}\right)$$

$$h_0 = 1.0\left(\frac{7.13^2}{2 * 32.2}\right)$$

$$h_0 = 0.79$$

$$[h_0 = 0.79\ ft\ @Outlet\ Headwall]$$

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Compute the Friction Headloss – Proposed 24” Storm Drain (Outlet Headwall to Catch Basin #1):

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_f = Pipe\ Friction\ Headloss$

$L = Length\ of\ Storm\ Drain\ Pipe = 24\ ft$

$Q = Storm\ Drain\ Design\ Discharge = 22.4\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter = 2.0\ ft$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{22.4}{\pi * \left(\frac{2.0^2}{4}\right)} = 7.13 \frac{ft}{s}$$

$n = Manning's\ Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{2.0}{4} = 0.50\ ft$$

$S_f = Friction\ Slope$

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

$$S_f = 0.0049\left(\frac{7.13^2}{2 * 32.2 * 0.50^{\frac{4}{3}}}\right)$$

$$\left[S_f = 0.0097 \frac{ft}{ft}\right]$$

$h_f = S_f\ L$

$h_f = 0.0097 * 24$

$h_f = 0.23$

$$\left[h_f = 0.23\ ft\ @Outlet\ Headwall\ to\ Catch\ Basin\ \#1\right]$$

Compute the Headloss through Catch Basin #1:

Bend Headloss:

$$h_{mh} = k_b \left(\frac{V^2}{2g}\right)$$

$(Equation\ 4.12 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_{mh} = Headloss\ at\ Catch\ Basin\ due\ to\ Bend$

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

$\gamma = Deflection\ Angle$

$\gamma = 50°$

$k_b = Bend\ Loss\ Coefficient$

$k_b = 0.36$

$(Figure\ 4.10 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge = 20.4\ cfs$

$D = Upstream\ Proposed\ Storm\ Drain\ Pipe\ Diameter = 2.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{20.4}{\pi * \left(\frac{2.0^2}{4}\right)}$$

$$\left[V = 6.49 \frac{ft}{s}\right]$$

$$h_{mh} = k_b \left(\frac{V^2}{2g}\right)$$

$$h_0 = 0.08 \left(\frac{6.49^2}{2 * 32.2}\right)$$

$$h_0 = 0.05$$

$$[h_{mh} = 0.05\ ft\ @Catch\ Basin\ \#1]$$

Compute the Friction Headloss – Proposed 24” Storm Drain (Catch Basin #1 to Manhole #1):

$h_f = S_f\,L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_f = Pipe\ Friction\ Headloss$

$L = Length\ of\ Storm\ Drain\ Pipe = 91\ ft$

$Q = Storm\ Drain\ Design\ Discharge = 20.4\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter = 2.0\ ft$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{20.4}{\pi * \left(\frac{2.0^2}{4}\right)} = 6.49\frac{ft}{s}$$

$n = Manning'sRoughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{2.0}{4} = 0.50\ ft$$

$S_f = Friction\ Slope$

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

$$S_f = 0.0049\left(\frac{6.49^2}{2 * 32.2 * 0.50^{\frac{4}{3}}}\right)$$

$$\left[S_f = 0.0081\frac{ft}{ft}\right]$$

$h_f = S_f\,L$

$h_f = 0.0081 * 91$

$h_f = 0.74$

$$\left[h_f = 0.74\ ft\ @Catch\ Basin\ \#1\ to\ Manhole\ \#1\right]$$

Compute the Headloss through Manhole #1:

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\left(\frac{V^2}{2g}\right)$$

$(Equation\ 4.11 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_{mh} = Headloss\ at\ Manhole$

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

$Q = Upstream\ Storm\ Drain\ Design\ Discharge = 13.6\ cfs$

$D = Upstream\ Proposed\ Storm\ Drain\ Pipe\ Diameter = 2.0\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{13.6}{\pi * \left(\frac{2.0^2}{4}\right)} = \frac{13.6}{3.14}$$

$$\left[V = 4.33\frac{ft}{s}\right]$$

$$h_0 = 0.05\left(\frac{V^2}{2g}\right)$$

$$h_0 = 0.05\left(\frac{4.33^2}{2 * 32.2}\right)$$

$$h_0 = 0.01$$

$$[h_{mh} = 0.01\ ft\ @Manhole\ \#1]$$

Compute the Friction Headloss – Proposed 18" Storm Drain (Manhole #1 to Manhole #2):

