EXHIBIT A

Surface Water Plan (SWP)
For
City of Sierra Vista

Part 1 – Hydrology
&
Part 2 - Existing Conditions and
Preliminary Analysis of Flood and Erosion Control Alternatives

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EXECUTIVE SUMMARY

Background

This document presents the results of a study conducted by CMG Drainage Engineering, Inc. (CMG) in cooperation with the City of Sierra Vista Public Works Department (COSV) to update the community's Surface Water Plan (SWP) for stormwater runoff control. The COSV has grown significantly since the date of the most recent SWP (2006) and there have been many changes to some of the watercourses, both natural and man-made, including channelization, roadway culvert modifications, enlargement of detention basins on the Ft. Huachuca Military Reservation, and channel entrenchment. The purpose of the Surface Water Plan Update is to guide the planning, construction, location, and function of future surface water conveyance systems and erosion control measures within the City of Sierra Vista.

Flood History

Historical flooding of structures along the major watercourses (which are the focus of this study) has been limited and generally confined to areas such as Soldier Creek, Fry Town and the Sulger subdivision. This report assesses possible solutions for these locations.

The effective FEMA floodplain mapping indicates that several structures are vulnerable to flooding along other watercourses during the 100-year storm, but they generally tend to be local and associated with inadequate structures such as roadway culverts and undersized channels.

Floodplain Mapping

All of the water courses within the study area have previously been mapped with FEMA flood hazard boundaries. The age of the mapping varies but most date back 10 to 20 years or more. The only washes that have more recent mapping are Soldier Creek, Country Club Drainageway, the 3rd Street/Buena #3 Drainageway and portions of the Town & Country School Drainageway/Kings Manor Wash.

The City of Sierra Vista, in recognition of the need to update the mapping, requested FEMA to remap the flood hazard boundaries using most current (2009) topographic mapping. That study was ongoing at the time of this report although the community has reviewed and commented on the preliminary results.

A review of the effective and proposed FEMA mapping, and information provided by the COSV found the following locations where existing structures are vulnerable to flooding during the 100-year storm event.

Locations where Several Overbank Structures are within the FEMA Floodplain

Watercourse Name	Location	Approximate Number of Structures within the Floodplain
Soldier Creek	Garden Avenue to SR-90	32
Fab Avenue Wash	Upstream of Fry Blvd. for a distance of about 600 feet	7 (4 are commercial)
Vista Village Drainageway	7th Street to inlet of culvert upstream of Canyon Drive	50
3rd St/Buena #3 Wash	Sulger Subdivision	36
Woodcutters Canyon	Upstream of Fry Blvd to Lenzner Avenue	5 (3 are commercial)
Montebello/Kings Manor Wash	Colombo Dr to Fry Blvd (SR-90) and Avenida Escuela to Camino Real	27
Coyote Wash	Upstream of Camino Real for about ½ mi	5
South Garden Drainageway	Vicinity of Cashway Mini-Storage units upstream of SR-92	Mini-Storage Units only

Channel Erosion

Several of the watercourses passing through the COSV have been experiencing channel bottom erosion for many decades. The causes are related to increases in storm water runoff volume from urban areas, higher sediment transport capacity associated with channel entrenchment, and deposition of sediment within the Ft. Huachuca detention basins. Long-term channel bottom erosion is considered one of the most important drainage related issues to be addressed by this SWP because it threatens channel stability and can cause infrastructure failure during future floods.

As is, erosion (degradation) is expected to continue at a rate determined by the frequency and magnitude of future stream flows; unless counter measures such as additional erosion control structures are installed. The potential consequences of degradation are the undermining of infrastructure within the wash environments including underground utilities, bank protection, culverts and bridge foundations.

The figure below shows an example of degradation along Coyote Wash about one-third mile downstream of Foothills Drive. The City identified the necessity to place temporary erosion control measures (broken concrete) to prevent headcutting from propagating into the upstream reach where bank protection and a sewer line crossing of the wash are located.

Channel Erosion along Coyote Wash

Bank erosion is also evident at several locations particularly where degradation has occurred. One such example is along the 3rd Street drainageway between Fry Blvd. and Lenzner Avenue where gradual bank erosion threatened a regional sewer line. The City installed about 165 feet of gabion bank protection to mitigate this threat.

Summary of Findings for Existing Conditions

Work completed as a part of the existing conditions SWP analyses has concluded that long-term degradation presents the greatest threat to property and infrastructure stability throughout most of the community. The threatened structures include roads, culverts, bank protection and utility lines that either cross the washes or run parallel to and near the banks. Associated with degradation is an increase in the potential for lateral migration to threaten existing structures that are nearby, but not presently located within the washes.

The community has over recent decades taken actions to control degradation by constructing several grade control structures to prevent headcut propagation. Some of these structures are well engineered while many others appear to be measures installed as an emergency action; those most usually being dumped concrete and rock or broken concrete. Field observations found the condition of these structures to vary, some being in reasonably good condition while others show evidence of potential failure.

While a potential for flooding of structures is present as identified by both the effective and proposed FEMA floodplain mapping, historical flood records (which are limited) indicate that flooding risk is relatively isolated and is not represented as a larger sum of the landmass of the City. Information derived from these sources indicate that past flood damages have been mostly associated with erosion, sediment deposition, debris deposits, perimeter fencing and landscaping. Areas where more frequent flooding occurs are locations such as the Sulger subdivision and Fry Town where widespread shallow flooding occurs. Surface flows through these locations occurs due to the absence of adequate drainage structures such as channels and underground storm drains. Remedying these conditions through fully developed communities can be difficult without large expenditures and neighborhood disruptions.

Identified Flood and Erosion Control Priorities

Priorities identified from the Existing Conditions Study results and Review of Draft FEMA floodplain maps include:

1. Fab Avenue/Fry Town and Vista Village Drainageways Flood Mitigation
2. Coyote Wash – Avenida del Sol to Foothills Drive Grade Stabilization
3. 3rd Street Drainageway – Coronado Drive to Fry Blvd Grade Stabilization
4. Soldier Creek -Buffalo Soldier Trail to SR-90 Flood Mitigation
5. Charleston Wash – Fry Blvd to Colombo Street Grade Stabilization
6. Coyote Wash – Camino Rancho to Town and Country Drive Grade Stabilization
7. Sulger Subdivision Flood Mitigation

Alternative solutions that address the current storm water runoff conditions in Fry Town and the Sulger subdivision are limited due to the dispersed nature of the stormwater runoff sources. One possible alternative would be to lower the street elevations and install curbs to contain flow, however, a much more detailed analysis of this approach is required to determine benefits and cost. Lot to lot drainage inherent in the subdivision design is not fully resolved by this approach.

The feasibility of mitigating flooding within the COSV is questionable due to cost, limited benefits relative to cost, and logistics. All alternatives will require acquisition of occupied property and disruption of neighborhoods to widen channels, install underground storm drains within the streets, along with installing curbs to direct flows and prevent flow onto private property.

Mitigation of channel degradation should be the highest priority for the COSV to prevent infrastructure failures as described above, and to limit future erosion.

Surface Water Plan (SWP)
For
City of Sierra Vista

Part 1 - Hydrology

Sections 33-36, T21S, R20E,
Sections 11,13-15, 19-36, T21S, R21E,
Section 30 T21S, R22E,
Sections 1,12,13,14,23-26,34-36 T22S, R19E,
Sections 1-36 T22S, R20E,
Sections 1-34 T22S, R21E,
Sections 1,2,11-13, T23S, R19E,
Sections 1-12,15-21 T23S, R20E,
G&SRB&M, Cochise County, Arizona

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SECTION 1: INTRODUCTION

1.1 Study Area

The study area for Part 1 (Watershed Hydrology) of this Storm Water Plan (SWP) includes the following named water courses within the corporate limits of the City of Sierra Vista, Arizona.

- Ramsey Canyon Wash
- Garden Canyon Wash
- Charleston Wash
- Coyote Wash
- Soldier Creek
- Murray Springs Wash
- Lewis Springs Wash
- Graveyard Gulch
- Vista Village Drainageway

The western (upstream) study limit coincides with Fort Huachuca while the downstream study limit extends to the San Pedro River. A map showing the study area is provided in Figure 1. Figure 1 also shows the study wash names and study limits.

1.2 Study Purpose

The purpose of the Surface Water Plan Update is to guide the planning, construction, location, and function of surface water conveyance systems within the City of Sierra Vista. In our high desert environment, surface water runoff is a valuable resource that needs to be protected and actively managed to provide the greatest benefit for the public and the ecosystems within the watershed. The presence of development within the watershed alters the dynamics of surface water generation and flow, but this is not necessarily a detriment.

Development increases the amount of impervious surface within a watershed. Impervious surfaces can cause more stormwater to runoff from a developed site if detention is not provided. Storm water leaving a developed site is generally concentrated as a result of the development. Once runoff has been concentrated, its impact on the environment becomes more pronounced. Concentrated flows have higher velocities which increase erosion and sediment transport. However, with proper management, the additional runoff created by development can be beneficial to downstream ecosystems.

This report documents Part 1 of the City of Sierra Vista's 2021 Surface WaterPlan (SWP). Part 1 addresses watershed hydrology and the determination of peak discharge rates for the 10-, 50-, 100- and 500-year return period storm events. Hydrologic modeling for Part 1 of the SWP was conducted by City staff and documented herein by CMG Drainage Engineering, Inc. at the City's request. Please note that Part 1 covers more watersheds and has greater study limits than Part 2. The study area boundaries shown on Figures 1 in Parts 1 and 2 differ for that reason.

It should be noted that the hydrologic modeling results (peak discharge rates) presented in this report have been used by the City of Sierra Vista for FEMA floodplain mapping, design of public infrastructure and land development project since 2016. The effective FEMA floodplain mapping for the 3rd Street Drainageway and Country Club Wash are based on peak discharges documented in this Part 1 report. It is the City's intent to use the hydrologic results presented in the report for any future revisions to the effective FEMA floodplain mapping and for design of private or public infrastructure within the wash environments.

1.3 Previous Surface Water Plan

The City's Surface Water Plan was developed in the mid-1980's by Simons, Li and Associates. The plan analyzed all aspects of surface water within the watersheds impacted by development within the City. The plan provided the following data:

- Estimated 100-year peak flows for conditions as they existed in 1986.
- Estimated 100-year peak flows for ultimate build-out conditions.
- Identified beneficial locations for regional detention basins.
- Estimated the reduction in 100-year peak flows due to the proposed regional detention basins.
- An inventory of existing drainage channels and culverts and an estimate of their hydraulic capacity
- Identification of areas with high potential for flooding
- A general discussion of the erosion potential in local washes
- Introduction of the concepts of establishing Natural Drainage Maintenance Corridors (NDMC) and Flood and Erosion Control Corridors (FECC) to mitigate the impact of erosion on new development.
- Filed a Surface Water Appropriation with the Arizona Department of Water Resources to begin perfecting the City's right to claim and use unclaimed surface water.

The next update to the City's Surface Water Plan was dated 1998 and titled, "Technical Data Support Notebook – Flood Study for the City of Sierra Vista AZ- Hydro-Sciences Southwest, Inc June 1998".The purpose of this update was to compile and augment previous efforts to delineate floodplains in the City and compile into a set of work maps.

The latest update to the City's Surface Water Plan was dated 2006 and titled, "Cochise County Flood Control / Urban Runoff Recharge Plan- Appendix A – Hydrology and Flood Control – Stantec Consulting Inc. April 2006". This report presents the procedures and results of studies conducted jointly by Stantec Consulting, Inc. and Geosystems Analysts, Inc. to evaluate the potential flood control and incidental recharge benefits associated with construction of regional detention basins.

No other Surface Water Plan updates occurred between 2006 and present.

SECTION 2: Data Description

This study is based on high quality Light Detection and Ranging (LIDAR) data provided by Fort Huachuca dated 2010. LIDAR data is gathered using an airborne laser to measure the distance to objects on the ground. This method of data gathering provides accurate terrain elevations. The data provided also included high resolution aerial topography. Fort Huachuca was kind enough to share this valuable data with the City of Sierra Vista. The study area includes the Huachuca Mountains which form the western boundary of the watershed contributing to the City of Sierra Vista. The eastern boundary of the study area varies, but it includes everything within the current city limits.

Analysis of the data gathered provided the following information:

- Digital Elevation Models (DEM)- DEMs are raster files which include elevation on a predetermined grid spacing. The topography between known points is then determined by the difference in elevation between a center point and the eight surrounding points. The closer the grid spacing, the more accurate the topographical map. USGS provides national coverage in 10 to 30-meter resolution. The data used in this study has a resolution of 1.5 meters. The data is divided into 133 separate files which cover the study area. The original spatial projection of the data is WGS_1984_UTM_Zone_12N. The distances are in meters. The vertical datum of the elevations is NAVD 88, metric.

- Contour mapping- maps with a 0.5-meter contour interval were created and included in the original data set from Fort Huachuca. The contour interval is increased to 6 meters in the Huachuca Mountains due to the steepness of the slopes. Again, the information is divided into 133 files to reduce the data load that would have resulted from larger files. The files are in shapefile format for use with GIS platforms.

- Aerial photographs- excellent quality, high resolution aerial photographs help modelers visualize ground conditions and provide information that allows the extraction of data related to impervious surfaces and structures. The color photographs have a resolution of 10 centimeters per pixel. In comparison, USGS produces Digital Orthophoto Quad photos in 1-meter pixel resolution.

- Impervious surfaces and structures- Impervious surfaces such as roads and concrete were identified using a computerized scanning technique. This technique was also used to identify structures. The information was made available as shapefiles for use in GIS based applications.

- Geographic positioning- for spatial data to be useful, it must be assigned to a coordinate system that locates the information on the earth's surface. The data provided by Fort Huachuca incorporates a Geographic coordinate system, WGS_1984_UTM_Zone_12N. This is a global coordinate system used for locating points on a planet-wide scale. A local projection is also used to reduce the distortion inherent in converting a round planet to a flat map. Assigning the spatial location systems to the data allows for a seamless reconstruction of the data files.

- Locally, the City of Sierra Vista and Cochise County use a different geographical projection known as the Cochise County Low Distortion Projection. This projection is based on the North American Datum of 1983 (NAD 83). It is a different projection than that of Fort's data, but GIS is capable of using data from different projections in the same file. COSV staff performed the reprojection, as needed. GIS has the capability to bring files in different projections into the same GIS file, so they work seamlessly. The end result is that all data would be able to be processed in the same GIS file.

Modeling hydrology in the Sierra Vista area required additional information. The necessary data is available from the following sources:

- Soils data- The most important technological advancement related to the Plan update is the availability of high-quality soil data. A comprehensive soil survey of the Sierra Vista watersheds was recently published by NRCS* (survey is undated, but data collection was completed in 2000). The paper report was published as the Soil Survey of Cochise County, Arizona, Douglas-Tombstone Part 7 a few years after the data was gathered. A second study, Soil Survey of Santa Cruz and parts of Cochise and Pima Counties, Arizona10, was published (on paper) in the early 1970's. Soil data available in these surveys is probably the most significant factor that will influence the accuracy of the hydrologic calculations presented in this study. The soil mapping available previously was limited to a small map that showed all of Cochise County at a scale of 1 inch equals 8 miles. This map only identified two soil map units within the Sierra Vista watershed. The new survey identifies 68 soil map units, shows their locations, provides detailed information on soil types and soil layers, and describes numerous properties of the soil units, including the parameters that affect their hydrologic properties. Hydrologic properties of soils determine the amount of water that will run off a watershed during a storm. Data from this soil survey was used to model some of the southernmost watersheds in the Sierra Vista area. The data contained in these studies allows more accurate hydrologic modeling in Sierra Vista. The data is now available digitally from the Soil Survey Geographic (SSURGO) Database[11]. The data is a collection of database files and shapefiles. The shapefiles can be used in conjunction with the Fort data in a GIS platform. Figure 3 maps the soils types for each watershed.

- Parcel boundaries- the City of Sierra Vista and Cochise County keep a current database of parcel boundaries based on land ownership and political subdivisions. The data is in the form of shapefiles. This allows the information to be easily imported into a GIS program. The parcel boundaries make it easier for the user to navigate and orient himself within the data frame without the data load associated with aerial photos.

- Land Cover- The type of vegetation covering the land surface has a significant impact on the amount of storm water runoff from a subbasin. Grasses, trees, urban landscaping and bare ground all influence the amount of rainfall that is trapped and stored on a parcel of land. In hydrologic terms, the water trapped by vegetation is called interception. It is a distinct process from rainfall absorption by soil or depression storage. Leaves and vegetative ground litter trap and hold moisture quickly, greatly influencing the amount of runoff from small storms or at the beginning of a large storm. Vegetative cover information is available at no cost from the National Land Cover Database (NLCD)9. The Land Cover Database has 16 class land cover classifications at a spatial resolution of 30 meters. The information is based on Landsat Enhanced Thematic Mapper satellite data. Rainfall interception capacities were assigned to each land cover classification.

SECTION 3: Hydrologic Modeling

Hydrologic models created herein were created using the HEC-HMS program from the U.S. Army Corps of Engineers (the Corps). This software was chosen for several reasons- it is widely used amongst hydrologic modelers; the output files are easy to customize and easily read; the Corps has developed free extension programs that allow HEC-HMS to be used in conjunction with ArcGIS (HEC-GeoHMS); and another extension program, ArcHydro. Producing the results of the watershed models in HEC-HMS format will allow the data to be readily distributed to potential users. Models may be modified to suit individual situations or analyze numerous scenarios.

The hydrologic modeling method is based on the 2014 ADOT Highway Drainage Design Manual[13] and the Drainage Design Manual for Maricopa County, 2011. The methodologies presented in these manuals are reasonably conservative, clearly presented, appropriate for Arizona, and largely supported by the functions within HEC-HMS. The components of a hydrologic model are:

1. Design Storm Frequency
2. Unit Hydrograph
3. Loss Parameters
4. Channel Routing Method

The following descriptions are only intended as a convenient summary of the model parameters.

3.1 Design Storms

All design storms must incorporate these components:

- Frequency- chance of storm occurring, usually 1% or 100-year storm for floodplain modeling.
- Duration- length of storm in hours
- Rainfall Distribution- pattern in which precipitation occurs during the storm.
- Precipitation Depth- total depth of precipitation associated with the storm duration and frequency.
- Depth Area Reduction Factors- as the size of a watershed increases, the likelihood of the storm having a maximum intensity over the entire watershed decreases. To keep models from being overly conservative, the total rain fall is reduced by a factor directly proportional to the size of the watershed.

3.2 Storm Frequency

For the purpose of the SWP, a frequency of 100-years will be used. This is typically required for floodplain studies and by the Development Code. In addition to the model for the 100-year storm, hydrologic models have been created for the 10-year and 50-year storms.

3.3 Storm Duration

Storm duration needs to be matched to the size of the watershed. The storm duration should not be shorter than the time of concentration for the watershed. Some agencies simply require a longer storm duration to assure that the time of concentration is no longer than the duration. Other agencies impose watershed size limits to shorter storms. For example, ADOT used to require a 24-hour storm for watersheds greater than 1 square mile.

The areas of the watersheds considered in Sierra Vista are generally less than 20 square miles. However, Garden Canyon Wash is approximately 32 square miles. A rough check of the Garden Canyon Wash watershed and the Coyote Wash watershed (the second largest and flattest) show the approximate times of concentration are about 4 hours. Therefore, a 6-hour storm distribution is feasible for the purpose of the SWP.

A second consideration in regard to storm length is the type of storms that occur in Sierra Vista. Winter storms are generally long duration, less intense storms that occur over large areas. Summer thunderstorms are usually briefer, more intense and occur in a limited area. The largest storms recorded by NOAA in the Sierra Vista area occur more often in the summer, but the largest rainfall depth recorded was in January (3.9 inches in 1905). For the purpose of the Surface Water Plan, both the 24-hour and 6-hour storms are to be considered for all watersheds.

3.4 Rainfall Distribution

The rainfall distribution reflects variations in rainfall intensity that occurs during the storm. Most rainfall distributions used in regulatory storms start rather slowly, have an intense rainfall near the middle of the storm and then taper off as the storm ends. The rainfall distribution has a **significant** impact on peak flows. For example, the Soldier Creek watershed had a peak flow of 7,344 cubic feet per second (cfs) when the storm distribution was an Alternating Block hyetograph. The flow for the same watershed was 4,953 for an SCS Type II distribution. Both of these storms were 24-hours long with the same total precipitation and same physical characteristics used for the watershed.

As discussed in the previous paragraph, the storm durations to be considered for the Surface Water Plan are 6-hours long and 24-hours long. The SCS Type II 24-hour storm distribution is a commonly used storm distribution for many parts of the United States, such as the Maricopa County Flood Control District. Therefore, the Surface Water Plan modeling will incorporate the Type II distribution.

Storm durations of 6 hours are not all that common, but Maricopa County has four, 6-hour storm distributions that were based on a storm that occurred on August 19, 1954, in Queen Creek, Arizona and another statistically based curve similar to the Queen Creek curves. These 6-hour storm durations, called Pattern 1, Pattern 2, etc., are suitable for use the Sierra Vista area for several reasons:

- Size of the watersheds considered in the surface water plan- most watersheds are too big to be modeled with a 1-hour storm.
- Geographically and meteorologically, there is the potential for a storm similar to the Queen Creek Storm to occur in Sierra Vista.
- Initial model results indicated that the 24-hour storm distributions might be underestimating peak flows from some of the smaller watersheds.
- These 6-hour storms are more representative of a thunderstorm on a watershed scale.

The 6-hr or 24-hr storm pattern selection is based on the contributing drainage areas, as indicated in the Maricopa County Hydrology Manual. The higher the drainage area, the higher the pattern number. The storm patterns selected for the 6- and 24-hour storm durations for each watershed are summarized in Table 1 below. Watershed boundaries, sub-area boundaries and concentration point locations are shown in Figure 2.

Table 1 – Storm Patterns for HEC-HMS Modeling

Watershed	Area (sm)	6-hr Storm	24-hr Storm
Soldier Creek	10.465	Pattern 2	Type II
Graveyard Gulch	0.405	Pattern 1	Type II
Vista Village Drainageway	0.943	Pattern 1	Type II
Charleston Wash	16.759	Pattern 2	Type II
Coyote Wash	19.160	Pattern 3	Type II
Murray Springs Wash	5.213	Pattern 2	Type II
Lewis Springs Wash	7.486	Pattern 2	Type II
Garden Canyon Wash	32.437	Pattern 3	Type II
Ramsey Canyon Wash	20.974	Pattern 3	Type II

3.5 Precipitation Depths

Sierra Vista is somewhat unique in that many of the local watersheds originate in the Huachuca Mountains. The mountains typically experience more rainfall than the alluvial plain east of the mountains. They not only receive rainfall more often, but they can also receive heavier rainfall during a storm event that covers the entire watershed.

NOAA produces rainfall depth charts for the United States. The data is available using interactive maps. The user simply selects the average elevation of the watershed and downloads rainfall depths for different frequencies and durations of storms. Two sets of data were used for modeling in the SWP- mountain precipitation depths and valley precipitation depths. The precipitation depths were based on the average elevations in the mountains (calculated average elevation of Miller Peak and the base of the foothills- 7,224 feet) and in the valley (average elevation of the base of the foothills and the San Pedro River- 4,452 feet).

The 100-year precipitation depths for the mountain and valley areas are given in Table 2 below. The City is located within the Valley region of the watersheds.

Table 2 - 100-Year Storm Precipitation Depths

100-Year Storm Precipitation Depths (inches)		
Storm	**100-year, 6-HR**	**100-year, 24-HR**
Mountain Design Storm	4.90	5.50
Valley Design Storm	3.35	3.86

It should be noted that current precipitation depths for the 100-year, 24-hour and 100-year, 6-hour storms are roughly the same as the precipitation depths used in the original Surface Water Plan from 1985. Comparisons of the isopluvial maps show very little difference in precipitation depths.

3.6 Depth Area Reduction Factors

As watershed size increases, it is less and less likely that the full precipitation depth will fall evenly over the entire watershed. Therefore, reduction factors are commonly used to reduce the total precipitation depth over the entire watershed. The following depth area reduction factors for the 6-hour duration storm were applied to the Sierra Vista watersheds as indicated below in Table 3. These depth area reduction factors were taken from the Drainage Design Manual for Maricopa County, 2011. Interpolated Watershed Depth-Area Reduction Factors for each of the study watersheds are provided in Table 4.

Table 3- 6-Hour Storm Depth Area Reduction Factors

6-Hour Storm Depth Area Reduction Factors *From Table 2.1, MCFCD Hydrology Design Manual, 2011*		
Area (square miles)	**DAR**	**Pattern Storm**
0	1	Pattern 1
0.5	0.994	Pattern 1
1	0.987	Pattern 1
2.8	0.975	Pattern 2
5	0.96	Pattern 2
10	0.94	Pattern 2
16	0.922	Pattern 3
20	0.91	Pattern 3
30	0.89	Pattern 3
40	0.87	Pattern 3

Table 4 - Interpolated Watershed Depth-Area Reduction Factors for the 6-Hour Storm

Sierra Vista 6-Hour Storm Interpolated Watershed Depth-Area Reduction Factors			
Interpolated DAR	**Watershed**	**Watershed Area (sm)**	**Storm Distribution**
0.939	Soldier Creek	10.465	Pattern 2
0.995	Graveyard Gulch	0.405	Pattern 1
0.988	Vista Village Drainageway	0.942	Pattern 1
0.925	Charleston Wash	15.040	Pattern 2
0.913	Coyote Wash	19.162	Pattern 3
0.959	Murray Springs Wash	5.213	Pattern 2
0.950	Lewis Springs Wash	7.468	Pattern 2
0.885	Garden Canyon Wash	32.438	Pattern 3
0.908	Ramsey Canyon Wash	20.974	Pattern 3

The following depth area reduction factors for the 24-hour duration storm were applied to the Sierra Vista watersheds as indicated below in Table 5. These depth area reduction factors were taken from the Drainage Design Manual for Maricopa County, 2011. Interpolated Watershed Depth-Area Reduction Factors for each of the study watersheds are provided in Table 6.

Table 5 - 24-Hour Storm Depth Area Reduction Factors

24-Hour Storm Depth Area Reduction Factors *From Table 2.2, MCFCD Hydrology Design Manual, 2011*	
Area (sm)	**DAR**
0	1
1	0.995
5	0.975
10	0.95
20	0.918
30	0.9
40	0.887

Table 6 -Interpolated Watershed Depth-Area Reduction Factors for the 24-Hour Storm

Sierra Vista 24-Hour Type II SCS Storm Interpolated Watershed Depth-Area Reduction Factors		
Interpolated DAR	**Watershed**	**Watershed Area (sm)**
0.949	Soldier Creek	10.465
0.998	Graveyard Gulch	0.405
0.995	Vista Village Drainageway	0.942
0.934	Charleston Wash	15.040
0.921	Coyote Wash	19.162
0.974	Murray Springs Wash	5.213
0.963	Lewis Springs Wash	7.468
0.897	Garden Canyon Wash	32.438
0.916	Ramsey Canyon Wash	20.974

3.7 Storm Hyetographs

A storm hyetograph depicts the precipitation depth as a function of time. In this Surface Water Plan, it is represented as an amount of precipitation that falls during sequential 15-minute periods during the storm event. Complete hyetographs can be found in Appendix B. There are thirty-six separate hyetographs to customize rainfall patterns for mountain subbasins, city subbasins, 24-hour storms, 6-hour storms, the various watersheds modeled.

3.8 Existing Condition Hydrology

Hydrologic models represent the runoff conditions resulting from the storm hyetograph within a defined watershed. The volume and peak flow of runoff resulting from a given storm are influenced by:

- The unit hydrograph selected,
- Absorption or lack of absorption of rainwater by the soil, vegetation, or impervious areas,
- Speed at which runoff is able to move through the watershed,
- The routing of concentrated flow through downstream subbasins,
- The physical characteristics of the watershed- slope, roughness characteristics, shape of contributing area (long and thin versus nearly round),

The headwaters of most of the watersheds are located in the Huachuca Mountains, west of Sierra Vista. The modeling is terminated at the confluence of the local washes with the San Pedro River. The purpose of the Surface Water Plan Part 1 is to determine peak storm water flows with the annexation limits of the City of Sierra Vista

3.9 Unit Hydrographs

The Clark Unit Hydrograph was chosen for use with the Surface Water Plan. The Clark Unit Hydrograph is used by both ADOT, and the Flood Control District of Maricopa County as documented in their hydrology manuals.

To utilize the Clark Unit Hydrograph procedure, the watershed size is recommended to be less than about 5 square miles in size as indicated in the Maricopa Hydrology Manual. A unit hydrograph is derived from or is representative of a specific watershed; therefore, a unit hydrograph is a lumped parameter that reflects all of the physical characteristics of the watershed that affect the time rate at which rainfall excess drains from the land surface.

3.10 Watershed Soils

Detailed soil data is probably the most significant contribution to this update of the Surface Water Plan. Using the detailed information from two United States Department of Agriculture, Natural Resources Conservation Service (NRCS) soil surveys in the Sierra Vista area; *Santa Cruz and Parts of Cochise and Pima Counties, Arizona,* and *Soil Survey of Cochise County, Arizona, Douglas-Tombstone Part* allows development of a model without overly conservative soil absorption factors.

Runoff enters soil at varying rates of absorption. The speed and quantity of water that is absorbed into *saturated* soil is represented by the saturated hydraulic conductivity, XKSAT. XKSAT is the rate of speed (in inches) of water that enter the soil per hour (in/hr). The soil survey lists types of soils found in the watershed, shows a range of depths at which the soils are found, and provides a textural description of the soil in each layer. The soil survey does list soil permeability, this is related to saturated hydraulic conductivity, but it is not the same. The hydrologic models require specific soil parameters to calculate rainfall runoff. These parameters are Initial Water Content, Saturated Water Content, Suction, Conductivity (XKSAT), and Natural Impervious Area (rock outcrops). While these hydrologic parameters are not given in the soil surveys, the information needed to determine the parameters is given in the surveys.

Values for hydrologic properties of the soils in the watershed were taken from the 2014 ADOT Hydrology Manual, Appendix C. ADOT compiled all the NRCS (SCS) soil surveys in the State of Arizona and determined Green and Ampt soil parameters for each soil type listed in each soil survey. These values are ready to be used directly by HEC-HMS models for subbasins containing a single soil type. However, nearly every subbasin is comprised of a variety of soils. For these cases, ADOT specifies a methodology for combining the soil parameters to represent a uniform soil type throughout the subbasin. For the parameters of Suction and Conductivity, ADOT's equation 3.1 is used to find the geometric mean of the different values present in the watershed. For the other soil parameters, a simple weighted average is used to determine the combined value within the subbasin. The reader is referred to the 2014 ADOT Hydrology Manual for a complete discussion of this topic. The soils types within the study watersheds can be examined on Figure 3.

3.11 Vegetation

Vegetation slows or reduces runoff by providing litter which absorbs runoff; by intercepting rainfall via leaves, and branches; and by holding soil in place using roots and possibly larger pieces of litter. The effect of vegetation on runoff production is fairly consistent and predictable. Different types of vegetation and the concentration of the vegetation combine to intercept a predictable portion of rainfall. This is usually expressed as inches of rainfall intercepted.

Land cover information is available from National Land Cover Database (NLCD). It is based on 2011 satellite images. Surface cover characteristics are presented in 30-meter grid squares. Of the total types of land cover identified in the database, ten different types of land cover occur in the Sierra Vista area. Local interception coefficients were determined using range cover information for from the soil survey, and a search of available literature, aerial photographs from September 2008; and consideration of the information in the ADOT and Maricopa County Hydrology Manuals. The manuals typically do not separate vegetative interception from surface storage (discussed below). Due to the lack of naturally occurring vegetation in Maricopa County and many parts of Arizona, this is not surprising. However, the Sierra Vista area is heavily vegetated. As seen in the 2008 aerial photos, many areas approach 100% vegetative cover when plant litter and tree canopies are considered. For these models, vegetative interception factors will be used as follows:

Table 7 - Vegetative Cover Abstractions for Land Cover Defined by NLCD

Vegetative Cover Abstractions for Land Cover Defined by NLCD		
Cover Designation from NLCD	**Cover Description**	**Weighted Interception Factors (in)**
11	Open Water	0.00
21	Developed, Open Space	0.22
22	Developed, Low Intensity	0.17
23	Developed, Medium Intensity	0.08
24	Developed, High Intensity	0.04
31	Barren Land (Rock/Sand/Clay)	0.04
42	Evergreen Forest	0.38
43	Mixed Forest	0.22
52	Shrub/Scrub	0.26
71	Grassland/Herbaceous	0.17

Local interception rates were entered into the GIS layer that contains the cover information. In this way, only the effects of vegetation were considered when extracting the data from the GIS map. Even though some of the land cover elements were described as developed, the interception coefficients used in the model did not account for impervious areas. Modeling of impervious areas is described below.

3.12 Impervious Areas

The original LIDAR data from the Fort was interpreted to create polygons that represented impervious areas (asphalt and concrete) and structures. Asphalt, concrete, and structures are all impervious and have similar effects on runoff production. For the purpose of this discussion, they will be lumped together as impervious areas. The software used to interpret the impervious areas had limitations due to several conditions:

- Impervious areas with overhanging trees were not considered impervious. The software identified the tree and did not consider it to be impervious. This was especially prevalent in residential areas.
- Cars were not considered impervious, but they are almost always parked on an impervious surface.
- Streets in residential areas were consistently underestimated. Sidewalks, curbs and gutters were rarely identified. Frequently, strips of asphalt adjacent to gutters were not identified, either.
- Rural areas in the Sierra Vista watershed are frequently occupied by mobile homes. Generally, water can enter under the homes and be absorbed by the soil underneath (at least to a certain extent). The mobile home would be identified as impervious, but due to the absorption underneath, it probably should be considered pervious.
- Gravel areas were sometimes identified as impervious.
- Areas with detention basins- commercial plots and a few subdivisions- should be modeled as bare soil. This will produce a relatively accurate peak flow, but it will underestimate the actual volume of runoff.

  Despite its drawbacks, the impervious layers still proved to be quite useful. The following strategies were used to try to improve the accuracy of the information available:

- A study of residential areas showed that the LIDAR data was about 18% deficient in identifying impervious areas. Residential areas were easily identified with a separate polygon layer. The impervious surface area in these residential tracts was increased by 18%.
- Impervious polygons were ignored in rural areas. Instead, aerial photos were used to roughly estimate actual impervious areas.
- In cases where storm runoff is detained, the impervious areas from this layer were deleted.

- Clean-up of known conditions was performed on a case-by-case basis in some locations.

3.13 Surface Storage

The final rainfall abstraction considered in the models is surface storage. Surface storage is the retention of rainfall in small depressions that occur on the surface. For the purposes of the hydrology models, surface storage was calculated based on the slopes of the soil groups identified in the soil survey. Each soil complex had a “typical slope” in the soil survey. The following surface storage coefficients were assigned based on a discussion in the Maricopa County Hydrology Manual (page 4-5, first paragraph).

Table 8 - Surface Storage Coefficients by Land Slope

Surface Storage Coefficients by Land Slope	
Slope	**Interception (in)**
0% to 1%	0.11
2%	0.075
3%	0.045
4%	0

3.14 Existing Conditions Hydrology Results

Hydrologic models for each watershed were created using the parameters described above. In all cases, the watershed was analyzed with a 6-hour storm and a 24-hour storm. All 24-hour storms had a SCS Type II distribution. The 6-hour storm distribution is determined by the overall size of the watershed. Smaller watersheds are analyzed with a Pattern 1 storm; larger watersheds use patterns with higher numbers. The final design storm for the watershed is the storm that produces the largest flow at the terminus. The final 100-Year Storm peak flows at the downstream watershed boundary are summarized in Table 9 below. Each of the major watercourses listed in Table 9 have named tributary washes. For example, the Buena No. 3 3rd Street Drainageway and Woodcutters Canyon Wash are tributaries to Charleston Wash. Figure 2 delineates watershed sub-area boundaries. Tables provided in Appendix D provide the sub-area labels, drainage areas and peak discharge rates for each of sub-areas delineated on the watershed map. Appendix E includes an electronic version of the watershed map and soils maps which are more readable than Figures 2 and 3 of this report. Appendix E includes the HEC-HMS models for the study washes.

Table 9 – 100-year Peaks Flows at Downstream Watershed Boundary

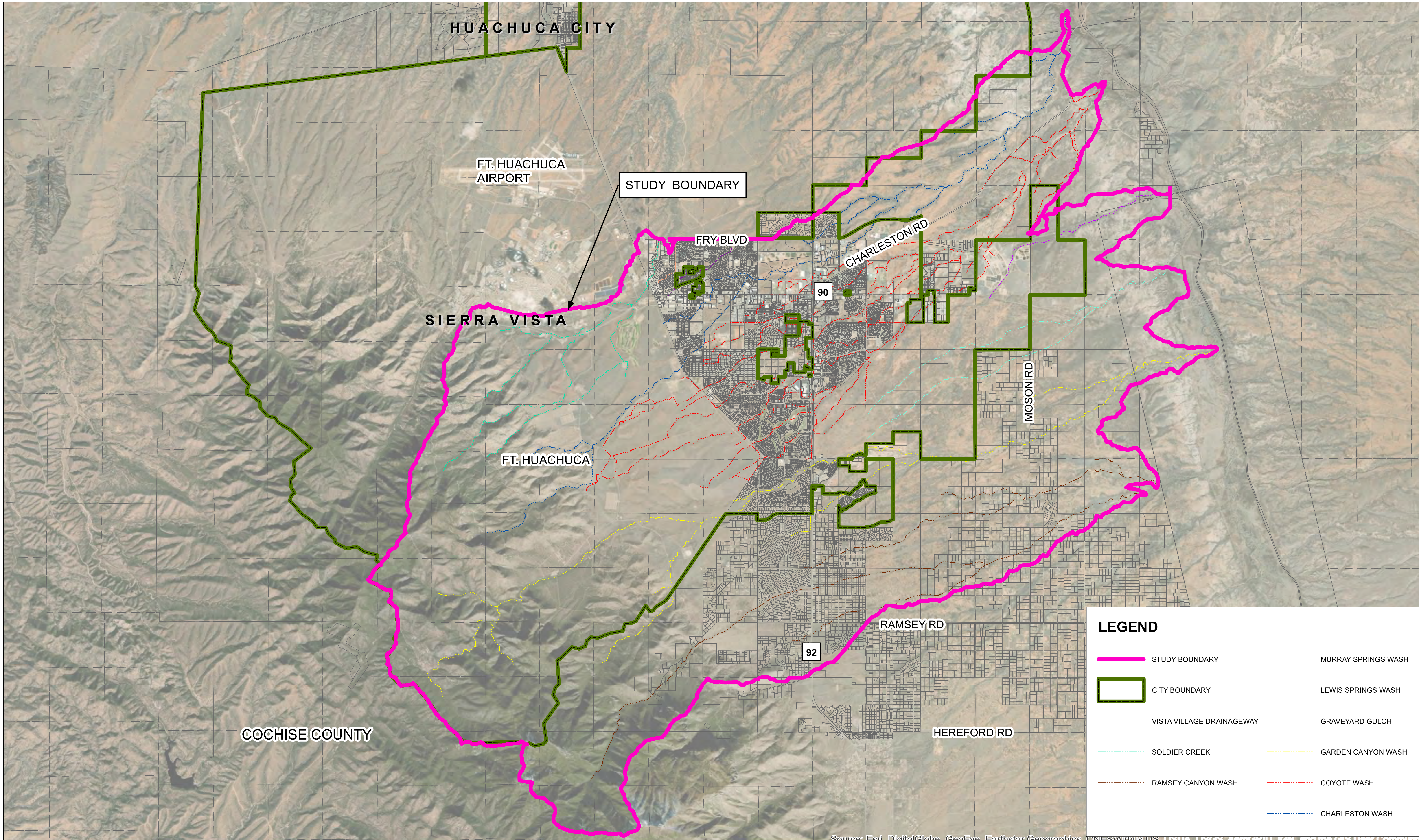
Watershed	Watershed Area (sq. mi.)	100-Year Design Storm Duration	Design Storm Name	Design Flow at Terminus[1] (cfs)
Soldier Creek	10.466	6-hour	Pattern 2	5,045
Graveyard Gulch	0.405	6-hour	Pattern 1	541
Vista Village Drainageway	0.943	6-hour	Pattern 1	732
Charleston Wash	15.040	6-hour	Pattern 2	4,763
Coyote Wash	19.162	6-hour	Pattern 3	pending
Murray Springs Wash	5.213	6-hour	Pattern 2	1,582
Lewis Springs Wash	7.486	6-hour	Pattern 2	2,039
Garden Canyon Wash	32.437	24-hour	Type II	10,716
Ramsey Canyon Wash	20.974	6-hour	Pattern 3	7,815

[1] Terminus for Soldier Creek, Graveyard Gulch and Vista Village Drainageway is SR 90 Bypass. All other models terminate at the San Pedro River

The summary table indicates that the smaller watersheds produce higher peak flows in the shorter, more intense 6-hour storm. Only the largest watershed, Garden Canyon Wash, had a higher peak flow for the 24-hour storm.

Detailed tables that provide peak flows for the 10-, 50-, and 100-year at each sub-area concentration point shown on Figure 2 are provided in Appendix D.

Appendix A
Figures



N.T.S.

FIGURE 1: LOCATION MAP

Portions of Sections 33-36, T21S, R20E, Sections 11,13-15, 19-36, T21S, R21E, Section 30 T21S, R22E, Sections 1,12,13,14,23-26,34-36 T22S, R19E, Sections 1-36 T22S, R20E, Sections 1-34 T22S, R21E, Sections 1,2,11-13, T23S, R19E, Sections 1-12,15-21 T23S, R20E, G&SRB&M, Cochise County, Arizona

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DATE: 01/15/2021

Document Path: Z:\PROJECTS\2020\20-005_City_of_Sierra_Vista_Charleston_Wash_CLOMR\HEC-HMS_v4.2_MasterPlan_Rpt\loc_map.mxd

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR CHARLESTON WASH

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
WC_08	1.925	1050.7	865.4	483.8
WC_07	0.801	471.9	387.4	214.9
WC_06	0.732	652.7	538.1	300.8
WC_J_6_7_8	3.458	2016.7	1656.2	918.6
WC_R_5	3.458	1995	1640.7	913.5
WC_05	0.892	236.6	172.6	45.9
WC_J_DB	4.35	2226	1801.9	956.3
WC_Det_Bas	4.35	1722.9	1227.5	280
WC_R_4	4.35	1710.1	1217	279.9
WC_04	0.351	329.8	271.1	147.5
WC_J_4	4.701	1724.8	1226.6	280.1
WC_R_3	4.701	1703.3	1218.2	279.9
WC_03	0.076	40	29.6	8.1
WC_J_3	4.777	1705.4	1219.7	279.9
WC_R_2	4.777	1705	1210.7	279.9
WC_02	0.153	238.5	109.1	53.4
WC_J_2	4.93	1709.3	1213.2	279.9
WC_R_1	4.93	1701.3	1205.9	279.8
Bu3ST_05	0.381	161.4	117.4	31.1
Busby_Det_Bas	0.381	0	0	0
Bu3ST_R_04ABDB	0.381	0	0	0
Bu3ST_04A	0.144	165.9	138.5	79.5
Bu3ST_04B	0.073	82	67.5	36
Bu3ST_04C	0.034	47.3	39.5	21.7
Bu3ST_04D	0.029	22.1	16.5	4.1
Bu3ST_R_04C	0.029	20.7	15.1	3.8
Bu3ST_J_04C	0.063	61.7	49.2	22.7
Bu3ST_R_04B	0.063	61.3	49	22.6
Bu3ST_J_04B	0.136	143.3	116.5	58.6
Bu3ST_R_04A	0.136	139.4	112.8	54.6
Bu3ST_J_4A	0.661	305.3	251.3	134.1
Bu3ST_R_03B	0.661	304.1	250.1	133.2
Bu3ST_03B	0.089	108.9	92.7	58.2
Bu3ST_J_03B	0.75	413	342.8	191.3
Bu3ST_R_3A	0.75	395.6	326.1	181.4
Bu3ST_03A	0.179	219	187.6	120.8
Bu3ST_J_3A	0.929	614.6	513.6	297.9
Bu3ST_R_2	0.929	596.5	500.7	295.5
Bu3ST_02	0.182	173	145.1	86.3
Bu3ST_J_2	1.111	769.5	645.8	381.9
Bu3ST_R_1	1.111	759.3	634.5	370.7
Bu3ST_01	0.17	175.2	146	83.1
Bu3ST_J_01	1.281	927.2	774.2	450.9
WC_J_Bu3ST	6.211	1740.4	1231.4	625.1
CH_R_WC_1	6.211	1715.2	1227.3	610.6
WC_01	0.119	133.6	112.6	67.6
CH_J_WC_01	6.33	1718.8	1229.1	660.2
CH_R_4B	6.33	1700.6	1219.7	658.8
CH_05	0.264	209	173.2	97.3
CH_R_4B_5	0.264	206.1	170.7	95.6
CH_04B	0.152	69.6	52.5	17
CH_J_4B	6.746	1801.2	1408.6	770.1
CH_R_4A	6.746	1792.6	1393	747.9
CH_04A	0.117	51.6	38.1	10.2
CH_J_4A	6.863	1844.3	1431.2	758.1
CH_R_3	6.863	1805.6	1389.8	753
CH_03	0.205	111.5	86.1	33.2
CH_J_3	7.068	1917.1	1475.9	784.2
CH_R_2	7.068	1878.2	1464.5	768.8
CH_02	0.296	174.9	140.3	68.5
CH_J_2	7.364	2053.1	1604.9	837.3
CH_R_1	7.364	2038.3	1576.1	819.9
RCE_01	0.829	587.4	486.9	278.1
CTY_01	0.7	290.3	226.9	97
CTY_01_J	0.7	290.3	226.9	97
RCE_R_1	0.7	284.7	223	96.5
CH_01	0.359	180.2	145.8	76.3
CH_J_RCE_1	9.252	3016.1	2365.3	1224.8
B3N_R_8	9.252	2972.2	2308.2	1212.1
B3N_09	0.624	285.3	234.9	132.4
B3N_08	0.446	234.2	191.4	104
B3N_J_8_9	10.322	3486.9	2730.3	1445.8
B3N_R_7	10.322	3476.5	2708.9	1428.7
B3N_06	0.96	321.3	260.9	140
B3N_07	0.849	247.7	195.4	87.2
B3N_J_6_7	12.131	4030.7	3153.2	1649
B3N_R_5	12.131	3999.6	3124.2	1621
B3N_04	0.826	240.5	193.1	98.5
B3N_05	0.191	97.3	73.5	23.4
B3N_J_4_5	13.148	4278.9	3345	1725.6
B3N_R_3	13.148	4239.1	3333.5	1714.4
B3N_03	0.949	358.7	277.6	109.3
B3N_02	0.797	388.9	322.1	185.7
B3N_J_2_3	14.894	4941.4	3895.7	1973.8
B3N_R_1	14.894	4865.9	3829.9	1981
B3N_01	0.146	46.5	34.3	9.2
SPR	15.04	4897.8	3853.2	1986.4

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR COYOTE CREEK

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
SG_02	1.586	380.3	269.9	57.6
SG_DET_BAS	1.586	377.1	268	57.5
SG_R_01	1.586	373.6	265.2	57.2
SG_01	0.677	547	460.7	278.4
SW_02	0.654	140.8	100.4	21.6
SW_DET_BAS	0.654	84.1	59.2	12.4
SW_R_01	0.654	84.2	59.2	12.4
SW_01	0.545	234.1	184.8	82.8
S_SG_DB_J	3.462	910.3	724.7	361.4
SUM_SGAR_DET	3.462	903.6	721	363
S_SG_R_01	3.462	897.6	717.1	354.2
CW_19	0.606	172.9	127.6	33.8
CW_J_19	0.606	172.9	127.6	33.8
CW_R_18	0.606	171.4	126.4	33.7
CW_18	0.485	95.5	63.2	4.9
7th_St_DB_J	1.091	258.8	182.4	37.1
7th_St_DET_BAS	1.091	50.8	38.6	10.7
CW_R_17	1.091	50.8	38.5	10.7
CW_17	0.411	283.6	227	105.1
CW_J_17	1.502	284.2	227.4	105.1
CW_R_16	1.502	278.8	220.3	102.8
CW_16	0.556	274.7	218.3	97.4
S_SG_01A	0.394	176.9	136.9	52.2
CW_J_S_SG	5.914	1563.2	1256	600
CW_R_15	5.914	1551.4	1240.9	590.9
CW_15	0.591	375.6	302.8	148.5
CW_J_15	6.505	1830.3	1462.1	703.4
CW_R_14	6.505	1819.2	1450.3	689.2
CW_14	0.127	115.6	95.7	52.2
CW_J_14	6.632	1861.7	1486	712
CW_R_13	6.632	1838.5	1459.9	710.1
MB_02	0.485	173.6	137	63.2
MB_03	0.228	138.6	111.7	53.8
MB_01	0.179	67.7	51.3	18.5
MB_R_2	0.179	66.4	50.5	18.3
MB_J_2_3	0.892	335.7	263.5	119.6
KM_R_MB90	0.892	333.6	262	118.9
MB90_01	0.486	406.7	329.3	158.1
KM_J_MB90	1.378	582	451.7	239.1
KM_R_MB_Colom	1.378	575.1	448.1	237.1
MB_Colom	0.021	19	15	5.8
KM_J_Colom	1.399	587	457.2	240.9
MBGC_R_01	1.399	585.1	456.8	239.9
FS3_03	0.3	286.1	244.2	155.7
ROS_DB_J	0.3	358.6	304	190.3
ROSTRON_DET_BAS	0.3	209.4	167	94.2
FS3_05	0.183	139.6	116.5	67.1
FS3_04	0.022	5.9	3.3	2.1
LOWES_SPLIT	0.205	72.4	59.8	34.6
FS3_R_02	0.205	71.3	58.9	34
FS3_02	0.018	17.9	15.2	9.2
FS3_R_01	0.523	266.1	213.6	123.8
FS3_01	0.249	90.4	71.1	31.8
FS3_DB_J	0.772	355.9	281.3	154.5
FIRE3_DET_BAS	0.772	335.8	267.3	144.2
MBGC_01	0.403	434.1	375.9	251.6
ROS_R_01	2.574	1111.4	896	513.7
ROS_01	0.418	148.7	110.8	30.6
CW_13	0.163	33.3	23	3.3
ROS_J_CW	9.787	3073.6	2446.7	1197.3
CW_R_12_ROS	9.787	3050	2425.1	1184.1
CC_01	0.533	311.7	253.5	131.2
CC_02	0.437	154.7	105.5	15.4
CC_DET_BAS	0.437	71.8	56.3	12.8
CC_R_01	0.437	71.7	56.1	12.6
CC_J_01	0.97	329.9	260.7	131.3
MM_R_07	0.97	328.2	260.2	130.3
MALL_01B	0.161	184.3	158.7	103.9
Oakmont	0.082	97.5	84.4	56.6
MALL_DET_BAS_1	0.082	96.6	83.5	55.6
MALL_DET_BAS_2	0.243	280.7	244	158.6
MALL_DET_BAS_3	0.243	280.6	241.4	158.9
MALL_DET_BAS_4	0.243	272.9	233.9	154.2
MALL_DET_BAS_5	0.243	272.1	234.2	156.5
Greenbrier	0.045	29.1	23.9	12.6
MAll_DB_J_Green	0.288	301.2	258.1	169.1
MM_R_09	0.288	292.5	250.5	158.8
MM_07	0.162	52	36.9	7.6
MM_09	0.046	23.6	18.8	8.8
MM_J_MALL	1.466	629.5	516.2	283
MM_R_05	1.466	613.3	498.8	269.1
MM_08	0.397	246.7	199.8	104
MM_J_08	0.397	246.7	199.8	104
MM_R_06	0.397	244.2	196.4	102.6
MM_05	0.539	366.3	304.5	175.4
MM_06	0.497	74.1	51.6	10.6
MM_J_5_6	2.899	1116.5	882.3	445
MM_R_04	2.899	1104.3	872.9	447.3
MM_04	0.448	234.4	192.6	105.9
CVN_01	0.366	151.3	127.8	79.8
MM_J_CVN	3.713	1456.3	1166.9	621.4
MM_R_03	3.713	1451.3	1163.1	613.9
MM_03	0.318	142.8	114.2	54.6
MV_01	0.308	153.3	122.3	55.3
MM_J_MV	4.339	1677.5	1349.6	692.2
MM_R_01	4.339	1672.2	1341.4	688.4
MM_02	0.267	120.8	99	54.4
MM_01	0.23	76.7	59.7	24.2
MM_J_1_2	4.836	1852.6	1485.3	759.2
CW_12	0.427	120.7	90.6	27.3
CW_J_MM	15.05	5007.6	3978	1965.5
CW_R_10	15.05	4940.8	3934.6	1940.9
CW_11	0.288	69.5	51.7	17.7
CW_10	0.129	45	32.8	8.6
CW_J_10_11	15.467	5044	4004.1	1963.9
CW_R_08	15.467	5028	3987.2	1941.1
CW_09	1.155	255	188.6	51.9
CW_08	0.624	154.6	109.3	20
CW_J_8	17.246	5424.6	4272.4	2010.8
CW_R_07	17.246	5404.9	4271.9	2004.3
CW_06	0.217	66.4	45.1	4.1
CW_07	17.246	5.8	33	47
CW_J_6_7	0.217	2010.2	4322	5479.3
CW_R_04	0.216	1993.8	4291.8	5454
CW_05	17.679	8.7	67.4	100.5
CW_04	17.679	0	12.1	19.2
CW_J_2_3	0.569	2001.7	4362.4	5560.3
CW_R_02	0.072	1985.8	4315	5507.2
CW_03	18.320	9.8	69.8	100.3
CW_02	18.320	0.2	42.7	74.3
CW_J_01	0.415	1991.9	4373.3	5611.8
CW_R_01	0.247	1985.7	4373.2	5575.7
CW_01	18.982	0.1	37.3	57.6
SPR_Outlet	18.982	1985.7	4382.8	5591.1

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR SOLDIER CREEK

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
SubSC_FH7	1.688	617.4	479.6	195.3
SubSC_FH5	1.506	1415.9	1182.1	695
J_S_7	3.194	2005.2	1641.5	874.6
Reach FH3	3.194	2001.5	1636.3	872.6
SubSC_FH11	1.479	985.4	810.1	454
J_11	1.479	985.4	810.1	454
Reach FH9	1.479	963.8	807.5	452.9
SubSC_FH9	0.846	418.3	326.9	145.6
SubSC_FH3	0.151	69.4	52.7	17.8
J_9_10	5.769	3131.7	2508	1283
Reach FH1	5.769	3184.1	2555.9	1280.5
SubSC_FH10	1.043	564.3	458.4	250.9
SubSC_FH8	0.539	404.1	335.6	180.6
J_8_10	1.582	950	777.6	404
Reach FH6	1.582	946.4	776.2	403.1
SubSC_FH6	0.536	215.5	173.7	65.9
SubSC_FH4	0.399	159.9	115.3	29.8
J_4_6	2.519	1288.9	1039.1	546.6
Reach FH2	2.519	1284.1	1037.6	545.3
SubSC_FH2	0.866	316.2	237	71
SubSC_FH1	0.482	162.9	134.1	64.1
J_SC_BST	9.636	4888.6	3902.5	1923.3
Reach BST	9.636	4881.3	3897.9	1920
SubSC_BST	0.26864	204.9	170.4	99.5
J_SC_BST	9.90464	4965.7	3962.3	1958.8
Reach KAY	9.90464	4961.6	3971.4	1957.6
SubSC_KAY	0.395	441.4	374.6	225.9
J_SC_KAY	10.29964	5045.1	4017.3	1981.9
Reach SR90	10.29964	5030.2	4006.9	1977.7
SubSC_SR90	0.176	209.1	176.4	107.6
J_SC_SR90	10.46564	5044.7	4018.8	1982.1

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR MURRAY SPRINGS WASH

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
MSW_04	1.34	516.6	392.6	140.1
MSW_R_3	1.34	510.2	387.5	138.5
MSW_03	0.611	268.5	203.4	70
MSW_J_3	1.951	660.6	518.6	171.1
MSW_R_2	1.951	654.8	512	169.3
MSW_02	1.654	584.9	442.6	135.8
MSW_J_2	3.605	1220	910.3	283
MSW_R_1	3.605	1217	902.3	263
MSW_01	1.608	467.1	344.9	70
SPR	5.213	1581.5	1144.2	291.4

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR LEWIS SPRINGS WASH

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
LSW-07	2.226	968.5	783.5	343.8
LSW-06	0.046	20	15.3	2.5
LSW-R-7	0.046	19.4	20.8	2.5
LSW-J-7	2.274	965.5	783.5	343.2
LSW-R-6	2.274	963.4	782.9	342.9
LSW-05	0.774	266.3	215.6	70.7
LSW-J-5	3.048	1182.9	920.2	389.1
LSW-R-5	3.048	1168.9	913.8	383.9
LSW-04	1.546	294.6	218.8	67.8
LSW_05	0.14	45.7	33.9	10.3
LSW-R-4	0.14	44.8	33.1	11.4
LSW-03	1.486	292.6	218.7	67.1
LSW_J_4	0.253	101.4	74.7	20.7
LSW-J-4	1.771	338.5	250.3	74.8
LSW-R-3	1.771	337.1	249.3	74.1
LSW-J-3	4.972	1561.4	1189.6	453
LSW-03	0.159	64.6	45.6	5.2
LSW-J-2-3	5.263	1555.8	1189.8	453.4
LSW-R-1	5.261	1546.6	1180.5	450.5
LSW_01	2.203	830.6	680.6	315.9
SPR	7.486	2239.6	1750.2	648.7

HYDROLOGIC RESULTS FOR THE 24-HR STORM FOR GARDEN CANYON WASH

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
GCW_19	3.442	1580.6	1571	911.1
GCW_18	1.867	1117.7	941.8	556.8
GCW_J_18_19	5.309	2565	2502.1	1464.5
GCW_R_17	5.309	2984.6	2500.9	1467.7
GCW_16	2.603	1602	1564	924.1
GCW_17	0.189	205.6	174.2	105.3
GCW_J_17_18	8.061	4798.2	4023.2	2357.4
GCW_R_16	8.061	4769.9	4013.8	2343.4
GCW_15	1.319	774.3	798.6	351.8
GCW_14	2.4	1067.6	898.8	546.2
GCW_13	1.174	611.1	514	305.6
GCW_J_14_15	3.574	1635.3	1371.3	822.5
GCW_R_13	3.574	1633.3	1371.1	820.5
GCW_12	1.159	723.5	585.7	346.1
GCW_J_12_13	18.43	5856.8	7436.2	4281.2
GCW_R_11	18.43	8922.8	7428.4	4270.2
GCW_10	1.166	272.1	230.7	132.4
GCW_11	0.499	138.7	113.1	60.9
GCW_J_10_11	20.097	9210.2	7662.5	4361.6
GCW_R_8	20.097	9181.4	7640.1	4355.6
GCW_08	2.85	996.7	820.6	410.9
GCW_J_8	22.947	9469.6	7865.1	4468.6
GCW_R_6	22.947	9460.6	7860.5	4465.9
GCW_09	1.469	604.7	338.4	205.8
GCW_07	1.469	404.1	337.6	205.8
GCW_06	1.139	229.6	195	83.5
GCW_07	0.957	187.8	159.3	64.5
GCW_J_6_7	26.221	10115.7	8385.4	4762.5
GCW_R_5	26.221	10096.2	8376.4	4740.5
GCW_05	1.344	253.1	198.9	80.7
GCW_J_5	27.565	10292	8494.4	4797.3
GCW_R_3	27.565	10250.9	8480.7	4774.5
GCW_04	1.524	378.4	316.4	179.6
GCW_03	1.207	164.9	141.9	56.2
GCW_J_3_4	31.361	10603.3	8773.1	4890.1
GCW_R_1	31.361	10603.3	8755.6	4909.1
GCW_02	1.085	353.3	285.6	155.1
GCW_01	1.076	257.3	206.4	105.7
GCW_J_1	32.437	10716.6	8813.9	4913.9
SPR	32.437	10716.6	8813.9	4913.9

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR RAMSEY CANYON WASH

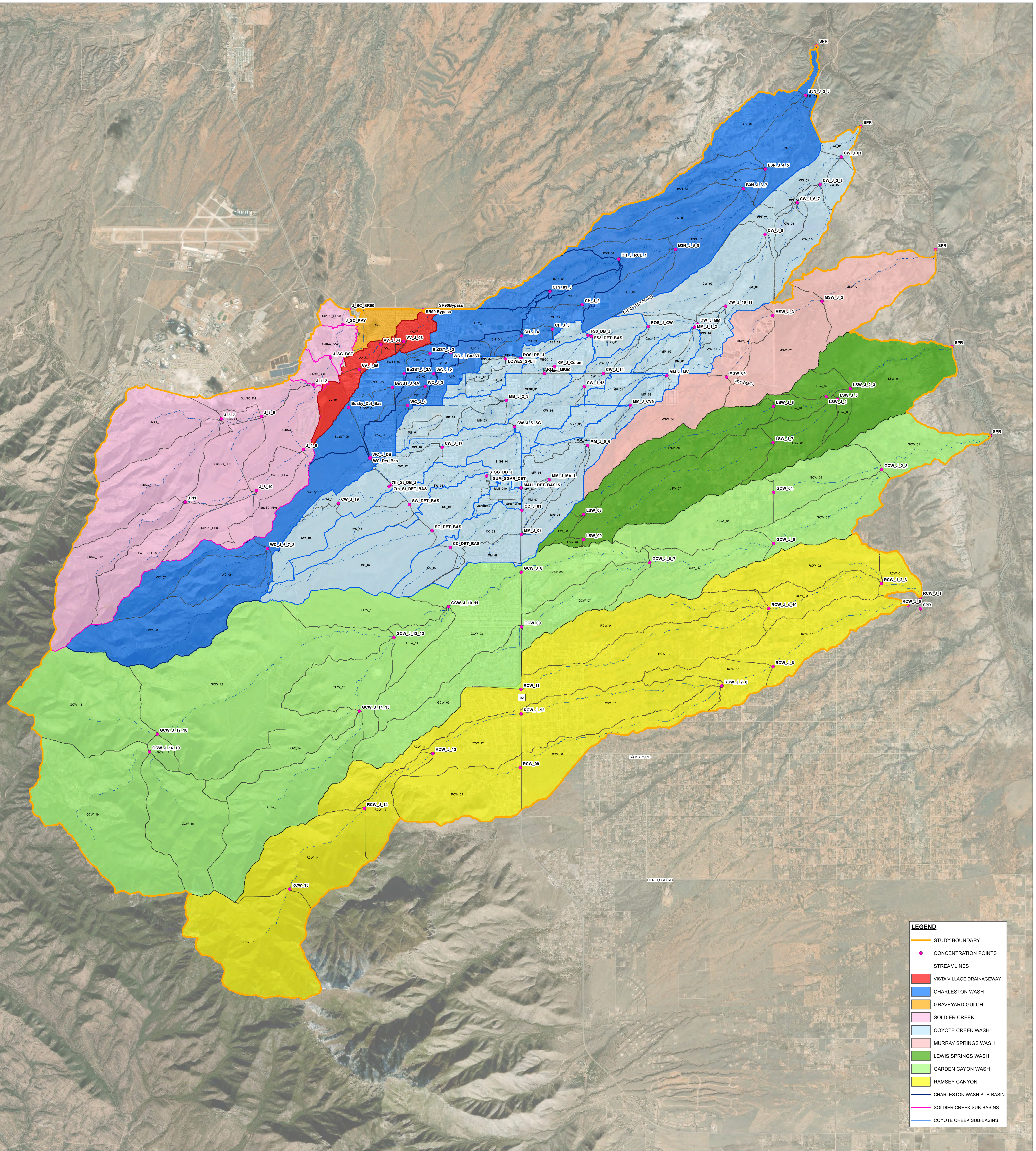
Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
RCW_15	2.894	3161.7	2736.7	1828.7
RCW_R_14	2.894	3165.1	2718.9	1817.7
RCW_14	1.961	1744.8	1456.1	855.8
RCW_J_14	4.855	4869.8	4176	2670.5
RCW_R_13	4.855	4852.6	4153.1	2650.8
RCW_13	0.586	357	286.3	144.6
RCW_J_13	5.241	5188.3	4401.3	2786.4
RCW_R_12	5.241	5170.7	4392.6	2738.4
RCW_12	1.05	294.3	199.7	94.4
RCW_J_12	6.291	5399.8	4543.7	2774.2
RCW_R_9	6.291	5391.4	4508.4	2740.4
RCW_09	0.751	250.4	206	103.6
RCW_08	1.584	431.7	281.1	87.7
RCW_07	1.584	430.1	278.3	87.6
RCW_R_8	1.514	150.9	93.1	15.4
RCW_J_8_9	10.618	5573.3	4670.4	2660.8
RCW_R_7	10.618	5561.6	4634.5	2620.9
RCW_06	0.733	374.6	309.6	171.4
RCW_J_6_7	11.553	5700.6	4955.1	2823.1

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR GRAVEYARD GULCH

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
GG	2.405	540.8	463.2	205.4
SR90 Bypass	2.405	540.8	463.2	205.4

HYDROLOGIC RESULTS FOR THE 6-HR STORM FOR VISTA VILLAGE DRAINAGEWAY

Hydrologic Element	Drainage Area (MI^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
VV_05	0.387	206.9	171.1	101.3
VV_J_05	0.387	206.9	171.1	101.3
VV_R_05	0.387	203.4	169.3	100.3
VV_04	0.774	367.6	264.8	158
VV_J_04	0.561	347.8	291.8	163.4
VV_R_04	0.561	345.6	290.6	162.3
VV_03	0.074	160.8	137.1	80.9
VV_J_03	0.635	395	330	244.6
VV_R_02	0.635	446.6	379.8	224.8
VV_02	0.019	44.6	38.5	23.6
VV_J_02	0.654	484.8	413.5	236.5
VV_R_01	0.654	473.5	397.6	236.5
VV_01	0.289	256.6	219.6	141.2
SR90Bypass	0.943	731.5	617.2	392



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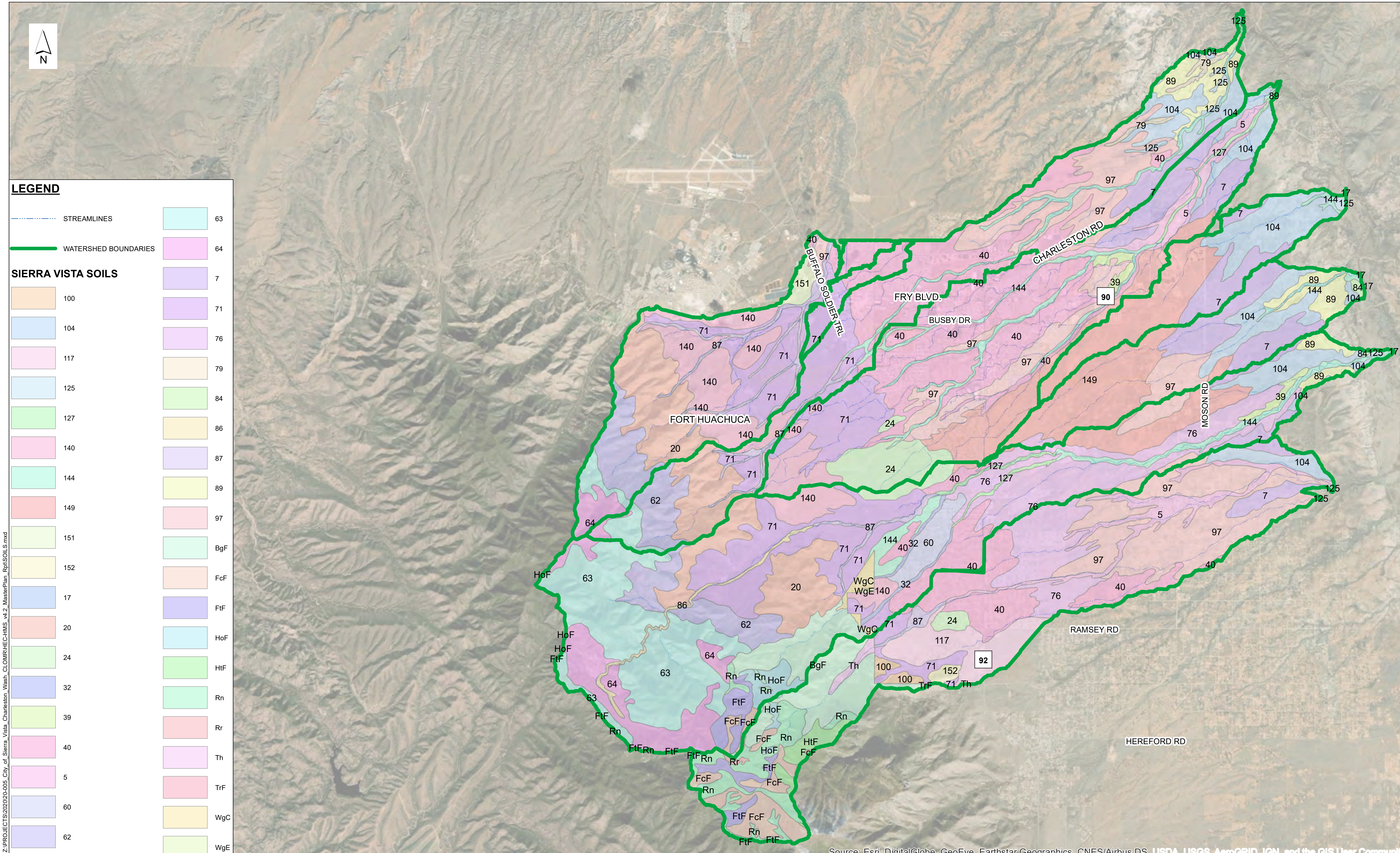
5,900 2,950 0 5,900 Feet

FIGURE 2: WATERSHED BOUNDARY MAP

CMG DRAINAGE ENGINEERING, INC.

PROJECT: 20-005 DATE: 01/18/2021



8,000 4,000 0 8,000 Feet

FIGURE 3:
SOILS CLASSIFICATION MAP

CMG DRAINAGE ENGINEERING, INC.
3555 N Mountain Ave., Tucson Arizona 85719
Phone: (520) 882-4244, Fax: (520) 888-1421

PROJECT NO.: 20-005 DATE: 01/15/20210

Document Path: Z:\PROJECTS\2020\20-005_City_of_Sierra_Vista_Charleston_Wash_CLOMR\HEC-HMS_v4.2_MasterPlan_Rpt\SOILS.mxd

Appendix B
Storm Hyetographs

2.4.2 6-hour Storm Distribution

The 6-hour storm distributions are used for flood studies and design of stormwater drainage facilities in Maricopa County of drainage areas less than 20 square miles, except for on-site stormwater storage facilities (see *Policies and Standards Manual*). These distributions would also be used for drainage areas larger than 20 square miles and smaller than 100 square miles by critically centering the storm over all or portions of the drainage area to estimate the peak flood discharges that could be realized on such watersheds due to the occurrence of a local storm over the watershed.

The Maricopa County 6-hour local storm distributions consist of five dimensionless storm patterns. Pattern No. 1 represents the rainfall intensities that can be expected in the "eye" of a local storm. These high, short-duration rainfall intensities would only occur over a relatively small area near the center of the storm cell. Pattern No. 1 is an offset, dimensionless form of the hypothetical distribution derived from rainfall statistics found in the NOAA Atlas for the Western United States, Arizona (Miller et al. 1973) and Arkell and Richards (1986) for the Phoenix Sky Harbor Airport location. Pattern Numbers 2 through 5 are modifications of the U.S. Army Corps of Engineers (1974) analysis of the Queen Creek storm of 19 August 1954. The dimensionless form of these 6-hour storm distributions are shown in and Table 2.4.

Inspection of the storm patterns indicates that the peak rainfall intensities are much greater for Pattern No. 1 than for the other pattern numbers, and that peak rainfall intensity decreases as the pattern number increases. The selection of the pattern number is based on the size of the drainage area under consideration, as shown in Figure 2.5. As illustrated by Figure 2.5, the maximum rainfall intensities, averaged over the entire drainage area, decrease as the size of the drainage area increases. This is to account for the spatial variability of local storm rainfall wherein the maximum rainfall intensities occur at the relatively small eye of the storm but that the average rainfall intensities over the storm area decrease as the storm area increases.

Table 2.4
6-HOUR DISTRIBUTIONS

Time, in hours	Percent of Rainfall Depth				
	Pattern 1	Pattern 2	Pattern 3	Pattern 4	Pattern 5
0.00	0.0	0.0	0.0	0.0	0.0
0.25	0.8	0.9	1.5	2.1	2.4
0.50	1.6	1.6	2.0	3.5	4.3
0.75	2.5	2.5	3.0	5.1	5.9
1.00	3.3	3.4	4.8	7.1	7.8
1.25	4.1	4.2	6.3	8.7	9.8
1.50	5.0	5.1	7.6	10.5	11.9
1.75	5.8	5.9	9.0	12.5	14.1
2.00	6.6	6.7	10.5	14.3	16.2
2.25	7.4	7.6	11.9	16.0	18.6
2.50	8.7	8.7	13.5	17.9	21.2
2.75	9.9	10.0	15.2	20.1	23.9
3.00	11.8	12.0	17.5	23.2	27.1
3.25	13.8	16.3	22.2	28.1	32.1
3.50	21.6	25.2	30.4	36.4	40.8
3.75	37.7	45.1	47.2	50.0	51.5
4.00	83.4	69.4	67.0	65.8	62.7
4.25	91.1	83.7	79.6	77.3	73.5
4.50	93.1	90.0	86.8	84.1	81.4
4.75	95.0	93.8	91.2	88.8	86.4
5.00	96.2	95.0	94.6	92.7	90.7
5.25	97.2	96.3	96.0	94.5	93.0
5.50	98.3	97.5	97.3	96.4	95.4
5.75	99.1	98.8	98.7	98.2	97.7
6.00	100.0	100.0	100.0	100.0	100.0

Figure 2.4
6-HOUR MASS CURVES FOR MARICOPA COUNTY

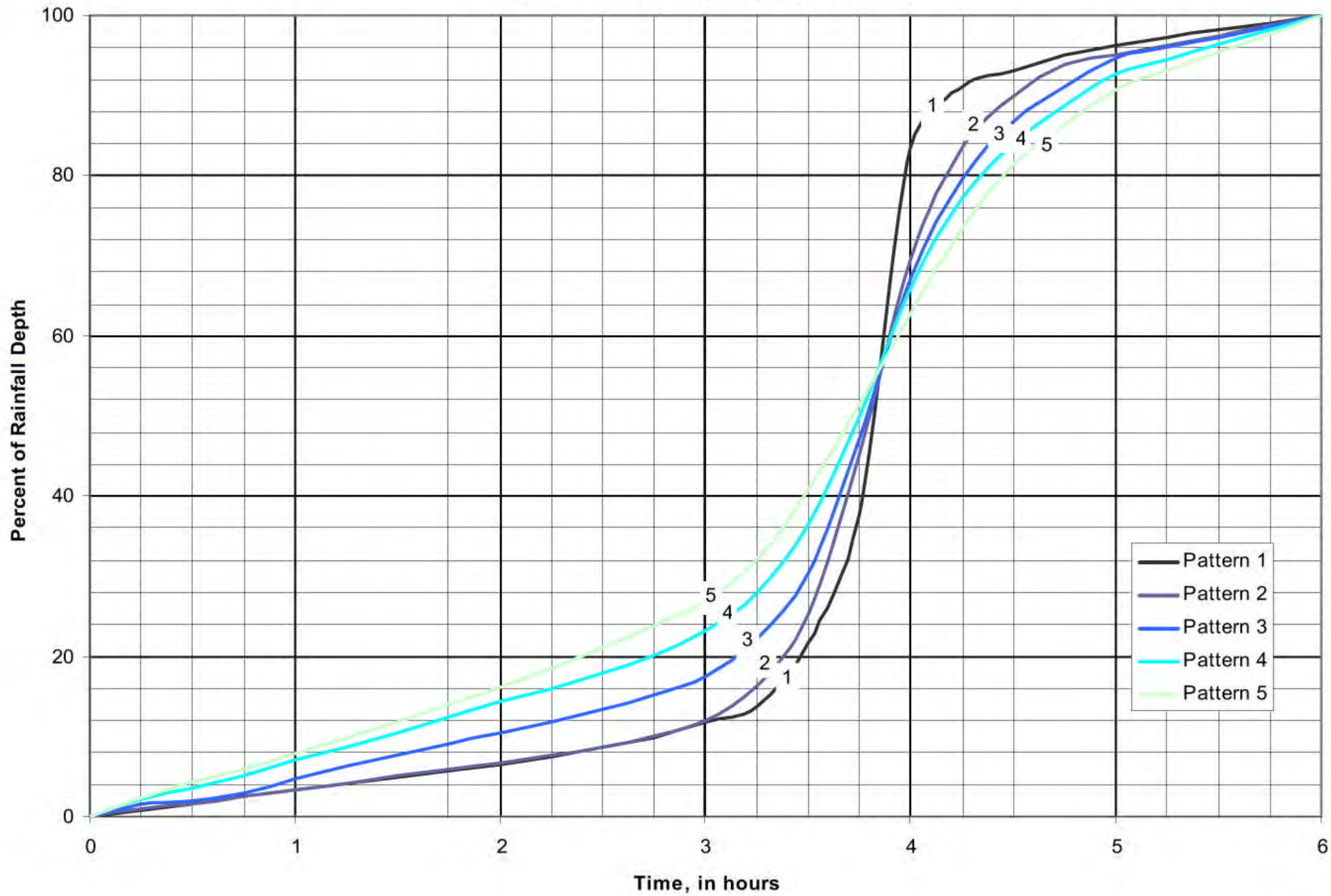
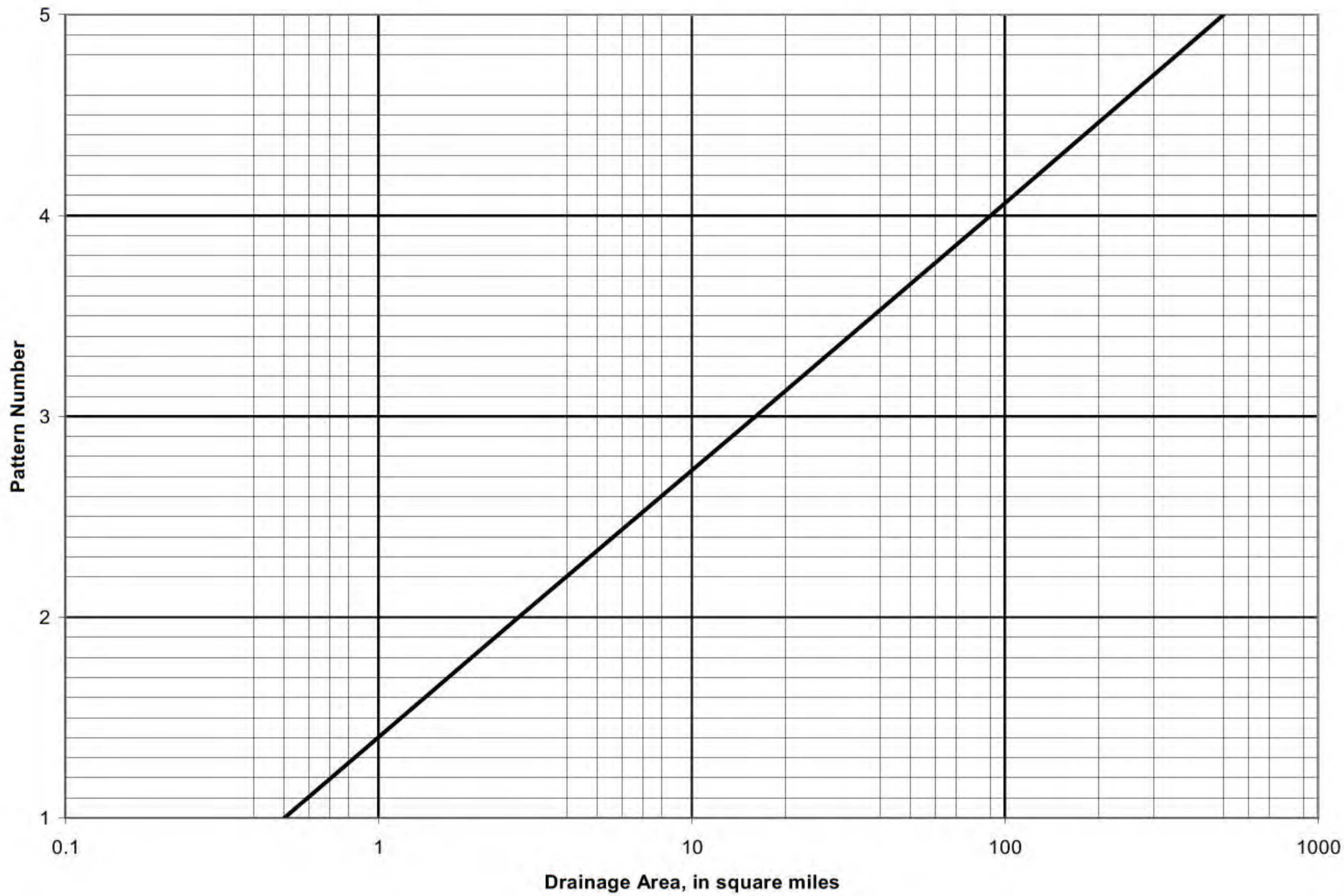