$h_f = S_f\ L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_f = Pipe\ Friction\ Headloss$

$L = Length\ of\ Storm\ Drain\ Pipe = 136\ ft$

$Q = Storm\ Drain\ Design\ Discharge = 13.6\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter = 2.0\ ft$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{13.6}{\pi * \left(\frac{1.5^2}{4}\right)} = \frac{13.6}{3.14} = 4.33 \frac{ft}{s}$$

$n = Manning'sRoughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

$$R = \frac{D}{4} = \frac{2.0}{4} = 0.50\ ft$$

$S_f = Friction\ Slope$

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

$$S_f = 0.0049\left(\frac{4.33^2}{2 * 32.2 * 0.50^{\frac{4}{3}}}\right)$$

$$\left[S_f = 0.0036 \frac{ft}{ft}\right]$$

$h_f = S_f\ L$

$h_f = 0.0036 * 136$

$h_f = 0.49$

$$\left[h_f = 0.49\ ft\ @Manhole\ \#1\ to\ Manhole\ \#2\right]$$

Compute the Combined Headloss at Manhole #2:

At this junction, compute the headloss associated with the straight-through manhole loss and the lateral inflow at the manhole. The combined headloss is the total headloss at Manhole #2.

Straight-Through Manhole Loss:

$$h_{mh} = 0.05\left(\frac{V^2}{2g}\right)$$

$$(Equation\ 4.11 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$$

where,

$h_{mh} = Headloss\ at\ Manhole$

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

$Q = Upstream\ Storm\ Drain\ Design\ Discharge = 11.3\ cfs$

$D = Upstream\ Proposed\ Storm\ Drain\ Pipe\ Diameter = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{11.3}{\pi * \left(\frac{1.5^2}{4}\right)} = \frac{11.3}{1.77}$$

$$\left[V = 6.38\frac{ft}{s}\right]$$

$$h_{mh} = 0.05\left(\frac{V^2}{2g}\right)$$

$$h_{mh} = 0.05\left(\frac{6.38^2}{2 * 32.2}\right)$$

$$h_{mh} = 0.03$$

Gavan & Barker Inc.

Junction Headloss (Lateral Inflow):

$$h_{hj} = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3 cos\theta)}{(A_1 + A_2)g} + \left(\frac{V_1^2}{2g}\right) - \left(\frac{V_2^2}{2g}\right)$$

$(Equation\ 4.10b - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_{hj} = Headloss\ at\ Pipe\ Junction\ with\ lateral\ inflow$

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

$A_1 = Upstream\ Flow\ Area\ (18"\ Storm\ Drain)$

$$A_1 = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.5^2}{4}\right) = 1.77\ ft^2$$

$A_2 = Downstream\ Flow\ Area\ (24"\ Storm\ Drain)$

$$A_2 = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{2.0^2}{4}\right) = 3.14\ ft^2$$

$A_3 = Lateral\ Flow\ Area\ (15"\ Storm\ Drain)$

$$A_3 = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

$Q_1 = Upstream\ Design\ Flow\ Rate = 11.3\ cfs$

$Q_2 = Downstream\ Design\ Flow\ Rate = 13.6\ cfs$

$Q_3 = Lateral\ Inflow\ Rate = Q_2 - Q_1 = 13.6 - 11.3 = 2.3\ cfs$

$V_1 = Upstream\ Flow\ Velocity$

$$V_1 = \frac{Q_1}{A_1} = \frac{11.3}{1.77} = 6.38 \frac{ft}{s}$$

$V_2 = Downstream\ Flow\ Velocity$

$$V_2 = \frac{Q_2}{A_2} = \frac{13.6}{3.14} = 4.33 \frac{ft}{s}$$

$V_3 = Lateral\ Flow\ Velocity$

$$V_3 = \frac{Q_3}{A_3} = \frac{2.3}{1.23} = 1.87 \frac{ft}{s}$$

$\theta = Angle\ between\ lateral\ and\ main\ storm\ drain = 90° \ (Figure\ 4.7)$