Figure 2.5
AREA VERSUS PATTERN NUMBER FOR MARICOPA COUNTY



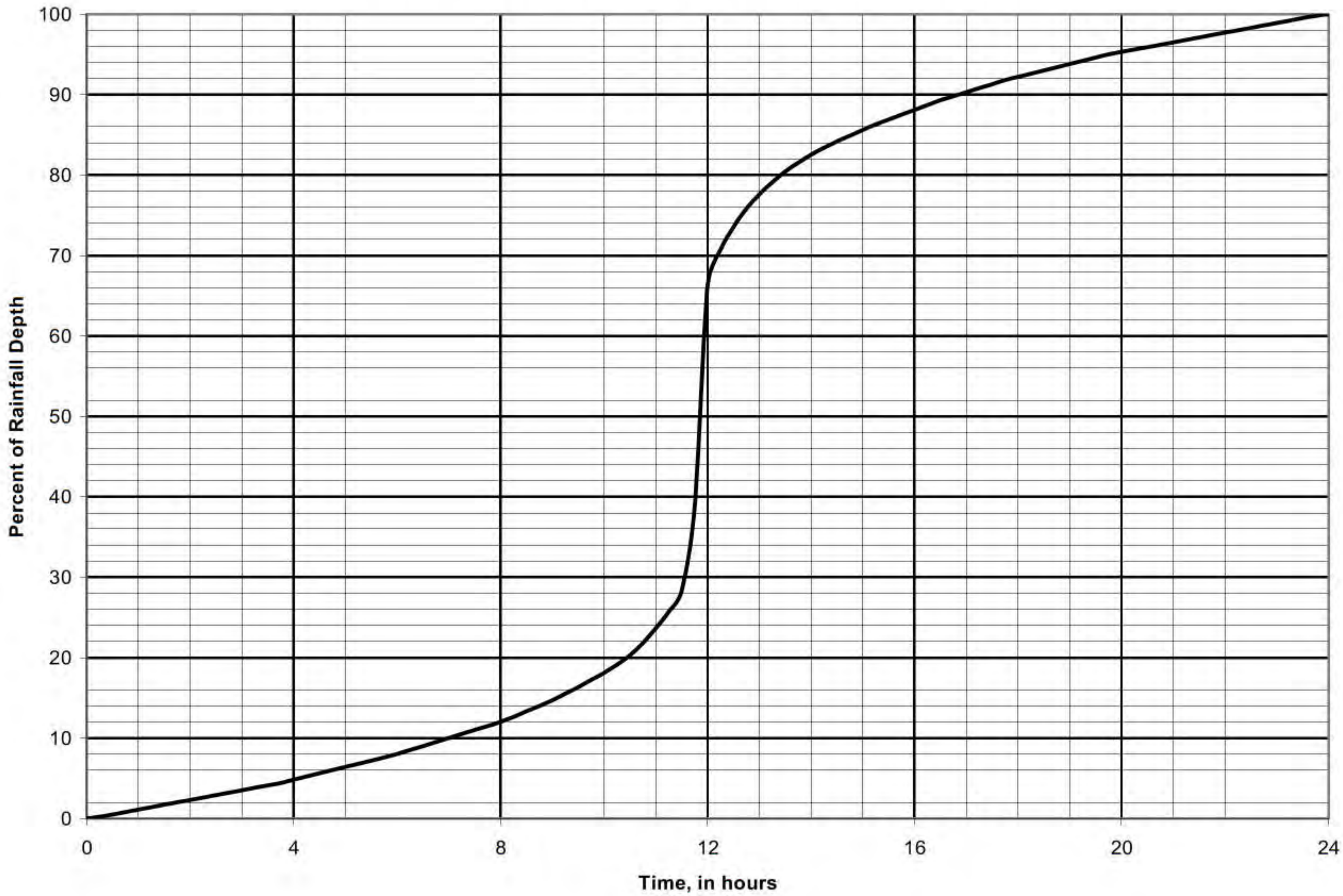
2.4.3 24-hour Storm Distribution

The 24-hour storm distribution that is to be used for flood studies and design of stormwater drainage facilities in Maricopa County is the SCS Type II distribution. This distribution is shown in Table 2.5 and Figure 2.6. The 24-hour storm distribution is used for flood studies of drainage area larger than 100 square miles (see *Policies and Standards Manual*). This distribution is also to be used in combination with the 6-hour storm distribution for drainage areas between 20 and 100 square miles to determine whether a local storm or a general storm will produce the greatest flood peak discharges or the maximum flood volumes.

Table 2.5
24-HOUR DISTRIBUTION

Time hours	Rainfall Depth %
0.00	0.0
0.25	0.2
0.50	0.5
0.75	0.8
1.00	1.1
1.25	1.4
1.50	1.7
1.75	2.0
2.00	2.3
2.25	2.6
2.50	2.9
2.75	3.2
3.00	3.5
3.25	3.8
3.50	4.1
3.75	4.4
4.00	4.8
4.25	5.2
4.50	5.6
4.75	6.0
5.00	6.4
5.25	6.8
5.50	7.2
5.75	7.6
6.00	8.0
6.25	8.5
6.50	9.0
6.75	9.5
7.00	10.0
7.25	10.5
7.50	11.0
7.75	11.5
8.00	12.0
8.25	12.6
8.50	13.3
8.75	14.0
9.00	14.7
9.25	15.5
9.50	16.3
9.75	17.2
10.00	18.1
10.25	19.1
10.50	20.3
10.75	21.8
11.00	23.6
11.25	25.7
11.50	28.3
11.75	38.7
12.00	66.3
12.25	70.7
12.50	73.5
12.75	75.8
13.00	77.6
13.25	79.1
13.50	80.4
13.75	81.5
14.00	82.5
14.25	83.4
14.50	84.2
14.75	84.9
15.00	85.6
15.25	86.3
15.50	86.9
15.75	87.5
16.00	88.1
16.25	88.7
16.50	89.3
16.75	89.8
17.00	90.3
17.25	90.8
17.50	91.3
17.75	91.8
18.00	92.2
18.25	92.6
18.50	93.0
18.75	93.4
19.00	93.8
19.25	94.2
19.50	94.6
19.75	95.0
20.00	95.3
20.25	95.6
20.50	95.9
20.75	96.2
21.00	96.5
21.25	96.8
21.50	97.1
21.75	97.4
22.00	97.7
22.25	98.0
22.50	98.3
22.75	98.6
23.00	98.9
23.25	99.2
23.50	99.5
23.75	99.8
24.00	100.0

Figure 2.6
24-HOUR MASS CURVE FOR MARICOPA COUNTY (SCS TYPE II)



Appendix C
Hydrologic Properties of Watershed Soils

Soil Parameters from ADOT Highway Draiange Design Manual, Volume 2, Hydrology, Second Edition, 2014, Appendix B							
		Initial Content (Volume Ratio)					
Mapunit Symbol	Mapunit Name	Wilting Point (Dry)	Field Capacity (Normal)	Saturated Content (Volume Ratio)	Suction (in)	Conductivity (in/hr)	Natural Impervious
5	Baboquivari-Combate complex, 0 to 3 percent slopes	0.11	0.23	0.42	7.81	0.31	0
7	Bella fine sandy loam, 1 to 10 percent slopes	0.09	0.22	0.41	13.10	0.31	0
8	Blakeney-Luckyhills complex, 3 to 15 percent slopes	0.07	0.16	0.39	5.02	0.59	0
17	Brookline-Fluvaquents-Riverwash complex, 0 to 3 percent slopes	0.04	0.12	0.42	3.70	1.24	0
18	Brunkcow-Chiricahua-Andrada complex, 3 to 20 percent slopes	0.13	0.22	0.42	5.53	0.28	0
19	Brunkcow-Chiricahua-Lampshire complex, 15 to 60 percent slopes	0.14	0.24	0.43	6.83	0.20	0
20	Budlamp-Woodcutter complex, 15 to 60 percent slopes	0.09	0.19	0.42	4.74	0.38	0
24	Carbine very gravelly loam, 3 to 30 percent slopes	0.09	0.23	0.44	9.82	0.38	0
32	Combate loamy sand, 0 to 5 percent slopes	0.04	0.09	0.42	0.34	1.55	0
39	Courtland-Diaspar complex, 0 to 3 percent slopes	0.07	0.16	0.42	3.75	0.73	0
40	Courtland-Sasabe-Diaspar complex, 1 to 8 percent slopes	0.09	0.18	0.41	4.75	0.50	0
58	Elgin-Stronghold complex, 3 to 20 percent slopes	0.20	0.31	0.46	8.06	0.08	0
60	Eloma-Caralampi-White House complex, 1 to 15 percent slopes	0.20	0.33	0.44	13.83	0.06	0
62	Far-Hogris association, 15 to 60 percent slopes	0.08	0.17	0.44	2.02	0.67	0
63	Far-Huachuca-Hogris association, 15 to 70 percent slopes	0.09	0.21	0.44	6.02	0.35	0
64	Far-Huachuca-Hogris association, moist, 15 to 70 percent slopes	0.09	0.21	0.44	6.02	0.35	0
71	Gardencan-Lanque complex, 0 to 5 percent slopes	0.09	0.17	0.42	3.81	0.61	0
76	Graveyard-Sierravista complex, 0 to 8 percent slopes	0.07	0.16	0.40	5.75	0.62	0
79	Guest silty clay loam, 0 to 3 percent slopes	0.19	0.35	0.45	22.19	0.06	0
84	Guest-Riveroad association, 0 to 1 percent slopes	0.17	0.28	0.44	8.30	0.11	0
86	Haplustolls-Fluvaquents association, mesic, 0 to 4 percent slopes	0.05	0.13	0.44	0.40	0.92	0
87	Haplustolls-Fluvaquents association, thermic, 0 to 4 percent slopes	0.07	0.15	0.44	2.80	0.80	0
89	Kaboom-Reeup complex, 0 to 45 percent slopes	0.19	0.33	0.43	17.63	0.05	0
97	Libby-Gulch complex, 0 to 10 percent slopes	0.18	0.29	0.44	10.28	0.09	0
98	Luckyhills loamy sand, 0 to 5 percent slopes	0.03	0.09	0.41	0.46	1.57	0
99	Luckyhills-McNeal complex, 3 to 15 percent slopes	0.13	0.23	0.41	8.60	0.18	0
100	Lutzcan-Yarbam complex, 25 to 50 percent slopes	0.15	0.29	0.44	12.77	0.10	0
101	Mabray-Chiricahua-Rock outcrop complex, 3 to 45 percent slopes	0.18	0.31	0.45	13.53	0.08	15
102	Mabray-Rock outcrop complex, 3 to 45 percent slopes	0.13	0.26	0.43	12.81	0.13	30
104	Major complex, 0 to 5 percent slopes	0.07	0.19	0.40	10.22	0.43	0
108	McAllister-Stronghold complex, 3 to 20 percent slopes	0.15	0.24	0.41	7.04	0.15	0
113	Nolam-Libby-Buntline complex, 1 to 10 percent slopes	0.13	0.24	0.40	9.86	0.14	0
117	Oversight-Lanque complex, 1 to 5 percent slopes	0.07	0.15	0.43	3.63	0.92	0
121	Pits	0.00	0.04	0.43	0.12	2.00	0
125	Riveroad and Ubik soils, 0 to 5 percent slopes	0.10	0.27	0.42	22.13	0.16	0
127	Riverwash-Bodecker complex, 0 to 3 percent slopes	0.03	0.11	0.40	1.94	1.29	0
129	Sasabe complex, 0 to 3 percent slopes	0.09	0.21	0.40	9.46	0.32	0
136	Sutherland-Mule complex, 3 to 15 percent slopes	0.08	0.17	0.42	3.96	0.47	0
140	Terrarossa complex, 0 to 45 percent slopes	0.10	0.20	0.40	6.87	0.30	0
141	Terrarossa-Blacktail-Pyeatt complex, 1 to 40 percent slopes	0.13	0.26	0.42	13.81	0.16	0
144	Ubik complex, 0 to 3 percent slopes	0.08	0.21	0.41	12.44	0.37	0
149	Vana fine sandy loam, 1 to 10 percent slopes	0.09	0.17	0.40	4.56	0.55	0
150	Vana-Moco complex, 1 to 5 percent slopes	0.17	0.28	0.42	11.65	0.11	0
151	White House complex, 1 to 30 percent slopes	0.20	0.33	0.45	13.79	0.07	0
152	Yarbam-Rock outcrop complex, 25 to 60 percent slopes	0.09	0.22	0.43	11.09	0.34	30

An	Anthony soils	0.09	0.17	0.40	4.56	0.57	0
Ao	Anthony soils, very gravelly variants	0.07	0.15	0.40	3.64	0.70	0
AtF	Atascosa very gravelly sandy loam, 30 to 50 percent slopes	0.18	0.28	0.41	9.37	0.08	0
BaE	Barkerville-Gaddes complex, 10 to 30 percent slopes	0.06	0.14	0.42	2.69	0.88	0
BgF	Barkerville-Gaddes association, steep	0.06	0.14	0.42	2.62	0.90	15
BhD	Bernardino-Hathaway association, rolling	0.16	0.28	0.44	9.97	0.10	0
BoB	Bonita clay, 0 to 5 percent slopes	0.27	0.40	0.48	15.73	0.02	0
Ca	Calciorthids-Haplargids association	0.22	0.35	0.45	13.00	0.04	0
CbD	Canelo gravelly sandy loam, 0 to 20 percent slopes	0.07	0.16	0.42	3.51	0.75	0
CdE	Canelo very gravelly sandy loam, 20 to 40 percent slopes	0.06	0.15	0.40	4.06	0.53	0
CeD	Canelo cobbly sandy loam, 0 to 20 percent slopes	0.05	0.13	0.42	2.18	0.98	0
CgE	Caralampi gravelly sandy loam, 10 to 40 percent slopes	0.17	0.27	0.41	9.60	0.08	0
CgF2	Caralampi gravelly sandy loam, 10 to 60 percent slopes, eroded	0.17	0.27	0.41	9.60	0.08	0
ClB	Caralampi gravelly loam, brown variant, 1 to 5 percent slopes	0.16	0.30	0.44	14.36	0.12	0
CmE	Casto very gravelly sandy loam, 10 to 40 percent slopes	0.07	0.16	0.42	3.51	0.52	0
Cn	Cave gravelly sandy loam	0.07	0.16	0.40	4.28	0.54	0
CoE	Chiricahua cobbly sandy loam, 10 to 45 percent slopes	0.05	0.13	0.42	2.18	0.98	0
CrD	Chiricahua-Lampshire association, rolling	0.06	0.16	0.42	3.40	0.64	0
CsC	Comoro sandy loam, 5 to 10 percent slopes	0.07	0.16	0.41	4.20	0.78	0
CtB	Comoro soils, 0 to 5 percent slopes	0.07	0.16	0.40	4.41	0.70	0
CuC	Continental soils, 1 to 10 percent slopes	0.12	0.22	0.41	6.55	0.24	0
CvE2	Continental-Rillino complex, 1 to 40 percent slopes, eroded	0.07	0.17	0.40	5.76	0.53	0
EbC	Eba very gravelly sandy loam, 0 to 10 percent slopes	0.30	0.42	0.49	14.63	0.01	0
FaD	Fanno-Luzena association, rolling	0.24	0.37	0.47	14.50	0.04	0
FcF	Fanno soils, acid variants, 20 to 50 percent slopes	0.15	0.29	0.43	14.59	0.10	0
FrE	Faraway-Rock outcrop complex, 10 to 30 percent slopes	0.08	0.17	0.44	2.50	0.63	25
FrF	Faraway-rock outcrop complex, 30 to 60 percent slopes	0.08	0.17	0.44	2.50	0.63	30
FtF	Faraway-Tortugas-Rock outcrop association, steep	0.10	0.21	0.43	4.93	0.36	25
GaE	Gaddes very gravelly sandy loam, 5 to 30 percent slopes	0.08	0.17	0.41	4.07	0.72	0
GbB	Grabe-Comoro complex, 0 to 5 percent slopes	0.07	0.16	0.41	4.20	0.78	0
Ge	Grabe soils	0.08	0.19	0.41	7.42	0.50	0
GhD	Graham soils, 5 to 20 percent slopes	0.28	0.42	0.49	14.99	0.02	0
GhF	Graham soils, 20 to 50 percent slopes	0.28	0.42	0.49	14.99	0.02	0
Gu	Guest soils	0.18	0.31	0.45	13.34	0.10	0
HaF	Hathaway gravelly sandy loam, 20 to 50 percent slopes	0.07	0.16	0.42	3.51	0.75	0
HhE2	Hathaway soils, 1 to 40 percent slopes, eroded	0.07	0.16	0.42	3.38	0.65	0
HoF	Hogris-Telephone association, steep	0.11	0.21	0.42	5.84	0.28	0
HtF	Hogris-Telephone-Rock outcrop association, very steep	0.11	0.21	0.42	5.98	0.27	20
KbC	Kimbrough soils, 2 to 10 percent slopes	0.07	0.20	0.40	12.41	0.35	0
LaE	Lampshire very gravelly sandy loam, 0 to 25 percent slopes	0.07	0.16	0.42	3.24	0.57	0
LaF	Lampshire very gravelly sandy loam, 25 to 50 percent slopes	0.07	0.16	0.42	3.24	0.57	0
LcF	Lampshire-Chiricahua association, steep	0.11	0.22	0.43	7.61	0.27	0
LgF	Lampshire-Graham-Rock outcrop association, steep	0.20	0.33	0.46	13.77	0.06	30
LuD	Luzena gravelly loam, deep variant, 5 to 20 percent slopes	0.21	0.34	0.44	16.50	0.04	0
McF	Mabray-Chiricahua-Rock outcrop association, steep	0.09	0.20	0.43	5.29	0.40	30
Mg	Martinez gravelly loam	0.21	0.35	0.46	15.17	0.06	0
Pm	Pima soils	0.17	0.34	0.46	23.19	0.08	0
Pn	Pima clay loam, sandy clay loam subsoil variant	0.21	0.35	0.46	15.17	0.06	0
PoC	Pinalino gravelly sandy loam, 0 to 10 percent slopes	0.06	0.14	0.39	4.60	0.66	0
RlE2	Rillino soils, 8 to 40 percent slopes, eroded	0.07	0.18	0.40	7.26	0.44	0
Rn	Rock outcrop-Lithic Haplustolls association	0.17	0.27	0.41	9.60	0.09	50
Rr	Rock outcrop	0.30	0.42	0.48	11.42	0.01	90
ScD	Schrap very shaly clay loam, 5 to 20 percent slopes	0.17	0.31	0.42	18.12	0.05	0
ShF	Schrap cobbly clay loam, 20 to 50 percent slopes	0.20	0.33	0.44	16.48	0.05	0
SnD	Signal soils, 1 to 20 percent slopes	0.25	0.39	0.47	14.98	0.03	0
SoB	Sonoita gravelly sandy loam, 1 to 8 percent slopes	0.06	0.15	0.40	4.06	0.70	0
SoD	Sonoita gravelly sandy loam, 8 to 20 percent slopes	0.06	0.15	0.40	4.06	0.70	0
Th	Torrifluvents and haplustolls	0.11	0.21	0.45	2.00	0.49	0
TrE	Tortugas-Rock outcrop complex, 5 to 25 percent slopes	0.13	0.26	0.43	12.81	0.16	25
TrF	Tortugas-Rock outcrop complex, 25 to 60 percent slopes	0.13	0.26	0.43	12.81	0.16	40
W	Water	0.30	0.42	0.48	11.42	0.01	100
WgC	White House gravelly loam, 0 to 10 percent slopes	0.16	0.30	0.44	14.36	0.12	0
WgE	White House gravelly loam, 10 to 35 percent slopes	0.16	0.30	0.44	14.36	0.12	0
WhC	White House cobbly sandy loam, 1 to 15 percent slopes	0.05	0.14	0.42	2.57	0.88	0
WnC	White House-Bonita complex, 0 to 10 percent slopes	0.19	0.33	0.45	14.78	0.07	0
WoE	White House-Caralampi complex, 10 to 35 percent slopes	0.12	0.22	0.41	5.81	0.24	0
WtF	White house-hathaway association, steep	0.12	0.24	0.43	8.18	0.25	0

Mapunit Symbol	Mapunit Name	Slope Gradient - Dominant Component	Slope Gradient - Weighted Average	Bedrock Depth - Minimum	Flooding Frequency - Dominant Condition	Flooding Frequency - Maximum	Available Water Storage 0-25 cm - Weighted Average	Available Water Storage 0-50 cm - Weighted Average	Available Water Storage 0-100 cm - Weighted Average	Available Water Storage 0-150 cm - Weighted Average	Drainage Class - Dominant Condition	Drainage Class - Wettest	Hydrologic Group - Dominant Conditions	Mapunit Key
1	Altar-Mallet complex, 0 to 8 percent slopes	4	4		None	None	2.84	5.03	9.41	11.89	Well drained	Well drained	B	55052
2	Anthony-Maricopa complex, 0 to 5 percent slopes	3	3		None	Rare	2.75	5.5	8.98	11.89	Well drained	Well drained	B	54947
3	Arizo family-Riverwash complex, 0 to 3 percent slopes	2	2		Occasional	Frequent	1.05	1.7	3.72	6.22	Excessively drained	Excessively drained	A	55027
4	Ashcreek-Stanford complex, 0 to 10 percent slopes	5	5		Rare	Rare	3.85	7.85	15.9	21.76	Well drained	Well drained	D	55063
5	Baboquivari-Combate complex, 0 to 3 percent slopes	2	1.6		None	Rare	3.25	6.69	12.11	16.22	Well drained	Well drained	B	54934
6	Banshee complex, 0 to 5 percent slopes	2.5	2.5		None	None	3.47	6.88	12.02	12.67	Well drained	Well drained	D	55054
7	Bella fine sandy loam, 1 to 10 percent slopes	6	6		None	None	3.63	4.93	4.93	4.93	Well drained	Well drained	D	55012
8	Blakeney-Luckyhills complex, 3 to 15 percent slopes	9	9		None	None	3	4.28	6.5	8.73	Well drained	Well drained	D	55016
9	Bodecker and Comoro soils, 0 to 5 percent slopes	2.5									Well drained		B	55067
10	Bodecker very gravelly sandy loam, 0 to 2 percent slopes	1	1		Occasional	Occasional	1.06	1.56	2.56	3.2	Excessively drained	Excessively drained	A	54949
11	Bodecker very gravelly sandy loam, saline-sodic, 0 to 2 percent slopes	1	1		Occasional	Occasional	1.5	2.44	3.44	6.06	Excessively drained	Excessively drained	A	54936
12	Bonita clay, 0 to 1 percent slopes	0.5	0.5		Occasional	Occasional	3.75	7.5	15	22.5	Well drained	Well drained	D	54986
13	Bonita-Forrest complex, 1 to 8 percent slopes	4.5	4.5		None	None	4.4	8.15	16.26	24.65	Well drained	Well drained	D	55065
14	Borderland sandy clay loam, 1 to 10 percent slopes	5.5	5.5		None	None	3.81	7.56	8.76	8.76	Well drained	Well drained	D	55048
15	Borderline fine sandy loam, 2 to 15 percent slopes	9	9		None	None	3.8	7.6	13.1	18.14	Well drained	Well drained	B	55023
16	Boss, Krentz, and Paramore soils, and rock outcrop, 15 to 55 percent slopes	35									Well drained		D	55032
17	Brookline-Fluvaquents-Riverwash complex, 0 to 3 percent slopes	2	2		Frequent	Frequent	2.25	5.09	7.71	9.33	Somewhat poorly drained	Somewhat poorly drained	D	55019
18	Brunkcow-Chiricahua-Andrada complex, 3 to 20 percent slopes	12	12	20	None	None	3.18	4.65	4.65	4.65	Well drained	Well drained	C	55030
19	Brunkcow-Chiricahua-Lampshire complex, 15 to 60 percent slopes	38	38	23	None	None	2.99	5.12	5.17	5.17	Well drained	Well drained	D	55040
20	Budlamp-Woodcutter complex, 15 to 60 percent slopes	38	38	20	None	None	1.47	1.62	1.62	1.62	Well drained	Well drained	D	55000
21	Buntline clay loam, 0 to 2 percent slopes	1	1		None	None	4.75	7.59	7.59	7.59	Well drained	Well drained	D	54952
22	Caralampi sandy loam, 1 to 5 percent slopes	3	3		None	None	2.09	4.84	8.48	10.98	Well drained	Well drained	B	55057
23	Caralampi very gravelly sandy loam, 1 to 3 percent slopes	2	2		None	None	1.95	3.95	9.98	17.48	Well drained	Well drained	B	54954
24	Carbine very gravelly loam, 3 to 30 percent slopes	17	17		None	None	2.49	4.41	4.41	4.41	Well drained	Well drained	D	54997
25	Carbine-Hathaway complex, 3 to 45 percent slopes	24	24		None	None	3.02	3.95	5.06	5.89	Well drained	Well drained	D	55064
26	Cazador-Lesliecreek complex, 0 to 10 percent slopes	5	5		Frequent	Frequent	4.12	8	15.86	24.02	Well drained	Well drained	D	55074
27	Cherrycow-Blacktail complex, 3 to 30 percent slopes	16.5	16.5	102	None	None	3.46	6.27	8.17	8.83	Moderately well drained	Moderately well drained	D	55070
28	Cherrycow-Magoffin-Rock outcrop complex, 15 to 65 percent slopes	40	40	0	None	None	3.13	5.3	7.68	7.68	Moderately well drained	Moderately well drained	D	55044
29	Chorro-Doubleadobe-Gothard complex, 0 to 5 percent slopes	2.5	2.5		Frequent	Frequent	2.86	5.81	12.6	18.07	Well drained	Moderately well drained	B	55062
30	Chorro-Guest complex, 0 to 3 percent slopes	1.5	1.5		Rare	Rare	3.96	7.33	13.15	16.31	Well drained	Well drained	C	55045
31	Cogswell clay, saline-sodic, 0 to 2 percent slopes	1	1		Frequent	Frequent	3.27	6.02	12.33	17.2	Well drained	Well drained	C	54955
32	Combate loamy sand, 0 to 5 percent slopes	3	3		None	None	1.5	3	7.88	13.21	Well drained	Well drained	B	54982

33	Comoro sandy loam, 0 to 2 percent slopes	1	1		None	None	2.75	5.52	11.52	17.19	Well drained	Well drained	B	54981
34	Comoro sandy loam, saline-sodic, 0 to 2 percent slopes	1	1		None	None	2.6	4.6	8.6	12.6	Well drained	Well drained	B	54956
35	Contention, Crystalgyp, Monzingo, and Redington soils, breaks, 5 to 60 percent slopes	32.5									Well drained		D	55024
36	Contention-Ugyp soils complex, 0 to 5 percent slopes	3	3		None	Occasional	4.04	7.67	14.2	20.39	Well drained	Well drained	D	55025
37	Courtland sandy loam, 0 to 2 percent slopes	1	1		None	None	2.75	5.5	13.94	22.44	Well drained	Well drained	B	54950
38	Courtland sandy loam, saline-sodic, 0 to 2 percent slopes	1	1		None	None	2.4	4.4	10.36	16.36	Well drained	Well drained	B	55077
39	Courtland-Diaspar complex, 0 to 3 percent slopes	1	1		None	None	2.75	5.5	13.94	21.47	Well drained	Well drained	B	54992
40	Courtland-Sasabe-Diaspar complex, 1 to 8 percent slopes	5	3.4		None	None	3.38	7.47	16.11	24.39	Well drained	Well drained	B	54991
41	Crowbar-Brunopeak association, 1 to 40 percent slopes	8	14.5		None	None	2.44	4.02	6.69	9.36	Well drained	Well drained	B	55056
42	Deloro-Leyte-Lampshire complex, 3 to 55 percent slopes	29	29	23	None	None	1.85	2.55	2.55	2.55	Well drained	Well drained	D	55058
43	Denab-Castledome complex, 3 to 45 percent slopes	24	24	18	None	None	2.96	2.96	2.96	2.96	Well drained	Well drained	D	55075
44	Denied access													55028
45	Diaspar sandy loam, 0 to 2 percent slopes	1	1		None	None	2.75	5.5	11	16.5	Well drained	Well drained	B	54967
46	Diaspar sandy loam, saline-sodic, 0 to 2 percent slopes	1	1		None	None	2.3	4.3	8.3	12.3	Well drained	Well drained	B	55078
47	Dona Ana-Mohave complex, 1 to 5 percent slopes	3	3		None	None	4.21	8.76	11.87	11.87	Well drained	Well drained	B	55026
48	Doubleadobe sandy loam, 1 to 3 percent slopes	2	2		Frequent	Frequent	3.32	6.32	12.32	18.32	Moderately well drained	Moderately well drained	C	55079
49	Durazo loamy sand, 0 to 2 percent slopes	1	1		None	None	1.5	3	6	7.32	Somewhat excessively drained	Somewhat excessively drained	A	54951
50	Durazo loamy sand, saline-sodic, 0 to 2 percent slopes	1	1		None	None	1.45	2.7	5.2	6.3	Somewhat excessively drained	Somewhat excessively drained	A	55080
51	Durazo, saline-Sodic-Gothard complex, 1 to 15 percent slopes	7	5.4		None	Occasional	1.98	3.88	7	10.52	Somewhat excessively drained	Well drained	A	55081
52	Durazo-Courtland complex, 1 to 5 percent slopes	3	3		None	None	2.61	5.76	12.31	18.98	Somewhat excessively drained	Well drained	A	55061
53	Durazo-McAllister complex, 1 to 15 percent slopes	7	7		None	None	2.46	5.1	10.4	16.92	Somewhat excessively drained	Well drained	A	54989
54	Elfrida clay loam, 0 to 2 percent slopes	1	1		Occasional	Occasional	4.75	9.5	19	27.18	Well drained	Well drained	B	54968
55	Elfrida clay loam, saline-sodic, 0 to 2 percent slopes	1	1		Occasional	Occasional	4.65	8.15	15.15	21.16	Well drained	Well drained	B	55082
56	Elgin-McAllister-Stronghold complex, 1 to 8 percent slopes	4.5	4.5		None	None	4	8.37	15.94	21.21	Well drained	Well drained	B	55066
57	Elgin-Outlaw complex, 1 to 10 percent slopes	5.5	5.5		None	None	3.69	7.44	14.32	18.43	Well drained	Well drained	C	55049
58	Elgin-Stronghold complex, 3 to 20 percent slopes	12	12		None	None	2.84	5.42	8.94	12.22	Well drained	Well drained	C	54938
59	Eloma sandy loam, 1 to 10 percent slopes	5.5	5.5		None	None	2.32	4.32	9.04	13.25	Well drained	Well drained	C	55046
60	Eloma-Caralampi-White House complex, 1 to 15 percent slopes	8	8		None	None	3.23	5.94	10.56	13.75	Well drained	Well drained	C	55039
61	Epitaph very cobbly clay loam, 3 to 15 percent slopes	9	9	97	None	None	3.63	7.38	10.23	10.23	Well drained	Well drained	D	54942
62	Far-Hogris association, 15 to 60 percent slopes	43	39.9	41	None	None	1.88	3.1	3.72	4.35	Well drained	Well drained	D	55004
63	Far-Huachuca-Hogris association, 15 to 70 percent slopes	48	42.7	21	None	None	1.74	2.46	3.01	3.57	Well drained	Well drained	D	55005
64	Far-Huachuca-Hogris association, moist, 15 to 70 percent slopes	48	42.7	21	None	None	1.74	2.42	2.87	3.31	Well drained	Well drained	D	55006
65	Forrest clay loam, 1 to 3 percent slopes	2	2		None	None	4.35	8.1	15.84	25.34	Well drained	Well drained	C	54970
66	Forrest clay loam, saline-sodic, 1 to 3 percent slopes	2	2		None	None	4.05	7.05	13.17	20.17	Well drained	Well drained	C	55094
67	Forrest sandy loam, 1 to 3 percent slopes	2	2		None	None	3	6.75	14.49	23.99	Well drained	Well drained	C	54971

68	Forrest silt loam, 0 to 1 percent slopes	0.5	0.5		None	None	4.2	7.95	15.69	25.19	Well drained	Well drained	C	54972
69	Forrest silt loam, saline-sodic, 1 to 3 percent slopes	1.5	1.5		None	None	4.05	7.05	13.17	20.17	Well drained	Well drained	C	55097
70	Forrest-Bonita complex, 0 to 3 percent slopes	2	2		None	Occasional	4.19	8.38	16.74	25.63	Well drained	Well drained	C	54937
71	Gardencan-Lanque complex, 0 to 5 percent slopes	3	3		None	None	3.26	6.84	13.22	18.67	Well drained	Well drained	B	55008
72	Glendale very fine sandy loam, 0 to 2 percent slopes	1	1		Rare	Rare	4.03	8.78	17.65	26.73	Well drained	Well drained	B	54948
73	Gothard loam, 1 to 3 percent slopes	2	2		Frequent	Frequent	3.13	6.22	10.48	14.48	Well drained	Well drained	B	55098
74	Gothard sandy loam, 0 to 2 percent slopes	1	1		Frequent	Frequent	2.53	5.58	9.58	13.58	Well drained	Well drained	B	55099
75	Graham-Lampshire complex, 8 to 60 percent slopes	24	28	13	None	None	2.86	2.86	2.86	2.86	Well drained	Well drained	D	55033
76	Graveyard-Sierravista complex, 0 to 8 percent slopes	4	4		None	None	2.56	4.27	7.38	10.35	Somewhat excessively drained	Well drained	B	55007
77	Grizzle coarse sandy loam, 3 to 8 percent slopes	6	6	81	None	None	4.18	8.18	13.14	13.14	Well drained	Well drained	D	54935
78	Guest silty clay loam, 0 to 1 percent slopes	0.5	0.5		Occasional	Occasional	4.75	8.94	16.29	16.29	Well drained	Well drained	C	54974
79	Guest silty clay loam, 0 to 3 percent slopes	2	2		Occasional	Occasional	4.61	8.86	17.36	25.86	Well drained	Well drained	C	54994
80	Guest silty clay loam, saline-sodic, 0 to 1 percent slopes	0.5	0.5		Occasional	Occasional	4.25	7.33	12.72	12.72	Well drained	Well drained	C	55100
81	Guest silty clay, 0 to 1 percent slopes	0.5	0.5		Occasional	Occasional	3.75	7.5	15	22.5	Well drained	Well drained	C	54978
82	Guest silty clay, saline-sodic, 0 to 1 percent slopes	0.5	0.5		Occasional	Occasional	3.69	6.69	12.69	18.69	Well drained	Well drained	C	55101
83	Guest-Cogswell complex, saline-sodic, 0 to 1 percent slopes	0.5	0.5		Frequent	Frequent	3.72	6.64	12.74	18.4	Well drained	Well drained	C	55102
84	Guest-Riveroad association, 0 to 1 percent slopes	0.5	0.5		Rare	Rare	3.44	7.23	15.38	21.75	Well drained	Well drained	C	55014
85	Hantz silt loam, saline-sodic, 0 to 3 percent slopes	2	2		Occasional	Occasional	4.25	7.66	13.16	18.66	Well drained	Well drained	C	54940
86	Haplustolls-Fluvaquents association, mesic, 0 to 4 percent slopes	2	2		Occasional	Frequent	1.16	2.8	4.97	6.24	Somewhat poorly drained	Somewhat poorly drained	B	55001
87	Haplustolls-Fluvaquents association, thermic, 0 to 4 percent slopes	2	2		Occasional	Frequent	2.06	2.56	3.56	4.56	Somewhat poorly drained	Somewhat poorly drained	A	55002
88	Hayhollow-Rafter-Riverwash complex, 0 to 5 percent slopes	2.5	2.5		Frequent	Frequent	1.87	3.74	6.5	10.18	Well drained	Well drained	B	55047
89	Kaboom-Reeup complex, 0 to 45 percent slopes	3	10.9	33	None	None	4.18	6.72	8.54	8.54	Well drained	Well drained	D	54980
90	Kahn complex, 0 to 3 percent slopes	2	2		Rare	Rare	3.6	7.84	17.34	26.84	Well drained	Well drained	B	55017
91	Kahn-Zapolote complex, 1 to 15 percent slopes	8	8		Rare	Rare	4.43	8.68	17.19	25.69	Well drained	Well drained	D	55043
92	Karro loam, 1 to 3 percent slopes	2	2		Rare	Rare	4	8.42	17.92	27.42	Well drained	Well drained	B	54976
93	Karro loam, saline-sodic, 1 to 3 percent slopes	2	2		Rare	Rare	3.75	7.36	14.36	21.36	Well drained	Well drained	B	55103
94	Keysto-Riverwash complex, 1 to 5 percent slopes	3	3		Rare	Frequent	1.69	2.69	4.21	5.21	Well drained	Well drained	B	55021
95	Kuykendall-Rock outcrop complex, 3 to 45 percent slopes	24	24	0	None	None	3.75	6.9	6.9	6.9	Well drained	Well drained	D	55073
96	Lanque-Stanford complex, 0 to 5 percent slopes	2.5	2.5		Rare	Rare	2.77	5.8	13.13	21.63	Well drained	Well drained	B	55068
97	Libby-Gulch complex, 0 to 10 percent slopes	5	5		None	None	3.46	7.36	14.01	20.38	Well drained	Well drained	C	54993
98	Luckyhills loamy sand, 0 to 5 percent slopes	3	3		None	None	2.52	5.52	11.76	20.76	Well drained	Well drained	B	54939
99	Luckyhills-McNeal complex, 3 to 15 percent slopes	9	9		None	None	3.15	6.7	13.34	19.22	Well drained	Well drained	B	55034
100	Lutzcan-Yarbam complex, 25 to 50 percent slopes	38	38	23	None	None	2.38	3.74	3.74	3.74	Well drained	Well drained	D	55041
101	Mabray-Chiricahua-Rock outcrop complex, 3 to 45 percent slopes	24	24	0	None	None	2.55	4	4.05	4.05	Well drained	Well drained	D	55035
102	Mabray-Rock outcrop complex, 3 to 45 percent slopes	24	24	0	None	None	1.35	1.56	1.56	1.56	Well drained	Well drained	D	55036