$$h_{hj} = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3 cos\theta)}{(A_1 + A_2)g} + \left(\frac{V_1^2}{2g}\right) - \left(\frac{V_2^2}{2g}\right)$$

$$h_{hj} = \frac{2(13.6 * 4.33 - 11.3 * 6.38 - 2.3 * 1.87 * \cos(90°))}{(1.77 + 3.14)32.2} + \left(\frac{6.38^2}{2 * 32.2}\right) - \left(\frac{4.33^2}{2 * 32.2}\right)$$

$$h_{hj} = -0.1671 + 0.6321 - 0.2911$$

$$h_{hj} = 0.17$$

<u>Total Combined Headloss at Manhole #2:</u>

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

$h_{mh_{TOTAL}} = 0.03 + 0.17$

$h_{mh_{TOTAL}} = 0.20$

$[h_{mh} = 0.20\ ft\ @Manhole\ \#2]$

Compute the Friction Headloss – Proposed 18" Storm Drain (Manhole #2 to Manhole #3):

$h_f = S_f\,L$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_f = Pipe\ Friction\ Headloss$

$L = Length\ of\ Storm\ Drain\ Pipe = 142\ ft$

$Q = Storm\ Drain\ Design\ Discharge = 11.3\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter = 1.5\ ft$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{11.3}{\pi * \left(\frac{1.5^2}{4}\right)} = \frac{11.3}{1.77} = 6.38\frac{ft}{s}$$

$n = Manning'sRoughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

$S_f = Friction\ Slope$

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

$$S_f = 0.0049\left(\frac{6.38^2}{2 * 32.2 * 0.375^{\frac{4}{3}}}\right)$$

$$\left[S_f = 0.0115\frac{ft}{ft}\right]$$

$h_f = S_f\,L$

$h_f = 0.0115 * 142$

$h_f = 1.63$

$$\left[h_f = 1.63\ ft\ @Manhole\ \#2\ to\ Manhole\ \#3\right]$$

Compute the Combined Headloss at Manhole #3:

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 Manhole #3.

Compute the Bend Headloss at Manhole #3:

Bend Headloss:

$$h_{mh} = k_b \left(\frac{V^2}{2g}\right)$$

$$(Equation\ 4.12 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$$

where,

$h_{mh} = Headloss\ at\ Manhole\ due\ to\ Bend$

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

$\gamma = Deflection\ Angle$

$\gamma = 50°$

$k_b = Bend\ Loss\ Coefficient$

$k_b = 0.36$

$(Figure\ 4.10 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

$Q = Upstream\ Storm\ Drain\ Design\ Discharge = 8.3\ cfs$

$D = Upstream\ Proposed\ Storm\ Drain\ Pipe\ Diameter = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{8.3}{\pi * \left(\frac{1.5^2}{4}\right)} = \frac{8.3}{1.77}$$

$$\left[V = 4.69 \frac{ft}{s}\right]$$

$$h_{mh} = k_b \left(\frac{V^2}{2g}\right)$$

$$h_{mh} = 0.36 \left(\frac{4.69^2}{2 * 32.2}\right)$$

$$h_{mh} = 0.12$$

<u>Junction Headloss (Lateral Inflow):</u>

$$h_{hj} = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta)}{(A_1 + A_2)g} + \left(\frac{V_1^2}{2g}\right) - \left(\frac{V_2^2}{2g}\right)$$

(*Equation* 4.10*b* – *Drainage Design Manual for Maricopa County*, *Hydraulics*)

where,

h_{hj} = *Headloss at Pipe Junction with lateral inflow*

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

A_1 = *Upstream Flow Area* (18" *Storm Drain*)

$$A_1 = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.5^2}{4}\right) = 1.77\ ft^2$$

A_2 = *Downstream Flow Area* (18" *Storm Drain*)

$$A_2 = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.5^2}{4}\right) = 1.77\ ft^2$$

A_3 = *Lateral Flow Area* (15" *Storm Drain*)

$$A_3 = \pi * \left(\frac{D^2}{4}\right) = \pi * \left(\frac{1.25^2}{4}\right) = 1.23\ ft^2$$