103	Magoffin-Rock outcrop-Cherrycow complex, 0 to 15 percent slopes	7.5	7.5	0	None	None	2.75	4.81	7.09	7.17	Well drained	Moderately well drained	D	55071
104	Major complex, 0 to 5 percent slopes	3	3		Occasional	Occasional	3.56	6.56	14.52	23.16	Well drained	Well drained	B	54996
105	Mallet-Hooks complex, 0 to 5 percent slopes	2.5	2.5		Rare	Rare	3.45	7.1	14.44	20.32	Well drained	Well drained	B	55051
106	Marsh				Occasional	Occasional					Poorly drained	Poorly drained		55042
107	McAllister loam, 1 to 3 percent slopes	2	2		None	None	4.15	8.9	18.4	27.9	Well drained	Well drained	B	54977
108	McAllister-Stronghold complex, 3 to 20 percent slopes	12	12		None	None	2.31	4.69	9.3	13.86	Well drained	Well drained	B	54941
109	McNeal gravelly sandy loam, 1 to 3 percent slopes	2	2		None	None	3.53	7.28	15.17	22.27	Well drained	Well drained	B	54990
110	McNeal gravelly sandy loam, saline-sodic, 1 to 3 percent slopes	2	2		None	None	2.61	5.11	10.5	15.55	Well drained	Well drained	B	55104
111	Monzingo-Ugyp complex, 1 to 20 percent slopes	13	9.9		None	Rare	3.02	7.02	12.48	15.3	Well drained	Well drained	B	55022
112	Naco-Ruins soils complex, 1 to 5 percent slopes	3	3		None	None	2.75	5.33	10.84	16.83	Well drained	Well drained	D	55020
113	Nolam-Libby-Buntline complex, 1 to 10 percent slopes	6	6		None	None	2.62	4.62	8.26	12	Well drained	Well drained	B	55018
114	Outlaw-Epitaph-Paramore complex, 0 to 15 percent slopes	7.5	7.5	56	None	None	3.87	7.62	11.72	15.66	Well drained	Well drained	D	55031
115	Oversight gravelly sandy loam, 1 to 35 percent slopes	17.5	17.5		None	None	1.91	3.94	8.18	11.06	Well drained	Well drained	B	55053
116	Oversight sandy loam, calcareous, 1 to 20 percent slopes	10.5	10.5		None	None	2.61	4.26	6.26	7.4	Well drained	Well drained	B	55076
117	Oversight-Lanque complex, 1 to 5 percent slopes	3	3		None	None	2.64	4.85	9.02	13.43	Well drained	Well drained	B	55009
118	Pedregosa very gravelly fine sandy loam, 3 to 15 percent slopes	9	9		None	None	1.44	1.44	1.44	1.44	Well drained	Well drained	D	55037
119	Pedregosa-Tombstone complex, 3 to 20 percent slopes	11.5	11.5		None	None	2.14	3.82	4.26	4.71	Well drained	Well drained	D	55060
120	Perilla-Durazo complex, 0 to 3 percent slopes	2	2		None	None	2.06	4.11	8.26	12.04	Somewhat excessively drained	Somewhat excessively drained	B	54975
121	Pits				None	None								54987
122	Pits-Dumps complex				None	None								54988
123	Quiburi-Fluvaquents-Riverwash complex, 0 to 5 percent slopes	3	2.1		Frequent	Frequent	2.6	5.19	10.47	15.46	Moderately well drained	Somewhat poorly drained	D	55029
124	Rafter-Lanque complex, 0 to 5 percent slopes	2.5	2.5		Rare	Rare	2.27	4.13	7.02	9.58	Well drained	Well drained	B	55069
125	Riveroad and Ubik soils, 0 to 5 percent slopes	3									Well drained		B	55015
126	Riverwash, 1 to 10 percent slopes				Frequent	Frequent								55055
127	Riverwash-Bodecker complex, 0 to 3 percent slopes		2		Frequent	Frequent	1.65	2.7	4.96	7.18		Excessively drained		54943
128	Rock outcrop-Magoffin complex, 5 to 60 percent slopes		32.5	0	None	None	2.59	3.77	3.77	3.77		Well drained		55072
129	Sasabe complex, 0 to 3 percent slopes	2	2		None	Frequent	3.93	7.9	15.51	21.39	Well drained	Well drained	C	54973
130	Sasabe gravelly sandy loam, 0 to 2 percent slopes	1	1		None	None	2.14	5.89	13.88	21.88	Well drained	Well drained	C	54969
131	Sasabe gravelly sandy loam, saline-sodic, 0 to 2 percent slopes	1	1		None	None					Well drained	Well drained	C	55105
132	Schiefflin very stony loamy sand, 3 to 15 percent slopes	9	9	46	None	None	1.65	2.91	2.91	2.91	Somewhat excessively drained	Somewhat excessively drained	D	54944
133	Stronghold gravelly fine sandy loam, 1 to 3 percent slopes	2	2		None	None	2.9	5.9	11.9	18.42	Well drained	Well drained	B	54979
134	Stronghold-Bernardino complex, 10 to 30 percent slopes	20	20		None	None	2.97	5.64	8.87	12.09	Well drained	Well drained	B	54945
135	Surge-Rock outcrop complex, 3 to 45 percent slopes	24	24	0	None	None	2.76	2.76	2.76	2.76	Well drained	Well drained	D	55050
136	Sutherland-Mule complex, 3 to 15 percent slopes	9	9		None	None	2.04	3.94	5.69	7.44	Well drained	Well drained	D	55038
137	Swisshelm sandy loam, 1 to 3 percent slopes	2	2		None	None	2.75	6.5	12.74	21.75	Well drained	Well drained	B	54983
138	Swisshelm sandy loam, saline-sodic, 1 to 3 percent slopes	2	2		None	None	2.75	5.5	9.68	16.26	Well drained	Well drained	B	55106
139	Tenneco fine sandy loam, 0 to 2 percent slopes	1	1		Rare	Rare	3.8	7.8	15.8	21.5	Well drained	Well drained	B	55011

140	Terrarossa complex, 0 to 45 percent slopes	23	23		None	None	2.71	6.46	13.28	19.69	Well drained	Well drained	C	54998
141	Terrarossa-Blacktail-Pyeatt complex, 1 to 40 percent slopes	21	21		None	None	3.4	7.04	13.39	19.47	Well drained	Well drained	C	54999
142	Tombstone very gravelly fine sandy loam, 8 to 15 percent slopes	12	12		None	None	2.17	3.66	6.16	8.66	Somewhat excessively drained	Somewhat excessively drained	B	54946
143	Turquoise-Nugget complex, 3 to 45 percent slopes	24	24	13	None	None	1.71	1.9	1.9	1.9	Well drained	Well drained	C	55059
144	Ubik complex, 0 to 3 percent slopes	2	2		Occasional	Occasional	4.01	8.01	15.18	21.18	Well drained	Well drained	B	54995
145	Ubik loam, 1 to 3 percent slopes	2	2		Occasional	Occasional	4	7.91	15.41	22.81	Well drained	Well drained	B	54985
146	Ubik loam, saline-sodic, 1 to 3 percent slopes	2	2		Occasional	Occasional	3.47	6.22	11.72	17.12	Well drained	Well drained	B	55107
147	Ubik sandy loam, 1 to 3 percent slopes	2	2		Occasional	Occasional	3.1	7.01	14.51	21.91	Well drained	Well drained	B	54984
148	Ubik sandy loam, saline-sodic, 1 to 3 percent slopes	2	2		Occasional	Occasional	2.57	5.32	10.82	16.22	Well drained	Well drained	B	55108
149	Vana fine sandy loam, 1 to 10 percent slopes	6	6		None	None	3.08	4.95	4.95	4.95	Well drained	Well drained	D	55013
150	Vana-Moco complex, 1 to 5 percent slopes	3	3		None	None	4.27	7	10.78	14.56	Well drained	Well drained	D	54966
151	White House complex, 1 to 30 percent slopes	16	16		None	None	3.58	7.79	15.35	24.59	Well drained	Well drained	C	55003
152	Yarbam-Rock outcrop complex, 25 to 60 percent slopes	42.5	42.5	0	None	None	2.07	2.07	2.07	2.07	Well drained	Well drained	D	55010

Appendix D
Peak Discharge Rates for Sub-Area Concentration Points

Charleston Wash

Hydrologic results for the 6-hour storm for Charleston Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
WC_08	1.925	1050.7	865.4	483.8
WC_07	0.801	471.9	387.4	214.9
WC_06	0.732	652.7	538.1	300.8
WC_J_6_7_8	3.458	2016.7	1656.2	918.6
WC_R_5	3.458	1995	1640.7	913.5
WC_05	0.892	236.6	172.6	45.9
WC_J_DB	4.35	2226	1801.9	956.3
WC_Det_Bas	4.35	1722.9	1227.5	280
WC_R_4	4.35	1710.1	1217	279.9
WC_04	0.351	329.8	271.1	147.5
WC_J_4	4.701	1724.8	1226.6	280.1
WC_R_3	4.701	1703.3	1218.2	279.9
WC_03	0.076	40	29.6	8.1
WC_J_3	4.777	1705.4	1219.7	279.9
WC_R_2	4.777	1705	1210.7	279.9
WC_02	0.153	238.5	109.1	53.4
WC_J_2	4.93	1709.3	1213.2	279.9
WC_R_1	4.93	1701.3	1205.9	279.8
Bu3ST_05	0.381	161.4	117.4	31.1
Busby_Det_Bas	0.381	0	0	0
Bu3ST_R_04ABDB	0.381	0	0	0
Bu3ST_04A	0.144	165.9	138.5	79.5
Bu3ST_04B	0.073	82	67.5	36
Bu3ST_04C	0.034	47.3	39.5	21.7
Bu3ST_04D	0.029	22.1	16.5	4.1
Bu3ST_R_04C	0.029	20.7	15.1	3.8
Bu3ST_J_04C	0.063	61.7	49.2	22.7
Bu3ST_R_04B	0.063	61.3	49	22.6
Bu3ST_J_04B	0.136	143.3	116.5	58.6
Bu3ST_R_04A	0.136	139.4	112.8	54.6
Bu3ST_J_4A	0.661	305.3	251.3	134.1
Bu3ST_R_03B	0.661	304.1	250.1	133.2
Bu3ST_03B	0.089	108.9	92.7	58.2
Bu3ST_J_03B	0.75	413	342.8	191.3
Bu3ST_R_3A	0.75	395.6	326.1	181.4
Bu3ST_03A	0.179	219	187.6	120.8
Bu3ST_J_3A	0.929	614.6	513.6	297.9
Bu3ST_R_2	0.929	596.5	500.7	295.5
Bu3ST_02	0.182	173	145.1	86.3
Bu3ST_J_2	1.111	769.5	645.8	381.9
Bu3ST_R_1	1.111	759.3	634.5	370.7
Bu3ST_01	0.17	175.2	146	83.1
Bu3ST_J_01	1.281	927.2	774.2	450.9
WC_J_Bu3ST	6.211	1740.4	1231.4	625.1
CH_R_WC_1	6.211	1715.2	1227.3	610.6
WC_01	0.119	133.6	112.6	67.6
CH_J_WC_01	6.33	1718.8	1229.1	660.2
CH_R_4B	6.33	1700.6	1219.7	658.8
CH_05	0.264	209	173.2	97.3
CH_R_4B_5	0.264	206.1	170.7	95.6
CH_04B	0.152	69.6	52.5	17
CH_J_4B	6.746	1801.2	1408.6	770.1
CH_R_4A	6.746	1792.6	1393	747.9
CH_04A	0.117	51.6	38.1	10.2
CH_J_4A	6.863	1844.3	1431.2	758.1
CH_R_3	6.863	1805.6	1389.8	753
CH_03	0.205	111.5	86.1	33.2
CH_J_3	7.068	1917.1	1475.9	784.2
CH_R_2	7.068	1878.2	1464.5	768.8

Charleston Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
CH_02	0.296	174.9	140.3	68.5
CH_J_2	7.364	2053.1	1604.9	837.3
CH_R_1	7.364	2038.3	1576.1	819.9
RCE_01	0.829	587.4	486.9	278.1
CTY_01	0.7	290.3	226.9	97
CTY_01_J	0.7	290.3	226.9	97
RCE_R_1	0.7	284.7	223	96.5
CH_01	0.359	180.2	145.8	76.3
CH_J_RCE_1	9.252	3016.1	2365.3	1224.8
B3N_R_8	9.252	2972.2	2308.2	1212.1
B3N_09	0.624	285.3	234.9	132.4
B3N_08	0.446	234.2	191.4	104
B3N_J_8_9	10.322	3486.9	2730.3	1445.8
B3N_R_7	10.322	3476.5	2708.9	1428.7
B3N_06	0.96	321.3	260.9	140
B3N_07	0.849	247.7	195.4	87.2
B3N_J_6_7	12.131	4030.7	3153.2	1649
B3N_R_5	12.131	3999.6	3124.2	1621
B3N_04	0.826	240.5	193.1	98.5
B3N_05	0.191	97.3	73.5	23.4
B3N_J_4_5	13.148	4278.9	3345	1725.6
B3N_R_3	13.148	4239.1	3333.5	1714.4
B3N_03	0.949	358.7	277.6	109.3
B3N_02	0.797	388.9	322.1	185.7
B3N_J_2_3	14.894	4941.4	3895.7	1973.8
B3N_R_1	14.894	4865.9	3829.9	1981
B3N_01	0.146	46.5	34.3	9.2
SPR	15.04	4897.8	3853.2	1986.4

Soldier Creek

Hydrologic results for the 6-hour for Soldier Creek

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
SubSC_FH7	1.688	617.4	479.6	195.3
SubSC_FH5	1.506	1415.9	1182.1	695
J_5_7	3.194	2009.2	1641.5	874.9
Reach FH3	3.194	2001.9	1636.2	873.4
SubSC_FH11	1.479	985.4	810.1	454
J_11	1.479	985.4	810.1	454
Reach FH9	1.479	983.8	807.5	452.8
SubSC_FH9	0.945	416.3	326.9	145.6
SubSC_FH3	0.151	69.4	52.7	17.6
J_3_9	5.769	3191.7	2559	1285
Reach FH1	5.769	3184.1	2555.7	1280.5
SubSC_FH10	1.043	564.3	458.4	250.9
SubSC_FH8	0.539	404.1	335.6	193.6
J_8_10	1.582	950	777.6	434
Reach FH4	1.582	948.4	776.2	433.1
SubSC_FH6	0.538	215.9	172.9	92.3
SubSC_FH4	0.399	156.9	115.3	28.8
J_4_6	2.519	1288.3	1039.1	546.6
Reach FH2	2.519	1284.1	1037.6	545.3
SubSC_FH2	0.866	316.2	237	71
SubSC_FH1	0.482	168.6	134.1	64.1
J_1_2	9.636	4889.6	3901.5	1923.3
Reach BST	9.636	4881.3	3897.5	1920
SubSC_BST	0.26864	204.9	170.4	99.5
J_SC_BST	9.90464	4985.7	3982.3	1958.9
Reach KAY	9.90464	4981.6	3971.4	1957.6
SubSC_KAY	0.385	441.4	374.8	235.9
J_SC_KAY	10.28964	5049.1	4017.8	1981.9
Reach SR90	10.28964	5030.2	4006.9	1977.2
SubSC_SR90	0.176	209.5	176.4	107.6
J_SC_SR90	10.46564	5044.7	4018.8	1982.1

Coyote Wash

Hydrologic results for the 6-hour storm for Coyote Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
SG_02	1.586	380.3	269.9	57.6
SG_DET_BAS	1.586	377.1	268	57.5
SG_R_01	1.586	373.6	265.2	57.2
SG_01	0.677	547	460.7	278.4
SW_02	0.654	140.8	100.4	21.6
SW_DET_BAS	0.654	84.1	59.2	12.4
SW_R_01	0.654	84.2	59.2	12.4
SW_01	0.545	234.1	184.8	82.8
S_SG_DB_J	3.462	910.3	724.7	361.4
SUM_SGAR_DET	3.462	903.6	721	363
S_SG_R_01	3.462	897.6	717.1	354.2
CW_19	0.606	172.9	127.6	33.8
CW_J_19	0.606	172.9	127.6	33.8
CW_R_18	0.606	171.4	126.4	33.7
CW_18	0.485	95.5	63.2	4.9
7th_St_DB_J	1.091	258.8	182.4	37.1
7th_St_ DET_BAS	1.091	50.8	38.6	10.7
CW_R_17	1.091	50.8	38.5	10.7
CW_17	0.411	283.6	227	105.1
CW_J_17	1.502	284.2	227.4	105.1
CW_R_16	1.502	278.8	220.3	102.8
CW_16	0.556	274.7	218.3	97.4
S_SG_01A	0.394	176.9	136.9	52.2
CW_J_S_SG	5.914	1563.2	1256	600
CW_R_15	5.914	1551.4	1240.9	590.9
CW_15	0.591	375.6	302.8	148.5
CW_J_15	6.505	1830.3	1462.1	703.4
CW_R_14	6.505	1819.2	1450.3	689.2
CW_14	0.127	115.6	95.7	52.2
CW_J_14	6.632	1861.7	1486	712
CW_R_13	6.632	1838.5	1459.9	710.1
MB_02	0.485	173.6	137	63.2
MB_03	0.228	138.6	111.7	53.8
MB_01	0.179	67.7	51.3	18.5
MB_R_2	0.179	66.4	50.5	18.3
MB_J_2_3	0.892	335.7	263.5	119.6
KM_R_MB90	0.892	333.6	262	118.9
MB90_01	0.486	406.7	329.3	158.1
KM_J_MB90	1.378	582	451.7	239.1
KM_R_MB_Colom	1.378	575.1	448.1	237.1
MB_Colom	0.021	19	15	5.8
KM_J_Colom	1.399	587	457.2	240.9
MBGC_R_01	1.399	585.1	456.8	239.9
FS3_03	0.3	286.1	244.2	155.7
ROS_DB_J	0.3	358.6	304	190.3
ROSTRON_DET_BAS	0.3	209.4	167	94.2
FS3_05	0.183	139.6	116.5	67.1
FS3_04	0.022	5.9	3.3	2.1
LOWES_SPLIT	0.205	72.4	59.8	34.6
FS3_R_02	0.205	71.3	58.9	34
FS3_02	0.018	17.9	15.2	9.2
FS3_R_01	0.523	266.1	213.6	123.8
FS3_01	0.249	90.4	71.1	31.8
FS3_DB_J	0.772	355.9	281.3	154.5
FIRE3_DET_BAS	0.772	335.8	267.3	144.2
MBGC_01	0.403	434.1	375.9	251.6
ROS_R_01	2.574	1111.4	896	513.7
ROS_01	0.418	148.7	110.8	30.6
CW_13	0.163	33.3	23	3.3

Coyote Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
ROS_J_CW	9.787	3073.6	2446.7	1197.3
CW_R_12_ROS	9.787	3050	2425.1	1184.1
CC_01	0.533	311.7	253.5	131.2
CC_02	0.437	154.7	105.5	15.4
CC_DET_BAS	0.437	71.8	56.3	12.8
CC_R_01	0.437	71.7	56.1	12.6
CC_J_01	0.97	329.9	260.7	131.3
MM_R_07	0.97	328.2	260.2	130.3
MALL_01B	0.161	184.3	158.7	103.9
Oakmont	0.082	97.5	84.4	56.6
MALL_DET_BAS_1	0.082	96.6	83.5	55.6
MALL_DET_BAS_2	0.243	280.7	244	158.6
MALL_DET_BAS_3	0.243	280.6	241.4	158.9
MALL_DET_BAS_4	0.243	272.9	233.9	154.2
MALL_DET_BAS_5	0.243	272.1	234.2	156.5
Greenbrier	0.045	29.1	23.9	12.6
MAll_DB_J_Green	0.288	301.2	258.1	169.1
MM_R_09	0.288	292.5	250.5	158.8
MM_07	0.162	52	36.9	7.6
MM_09	0.046	23.6	18.8	8.8
MM_J_MALL	1.466	629.5	516.2	283
MM_R_05	1.466	613.3	498.8	269.1
MM_08	0.397	246.7	199.8	104
MM_J_08	0.397	246.7	199.8	104
MM_R_06	0.397	244.2	196.4	102.6
MM_05	0.539	366.3	304.5	175.4
MM_06	0.497	74.1	51.6	10.6
MM_J_5_6	2.899	1116.5	882.3	445
MM_R_04	2.899	1104.3	872.9	447.3
MM_04	0.448	234.4	192.6	105.9
CVN_01	0.366	151.3	127.8	79.8
MM_J_CVN	3.713	1456.3	1166.9	621.4
MM_R_03	3.713	1451.3	1163.1	613.9
MM_03	0.318	142.8	114.2	54.6
MV_01	0.308	153.3	122.3	55.3
MM_J_MV	4.339	1677.5	1349.6	692.2
MM_R_01	4.339	1672.2	1341.4	688.4
MM_02	0.267	120.8	99	54.4
MM_01	0.23	76.7	59.7	24.2
MM_J_1_2	4.836	1852.6	1485.3	759.2
CW_12	0.427	120.7	90.6	27.3
CW_J_MM	15.05	5007.6	3978	1965.5
CW_R_10	15.05	4940.8	3934.6	1940.9
CW_11	0.288	69.5	51.7	17.7
CW_10	0.129	45	32.8	8.6
CW_J_10_11	15.467	5044	4004.1	1963.9
CW_R_08	15.467	5028	3987.2	1941.1
CW_09	1.155	255	188.6	51.9
CW_08	0.624	154.6	109.3	20
CW_J_8	17.246	5424.6	4272.4	2010.8
CW_R_07	17.246	5404.9	4271.9	2004.3
CW_06	0.217	66.4	45.1	4.1
CW_07	17.246	5.8	33	47
CW_J_6_7	0.217	2010.2	4322	5479.3
CW_R_04	0.216	1993.8	4291.8	5454
CW_05	17.679	8.7	67.4	100.5
CW_04	17.679	0	12.1	19.2
CW_J_2_3	0.569	2001.7	4362.4	5560.3
CW_R_02	0.072	1985.8	4315	5507.2
CW_03	18.320	9.8	69.8	100.3
CW_02	18.320	0.2	42.7	74.3
CW_J_01	0.415	1991.9	4373.3	5611.8
CW_R_01	0.247	1985.7	4373.2	5575.7

Coyote Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
CW_01	18.982	0.1	37.3	57.6
SPR_Outlet	18.982	1985.7	4382.8	5591.1

Murray Springs Wash

Hydrologic results for the 6-hour storm for Murray Springs Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
MSW_04	1.34	516.6	392.6	140.1
MSW_R_3	1.34	510.2	387.5	138.5
MSW_03	0.611	268.5	203.4	70
MSW_J_3	1.951	690.8	516.6	173.1
MSW_R_2	1.951	684.5	517.1	171.9
MSW_02	1.654	594.9	442.6	135.8
MSW_J_2	3.605	1227	910.9	263.9
MSW_R_1	3.605	1217	903.3	263
MSW_01	1.608	487.1	344.9	70
SPR	5.213	1581.5	1144.2	291.4

Lewis Springs Wash

Hydrologic results for the 6-hour storm for Lewis Springs Wash

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
LSW-07	2.226	966.5	763.5	343.8
LSW-09	0.048	22	15.3	3
LSW-R-7	0.048	20	20.8	2.5
LSW-J-7	2.274	966.5	763.5	343.8
LSW-R-5	2.274	953.4	755.5	340.2
LSW-05	0.774	286.3	215.6	70.7
LSW-J-5	3.048	1182.9	920.2	389.1
LSW-R-3	3.048	1168.9	913.8	383.9
LSW-06	1.348	294.6	218.8	67.8
LSW_08	0.14	45.7	33.9	10.5
LSW-R-6	0.14	44.8	33.7	11.4
LSW-J-6	1.488	294.6	218.8	67.8
LSW-R-4	1.488	292.6	216.7	67.1
LSW_04	0.283	101.4	74.2	20.7
LSW-J-4	1.771	338.5	250.2	74.8
LSW-R-2	1.771	337.1	249.3	74.1
LSW_02	0.305	117.8	85.8	22.9
LSW_03	0.159	64.6	45.6	9.2
LSW-J-2-3	5.283	1555.6	1189.9	453.4
LSW-R-1	5.283	1548.8	1180.5	450.5
LSW_01	2.203	830.6	660.8	315.9
SPR	7.486	2238.6	1700.2	648.7

Garden Canyon Wash

Hydrologic results for the 24-hour storm for Garden Canyon Wash

ydrologic Element	Drainage Area (Mi²)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
GCW_19	3.442	1880.8	1571	911.1
GCW_16	1.867	1117.7	941.6	554.8
GCW_J_16_19	5.309	2985	2502.1	1464.5
GCW_R_17	5.309	2984.5	2500.9	1457.7
GCW_18	2.603	1862	1564	924.1
GCW_17	0.169	205.6	174.2	103.2
GCW_J_17_18	8.081	4798.2	4023.2	2357.3
GCW_R_12	8.081	4789.9	4012.8	2345.4
GCW_12	5.456	2163.3	1795.6	997.5
GCW_15	2.4	1067.6	898.8	546.2
GCW_14	1.174	611.1	514	305.8
GCW_J_14_15	3.574	1635.5	1373.1	827.5
GCW_R_13	3.574	1633.7	1371.1	825.7
GCW_13	1.319	725.5	585.3	320.9
GCW_J_12_13	18.43	8936.6	7438.2	4281.2
GCW_R_11	18.43	8922.8	7428.8	4273.1
GCW_10	1.168	272.1	220.7	122.4
GCW_11	0.499	138.7	113.1	50.8
GCW_J_10_11	20.097	9210.2	7663.5	4381.6
GCW_R_8	20.097	9181.4	7640.1	4366.5
GCW_08	2.55	990.7	820.8	410.4
GCW_J_8	22.647	9469.6	7885.1	4486
GCW_R_6	22.647	9460.6	7864.5	4468.6
GCW_09	1.469	404.7	338.4	205.8
GCW_R_7	1.469	404.1	337.4	205.3
GCW_06	1.138	239.8	196	89.3
GCW_07	0.967	197.9	159.3	64.5
GCW_J_6_7	26.221	10115.7	8395.3	4752.5
GCW_R_5	26.221	10086.2	8376.4	4740.5
GCW_05	1.344	232.1	189.9	92.2
GCW_J_5	27.565	10232	8489.4	4787.2
GCW_R_3	27.565	10205.9	8463.2	4774.5
GCW_04	1.524	385.3	316.6	180.4
GCW_R_2	1.524	384.6	316.4	179.6
GCW_03	1.207	184.9	147.8	56.2
GCW_02	1.065	353.3	285.6	155.1
GCW_J_2_3	31.361	10603.3	8773.1	4909.1
GCW_R_1	31.361	10633.9	8755.3	4893.3
GCW_01	1.076	257.3	206.4	105.7
GCW_J_1	32.437	10716.6	8813.9	4913.9
SPR	32.437	10716.6	8813.9	4913.9

Ramsey Canyon

Hydrologic results for the 6-hour storm for Ramsey Canyon

ydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
RCW_15	2.694	3161.7	2736.7	1826.7
RCW_R_14	2.694	3165.1	2718.9	1817.7
RCW_14	1.961	1744.8	1456.1	855.9
RCW_J_14	4.655	4909.8	4175	2673.6
RCW_R_13	4.655	4852.6	4133.1	2650.8
RCW_13	0.586	357	286.3	144.1
RCW_J_13	5.241	5188.3	4401.3	2784.4
RCW_R_12	5.241	5170.3	4392.6	2738.4
RCW_12	1.05	284.5	186.7	44.4
RCW_J_12	6.291	5399.8	4543.1	2774.2
RCW_R_7	6.291	5391.4	4508.4	2794.7
RCW_07	1.73	396.6	292.7	88.6
RCW_09	1.584	431.7	281.1	87.7
RCW_R_8	1.584	430.1	279.3	87.8
RCW_08	1.214	150.9	93.1	15.4
RCW_J_7_8	10.819	5972.3	4876.4	2882.6
RCW_R_6	10.819	5951.6	4834.5	2819.3
RCW_06	0.733	374.6	308.6	171.4
RCW_J_6	11.552	6100.8	4965.1	2870.5
RCW_R_5	11.552	6090.7	4950.1	2827.1
RCW_05	1.441	560.8	462.5	259
RCW_J_5	12.993	6395.6	5198.3	2930.3
RCW_10	2.425	639.1	500.8	227.4
RCW_04	1.82	364.1	274.9	88.8
RCW_11	1.248	273.6	195.9	64.6
RCW_R_10	1.248	272.9	195.9	64.5
RCW_J_4_10	5.493	1141.9	831.5	319.9
RCW_R_3	5.493	1138.2	832.3	318.4
RCW_02	1.36	439.2	345.4	160.9
RCW_03	0.793	343.3	283.4	159.6
RCW_J_2_3	7.646	1472.7	1055	413.5
RCW_R_1	7.646	1471.1	1053.5	411.3
RCW_01	0.335	158.6	119.2	34.3
RCW_J_1	7.981	1475.3	1055.6	411.4
S	20.974	7815.2	6161.7	3320.9

Graveyard Gulch

Hydrologic results for the 6-hour storm for Graveyard Gulch

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
GG	0.405	540.8	463.2	305.4
SR90 Bypass	0.405	540.8	463.2	305.4

Vista Village Drainageway

Hydrologic results for the 6-hour Vista Village Drainageway

Hydrologic Element	Drainage Area (Mi^2)	100-YR Peak Discharge (cfs)	50-YR Peak Discharge (cfs)	10-YR Peak Discharge (cfs)
VV_05	0.387	206.9	171.1	101.3
VV_J_05	0.387	206.9	171.1	101.3
VV_R_04	0.387	203.4	168.6	100.3
VV_04	0.174	287.1	244.6	156
VV_J_04	0.561	347.6	291.8	183.4
VV_R_03	0.561	345.8	290.6	182.3
VV_03	0.074	160.6	137.1	86.9
VV_J_03	0.635	460.7	393	244.6
VV_R_02	0.635	446.6	379.6	234.8
VV_02	0.019	44.8	38.2	23.7
VV_J_02	0.654	484.9	412.5	256.5
VV_R_01	0.654	472.9	397.6	250.8
VV_01	0.289	258.6	219.6	141.2
SR90Bypass	0.943	731.5	617.2	392

Appendix E
HEC-HMS Models
(Only provided in electronic format)

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Surface Water Plan (SWP)
For
City of Sierra Vista

Part 2-Existing Conditions and

Preliminary Analysis of Flood and Erosion Control Alternatives

Prepared for:
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CMG Project #21-001
May 15, 2023

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APPENDICES

SECTION 1: INTRODUCTION

This document presents the results of a study conducted by CMG Drainage Engineering, Inc. (CMG) in cooperation with the City of Sierra Vista Public Works Department (COSV) to update the community's Surface Water Plan (SWP) for stormwater runoff control. Prior to this, the most recent plan was completed in 2006 by Stantec and that study was preceded by another SWP completed by Simons, Li and Associates, Inc. in 1988. A summary overview of those studies is provided in Section II of this report.

The COSV has grown significantly since the date of the most recent SWP update by Stantec in 2006 and there have been many changes to some of the watercourses, both natural and man-made, including channelization, roadway culvert modifications, enlargement of detention basins on the Ft. Huachuca Military Reservation, and channel entrenchment due to long-term degradation which has been and will continue to occur along many of the watercourses. In addition, new studies have also been conducted by the COSV to update watershed hydrology and floodplain mapping based upon LIDAR mapping of the community in 2009.

It is for these reasons that the COSV decided this to be an appropriate time to again update the SWP. The scope of work for this update of the SWP is given in Appendix A of this report.

The first part of this SWP presents the methodology and results of hydrologic modeling to establish stream flow discharge rates at several concentration points along watercourses within the COSV. As noted in Section 1.2 of Part 1 – Watershed Hydrology, hydrologic modeling for Part 1 of the SWP was conducted by City staff and documented herein by CMG Drainage Engineering, Inc. at the City's request. The second part of this SWP addresses existing conditions including flood hazard areas, geomorphic processes such as erosion, degradation and bank erosion. The third part identifies areas of concern and concept level damage mitigation alternatives. The second and third parts follow below.

1.1 Study Purpose

The purpose of the Surface Water Plan Update is to guide the planning, construction, location, and function of future surface water conveyance systems and erosion control measures within the City of Sierra Vista. In our high desert environment, surface water runoff is a valuable resource that needs to be protected and actively managed to provide the greatest benefit for the public and the ecosystems within the watershed. The presence of development within the watershed alters the dynamics of surface water flows and sediment transport.

Development increases the amount of impervious surface within a watershed. Impervious surfaces can cause more stormwater to runoff from a developed site if detention is not provided. Storm water leaving a developed site is generally concentrated as a result of the development and sediment free. Once runoff has been concentrated, its impact on the environment becomes more pronounced. Concentrated flows have higher velocities which increase erosion and sediment transport capacity.

Surface drainage related problems are present throughout the City of Sierra Vista like most other communities in southern Arizona. Many of the problems are local, usually associated with erosion along roadside, public pathways and culvert outlets, and are addressed by periodic maintenance at limited cost.

The focus of this SWP is on issues and concerns related to potential flooding or erosion damages to public infrastructure, building structures and long-term stability of the study watercourses, rather than local drainage issues requiring periodic maintenance.

Historical flooding of structures within the community has been limited and generally confined to areas such as Soldier Creek, Fry Town and the Sulger subdivision. This report identifies the cause of this flooding and assesses possible solutions for these locations. The effective FEMA floodplain mapping indicates that several structures are vulnerable to flooding along other watercourses during the 100-year storm, but they generally tend to be isolated and associated with inadequate structures such as roadway culverts and undersized channels.

This study closely observes the historical and future impacts of channel erosion on the stability of existing drainage structures and public utilities within the wash environments.

Accompanying this report are electronic files that inventory available information such and culvert locations and sizes, grade control structure locations, stream profiles from HEC-RAS, HEC-HMS hydrology information such as watershed boundaries, concentration point locations and peak discharge rates for the 10-, 50-, and 100-year return period storms.

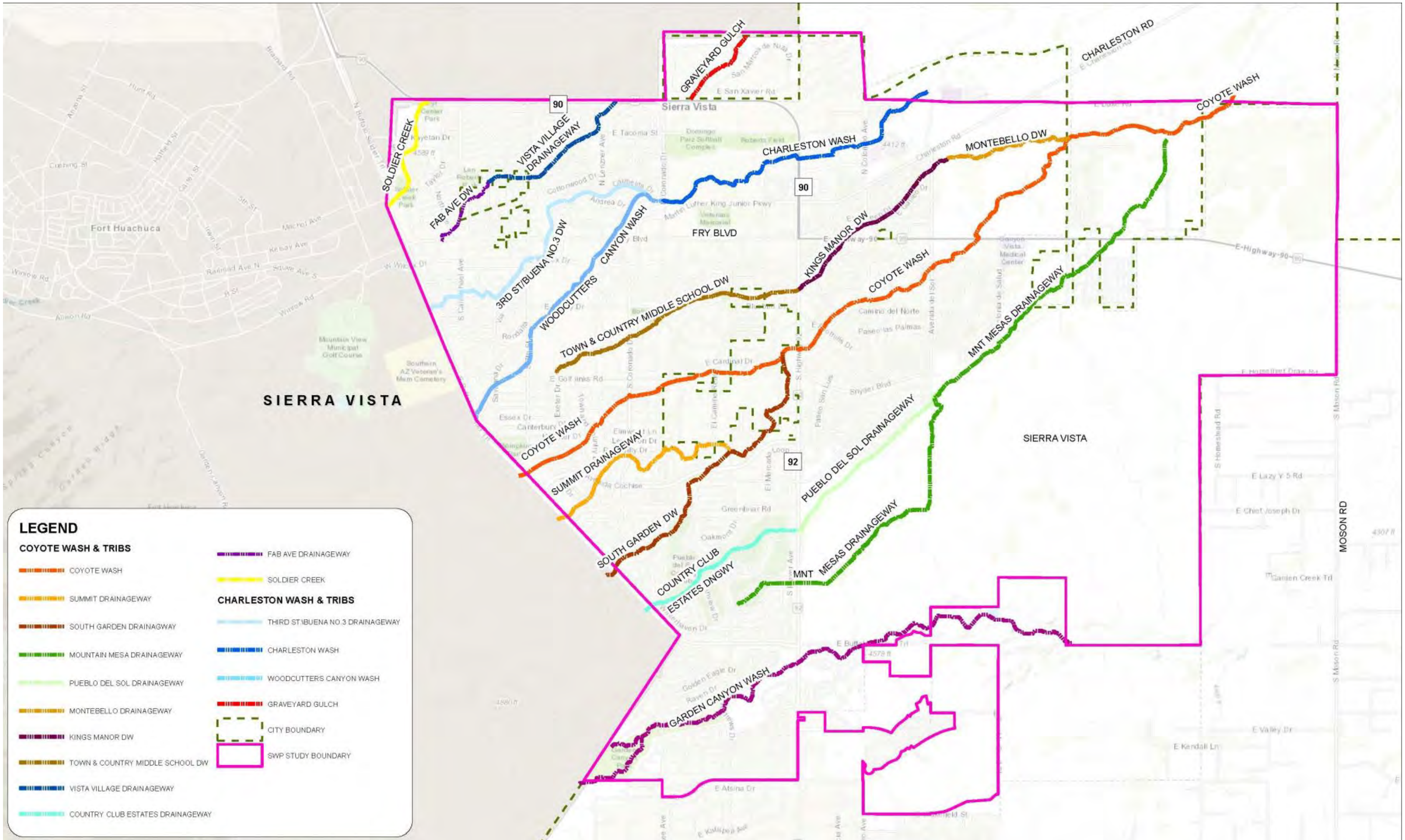
1.2 Description of Study Area

The study area for this SWP includes the following named watercourses within the corporate limits of the City of Sierra Vista, and unincorporated islands within the City limits.