Q_1 = *Upstream Design Flow Rate* = 8.3 *cfs*

Q_2 = *Downstream Design Flow Rate* = 11.3 *cfs*

Q_3 = *Lateral Inflow Rate* = $Q_2 - Q_1$ = 11.3 – 8.3 = 3.0 *cfs*

V_1 = *Upstream Flow Velocity*

$$V_1 = \frac{Q_1}{A_1} = \frac{8.3}{1.77} = 4.69\frac{ft}{s}$$

V_2 = *Downstream Flow Velocity*

$$V_2 = \frac{Q_2}{A_2} = \frac{11.3}{1.77} = 6.38\frac{ft}{s}$$

V_3 = *Lateral Flow Velocity*

$$V_3 = \frac{Q_3}{A_3} = \frac{3.0}{1.23} = 2.44\frac{ft}{s}$$

θ = *Angle between lateral and main storm drain* = 65° (*Figure* 4.7)

$$h_{hj} = \frac{2(Q_2V_2 - Q_1V_1 - Q_3V_3\cos\theta)}{(A_1 + A_2)g} + \left(\frac{V_1^2}{2g}\right) - \left(\frac{V_2^2}{2g}\right)$$

$$h_{hj} = \frac{2(11.3 * 6.38 - 8.3 * 4.69 - 3.0 * 2.44 * \cos(65°))}{(1.77 + 1.77)32.2} + \left(\frac{4.69^2}{2 * 32.2}\right) - \left(\frac{6.38^2}{2 * 32.2}\right)$$

$$h_{hj} = 0.5277 + 0.3416 - 0.6321$$

$$h_{hj} = 0.24$$

Total Combined Headloss at Manhole #3:

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

$h_{mh_{TOTAL}} = 0.12 + 0.24$

$h_{mh_{TOTAL}} = 0.36$

$[h_{mh} = 0.36\ ft\ @Manhole\ \#3]$

Compute the Friction Headloss – Proposed 18" Storm Drain (Manhole #3 to Manhole #4):

$$h_f = S_f L$$

$(Equation\ 4.6 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_f = Pipe\ Friction\ Headloss$

$L = Length\ of\ Storm\ Drain\ Pipe = 191\ ft$

$Q = Storm\ Drain\ Design\ Discharge = 8.3\ cfs$

$D = Proposed\ Storm\ Drain\ Pipe\ Diameter = 1.5\ ft$

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

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{8.3}{\pi * \left(\frac{1.5^2}{4}\right)} = \frac{8.3}{1.77} = 4.69\frac{ft}{s}$$

$n = Manning's Roughness$

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

$K = Pipe\ Roughness\ Coefficient$

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

$R = Hydraulic\ Radius$

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

$S_f = Friction\ Slope$

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

$$S_f = 0.0049\left(\frac{4.69^2}{2 * 32.2 * 0.375^{\frac{4}{3}}}\right)$$

$$\left[S_f = 0.0062\frac{ft}{ft}\right]$$

$$h_f = S_f L$$

$$h_f = 0.0062 * 192$$

$$h_f = 1.19$$

$$[h_f = 1.19\ ft\ @Manhole\ \#3\ to\ Manhole\ \#4]$$

Compute the Storm Drain Entrance Headloss at Manhole #4:

Entrance Headloss:

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

$(Equation\ 4.15 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

where,

$h_i = Headloss\ at\ Storm\ Drain\ Pipe\ Entrance$

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

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

$k_{en} = 0.20$

$(Table\ 5.10 - Drainage\ Design\ Manual\ for\ Maricopa\ County, Hydraulics)$

$Q = Storm\ Drain\ Design\ Discharge = 8.3\ cfs$

$D = Upstream\ Proposed\ Storm\ Drain\ Pipe\ Diameter = 1.5\ ft$

$V = Velocity\ of\ Flow$

$$V = \frac{Q}{A} = \frac{Q}{\pi * \left(\frac{D^2}{4}\right)} = \frac{8.3}{\pi * \left(\frac{1.5^2}{4}\right)} = \frac{8.3}{1.77}$$