- Garden Canyon Wash
- Charleston Wash
- Woodcutters Canyon Wash
- 3rd Street/Buena #3 Drainageway
- Montebello Drainageway
- Kings Manor Drainageway
- Town & Country Drainageway
- Coyote Wash
- Summit Drainageway
- South Garden Wash
- Mountain Mesa Drainageway
- Pueblo del Sol Drainageway
- Soldier Creek
- Murray Springs Wash
- Graveyard Gulch
- Vista Village Drainageway
- Fab Avenue Drainageway

The western (upstream) study limit coincides with Fort Huachuca which can also be generally described as Buffalo Soldier Trail, while the downstream study limit generally coincides with Moson Road. Unincorporated islands within the COSV corporate limits are also addressed. A map showing the study area is provided in Figure 1. Figure 1 also shows the study wash names and study limits.

Figure 1: Location Map



SECTION 2: Review of Previous Surface Water Plan Reports

It should be noted that the hydrologic results given in the previous study reports described below have been superseded by more recent studies completed by the City and FEMA. Several detention basins located on the Ft. Huachuca Military Reservation have been constructed or enlarged since the date of the reports that precede 2006. Also, some of the recommended flood control solutions identified in the previous SWP reports have since been constructed. Excerpts from the previous Surface Water Plan (SWP) reports are provided in Appendix B.

<u>The City of Sierra Vista Surface Water Plan Summary Report – Simons Li & Associates, Inc. January 27, 1988, p</u>urpose was to present a regional approach to the future management of surface water runoff within the study area, while at the same time addressing existing conditions and problems. The intent of the plan was also to provide means for protecting the public against the hazards of flooding and erosion while recognizing that storm water runoff as well as the natural systems are public amenities. The first phase of the study was an investigation of existing hydrologic and hydraulic conditions. Peak discharge rates for the major watercourses were calculated using the Pima County Method.The second phase was to develop and evaluate alternative surface water management schemes using information developed as a part of Phase 1. The third phase was to identify the preferred alternatives. More details regarding the results can be found in the study reports.

Most of the identified problem areas were related to undersized roadway culverts which could prevent all-weather access during floods or result in flow breaking out of the channel (listed in Table 2.3.2.1 of the SLA report). Undersized channels were Identified as being along portions of Soldier Creek, Charleston Wash at Coronado Drive, Woodcutters Canyon Wash upstream of Savannah Drive, and other locations. Bank erosion and channel bottom degradation was also identified as a concern and standardized erosion setbacks, based on contributing watershed area, were also recommended.

The third phase of the study developed surface water plan alternatives and a decision making model capable of comparing significant factors for ranking alternatives. The process involved qualitative ranking indices of the alternatives under evaluation. The indices were grouped together based on technical, economic, social and environmental factors. Weighting factors were also used to reflect relative importance. Appendix B includes a copy of tables from this report that summarize high potential flood hazard areas and alternatives for flood mitigation. Some of the recommendations, particularly construction of detention basins, have been implemented since the study date (1988).

<u>The Technical Data Support Notebook – Flood Study for the City of Sierra Vista AZ- Hydro-Sciences Southwest, Inc June 1998 p</u>urpose was to compile and augment previous efforts to delineate floodplains in the City and compile into a set of work maps. The report summarizes the results of the review of available hydrologic information. Hydrologic analyses results included information presented in the Surface Water Plan prepared by Simons Li and Associates, Inc., augmented by HEC-1 routing to simulate flood peaks affected by stormwater detention facilities. The study stated that although some of the major watercourses include

improved channels, the level of protection they afford is generally inadequate to convey storm water runoff during severe flooding conditions.

HEC-2 hydraulic modeling was conducted to develop to provide 100-year floodplain and floodway delineations for the major watercourses, with the intended purpose being a floodplain management tool for the community. Cross-sections were based on 1985 topography and the interval averaged 200-feet.

The study also identified possible locations for and evaluated several detention basin alternatives. Preliminary detention routing computations (using HEC-1) were also conducted to determine potential peak flow reductions for the downstream channel reaches.

<u>Cochise County Flood Control / Urban Runoff Recharge Plan- Appendix A – Hydrology and Flood Control – Stantec Consulting Inc. April 2006:</u> This report presents the procedures and results of studies conducted jointly by Stantec Consulting, Inc. and Geosystems Analysts, Inc. to evaluate the potential flood control and incidental recharge benefits associated with construction of regional detention basins. The study area included the City of Sierra Vista and the Ft. Huachuca Military Reservation. Hydrologic modeling was performed using the U.S Army Corps of Engineers HEC-HMS model although the report states that the results are considered appropriate for planning purposes but not intended to replace the requisite more detailed study efforts that should be performed for design purposes.

Detailed detention routings were conducted for 38 flood control facilities; 12 of which were in Cochise County, 16 were in the City and 9 were within Ft. Huachuca. The results of the detention modeling found that given the installation of all facilities, the targeted flood control objectives can generally be met. Estimated opinion of probable construction cost for each basin facility were calculated.

SECTION 3: Data Collection

Documents collected and reviewed as a part of this study include:

- City of Sierra Vista Surface Water Plan Summary Report – Simons Li & Associates, Inc. January 27, 1988
- Technical Data Support Notebook – Flood Study for the City of Sierra Vista AZ- Hydro-Sciences Southwest, Inc June 1998
- Cochise County Flood Control / Urban Runoff Recharge Plan- Appendix A – Hydrology and Flood Control – Stantec Consulting Inc. April 2006
- Light Detection and Ranging (LIDAR) data provided by Fort Huachuca in 2009
- Roadway/Culvert Plans for SR-90 and SR-92 provided by ADOT (dates vary)
- Field surveys and photographs of roadway culverts and grade control structures
- COSV GIS data base related to streets and roads, parcel boundaries and sewer systems

SECTION 4: Description of Primary Areas Vulnerable to Flooding and Erosion

The following locations are areas that have been determined to be the primary locations vulnerable to flooding and erosion as identified by CMG field investigations, COVS staff, and recent studies. Several other locations of concern are also present; discussions and recommended mitigation measures for these locations are described in Section 9 of this report.

Please note that COSV maintenance personnel have also identified several locations where erosion has been occurring near roadway crossings, recreational facilities such as bike paths, and urban drainageways that are not specifically discussed in this report since they are relatively minor and not included in the scope of work.

4.1 Sulger Subdivision Area Flooding

The Sulger subdivision is located south of Busby Drive between Carmichael Avenue and Judd Street; Timothy Lane is the south boundary of the subdivision. The subdivision was developed by lot splits in the 1960's and consists primarily of mobile homes although several ground set homes are present too. The offsite peak flows affecting the subdivision are not large (varying between 62 cfs and 143 cfs) but periodic flooding of portions of the subdivision occurs because there are no drainage facilities for conveyance of storm water. Storm water flows are conveyed along the subdivision streets which are not curbed and through residential yards at depths generally less than one foot.

A detention basin was recently constructed south of Timothy Avenue but there have not yet been a sufficient number of storms to assess the degree to which the basin has functioned to reduce flows through the subdivision. The above referenced peak discharge rates do not consider peak flow reductions that the recently constructed detention basin may provide. It is known though, that urban runoff from areas downstream of the basin remain so the detention basin does not function to capture all storm water entering the Sulger subdivision.

A few photographs of the subdivision and detention basin are provided below along with the FEMA flood hazard map.

Figure 4.1.1 - FEMA Floodplain Map for Sulger Subdivision

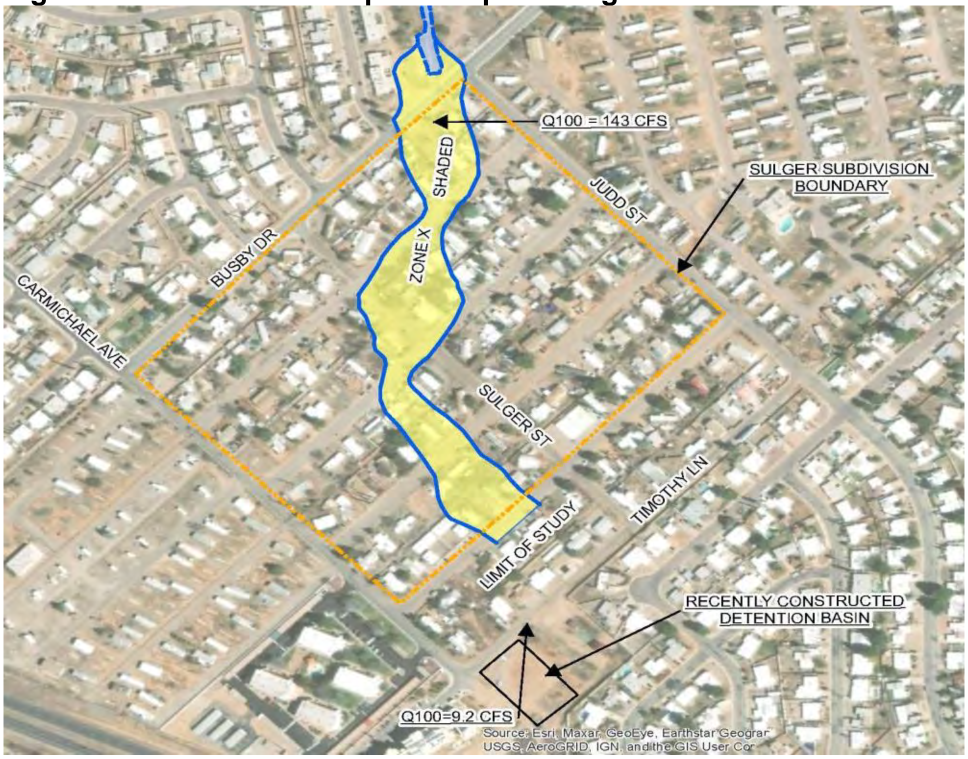


Figure 4.1.2 -Timothy Ln –Sulger Subdivision Drainage Swale between Residential Lots

Figure 4.1.3 - Jennifer Ln –Sulger Subdivision Drainage Ditch between residential lots

Figure 4.1.4 - Danny Lane – Sulger Subdivision Typical area of surface drainage through residential lot

Figure 4.1.5 - Busby Drive - Sulger Subdivision Typical area of surface drainage through residential lot

Figure 4.1.6 - Sulger Subdivision - Detention Basin South of Timothy Avenue

4.2 Fry Town Area Flooding

The Fry Town area is located south of Tacoma Street between Carmichael Avenue and 7th Street; Denman Avenue is roughly the south boundary of the area. The area began development in the 1930's and consists primarily of mobile homes although several ground set homes and commercial buildings are present as well. Surface drainage conditions are similar to that of the Sulger subdivision, which being due to the absence of, or under sized drainage structures, particularly in the areas south of Theater Drive.

A drainageway and a detention basin (referred to as the North Basin) are present between Fry Blvd. and Denman Avenue, but these structures terminate at the outlet of the basin into a narrow swale between residential lots on the west side of Carmichael Avenue. These drainage structures are collectively known as the Fab Drainageway (see Figure 4.2.1 and 4.2.2).

The Fab Drainageway flows discharge into a 66-inch diameter underground storm drain between Carmichael Avenue and Canyon Drive, but this drain is inadequate to contain the 100-year discharge of 207 cfs, so some of the flow must spread onto adjoining residential lots and Canyon Drive to the east. Surface flow conditions through the residential area continues northwesterly from Canyon Drive across 1st and 2nd Streets, and Theater Drive.

A man-made drainageway named Vista Village Drainageway begins at the outlet of the underground storm drain on the east side of 2nd Street about 200-feet north of Theater Drive (see Figure 4.2.3 through 4.2.7). This drainageway continues east across 3rd, 4th, 5th and 6th Streets to 7th Street to a drainageway large enough to contain the 100-year discharge which varies from 207 cfs at Fry Blvd. to 732 cfs at SR-90. The drainageway and all of the culverts or dip crossings between 2nd Street and 7th Street are undersized.

The scope of work for this SWP also includes a site specific assessment of drainage design alternatives for a City owned parcel of land (1.25 acres) located at the southeast corner of Fry Blvd. and Fab Avenue. The property is currently vacant and is within the 100-year floodplain for flows conveyed north along Fab Avenue to a box culvert beneath Fry Blvd. The 100-year discharge along Fab Avenue has been estimated to be 207 cfs; inadequate street capacity and backwater conditions at the Fry Blvd. culvert result in periodic storm water inundation of the site. CMG conducted a preliminary study to identify possible storm water mitigation alternatives. The study report is provided in Appendix G of this SWP.

Figure 4.2.1 – Fab Avenue Drainageway Location Aerial

Figure 4.2.2 - Fab Avenue Wash Looking Upstream from North Street

Figure 4.2.3 - Vista Village Drainageway Location Aerial

Figure 4.2.4 - Vista Village Drainageway Looking Downstream from 2nd Street

Figure 4.2.5 - Vista Village Drainageway Looking Downstream from 6th Street

Figure 4.2.6 - Vista Village Drainageway Looking Upstream from 7th Street

Figure 4.2.7 - Vista Village Drainageway Looking Downstream from Catalina Drive

4.3 Montebello/Kings Manor Wash North of Village Meadows School

This area of concern, identified by City staff is located along Montebello/Kings Manor Wash just upstream of Calle Portal. Field observations found that flow overtops the south bank along the reach extending about 500 feet plus upstream of Calle Portal due to inadequate channel capacity. Once flows leave the channel it does not return until downstream of Calle Portal because the south channel bank is higher than the south overbank. Preliminary floodplain mapping indicates that Nancy Hakes Park and a few residential structures on the south overbank could be inundated during the 100-year storm. Town and Country Middle School Drainageway crosses Calle Portal in a shallow dip section and the overbank topography does not lend to directing flow toward the channel. No records of past flooding in this area were discovered.

4.4 Kings Manor Wash Grade Controls at Savannah Springs Apartments and just upstream of SR-90

A grade control structure was installed at the downstream end of channelization and bank protection associated with the Savannah Springs Apartments in about 2005 (see Figure 4.4.1). The location of this structure is about 940 feet upstream (southwest) of State Route 90. The structure was constructed using concrete; the design dimensions of the structure are 1-foot wide by 15-feet deep, extending the full width of the channel. The top elevation of the structure was set at the channel invert at the time of construction, but field observations found that a 4-foot drop has developed on the downstream side of the grade control due to degradation. The grade control appears to be in good condition, but further degradation is expected which may compromise its stability. Periodic monitoring is recommended.

Figure 4.4.1 - Kings Manor Wash Grade Control at Savannah Springs Apartments

One of the factors important to the stability of the grade control at Savannah Springs Apartments is the stability of another grade control structure located about 800 feet downstream. This structure was installed to protect a sewer line crossing of the wash (see Figure 4.4.2 below). Field inspections found this grade control to be in good condition.

Figure 4.4.2 – Kings Manor Wash Grade Control located about 150-feet Upstream (south) of SR-90

4.5 Coyote Wash Headcut

A ± 8-foot deep headcut has been propagating upstream toward Foothills Drive and is located about 1/3 mile (1940-feet) downstream of street crossing. Approximately 10 years ago, the city recognized that this headcut would ultimately threaten existing bank protection and an underground utility crossing so dumped concrete was placed in an effort to arrest the degradation. Field observations found that the dumped concrete has functioned so far to prevent degradation into the upstream bank protection reach, but the materials' resistance to erosion during future floods is questionable. A photograph of the dumped concrete is provided below in Figure 4.5.1.

Figure 4.5.1 - Coyote Wash Headcut at Sewer Line Crossing 1940-ft downstream of Foothills Drive

4.6 Woodcutters Wash across from the Savannah Drive Intersection

Headcutting has been propagating upstream along the reach of Woodcutters Wash that parallels 7th Street south of Busby Drive. The reach of the wash downstream of the Savannah Drive intersection is unstable with bank erosion and bank sloughing evident due to degradation. The City installed a concrete rubble grade control structure at this location (about 860-feet downstream of the 7th Street culvert) to prohibit further headcut propagation into the upstream reaches. This structure appears to be functioning at the present time, however, further downstream degradation may uncut the structure at some time. Photographs of the structure and downstream channels conditions are provided below in Figures 4.6.1 and 4.6.2.

Figure 4.6.1 - Woodcutters Wash Headcut near across from the 7th St./Savannah Dr. Intersection

Figure 4.6.2 - Woodcutters Wash Headcut Downstream of Grade Control near7th Street/Savannah Drive Intersection

4.7 Soldier Creek – Garden Avenue to SR-90

The effective FEMA flood hazard map shows that several residential and commercial structures are within the floodplain and floodway along the reach between Garden Avenue and SR-90. The low flow channel has limited capacity due to shallow depth (4-6 feet) and a narrow channel width (20-30 feet). Soldier Creek is one of a few watercourses where native riparian vegetation still exists because degradation has been prevented by roadway culverts at Garden Avenue, Kayetan Drive and SR-90. However, removing existing structures from the floodplain is unlikely unless significant modifications to the channel are made to increase capacity or a levee is constructed. Both of these options will be costly, may cause environmental damage and possibly subject to Federal environmental regulations. A concept plan for channelization to contain the 100-year discharge is presented and discussed in Section 9 of this report.

Figure 4.7.1 - Photograph of Soldier Creek Overbank Area Subject to Flooding

4.8 Charleston Wash and 3rd Street Drainageway

Severe degradation has beenoccurring along Charleston Wash and the 3rd Street Drainageway between Colombo Avenue and Fry Blvd; the most severe that was observed throughout the community. Depth of channel entrenchment below top of bank elevations is commoly 10- to 15-feet. The channel has a lower width/depth ratio than other similar watercourses which likely contributes to higher flow velocities, sediment transport capacity thus increased erosions rates. Examples of this are shown in Figures 4.8.1-4.8.6 below.

Figure 4.8.1 - Charleston Wash downstream of Colombo Avenue

Figure 4.8.2 - Charleston Wash at Sewer Line crossing about 860-feet west of Avenida Escuela

Figure 4.8.3 - Charelston Wash Grade Control about 200-feet downstream of Coronado Drive

Figure 4.8.4 - 3rd Street Drainageway about 200-feet upstream of the Woodcutters Canyon Wash Confluence

Figure 4.8.5 – Grade Control along the 3rd Street Drainageway about 1000-feet Downstream of Fry Blvd.

Figure 4.8.6 - 3rd Street Drainageway about 560-feet Downstream of Fry Blvd.

SECTION 5: Channel Erosion and Lateral Migration Assessment

5.1 Background and Importance of Historical Changes

Several of the watercourses passing through the City have been experiencing channel bottom degradation for many decades as discussed and shown in photographs in Section 4. The causes are related to increases in storm water runoff volume from urban areas, higher sediment transport capacity associated with channel entrenchment, and deposition of sediment within the Ft. Huachuca detention basins. **Long-term channel bottom degradation is considered one of the most important drainage related issues to be addressed by this SWP because it threatens channel stability and can cause infrastructure failures during future floods.**

The most notable and important change to the washes experiencing degradation has been the incisement of the low flow channel. This incisement is caused primarily by man-made changes to the watersheds and wash environment including urbanization, channelization, floodplain encroachments and aggregate mining. The incisement has initiated a regime change from shallow braided channels where flow is dispersed and velocities are low, to a deeply incised single channel where flow is concentrated. Associated with these changes is an increase in flow velocity and sediment transport capacity.

Degradation resulting from the above described regime change usually results in headcutting to reduce velocity and slope, as the river system works toward energy reduction to balance sediment supply with transport capacity. Another natural process to achieve slope reduction is to increase the length of the flow path via an increase in meander amplitude. Bank erosion and bank sloughing are evidence of this process and can be observed along many of the study washes.

As is, degradation via headcutting is expected to continue at a rate determined by the frequency and magnitude of future stream flows; unless counter measures such as additional grade controls are installed. The potential consequences of degradation are the undermining of infrastructure within the wash environments including underground utilities, bank protection, culverts and bridge foundations, and an increase in the rate of bank erosion. Figure 5.1.1 below shows an example of degradation along Coyote Wash about 1/3 mile downstream of Foothills Drive. The City identified the necessity to place temporary erosion control measures (broken concrete) to prevent headcutting from propagating into the upstream reach where bank protection and a sewer line crossing of the wash are located.

Figure 5.1.1 - Headcut along Coyote Wash Downstream of Foothills Drive

A study of the wash profiles including field observations found striking differences in the level of historical degradation from one wash to another. Those washes experiencing the most degradation are highly urbanized watersheds and have detention basins located on Ft. Huachuca just west of Buffalo Soldier Trail. Detention basins are beneficial in achieving peak flow reduction, but they also trap sediment critical to maintaining downstream channel stability, and they prolong flow duration. Sediment trapped in the basins creates a clear water effect along downstream reaches thus increasing the streams capacity to entrain channel bottom and bank sediments. Table 1 below lists the observed channel conditions and highlight's locations, or channel reaches that remain vulnerable to degradation and erosion related damages. The reader should note that the conditions and measurements given in Table 1 are based primarily on 2009 topography.

Table 1: Summary of Historical Degradation and Relative Importance To Infrastructure Stability

Orange Highlights Rows Indicate Areas of Moderate Concern – Monitoring Recommended
Red Highlights Rows Indicate Areas of High Concern – Mitigation Measures Needed

Wash Name	Reach	Observed Changes	Comments
Soldier Creek	SR-90 to Garden Avenue	Profile stable	No detention basin on Fort
Fab Avenue Drainageway	Fry Blvd to near Canyon Drive	Profile stable	
Vista Village Drainageway	2nd Street to Catalina Street	Profile relatively stable upstream of Catalina St.	
	Catalina Street to SR-90	Severe degradation	6 grade control structures downstream of Catalina Dr. but headcutting continues to threaten the street dip crossings and downstream bank protection. One grade control present between Catalina St and Tacoma St
Graveyard Gulch	San Juan Capistrano to SR-90	4-feet of degradation downstream of SR-90	Otherwise, profile generally stable
Charleston Wash	Colombo Avenue to Lenzner Avenue	Severe degradation and bank erosion/sloughing	Several grade controls installed to protect street culverts and sewer line downstream of Coronado Dr
3rd Street / Buena #3 Drainageway	Coronado Dr. to Fry Blvd.	Severe degradation, bank erosion/sloughing	Several grade controls installed to protect street culverts and a sewer line downstream of Fry Blvd.
	Fry Blvd. to Busby Dr.	Profile relatively stable	Frequent culverts control degradation
Woodcutters Canyon Wash	Charleston Wash confluence to Fry Blvd.	Fully bank protected reach	Four grade control structures present
	Fry Blvd to Busby Drive	Profile relatively stable	Minor degradation at Wilcox and Busby Dr.
	Busby Dr. to 7th Street	Moderate degradation	Temporary grade control measures present across from Savannah Dr.

Wash Name	Reach	Observed Changes	Comments
Coyote Wash	Charleston Wash confluence to SR-90	Minor degradation	
	SR-90 to Avenida del Sol	Profile relatively stable	
	Avenida del Sol to Foothills Dr.	Headcutting with 8-foot drop midway between Avenida del Sol and Foothills Dr.	Concrete rubble grade control present, failure probable without periodic repairs or engineered structure
	Foothills Drive to SR-92	Profile relatively stable	
	SR-92 to Camino Rancho	5-ft drop at Camino Real, and 4-ft drop at Camino Rancho	Significant degradation expected particularly between SR-92 and Camino Real
	Camino Rancho to Coronado Drive	13 ft drop at Coronado Drive	Significant degradation expected to continue
	Coronado Drive to Town & Country Drive	Severe degradation	Three grade controls with 4-foot high drops present upstream of Coronado Dr. 2009 drop height at Coronado Drive was about 13-feet. Future headcut propagating toward Coronado Drive expected
	Town & Country Dr. to Buffalo Soldier Trail	Profile relatively stable	
South Garden Wash	SR-92 to South Wardell Rd.	Four foot drop at South Wardell Rd grade control	
	South Wardell Rd to upstream study limit	Profile relatively stable	Detention basin present on Fort
Summit Drainageway	Entire study reach	Profile relatively stable	Detention basin present on Fort
Country Club Estates Drainageway	Entire study reach	Profile relatively stable	Detention basin present on Fort
Montebello/ Kings Manor Wash	Guillo Cesare and Colombo Avenue	Moderate degradation (4 ft) at two dip crossings, Leonardo de Vinci Dr and Raffaele Avenue	
	Colombo Ave to SR-90	Moderate degradation (3 ft) at SR-90	

Wash Name	Reach	Observed Changes	Comments
	SR-90 to SR-92	4-ft drop at Savannah Springs Apartments grade control. 3-ft drop at SR-92 culvert outlet rubble	
	SR-92 to Calle Portal	Severe degradation	Grade control with an 8-ft drop at Avenida Escuela culvert inlet. 5.5-ft drop downstream of Calle Portal
	Calle Portal to Camino Real	Moderate degradation	Headcut development about 900-feet upstream of Calle Portal
Mountain Mesa Wash	SR-90 to upstream study limit	Profile relatively stable	No detention basin on Fort
Murray Springs Wash	Entire study reach	Profile relatively stable	No detention basin on Fort
Garden Canyon Wash	Downstream of SR-92	12-foot headcut downstream of SR-92	
	SR-92 to upstream study limit	Profile relatively stable	No detention basin on Fort
Orange Highlights Rows Indicate Areas of Moderate Concern – Monitoring Recommended			
Red Rows Indicate Areas of High Concern – Mitigation Measures Needed			

The washes that have experienced severe degradation include Vista Village, Charleston, Coyote, Garden Canyon, Woodcutters Washes, and the 3rd Street Drainageway (highlighted in red). Moderate degradation washes include some reaches of Woodcutters and Montebello/Kings Manor. Degradation along these washes may have been more severe if not for the presence of several grade control structures. Grade control structures as discussed herein may include roadway culverts, stabilized roadway dip crossings as well as concrete structures designed to protect underground utilities or other nearby structures. Some of these structures are temporary in nature since they are not engineered and possibly subject to failure during future floods.

Except for Garden Canyon Wash and Montebello/Kings Manor Wash, all washes experiencing severe degradation have an upstream detention basin on Ft. Huachuca which trap sediments. Most of the degradation along these washes is a result of the “clear water effect” which entrains sediments from the bottom and banks of channel reaches through the urban areas, and downstream thereof. This process will likely continue for decades to come, requiring additional structures to minimize damage potential.

Bank erosion is evident at several locations particularly where degradation and channel entrenchment has occurred. Usually, the magnitude of lateral bank movement has been minimal amounting to 30-feet or less, being caused by block caving of high vertical banks. Greater lateral erosion has been observed along the outside banks of meander bends and in some cases, has threatened public infrastructure. One such example is along the 3rd Street drainageway between Fry Blvd. and Lenzner Avenue where gradual bank erosion threatened a regional sewer line (see Figure 5.1.2 below, sewer line is in road where car parked). The City installed about 165 feet of gabion bank protection to mitigate this threat.

Figure 5.1.2 - 3rd Street Drainageway Bank Erosion near Sewer Line

Another example of significant bank erosion is along Charleston Wash in the vicinity of the Port Royale Apartments (APN#10716011N) where erosion at a meander bend resulted in the loss of 30- to 50-feet of bank where landscape improvements were located (see Exhibit below).

Figure 5.1.3 - Charleston Wash Bank Erosion at Port Royale Apartments

5.2 Description of Existing Grade Control Structures

Table 2 identifies the location of grade control structures or roadway crossings that function to prevent upstream propagation of headcuts. The condition of the grade controls varies greatly from engineered structures that include scour protection to dumped concrete rubble to gunited riprap. Drop heights based on 2009 topography are estimated and relative importance is noted based on the protection they provide to upstream structures. A statement regarding the observed condition of the grade control structures in 2021 has also been included in Table 2.

Relative importance is defined as follows:
Low – failure would not likely threaten upstream structures during a single flood event and repair cost would likely be minimal. Regular maintenance should suffice.

Moderate – there is evidence of widespread headcutting along the watercourse but distance to upstream infrastructure is sufficient to limit damages during a single flood event. However, left unrepaired, significant damage to upstream structures such as roads, bank protection and utility lines could occur.

High – Upstream infrastructure could be lost as a result of a single flood, and left unattended, headcutting depths and distances will be significant. Regional damage to watercourse conditions and structures could occur. Reconstruction and repair costs could be significant.

Grade Control Conditions are defined as follows:

Failing: structure foundation has been severely undermined with partial failure having already occurred or the structure stability is so compromised that near-term failure is likely. Lateral erosion has outflanked the structure such that degradation along upstream reaches can continue.

Poor: Foundation is being undermined to some degree and will likely continue yet the structure still functions as intended. Near-term failure appears to be unlikely. Partial failure of downstream aprons has occurred.

Good: Structure shows minimal or no signs of damage and continues to function as intended. Minor degradation could be present along the downstream channel reach but does not threaten structure stability.

Table 2: Grade Control Structure Locations and Condition

Vista Village Drainageway

Location	Drop Height (ft)	Engineered Grade Control Structure	Grade Control Material	Grade Control Condition	Relative Importance
At SR-90	About 4’	unknown	concrete	Poor	moderate
± 600 ft upstream of SR-90	Less than 3’	no	concrete	Poor	moderate
± 800 ft upstream of SR-90	Less than 3’	no	concrete	Poor	moderate
± 1000 ft upstream of SR-90	Less than 3’	no	concrete	Poor	moderate
± 1500 ft upstream of SR-90	Less than 4’	unknown	concrete	Poor	high
Downstream of Catalina Drive	3’ to 4’	No	Broken Concrete	Good	moderate
Between Catalina Dr and Tacoma St	± 4’	No	Grouted Riprap	Good	low
Between Tacoma Dr and 7th St	± 4’	No	Grouted Riprap	Good	low

Charleston Wash

Location	Drop Height (ft)	Engineered Grade Control Structure	Grade Control Material	Grade Control Condition	Relative Importance
Colombo Dr	± 4'	Unknown	Grouted Riprap	Good	High
SR-90	± 11'	no	concrete	Poor	moderate
Sewer Crossing about 860 ft upstream of Avenida Escuela alignment	± 8'	yes	concrete	Failing	high
about 200 ft downstream of Coronado Dr	± 9'	no	concrete	Poor	high

3rd Street Drainageway/Buena #3

Location	Drop Height (ft)	Engineered Grade Control Structure	Grade Control Material		Relative Importance
Just upstream of Woodcutters Wash confluence	± 4'	unknown	concrete	Poor	high
Lenzner Avenue	± 2'	no	Broken concrete	Failing	low
About 1000-ft downstream of Fry Blvd.	± 4'	no	concrete	Poor	high
About 560 ft downstream of Fry Blvd.	± 9'	no	Grouted Riprap	Good	high
Fry Blvd	± 3'	no	Grouted Riprap	Poor	moderate

Woodcutters Wash

Location	Drop Height (ft)	Engineered Grade Control Structure	Grade Control Material	Grade Control Condition	Relative Importance
3rd Street Drainageway	4’	yes	concrete	Good	high
540 ft upstream of 3rd Street drainageway confluence	Less than 3’	unknown	concrete	Good	high
920 ft upstream of 3rd Street drainageway confluence	Less than 3’	unknown	concrete	Good	high
1620 ft upstream of 3rd Street drainageway confluence	Less than 3’	unknown	concrete	Good	high
850 ft downstream of 7th Street culvert	± 5’	no	concrete rubble	Poor	high
7th Street culvert inlet	± 6’	yes	concrete	Good	high
Golf Links Road culvert inlet	± 2.5’	yes	concrete	Good	high

Coyote Wash

Location	Drop Height (ft)	Engineered Grade Control Structure	Grade Control Material	Grade Control Condition	Relative Importance
1600 feet downstream of Foothills Dr	± 8’	no	Broken concrete	Failing	Very high
Camino Real Dip Crossing	± 5’	no	Grouted and Broken concrete	Poor	moderate
Camino Rancho Dip Crossing	± 4’	no	Grouted and Broken concrete	Poor	moderate
Coronado Dr	± 13’ over 120’ downstream of culvert	no	Broken concrete and Grouted	Poor	high

			Riprap		
460 ft upstream of Coronado Dr	± 3’	no	Concrete CO Wall and Grouted Riprap apron	Good	moderate
960 ft upstream of Coronado Dr	± 5’	no	Concrete CO Wall and Grouted Riprap	Good	moderate
1330 ft upstream of Coronado Dr	± 5’	no	Concrete CO Wall and Grouted Riprap	Good	moderate
Town & Country Dr	± 5’	no	Grouted Riprap	Poor	moderate

Kings Manor Drainageway

Location	Drop Height (ft)	Engineered Grade Control Structure	Grade Control Material	Grade Control Condition	Relative Importance
Giulio Cesare Avenue	± 3’	yes	concrete	Good	high
Leonardo de Vince Dr dip crossing	± 5’	no	gunite	Good	high
Raffaele Avenue dip crossing	± 4’	no	gunite	Good	high
Colombo Avenue	± 3’	no	Concrete rubble & riprap	Good	moderate
SR-90	± 3’	no	Concrete rubble & riprap	Poor	moderate
330-feet upstream of SR-90	± 4’	yes	Gabions with Concrete cap	Good	high
Savannah Springs Apts ±940 ft upstream of SR-90	± 4’	yes	concrete	Good	high
SR-92	± 3.5’	no	Concrete rubble & riprap	Poor	moderate
Avenida Escuela culvert inlet	± 7’	no	gunite	Good	moderate
Calle Portal dip section	± 5.5’	no	gunite	Good	moderate
Coronado Dr	± 2’	no	gunite	Good	moderate

SECTION 6: Hydrology

Hydrologic models created in conjunction with this SWP were developed using the HEC-HMS program from the U.S. Army Corps of Engineers (the Corps). This software was chosen for several reasons- it is widely used amongst hydrologic modelers; the output files are easy to customize and easily read; the Corps has developed free extension programs that allow HEC-HMS to be used in conjunction with ArcGIS (HEC-GeoHMS); and another extension program, ArcHydro. Producing the results of the watershed models in HEC-HMS format allowed the data to be readily distributed to potential users. Models may be modified to suit individual situations or analyze numerous scenarios.

The hydrologic modeling method is based on the 2014 ADOT Highway Drainage Design Manual13 and the Drainage Design Manual for Maricopa County, 2011. The methodologies presented in these manuals are reasonably conservative, clearly presented, appropriate for Arizona, and largely supported by the functions within HEC-HMS. The **City of Sierra Vista's 2021 Surface Water Master Plan Part 1 – Hydrology** (which precedes Part 2 of this report) is a detailed report that describes the methodology and results of HEC-HMS modeling conducted to update the surface water hydrology for watersheds contributing storm water runoff to the COSV study area.

The design storm analyzed by the HEC-HMS modeling conducted by the City and FEMA uses both the 24-hour and 6-hour storms. More details regarding the watershed hydrologic modeling are given in the Part 1 report. It should also be noted that the 1-hour duration storm was used to conduct preliminary modeling for small (less than 100 acres) local watersheds that contribute to existing and proposed detention basins along the Fab Avenue Drainageway. Small, urbanized watersheds can yield peak discharges that are comparable to the 6-hour and 24-hours storms because of their higher rainfall intensity.

SECTION 7: Hydraulics/Floodplain Mapping

All of the water courses within the study area have previously been mapped with FEMA flood hazard boundaries. The age of the mapping varies but most date back 10 to 20 years or more. The only washes that have more recent mapping are the 3rd Street/Buena #3 Drainageway and Country Club Estates Drainageway.

The City of Sierra Vista, in recognition of the need to update the mapping, requested FEMA to remap the flood hazard boundaries using most current (2009) topographic mapping. That study was ongoing at the time of this report although the community has reviewed and commented on the preliminary results. This study (as well as the more recent map updates) is based on high quality Light Detection and Ranging (LIDAR) data provided by Fort Huachuca in 2009 LIDAR data is gathered using an airborne laser to measure the distance to objects on the ground. This method of data gathering provides accurate terrain elevations. The data provided also included high resolution aerial topography. The vertical datum used for these studies is the North American Vertical Datum of 1988 (NAVD88). The horizontal datum used for this study is: NAD_1983_HARN_StatePlane_Arizona_East_FIPS_0201_Feet_Intl (but converted to feet as a part of this study).

The effective FEMA flood hazard boundaries are contained within the channel banks along most watercourse reaches. A review of the effective and proposed FEMA mapping, and information provided by the COSV found the following locations where existing structures are vulnerable to flooding during the 100-year storm event (see Table 3).

Table 3: Locations where Several Overbank Structures are within the FEMA Floodplain

Watercourse Name	Location	Approximate Number of Structures within the Floodplain
Soldier Creek	Garden Avenue to SR-90	32
Fab Avenue Wash	Upstream of Fry Blvd. for a distance of about 600 feet	7 (4 are commercial)
Vista Village Drainageway	7th Street to inlet of culvert upstream of Canyon Drive	50
3rd St/Buena #3 Wash	Sulger Subdivision	36
Woodcutters Canyon	Upstream of Fry Blvd to Lenzner Avenue	5 (3 are commercial)
Montebello/Kings Manor Wash	Colombo Dr to Fry Blvd (SR-90) and Avenida Escuela to Camino Real	27
Coyote Wash	Upstream of Camino Real for about ½ mile	5
South Garden Drainageway	Vicinity of Cashway Mini-Storage units upstream of SR-92	Mini-Storage Units only

SECTION 8: Summary of Findings for Existing Conditions

Work completed as a part of the existing conditions SWP analyses has concluded that long-term degradation presents the greatest threat to property and infrastructure stability throughout most of the community. The threatened structures include roads, culverts, bank protection and utility lines that either cross the washes or run parallel to and near the banks. Associated with degradation is an increase in the potential for lateral migration to threaten existing structures that are nearby, but not presently located within the washes.

The community has over recent decades taken actions to control degradation by constructing several grade control structures to prevent headcut propagation. Some of these structures are well engineered while many others appear to be measures installed as an emergency action; those most usually being dumped concrete and rock or broken concrete. Field observations found the condition of these structures to vary, some being in reasonably good condition while others show evidence of potential failure.

While a potential for flooding of structures is present as identified by both the effective and proposed FEMA floodplain mapping, historical flood records (which are limited) do not indicate this to be a significant concern for the community. However, nuisance flooding of residential property is frequent within the Sulger subdivision and Fry Town areas.

Available information derived from these sources indicate that past flood damages have been mostly associated with erosion, sediment deposition, debris deposits, perimeter fencing and landscaping. Areas where more frequent flooding occurs are locations such as the Sulger subdivision and Fry Town where widespread, shallow flooding occurs. Surface flows through these locations occurs due to the absence of adequate drainage structures such as channels and underground storm drains. Remedying these conditions through fully developed communities can be difficult without large expenditures and neighborhood disruptions.

The City recently constructed a detention basin south of Timothy Street in an effort to reduce the volume of offsite flow that drains from the south through the Sulger subdivision. According to hydraulic modeling completed by City staff, the 100-year peak flow entering the south end of the subdivision will be reduced from 62 cfs to 9 cfs which will certainly benefit homes closest to the flowline which often drains through yards. However, storm water generated onsite and from areas downstream of the basin will continue to affect this residential area.

The Fry Town area which is affected by the Fab Avenue drainageway and Vista Village Drainageway shares many of the same drainage issues as the Sulger subdivision, which being inadequate drainage facilities through fully developed residential subdivisions. The most likely solution alternatives for this area will include some measure of detention to reduce flow peaks. However, flows generated downstream of existing or proposed basins will continue to be problematic.

SECTION 9: Development and Evaluation of Alternatives

9.1 Overview

Task 7 of the scope of work reads – "At City staff's direction, conduct preliminary assessment of alternatives such as additional regional flood detention basins, channelization, erosion control structures, land acquisition, and culvert redesign and construction. Detailed hydraulic modeling is not included in this scope; only location."

Alternative flood and erosion control methods typically include:

- Regional Detention Basins – specifically, enlarging existing basins on Ft. Huachuca and constructing a new basin for Soldier Creek. Possible benefits of enlarging the existing basins will require new topographic surveys of the basins located on Ft. Huachuca in order to conduct detailed hydraulic modeling of possible benefits of basin enlargements. No other locations within the City have been identified that could provide measurable downstream benefits, other than along the Fab Avenue Drainageway Basin just north of Fry Blvd. Most sites are too small, and the disbursed nature of flooding sources prevents a single basin from providing more than local benefits.
- Channelization – may provide benefits locally but neighborhood impacts will occur due to inadequate rights of way through developed communities. Loss of vegetation should be expected as well. Some locations where channelization may provide flood damage reduction are discussed in Section 9.6 of this report.
- Erosion Control Structures – are a high priority for long-term channel stability and prevention of infrastructure failures. Channel bottom degradation is a major concern for the City of Sierra Vista as demonstrated by the channel changes that have occurred during recent decades. Channel bank erosion is as well but has generally been localized. Both degradation and bank erosion have and will continue to threaten private and public property throughout the community unless preventative measures are taken. A detailed discussion on past erosion and problem area identification was given in Section 5 of this report. Section 9 discussions emphasize the need for repair of existing grade controls and provide some recommendations for new structures that will be needed to control future degradation.
- Culvert replacements, or new culverts at existing dip sections – Possibly beneficial for preventing flows from breaking out of channel and reducing street flooding. A reoccurring observation was flows breaking out of channels at existing dip sections and some undersized culvert locations. This appears to be due to channel shallowing at several dip section locations. However, the number of flood vulnerable structures within the mapped floodplains at these locations is limited. Adding culverts and deepening the channels will be costly relative to the number of structures that benefit. Local bank protection will also be needed upstream and downstream of new culvert structures to prevent lateral erosion. On the other hand, the question of emergency access also needs to be addressed when evaluating the needs and benefits associated with a culvert replacement. Results of the preliminary hydraulic modeling conducted by FEMA suggest that several of the existing culverts and dip crossings do not provide all-weather access. CMG reviewed the preliminary FEMA hydraulic models to estimate the capacity of the existing culverts and determine whether or not overtopping occurs. Results of this review are presented in Appendix H.

Several grade control structures have been installed by the City or County to prevent headcut migration that would threaten roadway or sewer line crossings. Each of these structures are unique in that a few are engineered with most having been installed by City maintenance crews to mitigate identified threats. Most are made of grout or grouted riprap and are constructed on slopes leading down to the channel invert; few have toe walls to account for degradation. The toe of the slope protection for many locations is now above the downstream channel invert elevation due to degradation.

It should be understood that the recommendations for repairs and reinforcement of existing grade control structures will be interim measures so further modifications may be needed after a period of several years, or several significant flow events. Their primary purpose is to prevent major failures that would require complete reconstruction or damage to the public infrastructure such as streets, bank protection and underground utilities that could fail. As such, it should be expected that modifications to these structures may be needed at some time after they were initially constructed. Several different cross-sections and profiles are offered for grade control structures depending on site conditions such as height of existing drops and potential for lateral migration to outflank a structure. However, other options may be possible and those should be reviewed at the time of final design. Costs could also vary based upon the type of material available at that time.

9.2 Identified Flood and Erosion Control Priorities

Priorities identified from the Existing Conditions Study results and Review of Draft FEMA floodplain maps include:

1. Fab Avenue/Fry Town and Vista Village Drainageways Flood Mitigation
2. Coyote Wash – Avenida del Sol to Foothills Drive Grade Stabilization
3. 3rd Street Drainageway – Coronado Drive to Fry Blvd Grade Stabilization
4. Soldier Creek -Buffalo Soldier Trail to SR-90 Flood Mitigation
5. Charleston Wash – Fry Blvd to Colombo Street Grade Stabilization
6. Coyote Wash – Camino Rancho to Town and Country Drive Grade Stabilization
7. Sulger Subdivision Flood Mitigation

The remainder of Section 9.2 briefly discusses the nature of flooding or erosion associated with priority areas and a more detailed assessment of the alternatives begins at Section 9.3.

<u>Fab Avenue/Fry Town and Vista Village Drainageways</u>
The Fab Avenue Drainageway and its downstream reach named Vista Village Drainageway do not have adequate capacity for containment of the 100-year discharge while residential and commercial developments are present immediately adjoining the channel banks. It is estimated that about 50 structures are located within the FEMA floodplain between Fry Blvd and 7th Street. Constraints associated with existing land uses likely limit the alternatives to increasing the capacity of the existing detention basin north of Fry Blvd and/or providing additional detention storage at downstream sites.

<u>Coyote Wash – Avenida del Sol to Foothills Drive Grade Stabilization</u>
Significant degradation has been occurring along Coyote Wash upstream of Avenida del Sol and is particularly evident along the reach about 1/3 mile downstream of Foothills Drive. Previous discussions have noted an 8-foot deep headcut just downstream of where a sewer line crosses the wash; there is also bank protection where residential structures are present extending west to near Foothills Drive. The sewer line and the bank protection are threatened with failure if the existing grade control measures fail. The existing measures consist of

dumped concrete that will not provide channel profile stability in the long term. Replacing the dumped concrete with an engineered grade control structure should be a high priority for the community.

3rd Street Drainageway – Coronado Drive to Fry Blvd Grade Stabilization

The 3rd Street Drainageway is grade stabilized at a point about 870-feet downstream of Fry Blvd where there is a grouted spillway with a height of about 13-feet. Culverts at Lenzner Avenue and Fry Blvd also act as grade controls; the existing drop heights at the locations are 5-feet and 8-feet, respectively and there is another 4-foot high grade control just above the confluence with Woodcutters Canyon Wash. The reach of greatest concern is upstream of this grade control where degradation and lateral migration continue.

The City installed about 165-feet of gabion bank protection along a reach of the drainageway where lateral migration threatened a sewer line and adjoining homes. It is expected that these processes will continue if additional erosion control measures are not installed. Controlling future erosion along this reach should be viewed as a high priority.

Soldier Creek - Buffalo Soldier Trail to SR-90

Overbank flooding occurs along this reach due to inadequate channel capacity. It is estimated that at least 32 residential structures and 4 commercial buildings are within the FEMA floodplain along this reach. Possible solution for mitigating the flood potential is a new detention basin west of Buffalo Soldier Trail, and/or widening the channel section to increase capacity.

Charleston Wash – Coronado Drive to Colombo Street Grade Stabilization

Severe degradation has occurred along Charleston Wash as evidenced by profile drops at Colombo Avenue (4-ft), SR-90 (11-ft), Coronado Drive (9-ft) and a grade control structure about 800-feet upstream of the Avenida Escuela alignment (8-ft). The number of existing structures close to the channel banks is limited but future bank erosion could become a threat. The primary goal for this watercourse should be reinforcement of the existing grade controls to prevent future failure of sewer lines, bank protection and streets.

Coyote Wash – Camino Rancho to Town and Country Drive Grade Stabilization

There are several grade control structures already in-place along this reach, however, degradation continues. The primary concern is that there are several homes in close proximity to the channel banks which may become more vulnerable to increased lateral erosion associated with future channel bottom degradation. The possible benefits of additional grade control structures or bank protection should be investigated.

The following sections of this report provide a more detailed description of the problem areas and preliminary mitigation measure approaches. Several concepts for modification of existing grade control structure or new structures are provided but are by no means the only potential solutions. The concepts provide a general measure of drop heights, toe down depths and scour protection but the materials (such as concrete) may be substituted with gabions or soil cement.

Sulger Subdivision Flood Mitigation

Storm water generated from developed areas to the south and west of the subdivision enters the Sulger subdivision. Periodic flooding of the subdivision occurs due to the absence of any drainage facilities and absence of curbs along the street. Lot to lot drainage is common and several of the homeowners have constructed small homemade channels to convey flow through their property. The absence of drainage facilities or easements for them along with the disbursed nature of flow limits the flood control alternatives for this area challenging. One possible solution evaluated by this study is an underground storm drain system.

9.3 Fab Avenue Drainageway and Vista Village Drainageway at Fry Town

Problem Description - Fry Town is located south of Tacoma Street between Carmichael Avenue and 7th Street; Denman Avenue is roughly the south boundary of the area. This area was developed over several decades beginning when Ft. Huachuca began to grow more rapidly in the mid-1900's, consisting primarily of mobile homes, ground set homes and small commercial buildings.

The man-made channel through the Fry Town area is named Fab Avenue Drainageway up to the inlet of a 66-inch diameter storm drain between Carmichael Avenue and Canyon Drive; and is named Vista Village Drainageway starting at the outlet of the storm drain at 2nd Street.

The Fry Town subdivisions which the Fab Avenue drainageway flows through is the area experiencing frequent flooding due to inadequate stormwater system capacity and nearby development that occurred without detention or drainage structures. Many of the streets throughout the area are not curbed, so flow may readily spread onto adjoining lots.

Flood volumes have increased over time due to continuing development within the contributing watershed; most of which was constructed without detention facilities.

The 100-year discharge as calculated by the City's HEC-HMS model at the intersection of Fab Avenue and Fry Blvd. is about 207 cfs (for a 6-hour storm duration), which is the location where stormwater first concentrates at a 2- 8' x 3' box culvert (see Figure 9.1 below). There are no stormwater drainage facilities upstream (south) of this location; flows reach this location as street flow and as overland sheet flow. Some of the flow passes through the 2 - 8' x 3' box culvert beneath Fry Blvd. while the remainder ponds at the intersection and drains north across the street surface when culvert capacity is exceeded. As noted in Section 4.2, a report addressing preliminary alternatives for this area was conducted by CMG (included in Appendix G of this SWP).

Figure 9.1- Two Cell 8’ x 3’ Box Culvert beneath Fry Blvd.

Flow leaving the Fry Blvd. culvert drains across a parking lot then between two commercial buildings via a driveway. After passing between two commercial buildings on the north side of the street, flow discharges into a man-made earthen channel that collects stormwater then drains it east toward North Avenue. A culvert beneath North Avenue discharges flow into a detention basin (hereafter referred to as the North Avenue Detention Basin). This basin presently has a capacity of about 4.5 acre-feet at top elevation. There is a 48-inch outlet culvert set at basin invert elevation along with an emergency spillway at 8.5-feet above basin invert to discharge flows at a reduced flow rate. Calculations completed as a part of this study determined that the 100-year storm peak inflow rate of 207 cfs is only reduced to 188 cfs at the basin outlet.

The earthen swale at the outlet of the North Avenue basin conveys flow to a location between residential lots on the west side of Carmichael Avenue where a portion of the flow discharges into a 66” diameter underground storm drain between Carmichael Avenue and Canyon Drive. Flows in excess of the storm drain capacity spread onto adjoining residential lots to the east, then onto Canyon Drive. Surface flows through the residential area continues northwesterly from Canyon Drive across 1st and 2nd Streets and Theater Drive.

A 5-foot wide concrete channel was constructed to accept stormwater from Carmichael Avenue and deliver it to the 66-inch diameter storm drain but this channel is insufficient to contain more than a small amount of flow. As such, most of the flow spreads onto the adjoining properties as it drains east toward the storm drain inlet. Overflows from the storm drain continue north through residential lots to Canyon Drive, then continue as overland flow until it reaches the Vista Village Drainageway at 2nd Street. The approximate floodplain limits for the area between the storm drain inlet and outlet are shown on Figure 9.2.

FIGURE 9.2 – Floodplain Area between 66-inch Storm Drain Inlet and Outlet

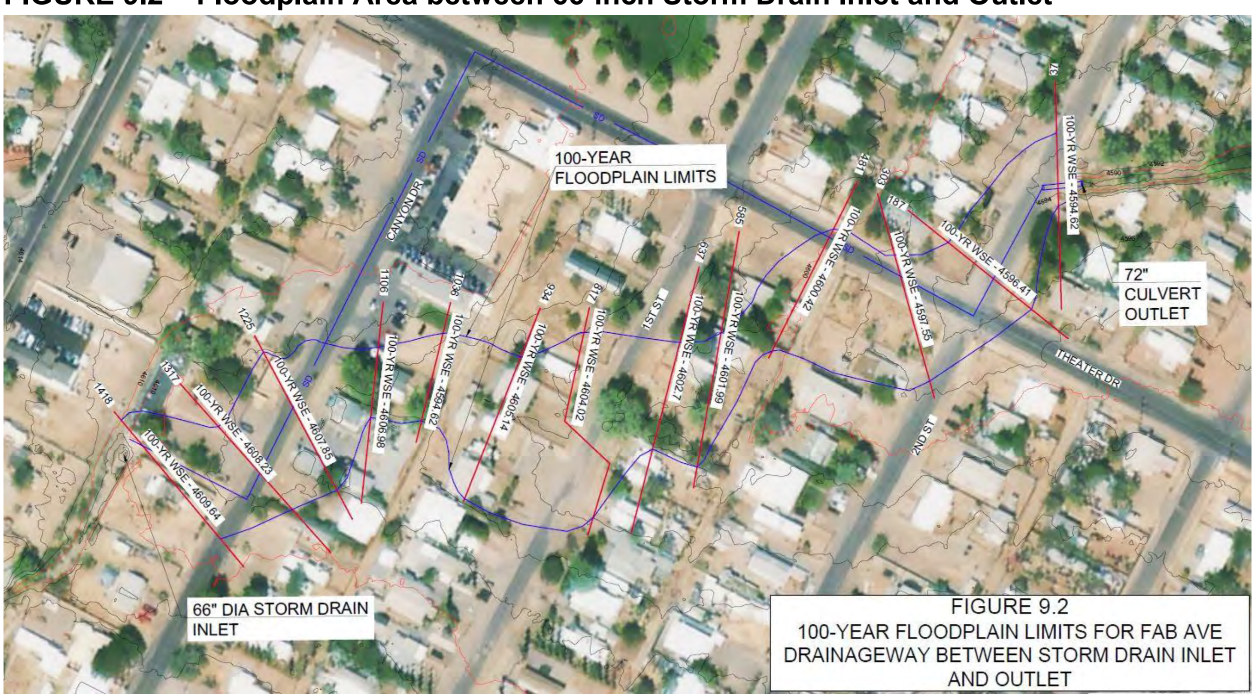


FIGURE 9.2
100-YEAR FLOODPLAIN LIMITS FOR FAB AVE
DRAINAGEWAY BETWEEN STORM DRAIN INLET
AND OUTLET

It is estimated that about 15 structures, mostly residential, are within the floodplain along the storm drain reach between Canyon Drive and 2nd Street. Photographs of storm drain inlet and outlet, and the drainageways constructed in conjunction with the underground drain are provided below as Figures 9.3 through 9.5.

Figure 9.3 - 5-ft Wide drainageway from Carmichael Avenue to 66-inch Storm Drain Inlet

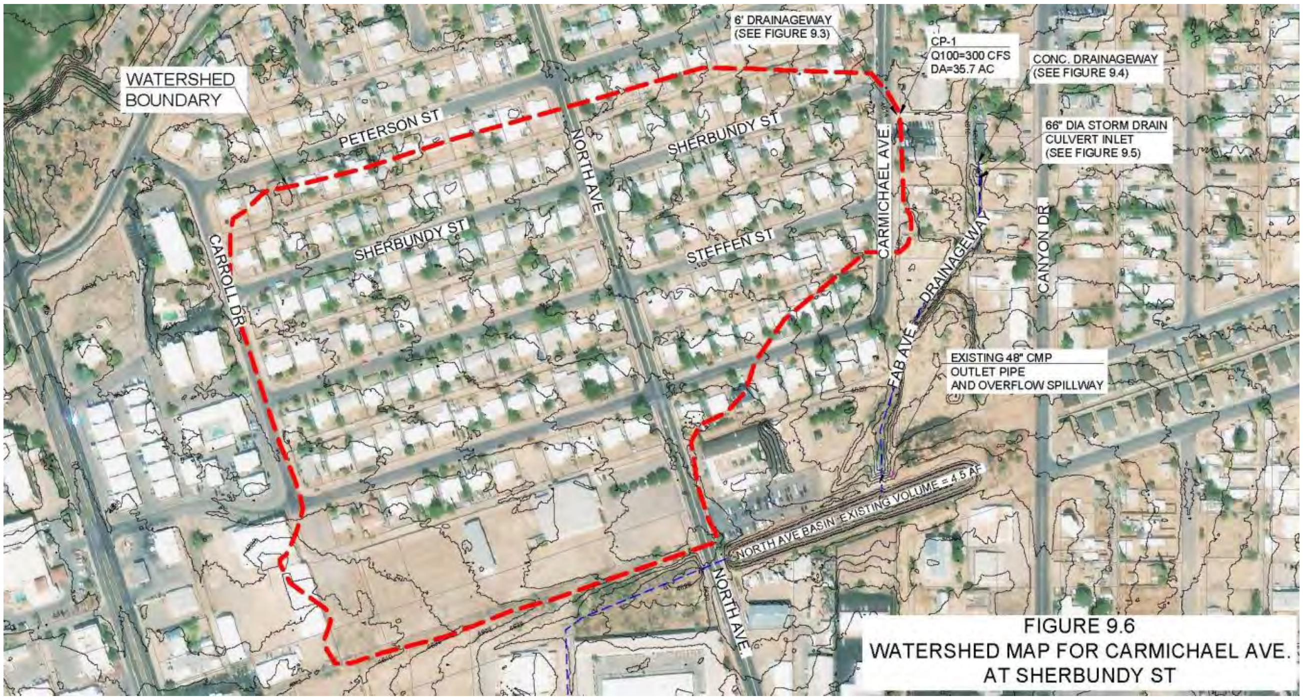
Figure 9.4 - Channel Connecting Fab Ave Drainageway to 66-inch Storm Drain Inlet

Figure 9.5 - Inlet to 66-Inch Storm Drain Connecting Fab Ave Drainageway to Vista Village Drainageway

The storm drain described above varies in diameter from 66-inches at the inlet to 78-inches at the outlet on 2nd Street north of Theatre Drive and is a corrugated metal pipe according to the plans prepared by Cella Barr and Associates. The total length of the storm drain is about 1600-feet. Construction plans for this storm drain were prepared by Cella Barr Associates in 1991. The capacity of this storm drain is estimated to be between 100 and 130 cfs, although that is not necessarily the available capacity at the Fab Avenue inlet since there are other inlets downstream thereof. A copy of the plans can be accessed from the electronic database developed in conjunction with this SWP update.

North Avenue Detention Basin provides enough capacity to reduce the 100-year storm peak inflow from 207 cfs to 188 cfs and lags the flow peak relative to stormwater runoff generated north thereof. However, areas downstream of the culvert inlet are still impacted significant stormwater runoff from residential subdivisions that do not include detention storage. As previously noted, the absence of curbs along most of the streets allows storm water to flow into and through residential lots. The most problematic location is along Carmichael Avenue at the intersection with Sherbundy Street. Calculations completed as a part of this study determined that the subdivision to the west extending as far south as Whitton Street, north to Peterson Street and west to Carol Drive can generate 300 cfs of stormwater runoff during the 100-year/1-hour event. Figure 9.6 shows the boundaries of this watershed and the location along Carmichael Avenue where the flow concentrates.

FIGURE 9.6 – Watershed Boundary for Area draining to Carmichael Avenue Channel



Vista Village Drainageway begins at the outlet of the storm drain on the east side of 2nd Street about 200-feet north of Theater Drive. This drainageway continues east across 3rd, 4th, 5th and 6th Streets to 7th Street to a drainageway large enough to contain the 100-year discharge which varies from 394 cfs at 2nd Street to 897 cfs at SR-90. The drainageway and all of the culverts or dip crossings between 2nd Street and 7th Street are undersized. The preliminary floodplain mapping indicates that about 30 to 35 structures (mostly residential) are within the 100-year floodplain between 2nd Street and 7th Street.

9.3.1 Description of Alternatives

Alternative solutions evaluated as a part of this SWP update included: (1) enlarging the North Avenue detention basin, and, (2) construction of another detention basin in the vicinity of the 66-inch storm drain inlet to reduce or mitigate overflows through the residential area between Carmichael Avenue and 2nd Street; along with installation of curbing on Carmichael Avenue to reduce flows through the residential lots between the street and the storm drain inlet.

9.3.2 Alternative 1 – Increase Capacity of the North Avenue Detention Basin

The current capacity of the basin is 4.5 acre-feet at a depth of 8.5-feet. A detention routing analysis was first conducted to determine the peak basin outflow rate for the existing basin during a 100-year 24-hour duration storm. This analysis was conducted using the Pima County Regional Flood Control District's PC-Route program. Results of this analysis determined that the current basin functions to reduce a peak inflow of 207 cfs to 188 cfs during this storm event.

Two possible approaches to increasing the capacity of the North Basin were evaluated. The first approach looked at lowering the basin bottom elevations as much as possible while maintaining gravity drainage to the outlet pipe. Preliminary calculations determined that the capacity of the North Avenue basin can be increased to about 4.98 acre-feet (about 10%) with minimal grading of the basin bottom elevations. Detention routing computations determined that this minimal increase in basin volume would have no effect on the peak outflow rate.

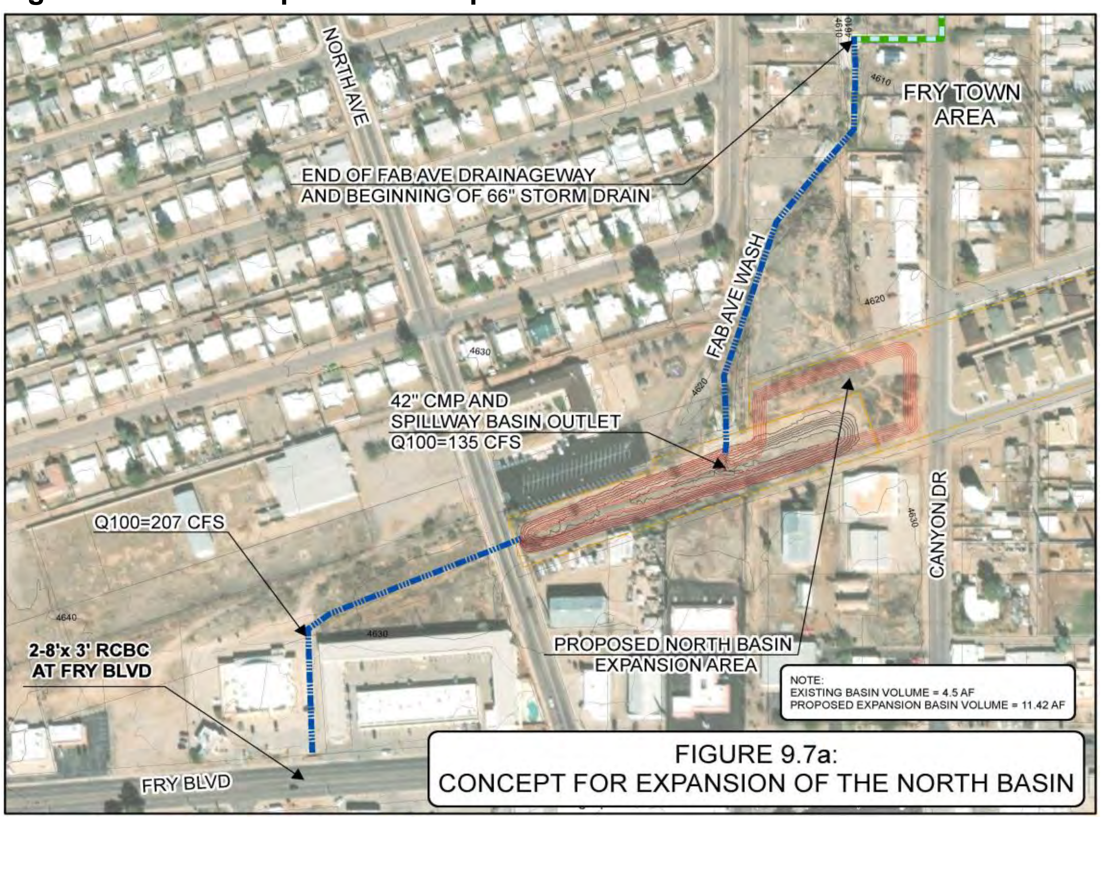
Another possible approach to increasing the capacity of the North Basin considers acquisition of an adjoining parcel of vacant land having an area of about 1.5 acres. This parcel adjoins the east end of the existing basin as shown on Figure 9.7a. The basin expansion would connect to the existing basin and provide offsets from top of basin to property line of 15-feet. The total capacity of the combined basin was determined to be 11.4 acre-feet at a depth of 8-feet. PC-Route computations determined the 100-year storm peak outflow rate to be 135 cfs which is about a 28% reduction of the existing basin peak outflow of 188 cfs. A detention routing computations spreadsheet for this alternative is provided in Appendix C.

The scope of this study does not include more detailed analyses of possible ways to further decrease the peak outflow rate which could possibly be achieved by other means such as modifying the basin outlet structure. However, the analyses conducted as a part of this study should provide a close approximation of potential peak flow reduction benefits associated with expanding the basin onto the east parcel.

The estimated cost for Alternative 1 (increase basin capacity to 11.4 acre-feet) is $297,660.

The cost estimate spreadsheet is provided in Appendix D.1.

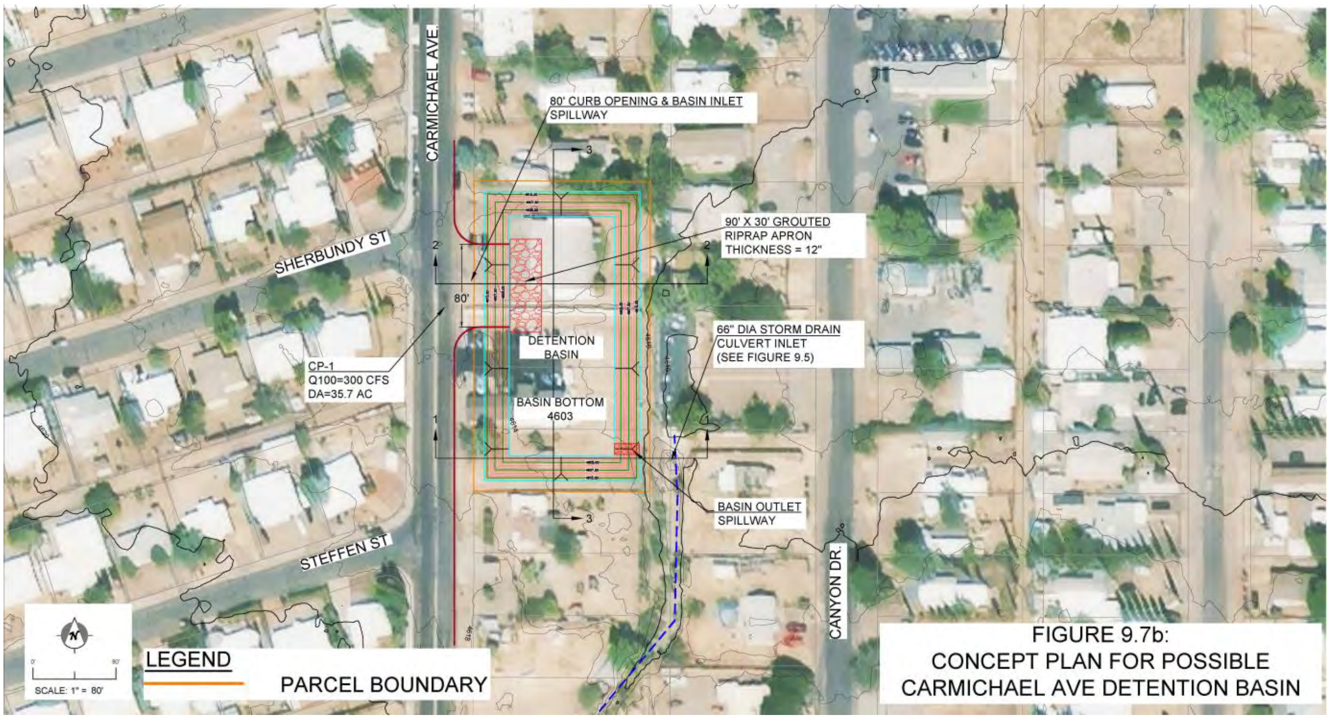
Figure 9.7a - Concept Plan for Expansion of the North Basin



9.3.3 Alternative 2 - New Detention Basin between Carmichael Ave & Canyon Drive

This alternative, to some degree, addresses storm water runoff generated from the subdivision to the west extending as far south as Whitton Street, north to Peterson Street and west to Carroll Drive. Storm water runoff from this area flows unconfined across Carmichael Avenue, through private properties, then toward the inlet to the 66-inch storm drain. Existing structures are in place to convey this flow, but they are undersized as is the storm drainpipe. Figures 9.3 through 9.5 above are photographs of these existing structures. Figure 9.7b provides a concept for the detention basin location, depth and size. Please note that this study does not address property ownership or real estate value so other nearby locations may be suitable as well. The north half of a basin at this location would occupy an abandoned commercial structure so no one would be displaced from this property. The south half is occupied by a church building.

Figure 9.7b - Concept Plan for Carmichael Avenue Detention Basin



A new detention basin (as shown in Figure 9.7b) and curbing along Carmichael Avenue, should help to attenuate flow peaks and improve the storm drain inlet capacity. A reduction in the frequency of overflows from the culvert to properties along Canyon Drive should also occur.

Preliminary storage routing computations for this basin configuration which occupies about 1-acre at a depth of 8-feet, determined that a peak inflow from a 1-hour duration (typical monsoon storm) of 291 cfs could be reduced to an outflow of 51 cfs assuming a 42-inch diameter outflow pipe set at basin bottom elevation. A detention routing spreadsheet for this alternative is provided in Appendix C.

The estimated cost for Alternative 2 is $435,535.

The cost estimate spreadsheet is provided in Appendix D.2.

9.3.4 Alternative 3: Increase Capacity of the Vista Village Drainageway 3rd St to 7th St.

According to the FEMA effective and draft FIS mapping revisions, overbank flooding along Vista Village Drainageway occurs between 3rd Avenue and 7th Avenue. Overbank flooding occurs along this reach due to inadequate channel capacity and storm water runoff generated by developed areas downstream of the North Avenue detention basin. The absence of curbs to contain some flow within the street sections is also a contributing factor.

The alternatives described above are not expected to fully mitigate overbank flows along the Vista Village Drainageway unless measures are also taken to increase channel capacity and enlarge culverts at 3rd and 6th Streets.

Review of the HEC-RAS model cross-sections found that the existing channel has a width of about 30-feet. Overbank flooding occurs because the channel depth is inadequate between 3rd Street and 7th Street, and because the culvert inverts are below the downstream channel inverts by 1.4 feet at 3rd St and by 3.6 feet at 6th St.

Lowering the channel invert profile to increase channel is not an option because of downstream constraints. Widening the channel would require acquisition of property along one side of the channel with the estimated width of this being about 30 feet. This likely means acquiring the full property since inadequate space would be left for a residential structure.

Detailed analyses of Alternative 3 were not conducted and a cost estimate has not been prepared for these reasons.

9.3.5 Comments on Alternatives 1, 2 and 3

Alternatives 1 and 2 are partial solutions for mitigating surface flows through private properties east of Carmichael Avenue. They function to decrease the frequency and magnitude of downstream flow peaks but are not 100-year storm solutions due to stormwater generated downstream and the dispersed nature of the flood sources affecting these areas.

Alternative 3 would require property acquisitions and removal of 7 to 9 of the 25 or so structures within the FEMA floodplain area between 3rd Street and 7th Street.

9.3.6 Vista Village Drainageway – SR-90 to Catalina Drive

Channel bottom degradation is occurring all along this reach of Vista Village Drainageway. The City has installed six grade controls within 1500 feet of SR-90 to control degradation and prevent headcutting into the reach between the Tacoma Street extension and Catalina Drive. The 2009 topography indicates drop heights at these street crossings of 3- to 4-feet. Available information suggests that these grade controls will continue to function as intended. Periodic inspections should occur to confirm this, and maintenance conducted when needed.

It is recommended that an engineered grade control structure be installed on the downstream side of Catalina Drive to replace existing concrete rubble and provide long-term stability, although damages to the channel or roadway are not imminent if regular maintenance of the existing riprap and broken concrete occurs. The existing channel slope downstream of Catalina Drive is about 1.4%; equilibrium slope was computed to be 1.1%. Long-term degradation at Catalina Dr. is estimated to be 6.6 feet. A typical cross-section of the grade control structure needed at this location is shown on Figure 9.8. Existing grade control structures (6) downstream of Catalina Drive should be inspected annually and after large floods to confirm that they continue to function as intended. The estimate of future degradation at Catalina Drive assume that these structures continue to function.

Figure 9.8 Typical Cross Section of the Catalina Dr. Grade Control Grade Control Structure

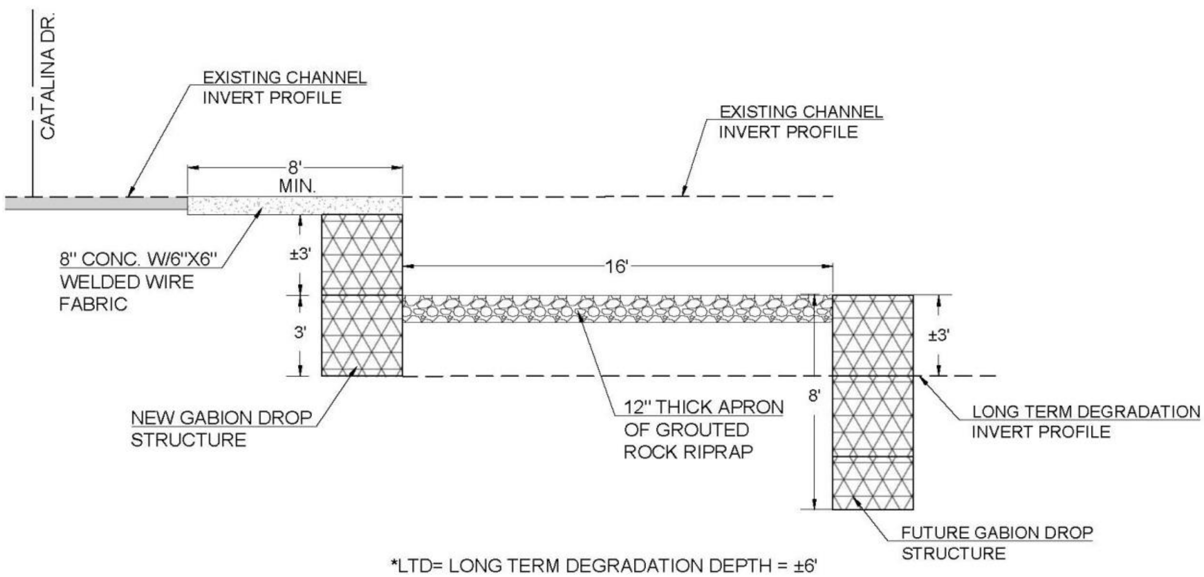


FIGURE 9.8
TYPICAL PROFILE FOR GABION DROP STRUCTURES ALONG VISTA VILLAGE DRAINAGE WAY AT CATALINA DR
N.T.S.

9.4 Coyote Wash

The primary concerns along Coyote Wash have to do with channel bottom erosion which threatens existing infrastructure such as roadway crossings and underground utility lines (as discussed in Sections 4.5 and 5.2 of this report); and in some locations, lateral bank erosion. The amount of channel bottom erosion that has occurred to date warrant mitigation measures to prevent infrastructure failures at some locations. Equilibrium slope computations were conducted to estimate future degradation so recommendations for mitigation measures can (to some degree) account for future changes as well. The science behind the equilibrium slope computations involves many variables so results can only be assumed to be an approximate measure.

Equilibrium channel slope was calculated using Equation 6.26 of the City of Tucson Drainage

Design Standards Manual. This methodology **estimates** the slope needed to balance sediment supply with sediment transport capacity. Long-term degradation is calculated as the difference between existing slope and equilibrium slope, times the reach length. It should be recognized that the equilibrium slope calculations are only an **estimate** of long-term degradation potential. Distance to the next downstream grade control structure, the estimated reduction of sediment supply due to entrapment in upstream basins, and channel slope are the primary variables affecting results. However, many other unquantifiable variables factor into the actual change. These factors are both natural processes and man-made influences, which the equations cannot fully account for. Equation 6.26 is given in Appendix E. Please note that the equation was developed to estimate degradation potential when a natural wash is to be channelized so changes to hydraulic conditions such as the roughness coefficient, discharge and channel width can be factored in. Pre- and post-channelization conditions are not considered in the SWP so these parameters in the equation do not apply. As such Equation 6.26 only considers distance to the next downstream grade control structure, the estimated reduction of sediment supply due to entrapment in upstream basins, and channel slope.

The existing channel slopes along Coyote Wash are as given in Table 4 below, along with the 2009 channel elevation change below drop structures, the average existing channel slope, calculated equilibrium slopes and estimated future degradation at the upstream end of each reach. **Please note that the long-term degradation estimates for the downstream most reaches have a high degree of uncertainty due to the large distances to the next downstream control point.**

Table 4: Long-Term Degradation Estimates for Coyote Wash

Reach Description	Reach Length (feet)	2009 Drop Height (feet)	Existing Slope (ft/ft)	Equilibrium Slope (ft/ft)	Future Long - Term Degradation Estimate (feet)
Kings Manor Wash Confluence to SR-90	5300	<2	0.011	0.0086	12.9
SR-90 to Avenida del Sol	1980	<2	0.0088	0.0069	3.9
Avenida del Sol to Sewer Line crossing at headcut location	3010	8	0.0074	0.0046	8.5
Sewer line crossing headcut to Foothills Dr.	1700	<1	0.010	0.0062	6.5
Foothills Drive to SR-92	1600	unknown	0.0097	0.0057	4.6
SR-92 to Camino Real	3460	5	0.012	0.074	16
Camino Real to Camino Rancho	1180	4	0.010	0.0062	4.5

Camino Rancho to Coronado Drive	2090	13	0.011	0.0068	8.8
Coronado Drive to Town and Country Drive	630*	5	0.010	0.0062	2.4
Town and Country Drive to Buffalo Soldier Trail	3170	unknown	0.019	0.012	23

* Distance is measured from upstream most grade control to Town and Country Drive

The equilibrium slope computation spreadsheets for Coyote Wash are provided in Appendix E.1. The following sections discuss reaches and structures most vulnerable to future degradation along with recommended mitigation measures.

9.4.1 Coyote Wash – Avenida del Sol to Sewer Line Crossing to 1/3 Mile Downstream of Foothills Drive

Significant channel bottom degradation has been occurring along most reaches of Coyote Wash upstream of Avenida del Sol. Channel invert elevation change between culvert outlets and downstream inverts is as high as 13 feet which indicate that severe degradation has been occurring over the last few decades since the culverts were installed.

The existing channel slope between Avenida del Sol and the concrete rubble that is presently serving as a temporary grade control structure is about 0.77%, although the slope within a few hundred feet of Avenida del Sol is 0.58%; possibly indicating that further headcutting should be expected along the reach extending to the existing dumped concrete serving as a temporary grade control. The distance between Avenida del Sol and this grade control structure is about 3,200 feet. Future degradation to a slope of 0.58% would suggest that an additional 8.5 feet of degradation could occur over the long-term.