$$\left[V = 4.69\frac{ft}{s}\right]$$

$$h_i = k_b\left(\frac{V^2}{2g}\right)$$

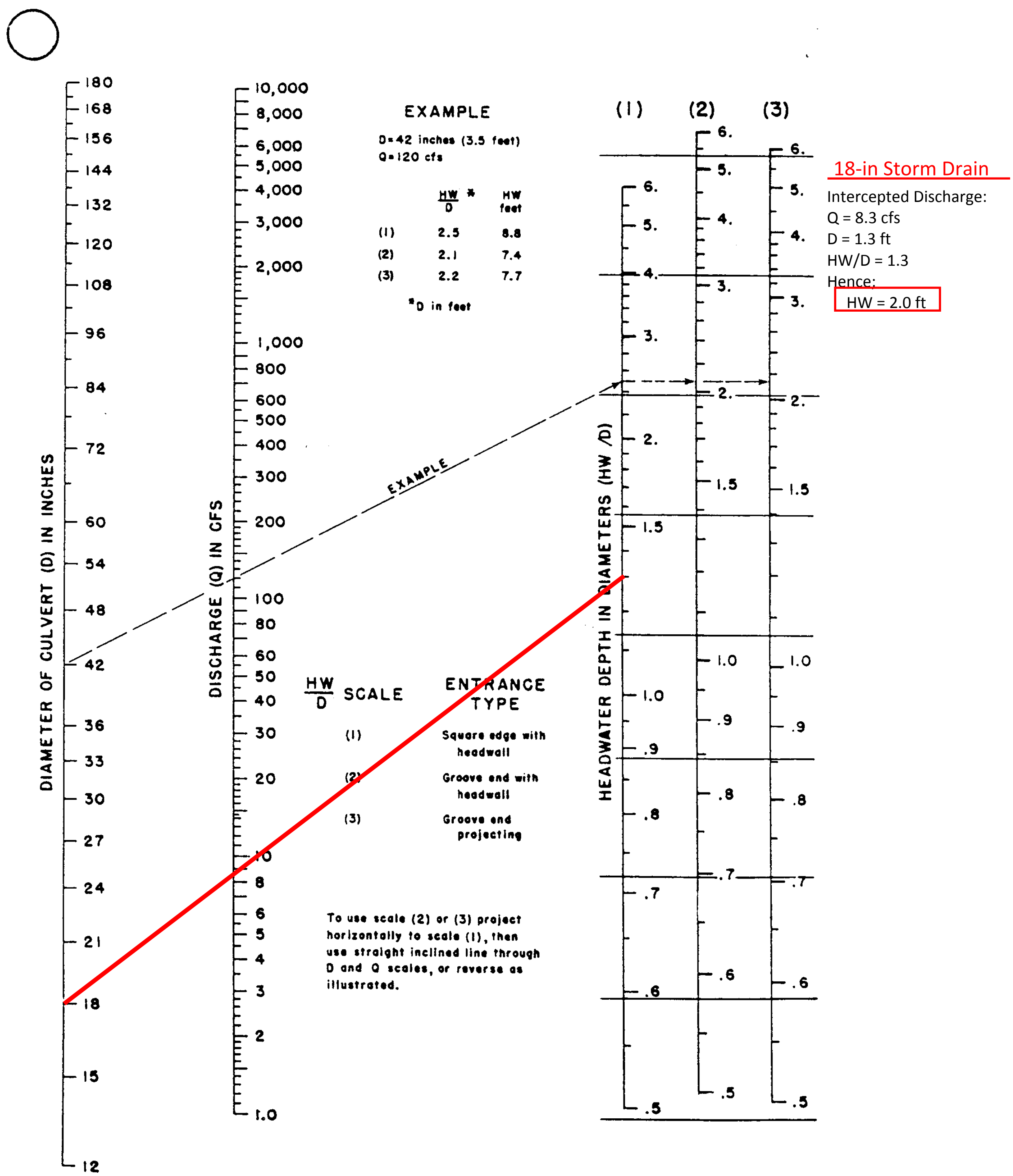
$$h_i = 0.20\left(\frac{4.69^2}{2 * 32.2}\right)$$

$$h_i = 0.07$$

$$[h_i = 0.07\ ft\ @Manhole\ \#4]$$

The inlet control headwater elevation for the peak discharge of 8.3 cfs and an 18-inch storm drain pipe is 2.3 feet. Refer to the end of these calculations for the inlet control headwater calculation nomograph.

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

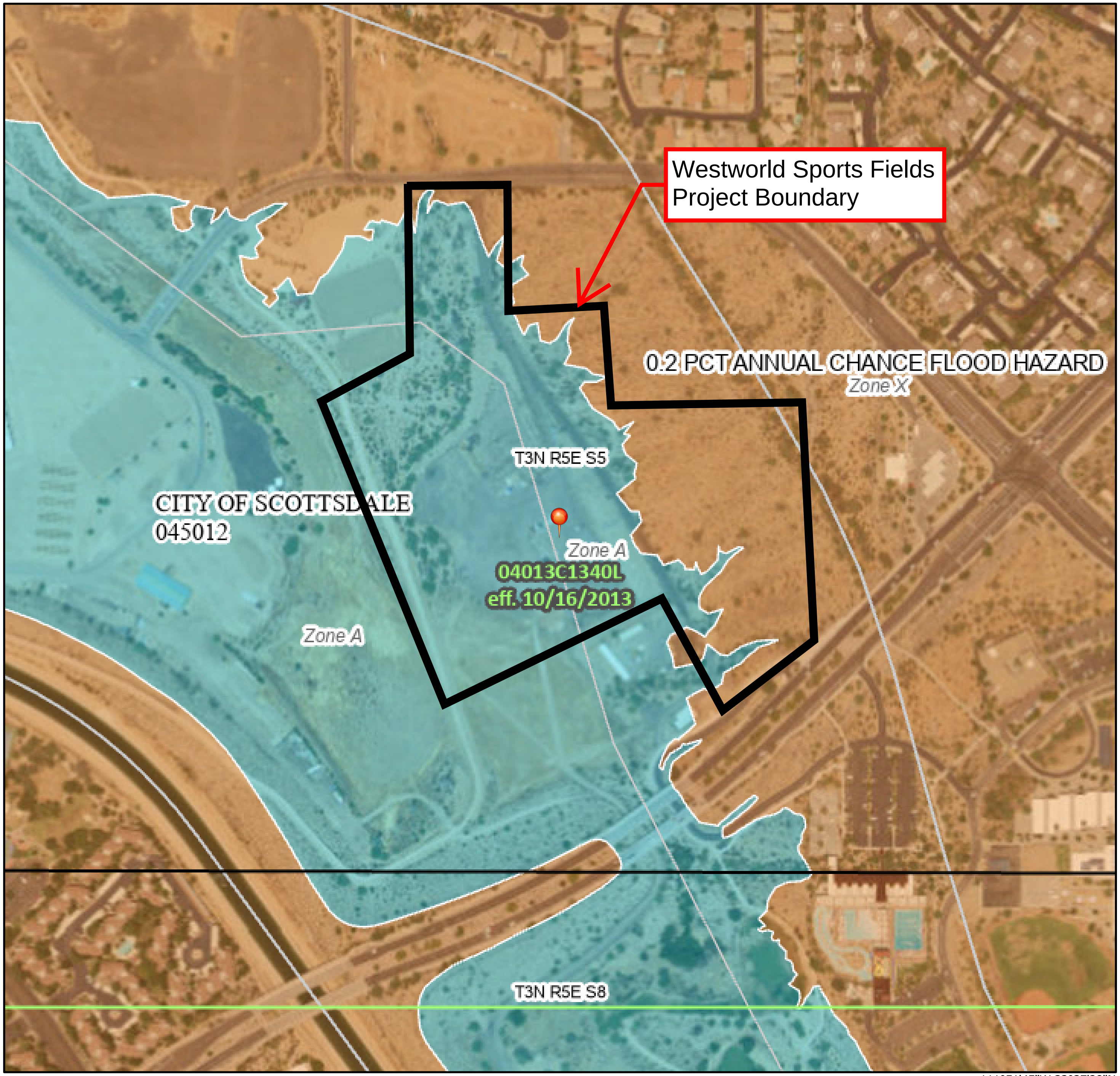


Appendix E: FEMA FIRMette

National Flood Hazard Layer FIRMette

FEMA

111°52'22"W 33°37'58"N



0 250 500 1,000 1,500 2,000 Feet

1:6,000

111°51'45"W 33°37'28"N

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 5/28/2021 at 5:12 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Appendix F: Digital Data

If pdf copy, click HERE to
download the Digital Data

[Digital Data CD]