Total long-term degradation at the sewer line which is the sum of past and predicted degradation at the existing broken concrete grade control, ranges between 14.1 feet and 17.5 feet.

Controlling degradation at this location is a high priority because of potential damages that could occur when the concrete rubble fails, including a sewer line breach, undercutting of the existing bank protection between the sewer line and Foothills Drive, and an increase in lateral erosion that can occur with degradation.

9.4.2 Description of Alternatives

The potential for more degradation and the presence of nearby infrastructure (a sewer line and bank protection) substantiates the need for an engineered grade control to prevent damage to these structures during future floods. Degradation can also lead to an increase in the rate of lateral erosion which portends the possibility of other damage to nearby property.

A grade control with a potential drop height of 14.1 feet to 17.5 feet requires one or more structures and a high degree of durability. The additional structures include bank protection along the downstream reach for a distance sufficient to extend beyond the zone of turbulence and grouted riprap or gabion aprons to control scour at the drop. Drops of lesser height will require similar structures but lesser toe downs, apron length and bank protection.

Normally, drop heights of 4-feet or more are not recommended but must be considered here because the existing drop height already exceeds this. The recommended approach for the sewer line crossing location is to install two or more drops of lesser height to achieve an overall drop height of 14.1- to 17.5-feet. The challenge here is to provide channel profile stability while recognizing that the time to long-term degradation conditions is unknown and flow frequency dependent. The least initial cost approach stabilizes the channel profile for existing conditions and for some degree of future degradation; allowing future additions to the structures when additional degradation occurs.

Drop structures can be constructed of soil cement or reinforced concrete and the slope face can vary from vertical to 4:1. Flatter face slopes are acceptable as well but become quite long which increases cost. Slopes constructed at 1:1 or flatter can be of reinforced concrete with a thickness of 6- inches or more, but long-term durability is questionable due to development of cracks and subsurface soil piping. This method is not recommended for drops higher than 3- to 4-feet.

Vertical concrete cutoff walls are more durable if reinforced with rebar and constructed at a width of 12-inchs or more. This method is also recommended for drops of 4-feet or less.

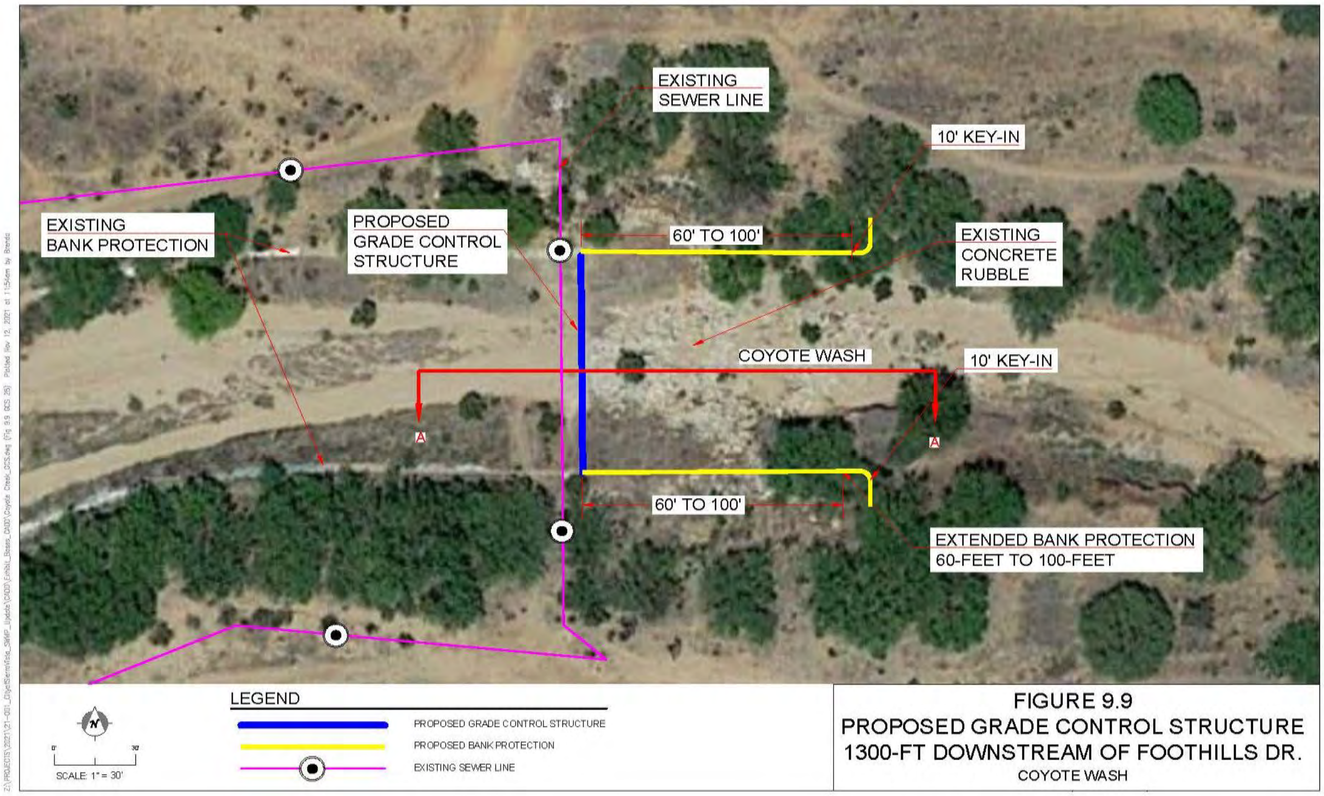
Soil cement is usually constructed at a 1:1 face slope at 8-foot thickness although, can be constructed at a 4:1 or flatter face slope at thicknesses of 18-inches or more. This material is recommended for all drop heights.

The bank to bank distance at the end of the existing concrete bank protection is about 125-feet. Any grade control structure will need to conform to this length and tie into the existing bank protection to insure stability.

CMG recommends that the grade control structure(s) be constructed of soil cement or as a vertical reinforced concrete wall with a drop height at each of 4-feet or less. In addition to cost, safety must be considered so a slope face of 4:1 or flatter should be considered if safety is a concern.

Figure 9.9 provides a plan view of the sewer line location and conceptual layout for future channel stabilization measures at this location. Three cross-sections' alternatives for an engineered grade control structure that were discussed above are provided on Figures 9.10 through 9.12.

FIGURE 9.9 – Plan View of Proposed Grade Control Structure at Coyote Wash Sewer Line Crossing



Grade Control Alternative 1 – Figure 9.10 depicts a grade control structure constructed of 8-foot thick soil cement with a 1:1 sideslope. This type of material and construction is common in southern Arizona but usually placed as bank protection, however, it has previously been used for grade controls. Extension of the existing bank protection at least 60-feet downstream of the drop is needed to prevent bank erosion within the zone of turbulence, or out flanking of the structure.

If Alternative 1 is the preferred approach, then it is recommended that the construction be a one-time event (without phasing based on the rate of degradation). This does not allow the City to defer some of the cost to a later date, but mobilization and site preparation expenses associated with setting up a soil cement batch plant are minimized.

Safety is also a concern due to the 1:1 sideslope which can only be addressed with warning signs. Flatter slopes are possible but will consume more space by requiring longer downstream bank protection and grouted riprap aprons.

The cost estimate spreadsheet is provided in Appendix D.3.

Estimated Construction Cost 2021 - $435,369

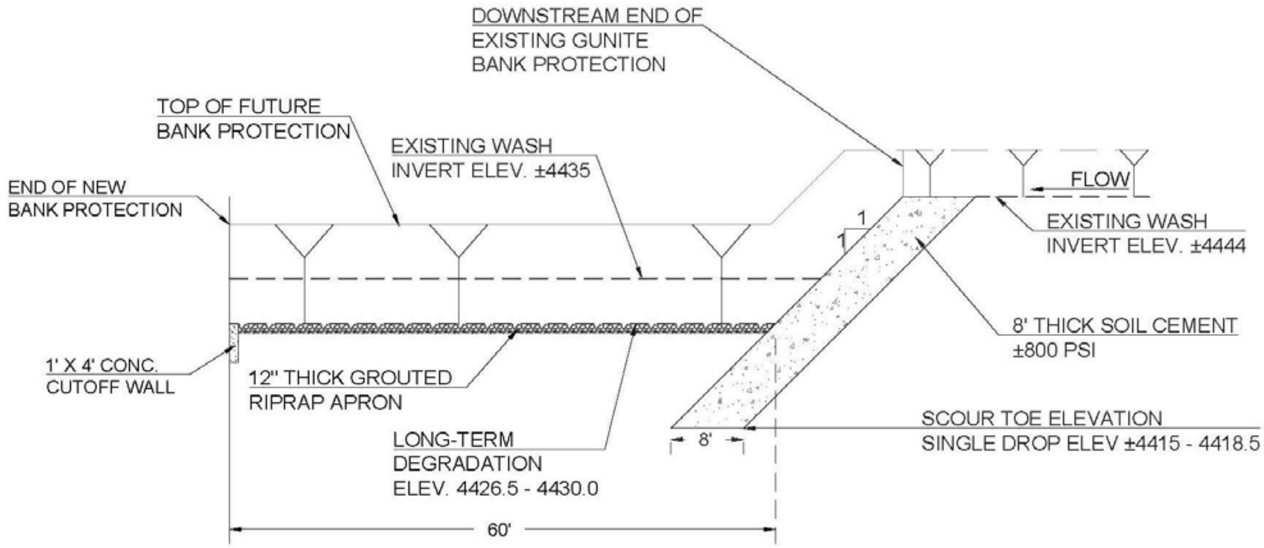


FIGURE 9.10
COYOTE WASH GRADE CONTROL STRUCTURE ALTERNATIVE 1
N.T.S

Grade Control Alternative 2 – Figure 9.11 depicts a grade control structure constructed as a series of reinforced concrete walls having a minimum thickness of one foot. This type of material and construction is common for grade controls having a vertical drop height of no more than 4-feet (preferably 3-feet). Extension of the existing bank protection for about 80- to 100-feet downstream of the first drop is needed to prevent bank erosion with the zone of turbulence, or out flanking of the structures. Grouted riprap is recommended between drops to prevent scour which then minimizes the depth of the walls below finished grade. The drop spacing as shown on Figure 9.11 is a minimum of 16-feet to 20-feet to provide access for maintenance purposes.

One advantage of Alternative 2 is that it can be phased based on the rate of degradation, which allows the City to defer some of the cost to a later date. Mobilization and site preparation expenses if construction occurs in two or more steps and concrete material costs will be greater since at least 3 concrete drop structures rather than one will be needed. Additional structures can be added if future degradation results in more than a 4-foot drop.

Safety is also a concern due to the vertical drops, but the height of each drop will only be 3- to 4-feet.

The cost estimate spreadsheet is provided in Appendix D.3.

Estimated Construction Cost 2021 - $315,260.

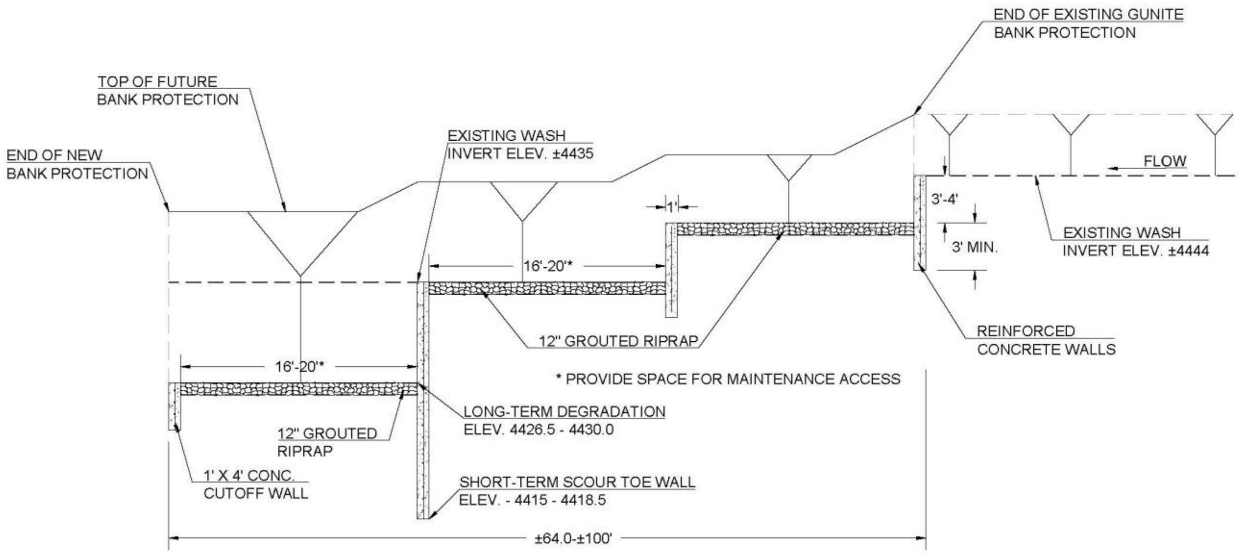


FIGURE 9.11
COYOTE WASH GRADE CONTROL STRUCTURE ALTERNATIVE 2
N.T.S.

Grade Control Alternative 3 – Figure 9.12 depicts a grade control structure constructed of 3-foot thick soil cement with a 4:1 sideslope, or flatter. Slope paving at thicknesses less than 8-feet can be constructed on milder slopes starting at 4:1. Bank protection for about 80- to 100-feet downstream of the drop is also needed to prevent bank erosion within the zone of turbulence, or out flanking of the structure. Public safety increases compared to Alternatives 1 and 2 due to the 4:1 grade control structure slope. Total length of the structures will be greater than Alternatives 1 and 2 due to the 4:1 grade control structure slope.

If Alternative 3 is the preferred approach, then it is recommended that the construction occur once to avoid additional mobilization and site preparation expenses associated with setting up a soil cement batch plant (without phasing based on the rate of degradation). In addition, a new toe wall will be required at the downstream limit of each drop which is an additional expense.

The cost estimate spreadsheet is provided in Appendix D.3.

Estimated Construction Cost 2021 - $331,815.

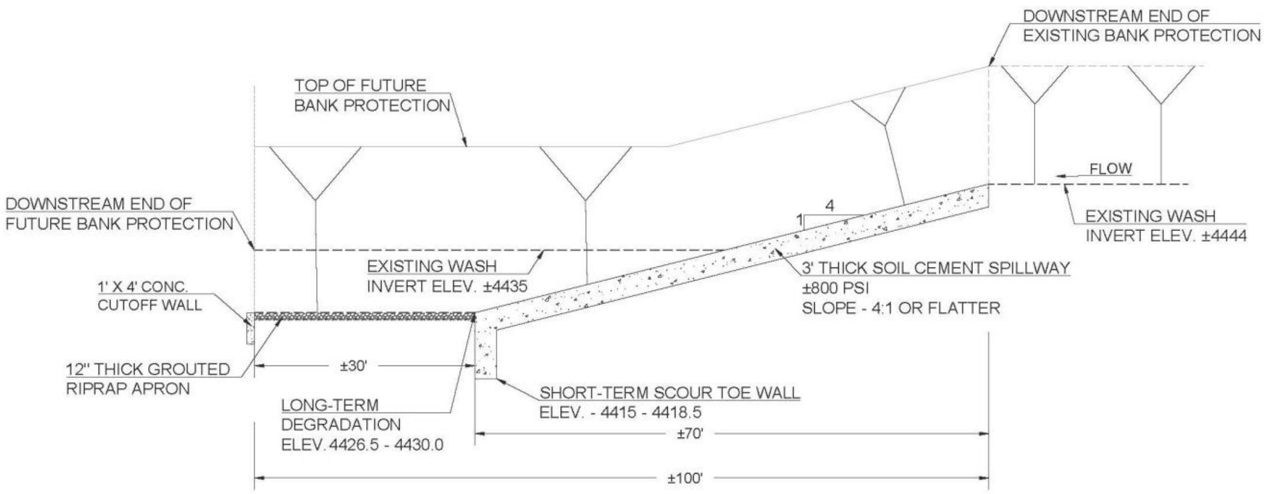


FIGURE 9.12

COYOTE WASH GRADE CONTROL STRUCTURE ALTERNATIVE 3
N.T.S.

9.4.3 Coyote Wash at Camino Real

The channel invert drop height downstream of the small diameter culvert at Camino Real was about 5 feet in 2009 and the existing grade control shows signs of failure. Equilibrium slope computations estimate a future equilibrium slope at 0.62% which implies significantly more degradation up to (16-feet) given the distance to the next downstream grade control.

Optimally, at least one or two more grade controls should be installed between SR-92 and Camino Real. Locations have not yet been identified but they should generally be equally spaced between the reach limits. The typical grade control cross-sections and profile shown on Figures 9.13 and 9.14 can be used with the top elevation conforming to channel bottom elevation and the toe of structure being at least 3 feet below the long-term degradation elevation.

Maintenance, monitoring and repairs of the drop structures at Camino Real is important. The repairs should include the addition of a cutoff wall at the toe of the drop to prevent undercutting that could result from scour and degradation. The top of the cutoff wall should conform to the downstream channel invert elevation and have a depth of 8-feet below the top which should be sufficient for many years, but another nearby structure could be needed when future degradation exceeds another 4-feet. Grout should be added between the top of the cutoff wall and the existing grouted rock as demonstrated on Figure 9.15.

A cost estimate for this is not provided since the construction scope is small and there are options for controlling future degradation along this reach. The ballpark estimate for the cost of minor channel stabilization measures like standalone grade control

structures is $10,000 to $40,000. Depending on material volume, material source and whether or not City maintenance staff conduct the work.

9.4.4 Coyote Wash at Camino Rancho

The channel invert drop height downstream of the dip crossing at Camino Rancho was about 4 feet in 2009 and the existing grade control shows signs of failure. Equilibrium slope computations estimate a future equilibrium slope at 0.62% which implies more degradation can be expected.

Maintenance, monitoring and repairs to the drop structures at Camino Rancho should suffice for the near-term. The repairs should include the addition of a cutoff wall at the toe of the drop to prevent undercutting that could result from scour and degradation. The top of the cutoff wall should conform to the downstream channel invert elevation and have a depth of 8-feet below the top. Grouted riprap should be added between the top of the cutoff wall and the existing grouted rock as demonstrated on Figure 9.15.

9.4.5 Coyote Wash – Camino Rancho to Coronado Drive

Upstream of the Camino Rancho dip section, there is a concern that degradation will continue further increasing the drop height at Coronado Drive which was about 13 feet in 2009. The average channel slope along this reach is about 1.1%. Equilibrium slope computations estimate a future slope of 0.88% which implies 8.8 feet of additional degradation could occur at Coronado Drive.

A minimum of two (preferably three) additional grade control structures should be constructed between Camino Rancho and Coronado Drive. Design drop height for these grade controls should be 3 to 4 feet and the typical cross-section and profile are shown on Figures 9.13 and 9.14. The location of these grade control should be that they are roughly spaced equally between Camino Rancho and Coronado Drive although some deviation is acceptable to account for access or property ownership constraints.

Maintenance, monitoring and repairs to the drop structures at Coronado Drive is especially important given the current drop height of 13-feet. The repairs should include the addition of a cutoff wall at the toe of the drop to prevent undercutting that could result from scour and degradation. The top of the cutoff wall should conform to the downstream channel invert elevation and have a depth of 8-feet below the top which should be sufficient for many years, but another nearby structure could be needed when future degradation exceeds another 4-feet. Grout should be added between the top of the new cutoff wall and the existing grouted rock as demonstrated on Figure 9.15.

9.4.6 Coyote Wash - Coronado Drive to Town and Country Drive

Three grade control structures have been constructed along the reach upstream of Coronado Drive to Town and Country Drive which have a total drop height of another 13 feet. Equilibrium slope for the 630 foot distance from the upstream most grade control to Town and Country is estimated to be about 0.62% which implies another 2.4 feet of long-term degradation at Town and Country Drive.

Maintenance, monitoring and repairs to the three drop structures between Coronado Drive and Town and Country Drive should probably suffice for the near-term. The repairs should include the addition of a cutoff wall at the toe of the drop to prevent undercutting that could result from scour and degradation. The top of the cutoff wall should conform to the downstream channel invert elevation and have a depth of 5-feet below the top. Grout should be added between the top of the cutoff wall and the existing grouted rock as demonstrated on Figure 9.15, if needed. These same comments apply to the existing drop structure at Town and Country Dr.

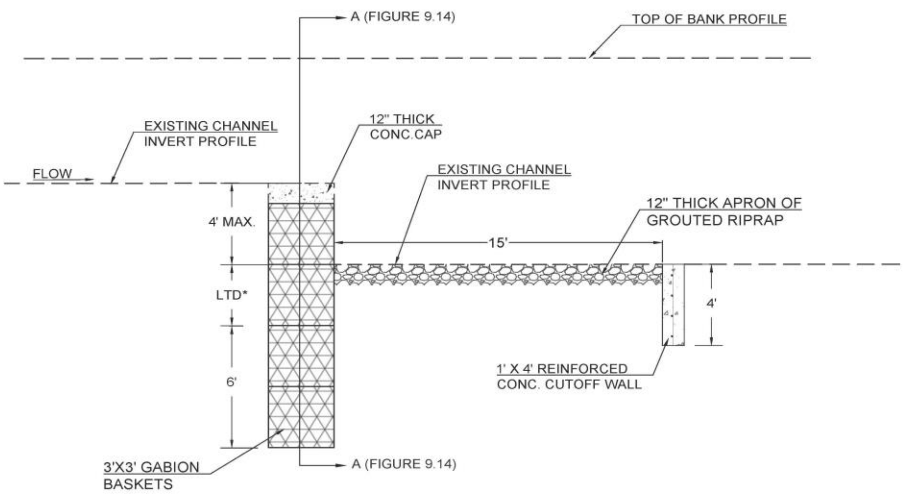


FIGURE 9.13

TYPICAL PROFILE FOR FOR NEW GRADE CONTROL STRUCTURES WITH DROP HEIGHT OF 4-FEET OR LESS

N.T.S.

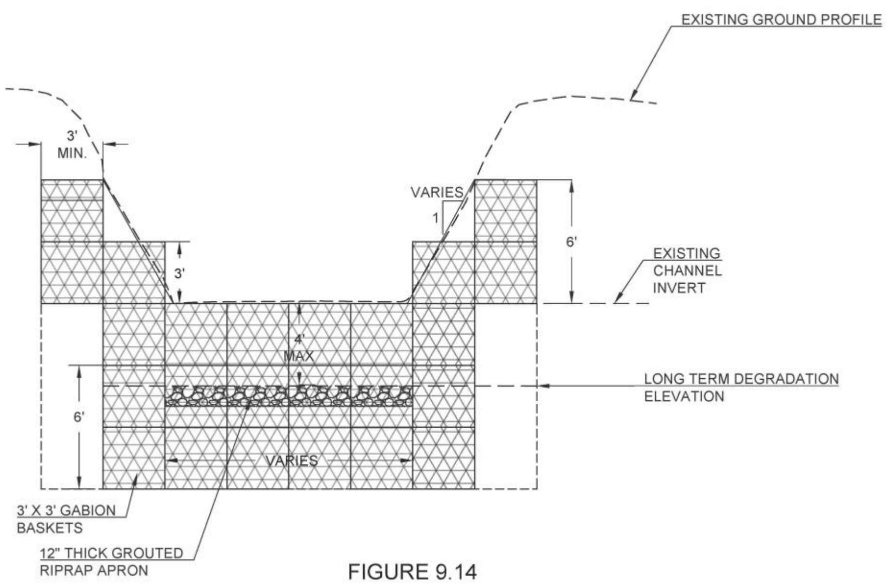


FIGURE 9.14

CROSS-SECTION A-A

TYPICAL SECTION FOR NEW GRADE CONTROL STRUCTURES WITH DROP HEIGHT OF 4-FEET OR LESS

N.T.S.

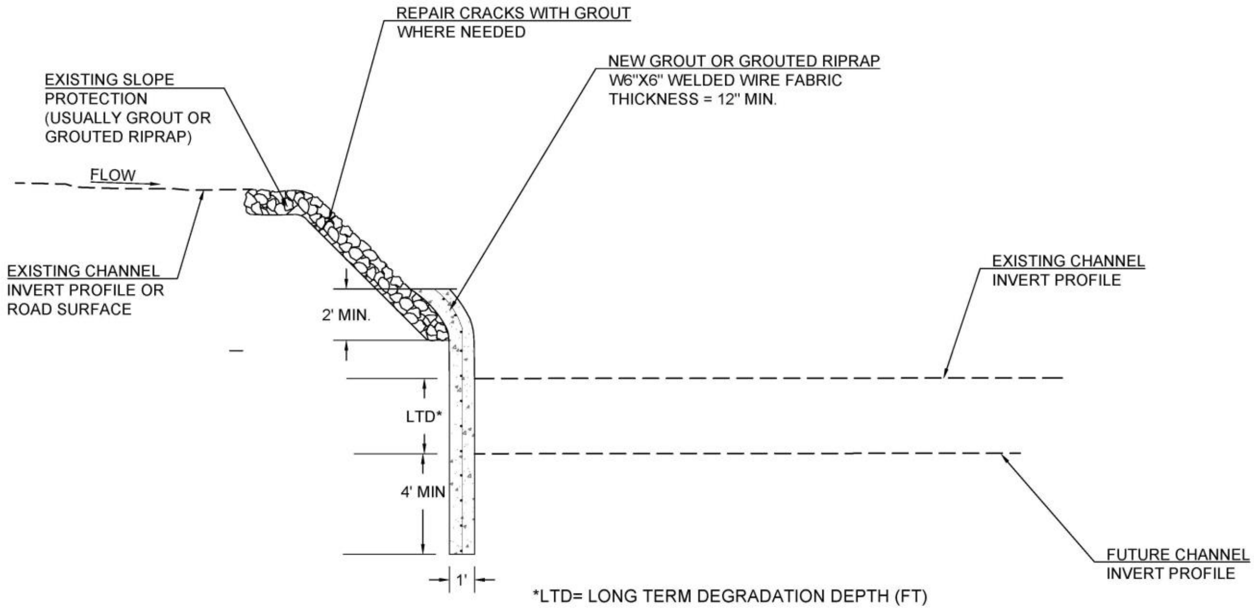


FIGURE 9.15
TYPICAL PROFILE FOR REINFORCEMENT OF EXISTING GRADE CONTROL STRUCTURES
N.T.S.

9.4.7 Coyote Wash - Town and Country Drive to Buffalo Soldier Trail

The equilibrium slope computations predict long-term degradation at Buffalo Soldier Trail to be 23 feet. This estimate seems contrary to fielded observations which found past degradation to be only about 2 feet. As such, no structural controls are necessary in the near term. The recommendation of this report is to conduct annual monitoring to determine if headcutting processes have begun, and at which time the need for grade control structures can be evaluated.

9.4.8 Coyote Wash Summary Recommendations

Severe degradation has been occurring along Coyote Wash upstream of Avenida Del Sol and is particularly evident along the reach about 1/3 mile downstream of Foothills Drive. Previous discussions have noted an 8-foot deep headcut just downstream of where a sewer line crosses the wash; there is also bank protection along banks where residential structures are present extending west to near Foothills Drive. The sewer line and the bank protection will be threatened if the broken concrete which is presently serving as the grade control fails. The existing measures are pieces of dumped concrete that will not provide channel profile stability over the long term. **Replacing the dumped concrete with an engineered grade control structure should be a high priority for the community.**

9.5 Charleston Wash / 3rd Street Drainageway – SR-90 to Fry Blvd.

Significant degradation has been occurring along Charleston Wash and 3rd Street Drainageway upstream of SR-90. The 3rd Street Drainageway is grade stabilized at a point about 560-feet downstream of Fry Blvd where there is a grouted spillway with a height of about

13-feet (see Figure 4.8.6). Gabion bank protection was also installed to control bank erosion adjoining the drop structure. These gabions appeared in good condition during the 2021 field inspections. In 1984/1985 the City constructed a grade control structure at this location to control ongoing degradation. According to the plans, the design drop height was about 3.5 feet and the downstream toe down for bank protection and the cutoff wall depth was 3-feet below channel invert elevation. In 2009, the drop height had increased to 13-feet which has required the City to periodically add concrete to the bottom of the drop to prevent degradation from undermining the structure. Long-term degradation computations which are discussed below, estimate that an additional 8.3 feet of degradation could occur in the future.

Culverts at Lenzner Avenue and Fry Blvd also act as grade controls; the existing drop heights at the locations are 6.3-feet and 8-feet, respectively. The reach of greatest concern is upstream Lenzner Avenue where degradation and lateral migration continue. The City installed about 165-feet of gabion bank protection along a reach of the drainageway where lateral migration threatened a sewer line and adjoining homes. The erosion occurred at a location along the outside bank of a meander bend where the force of flow was directed. The location for this bank erosion control project is shown on Figure 9.16.

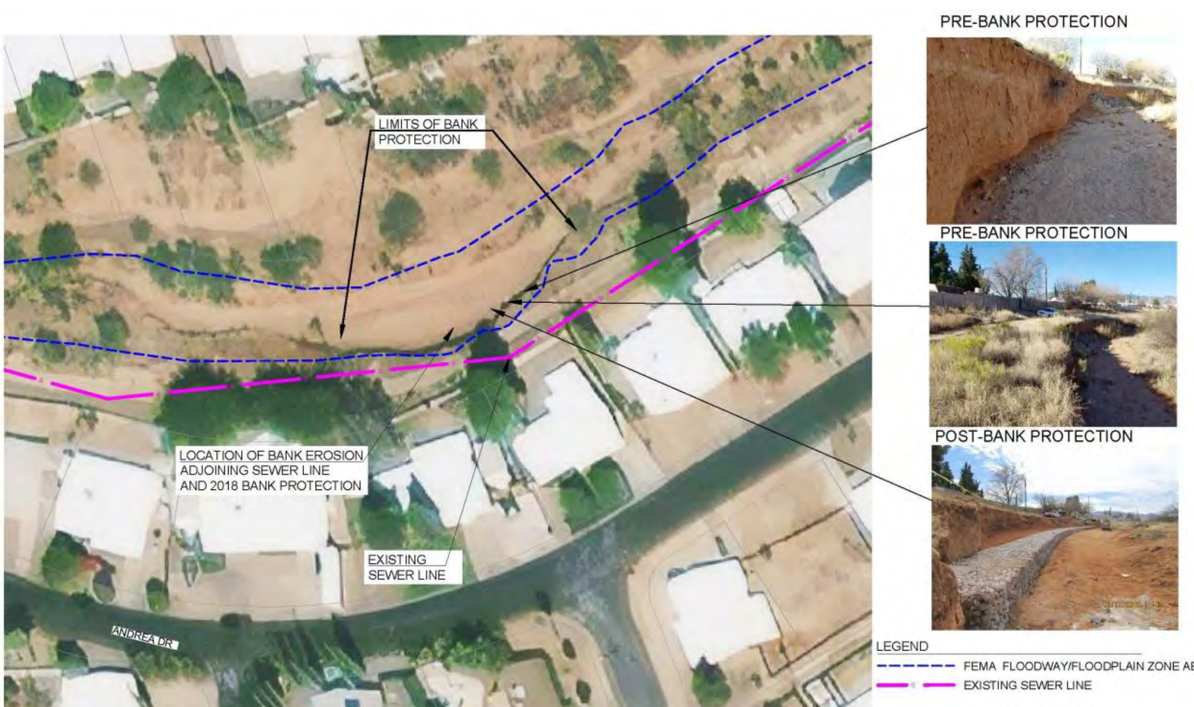


FIGURE 9.16
3RD ST DRAINAGEWAY BANK EROSION CONTROL PROJECT

Degradation along this wash has been associated with urbanization and clear water flows caused by upstream detention which traps sediment supplied from the watershed. Because of this, the sediment deficit is derived from the channel bottom and banks causing degradation and bank erosion. These erosional processes are expected to continue during future storm water flows.

Since these processes will continue, reinforcement of existing erosion control structures will be needed. Bank erosion and lateral migration should also be monitored annually and after major floods to ensure public infrastructure is protected until reinforcement measures can be installed.

9.5.1 Estimation of Future Degradation along Charleston Wash and the 3rd Street Drainageway

Equilibrium channel slope was calculated using Equation 6.26 of the City of Tucson Drainage Design Standards Manual. Please refer to section 9.4 and Appendix E of this report for additional information regarding Equation 6.26.This methodology estimates the slope needed to balance sediment supply with sediment transport capacity. Long-term degradation is calculated as the difference between existing slope and equilibrium slope, times the reach length.

The existing channel slopes along Charleston Wash and the 3rd Street Drainageway are as given in Table 5 below, along with the existing channel elevation change below drop structures, the average existing channel slope, calculated equilibrium slopes and estimated future degradation at the upstream end of each reach. Future degradation at the existing sewer line crossing between Coronado Drive and SR-90 was calculated with the culvert and cutoff wall at Avenida Escuela which was constructed in 2022.

Table 5: Long-term Degradation Estimates for Charleston Wash and 3rd Street Drainageway

Charleston Wash					
Reach Description	Reach Length (feet)	2009 Drop Height (feet)	Existing Slope (ft/ft)	Equilibrium Slope (ft/ft)	Future Long - Term Degradation Estimate (feet)
Colombo Avenue	>10,000 ft	4 ft	0.95%	Unknown	>10 ft
Colombo Ave to SR-90	2300 ft	11 ft	1.30%	0.77%	11.0 ft
SR-90 to Avenida Escuela	1330 ft	0 ft	0.85%	0.52%	4.3 ft
Avenida Escuela to Sewer Line Grade Control	900 ft	8 ft	0.85%	0.52%	2.9 ft
Sewer Line Grade Control to 200-ft downstream of Coronado Dr.	3915 ft	6 ft	0.70%	0.43%	10.5 ft
3rd Street Drainageway					
Coronado Dr. to Grade	800 ft	2.2 ft	0.51%	0.31%	1.4 ft

Control above Woodcutter Confluence					
Grade Control above Woodcutter Confluence to Lenzner Avenue	1660 ft	5 ft	0.77%	0.47%	4.9 ft
Lenzner Avenue to Grade Control Structure 460 feet downstream of Fry Blvd.	2955 ft	8.2 ft	0.73%	0.45%	8.3 ft
Grade Control Structure 460 feet downstream of Fry Blvd. to Fry Blvd.	460 ft	5.1 ft	1.00%	0.45%	2.5 ft

The equilibrium slope computation spreadsheets for Charleston Wash and the 3rd Street Drainageway are provided in Appendix E.2.

9.5.2 Description of Possible Locations for New Grade Control Structures

Existing drop heights, as well as estimates for additional degradation over long-term, substantiate the need for more engineered grade controls to prevent damage to structures during future floods. As previously noted, degradation can also lead to an increase in the rate of lateral erosion which portends the possibility of damages to nearby property and structures.

Possible locations for additional grade control structures were identified based on the long-term degradation estimates summarized in Table 5. Findings are summarized in Table 6 below. Please note that Table 6 only lists locations where grade control structures are not currently present. Recommendations for repair or replacement of some of the existing structures are given in Section 9 of this SWP.

Table 6: Possible Locations for new Grade Control Structures along Charleston Wash and 3rd Street Drainageway

Location	Comments
Extension of Giulio Cesare Ave across Charleston Wash	A grade control at this location could be incorporated into a future road crossing and significantly reduce the potential for future degradation at Colombo Avenue.
Downstream of the Sewer Crossing between Port Royale Apts and U of A property	Site is about 1400 ft downstream of Colombo Ave. Would significantly decrease degradation potential at Colombo Ave and protect sewer line from possible failure.
Downstream of Coronado Drive	Grade control structures are already present here, but long-term degradation estimates suggest another 10.5 ft of bed elevation lowering may occur. An additional grade control structure is recommended between Avenida Escuela and the existing grade control protecting the sewer line crossing.

9.5.3 Charleston Wash/3rd Street Drainageway Existing Structure Reinforcement Priority List

The recommendations for additional grade control structures given in Table 6 above are intended to protect existing infrastructure like sewer lines and reduce future degradation at existing grade controls. However, there is a need to reinforce some of the existing grade controls with or without new structures to prevent failure due to degradation. The priority list for modifying existing grade controls is described below.

9.5.3.1 Existing Grade Control Structure 560-feet Downstream of Fry Blvd.

This structure may be subject to an additional 8.3 feet of long-term degradation (without new structures) based on the equilibrium slope computations, bringing the total drop height at this location to 17.2 feet. Two alternatives are available, (1) add additional toe down to the existing structure in preparation for additional degradation of up to 8.3 feet, or (2) stabilize the existing structure as needed to protect it, but also provide new structure(s) downstream to reduce degradation potential below the existing structure.

The recommended approach is to install two new grade controls downstream of the existing structure and to modify the existing structure (see Figure 9.17). This is because parts of the existing structure are not engineered and could become more vulnerable to structural failure if the drop height increases another 8.3 feet. The recommendations are:

- Extend the existing concrete apron at the toe to the downstream limit of the existing gunite and gabion bank protection.
- Install a new grade control structure designed for another 4-feet of degradation at the downstream limit of the existing bank protection. Toe wall depth below existing grade should be 6-feet minimum.
- Install concrete bank protection between existing grade control and new grade control on the east bank where existing bank protection is hanging on the upper slope.

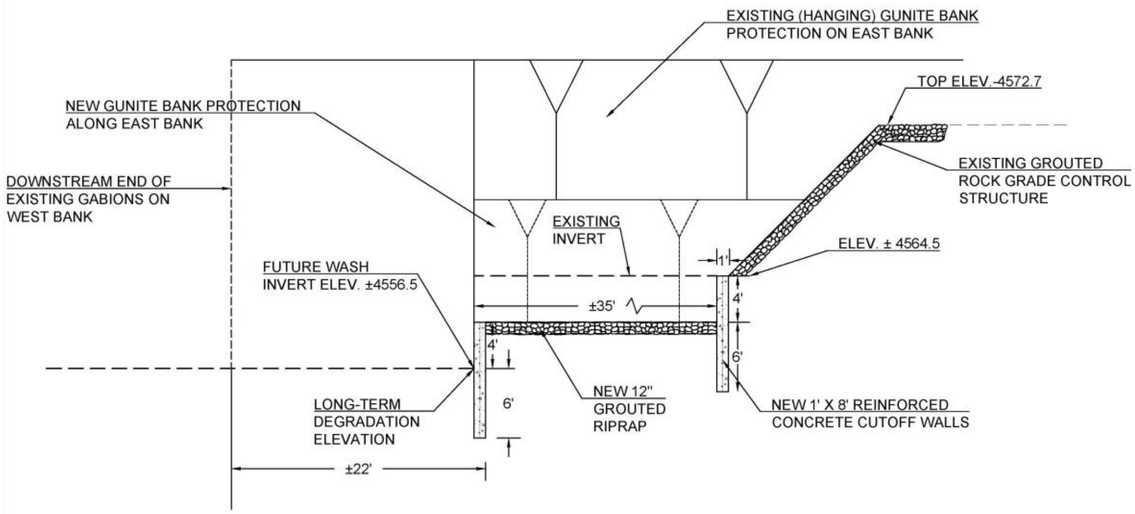


FIGURE 9.17
3RD STREET DRAINAGEWAY GRADE CONTROL STRUCTURE ±800' NORTH OF FRY BLVD. MODIFICATIONS
N.T.S.

9.5.3.2 Colombo Avenue Culvert Outlet

No grade control structure is present at the Colombo Avenue culvert outlet, only a grouted riprap apron which does not have a concrete cutoff wall at its downstream limit (see Figure 9.18). The existing drop height from the end of the riprap apron to channel invert is about 3-feet. Long-term degradation at this location cannot be estimated because there are no downstream structures that define the nearest grade stabilization point. Because of this, it should be anticipated that future degradation may be well in excess of 10-feet. The recommendation of this report is to install a new 10-foot reinforced concrete cutoff wall (±7-feet of which is below existing grade) or gabion baskets to provide some interim measure of scour protection at this location; with the expectation that another similar structure will be needed in the future. These grade control structure recommendations are shown on Figure 9.19 and 9.20.

FIGURE 9.18 - Colombo Avenue Culvert Outlet

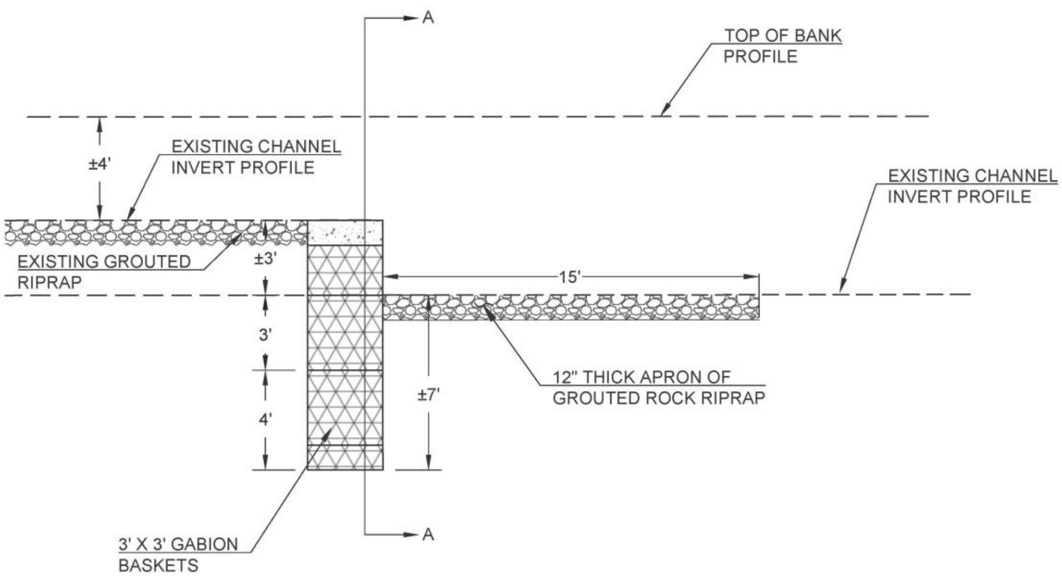


FIGURE 9.19
TYPICAL PROFILE FOR NEW GRADE CONTROL STRUCTURE
AT COLOMBO AVE CULVERT OUTLET
N.T.S.

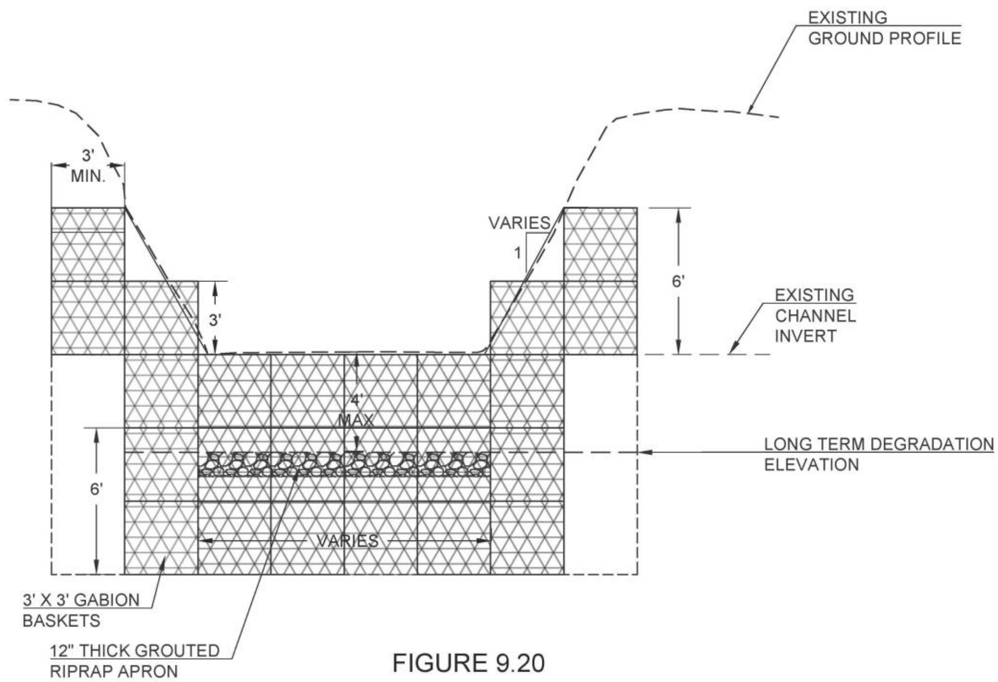


FIGURE 9.20

CROSS-SECTION A-A
TYPICAL SECTION FOR NEW GRADE CONTROL STRUCTURE
AT COLUMBO AVE CULVERT OUTLET
N.T.S.

9.5.3.3 Existing Grade Control Structure at Sewer Line Crossing Between Coronado Dr and SR-92

The existing grade control structure which protects the sewer line that crosses Charleston Wash between SR-90 and Coronado Drive currently has a drop height of about 9-feet. This drop height is about two feet more than the 2009 topography indicates. According to the as-built plans, the total height of the grade control is about 15-feet so the toe depth below the downstream channel elevation remains to be about 6-feet. According to the plans, the sewer line is only about 2 to 3' feet below existing grade just west of the gabion drop structure. Long-term degradation was estimated to be another 2.6 feet with the Avenida Escuela culvert and drop structure now in place.

Gabion bank protection present along the downstream reach has been damaged and should be replaced. It appears that the damage was caused by flows outflanking the gabions which caused erosion on their back side.

The critical importance of this structure suggests that the existing grade control should be repaired and augmented to ensure future degradation does not threaten failure. The recommended approach is to (1) remove the damaged gabions and replace with new gabions, (2) install new gabion bank protection along a 30-foot reach upstream of the drop structure to prevent outflanking and future damages to the bank protection along the downstream reach, and (3) construct a new grouted riprap apron and concrete cutoff wall to prevent future undermining of the existing drop structure. A profile for the new grade control is provided in Figure 9.21. The recommended toe down depth is 5-feet as shown on Figure 9.21.

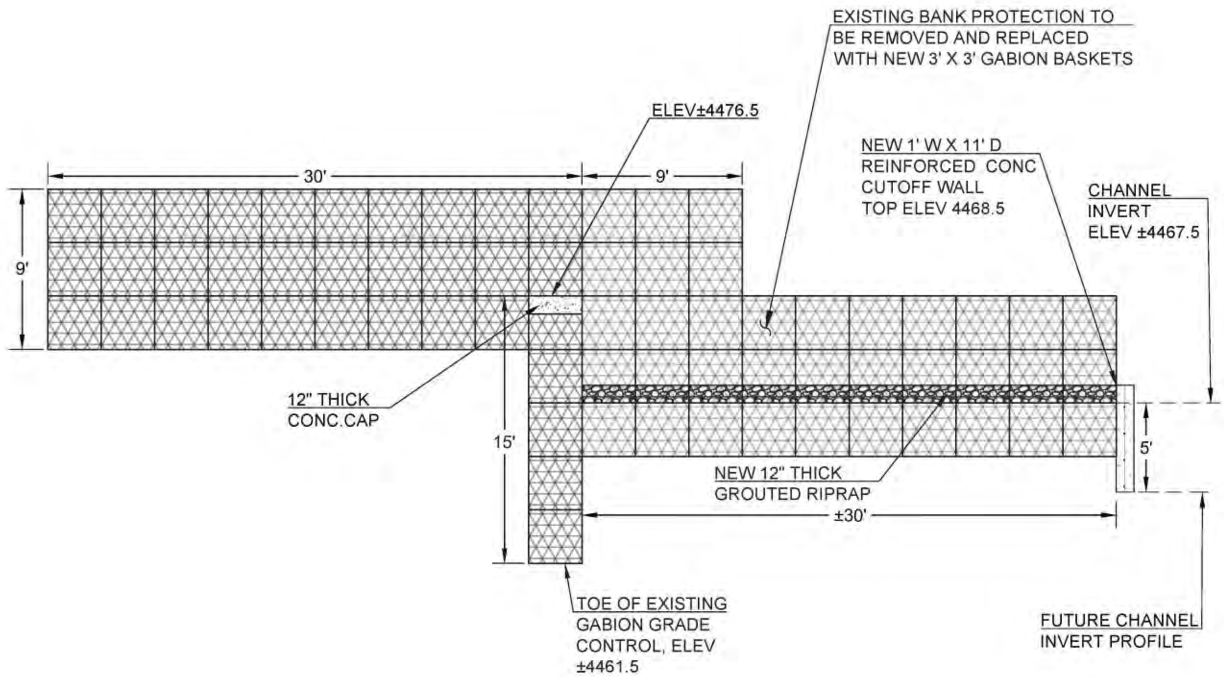


FIGURE 9.21
PROFILE OF EXISTING AND RECOMMENDED CHARLESTON WASH GRADE CONTROL AT SEWER LINE CROSSING
N.T.S.

9.5.3.4 Existing Grade Control 200-ft Downstream of Coronado Dr.

This grade control appears to have been installed to protect bank protection just downstream of Coronado Drive. The current drop height at this location is about 6-feet and field inspections indicate that the downstream channel invert is below the toe of the concrete by several feet. Modifications to this grade control are needed to prevent a future failure which could also undermine the concrete channel at the outlet of the Coronado Drive culverts. A photograph of existing conditions and a typical profile of the recommended reinforcement measures are given in Figures 9.22 and 9.23.

Figure 9.22 - Existing Grade Control 200-ft Downstream of Coronado Dr

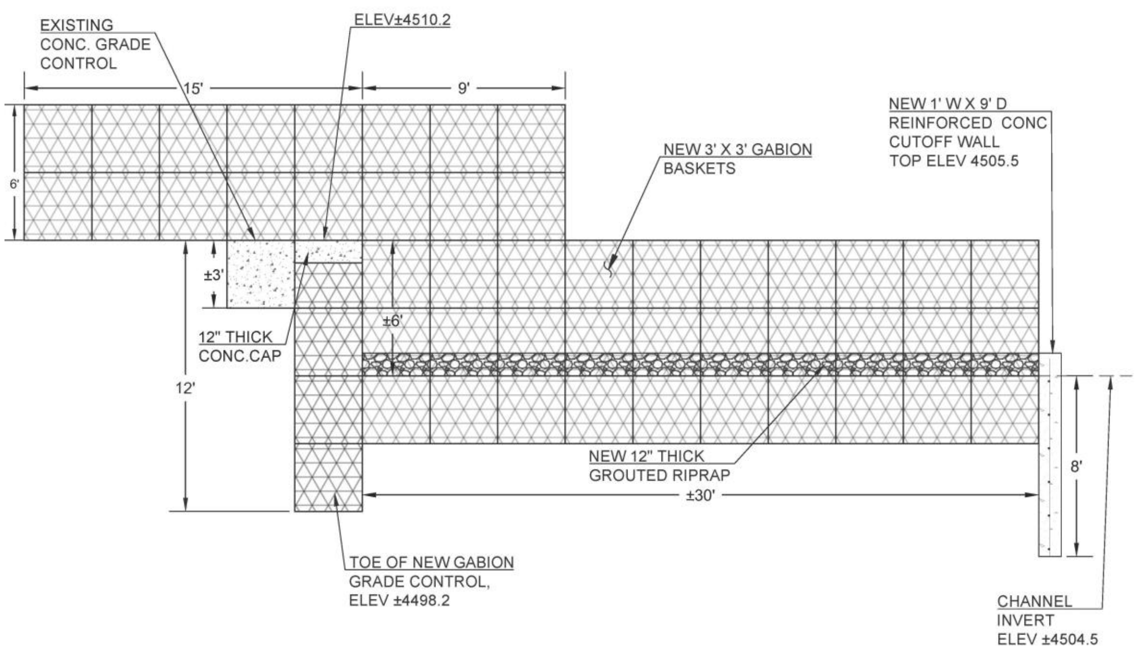


FIGURE 9.23
PROFILE OF RECOMMENDED CHARLESTON WASH GRADE CONTROL 200'
DOWNSTREAM OF CORONADO DR.
N.T.S.

9.5.3.5 Existing Grade Control 800-ft Upstream of Coronado Dr.

A grade control structure is present just above the confluence of 3rd Street Drainageway and Woodcutters Wash; located near the upstream end of the existing bank protection. This grade control is only about 2- to 3- feet deep and scour has already lowered the channel invert below the toe of the structure. Near-term failure is probable because flow is already passing beneath the bottom of the structure. Future degradation is only predicted to be another 2.2-feet; however, local short-term scour of up to 8.9 feet could occur due to flow passing over the drop. Therefore, it is recommended that a new reinforced concrete or gabion grade control with a toe depth of at least 9-feet be installed as shown on Figure 9.24. The horizontal limits of the grade control should tie into the existing bank protection. The toe down depth for the existing bank protection is unknown.

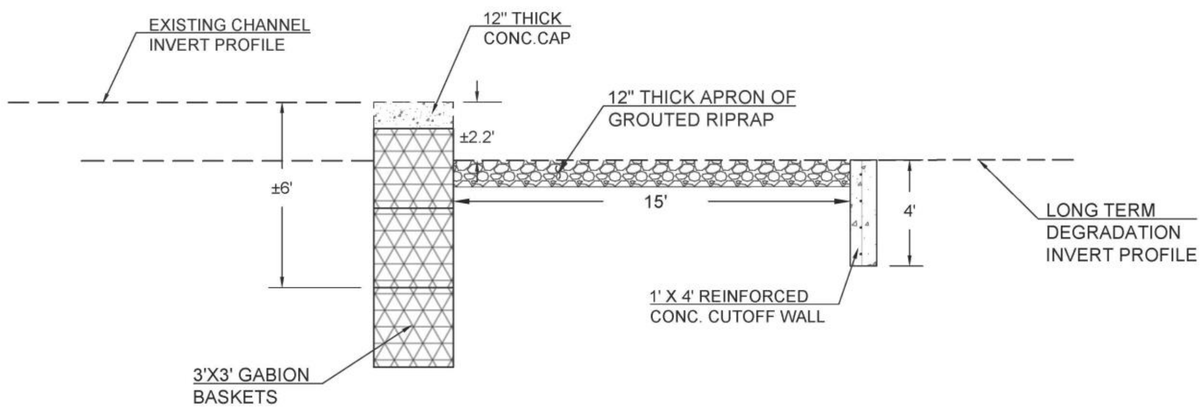


FIGURE 9.24
TYPICAL PROFILE FOR NEW GRADE CONTROL STRUCTURE
ALONG CHARLESTON WASH 800' UPSTREAM OF CORONADO DRIVE
N.T.S.

9.6 Sulger Subdivision Flooding

As discussed in Section 4.1 of this report, the Sulger subdivision has experienced shallow flooding through residential lots for several decades due to the absence of any storm water facilities other than the homemade channels and swales each property owner has constructed on their lots. Homeowners have found this necessary due to the absence of drainage facilities and curbs along the streets. Lot to lot drainage is common.

The City recently constructed a detention basin just south of Timothy Lane to capture flow draining into the subdivision. Detention routing computations as a part of this report estimated the basin outflow peak during the 100-year storm to be 9 cfs. Hydrologic modeling conducted by the City estimated that a 100-year discharge of 98 cfs enters the subdivision along its south boundary. This is not a large amount of flow but conveyance structures through the subdivision have much less capacity and are discontinuous. Homeowner comments regarding the summer storms of 2021 suggest that storm water runoff from areas of the watershed that do not drain to the newly constructed basin still pass through the subdivision and are the primary source of storm water affecting the subdivision.

Alternative solutions that address the current storm water runoff conditions are limited due to the dispersed nature of the stormwater runoff sources. One possible alternative would be to lower the street elevations and install curbs to contain flow, however, a much more detailed analysis of this approach is required to determine benefits and cost. Lot to lot drainage inherent in the subdivision design is not fully resolved by this approach.

Storm water runoff from areas between Carmichael Avenue and Buffalo Soldier Trail drain to Sheila Ln, Danny Ln and Jennifer Ln, then through the subdivision lots towards the Busby Drive/Judd Street intersection. Flow then crosses Busby Drive in a dip section and enters the upstream end of the 3rd Street Drainageway.

Three points of concentration were identified along the west subdivision boundary as shown on Figure 9.25. Offsite flows entering the subdivision at the above listed intersections were 91, 53 and 36 cfs respectively, for the 1-hour/100-year storms and 46, 30, and 21 cfs for the 10-year storm.

One alternative that was evaluated on a very preliminary basis was the construction of an underground storm drain system to collect offsite flows entering the subdivision along its west boundary (Carmichael Avenue).

Figure 9.25 also shows a concept plan for catch basins and storm drainpipes to collect storm water along Carmichael Avenue and at concentration points within the subdivision. Catch basin locations within the subdivision are located at low points and street intersections where flow may accumulate. Approximate storm drainpipe diameters are also given for capacities approximately equal to the 10-year storm discharges.

There are two important issues associate with this alternative and any other storm drain layout that probably makes this approach cost prohibitive; those being (1) curbing throughout most of the subdivision streets is needed to contain and direct flow to the catch basins, and (2) sewer lines are present along all streets. The scope of work for this study does not permit sufficient analyses to determine whether or not the sewer lines prohibit construction of a storm drain system.

An approximate cost estimate was made assuming design feasibility given some level of sewer line modifications will be needed and curbing is installed. **The preliminary cost estimate for the storm drain shown on Figure 9.25 is $1,062,721.** This estimate should be considered only as an order of magnitude cost given the unknowns associated with it, particularly utility relocations.

The cost estimate spreadsheet is provided in Appendix D.4

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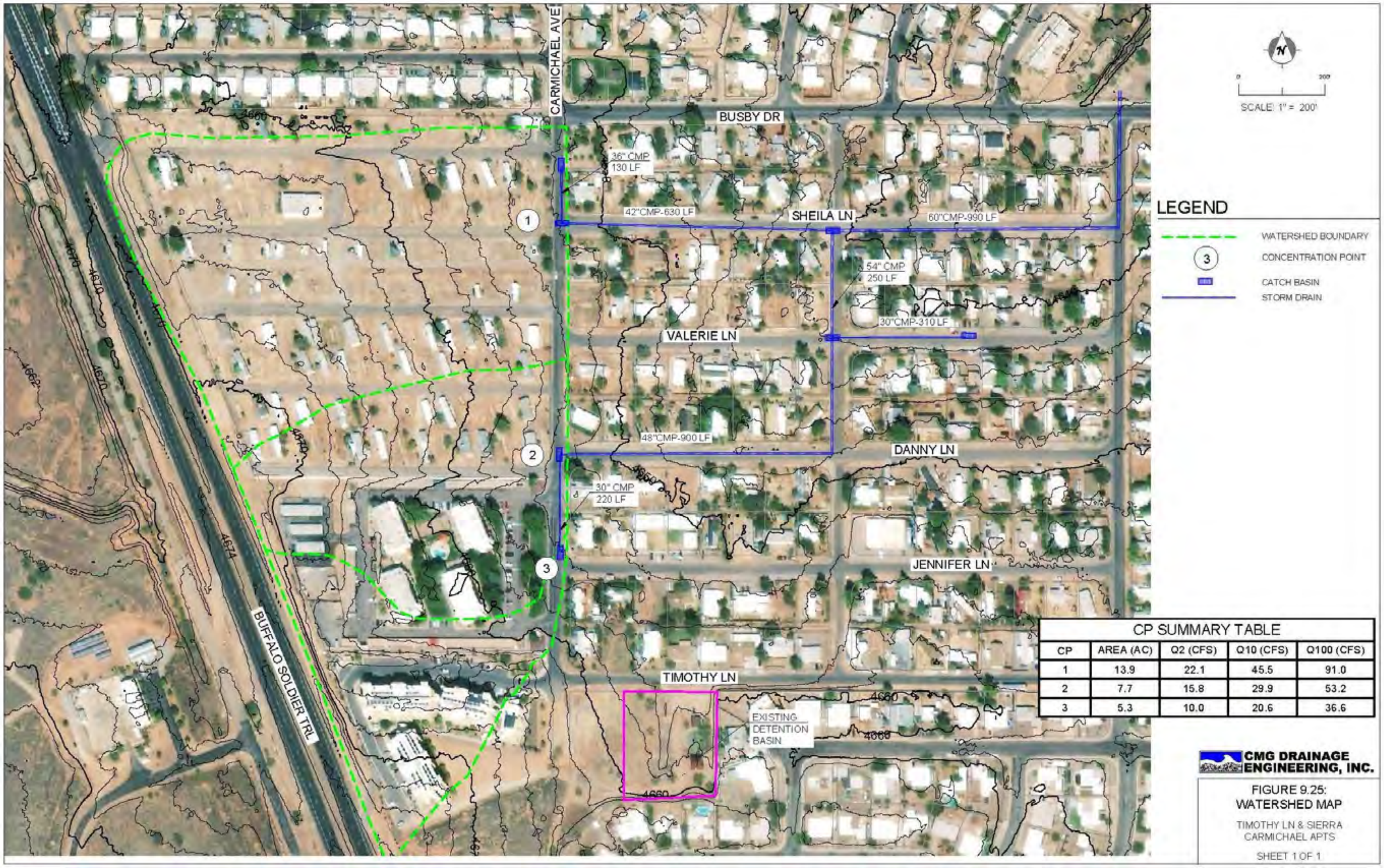


CP SUMMARY TABLE

CP	AREA (AC)	Q2 (CFS)	Q10 (CFS)	Q100 (CFS)
1	13.9	22.1	45.5	91.0
2	7.7	15.8	29.9	53.2
3	5.3	10.0	20.6	36.6

9.7 Soldier Creek - Buffalo Soldier Trail to SR-90

Problem Description - Hydrologic modeling using HEC-HMS has estimated the 100-year discharge for Soldier Creek to be in excess of 4,000 cfs. Overbank flooding occurs along this reach due to inadequate channel capacity and it is estimated that at least 32 residential structures and 4 commercial buildings are within the FEMA floodplain along this reach. Most of these structures are located east of Pfister Avenue between Tacoma Street on the south and Sycamore Drive on the north. Soldier Park separates these homes from the creek, but the residences are at an elevation only about 4- to 6-feet above channel invert and the overbank elevations do not slope toward the channel. Just north of Sycamore Drive, there is a significant contraction in the channel and floodplain width that is causing a backwater condition south thereof; this contraction causes the overbank flooding south to Tacoma Street. The preliminary floodplain modeling being conducted by FEMA confirms this backwater condition.

Alternative Solutions - Possible solution for mitigating the flood damage potential are a new detention basin west of Buffalo Soldier Trail on Ft. Huachuca, widening the channel section to increase capacity or, public acquisition of residential properties within the floodplain area.

Alternative 1 – Unlike many of the major washes to the south, flows emanating from the Soldier Creek watershed are not detained. Coordination with and support for a new basin on Ft. Huachuca must be solicited from the Department of the Army. A possible location for a 26.5 acre basin is shown on Figure 9.26. Preliminary calculations indicate the basin sizes and volumes given in Table 7 are needed to reduce downstream flows from 25% to 75%. Major peak flow reductions are needed to remove most of the existing structures from the FEMA floodplain area. The area for a possible basin shown on Figure 9.26 (which is about 26.3 acres) would need to be about three times larger to reduce the outflow peak to 3,000 cfs; given a basin depth of 5-feet. This alternative is clearly not feasible given the limited number of structures (± 32) within the floodplain. A cost estimate for this alternative is not provided for that reason

Table 7: Soldier Creek Detention Volume Estimates

Peak Flow Reduced to (cfs)	Required Basin Volume (acre-feet)	Estimated Basin Area* (acres)
3,000	362	87
2,000	544	131
1,000	726	174

* assumes a 5-foot basin ponding depth

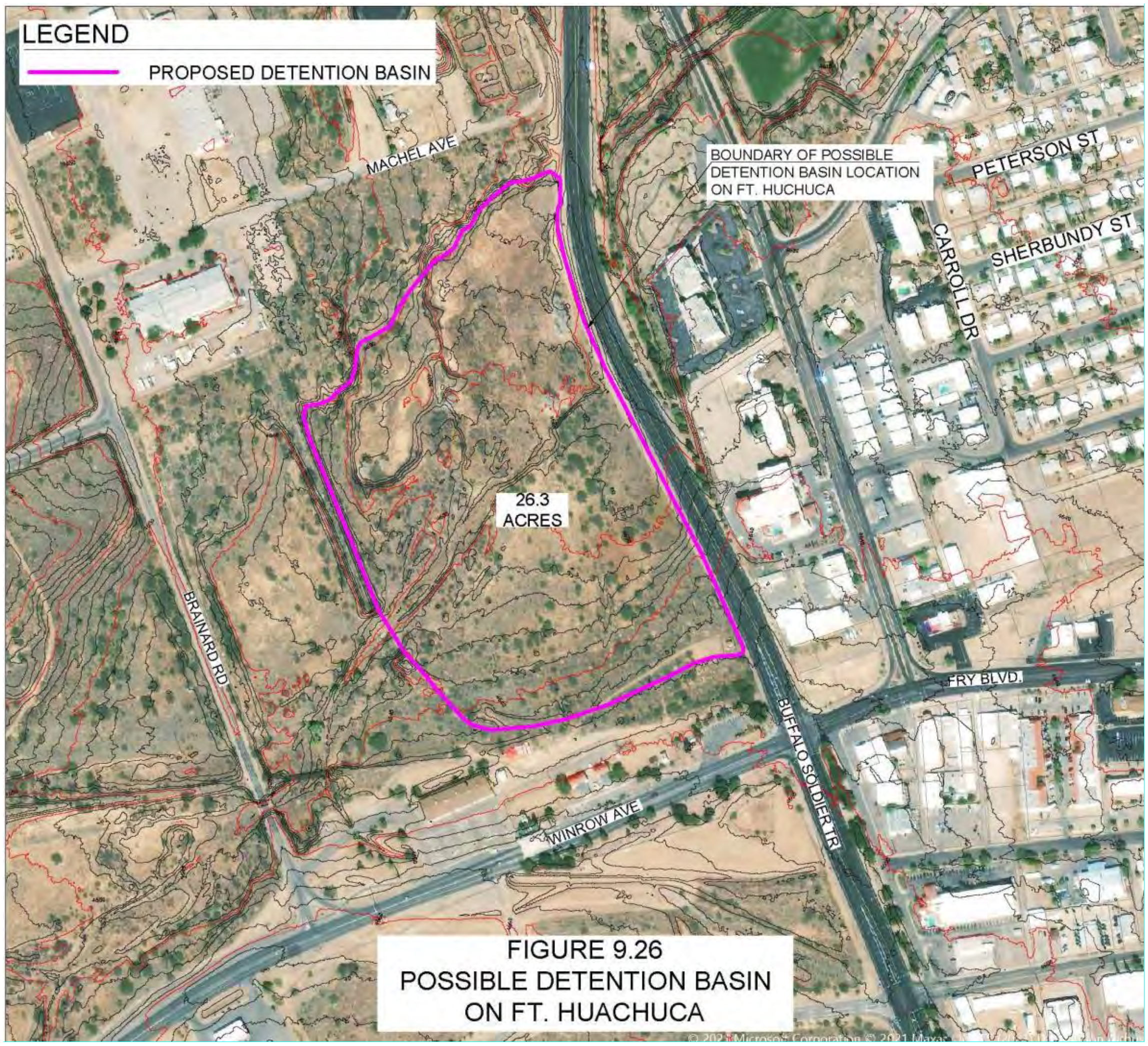


FIGURE 9.26
POSSIBLE DETENTION BASIN
ON FT. HUACHUCA

Alternative 2 – proposes widening the channel to a width of 80- to 100-feet along a 900- to 1100-foot long reach ending just north of Sycamore Drive to eliminate the backwater condition described above. The goal is to provide containment of the 100-year discharge of about 4,300 cfs south to Tacoma Street. This range of widths is based on a flow depth of approximately 6-feet. Deeper depths would require a berm or levee to contain flow which is not desirable due to regulatory issues and challenges associated with discharging local storm water through a levee embankment.

The channel widening will most likely, involve excavation of the east bank to reach a width of about 100-feet. This channelization would start just south of the Tacoma Street alignment tying into existing bank protection to the south, then extending north past the Sycamore Drive alignment around a sharp bend to the east, then terminating where the existing channel width is about 100-feet. Optimally, the widening could be substantially accomplished with minimal disturbance of natural vegetation, most notably the large Cottonwood trees that are present along this reach. Bank protection will be required to eliminate the risk of lateral erosion toward Pfister Avenue, the park trail and nearby homes. However, it is estimated that up to four residential structures could be sacrificed by the channelization.

Figure 9.27 provides an aerial view of the channelization reach, bank protection and residential structures that could be lost due to the channel widening. This figure also shows the existing 100-year floodplain limits and those for the post-channelization condition.

The estimated cost for Alternative 2 is $1,015,850.

The cost estimate spreadsheet is provided in Appendix D.5.

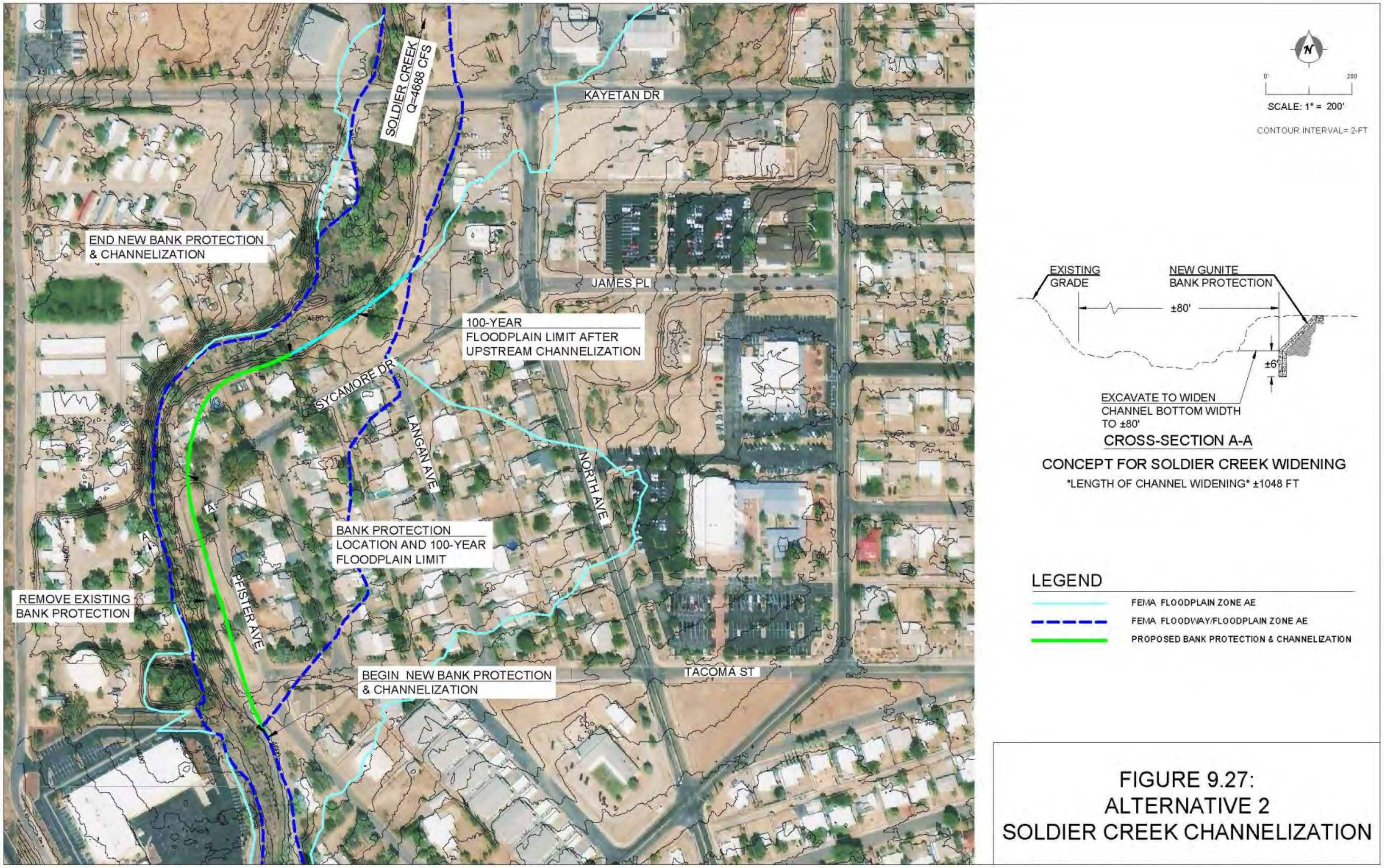


FIGURE 9.27:
ALTERNATIVE 2
SOLDIER CREEK CHANNELIZATION

Alternative 3 - Public Acquisition of Flood Vulnerable residential Structures. As previously noted, there are an estimated 32 residential structures and 2 commercial buildings within the 100-year floodplain of Soldier Creek. Alternative 3 considers public acquisition and demolition of the residential structures. The land would remain vacant for conveyance of storm water and possibly be used for expansion of Soldier Creek Park as long as any new structures do not obstruct flow.

9.8 Montebello/Kings Manor Wash

9.8.1 Estimation of Future Long-Term Degradation

Equilibrium channel slope was calculated using Equation 6.26 of the City of Tucson Drainage Design Standards Manual. This methodology estimates the slope needed to balance sediment supply with sediment transport capacity. Long-term degradation is calculated as the difference between existing slope and equilibrium slope, times the reach length.

The existing channel slopes along Montebello/Kings Manor Wash are as given in Table 8 below, along with the existing channel elevation change below drop structures, the average existing channel slope, calculated equilibrium slopes and estimated future degradation at the upstream end of each reach.

Table 8: Long-Term Degradation Estimates for Montebello/Kings Manor Wash

Reach Description	Reach Length (feet)	2009 Drop Height (feet)	Existing Slope (ft/ft)	Equilibrium Slope (ft/ft)	Future Long -Term Degradation Estimate (feet)
Coyote Wash Confluence to Giulio Cesare Avenue	5400	none	0.0094	0.0073	11.2
Giulio Cesare Avenue to Leonardo de Vinci	1200	5	0.012	0.0093	3.2
Leonardo de Vinci to Raffaele Ave	1160	4	0.0076	0.0059	2.0
Raffaele Ave to Colombo Avenue	760	3	0.015	0.012	2.5
Colombo Ave to SR-90	890	3	0.0096	0.0075	1.9
SR-90 to Grade Control 140-ft upstream of SR-90	140	3	0.025	0.020	0.8
Grade Control 140-ft upstream of SR-90 to Savannah Springs Apts grade Control	490	4	0.021	0.016	2.3
Savannah Springs Apt. Grade Control to SR-92	830	5	0.010	0.0078	1.8
SR-92 to Avenida Escuela	550*	<1	0.011	0.0086	1.3
Avenida Escuela to Calle Portal	1250	5	0.013	N/A	0.0**
Calle Portal to Camino Real	1690	1	0.0091	0.0071	3.4
Camino Real to Coronado Drive	2500	<1	0.013	0.010	7.2
Coronado Drive to Lenzner Avenue	2520	<1	0.014	0.011	7.8

* Distance is measured from existing grade control structure
** Concrete lined channel prevents degradation

The equilibrium slope computation spreadsheets for Montebello/Kings Manor Wash are provided in Appendix E.3.

Future degradation along Montebello/Kings Manor Wash is generally limited because of the shorter distances between existing grade controls. The accuracy of the degradation estimates for the downstream most reach is unknown because there are no structures to measure from, however, greater degradation potential exists because of this. Similar uncertainty exists for the reaches upstream of Camino Real; degradation estimates are higher due to the greater distances between grade stabilizing structures, but historical change has been minimal which leads to the conclusion that the computed values are probably high.

Mitigation measures for this wash should focus on reinforcing existing structures where past degradation has lowered the stream bed elevation below the toe of the grade control and where existing toe downs are shallow or absent. A description of some of the existing structures and recommendations for monitoring and mitigation are provided below.

9.8.1.1 Culvert Crossing at Guilio Cesare Drive

At the time of this study (October 2021) there was no grade control structure or vertical drop at this location. Long-term degradation at this location is estimated to be up to 11-feet, however, past change suggests this may be more than should be expected. The recommendation of this report is to monitor future change to determine if a grade control structure should be considered.

9.8.1.2 Grade Control Structures at Raffaele Drive and Leonardo de Vinci Drive

The street crossings at Raffaele Drive and Leonardo de Vinci Drive are dip sections with a concrete apron on the downstream side. The aprons are in good condition but there are scour holes at the toes that could cause some degree of failure during future floods. The depth or presence of a toe wall at the downstream end of these aprons is unknown. The recommendation of this report is to install a 7-foot deep concrete toe wall and a riprap apron at the downstream end to prevent scour from undermining the existing apron. A typical profile for the recommended modifications is given in Figure 9.28.

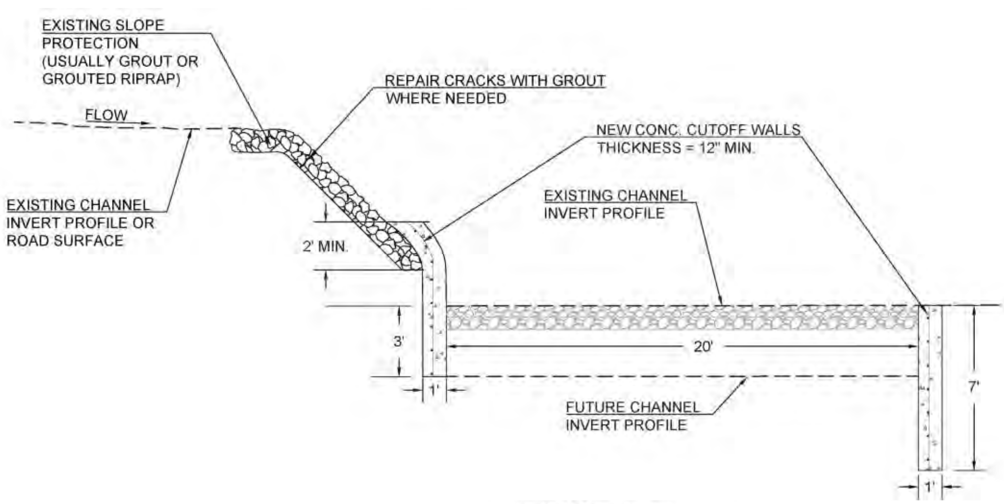


FIGURE 9.28

PROFILE FOR REINFORCEMENT OF EXISTING GRADE CONTROL STRUCTURE ALONG MONTEBELLO/KINGS MANOR WASH AT RAFFAELE DR. & LEONARDO DE VINCI DR.
N.T.S.

9.8.1.3 Drop at Outlet of Colombo Avenue Culvert

Presently, there is a 3-foot drop at the outlet of the 2-10’ x 3’ RCBC. A riprap apron was placed at the drop, but the stones have been washed downstream several feet leaving the outlet unprotected from scour. Inspection of the downstream channel and culvert conditions found no evidence of possible near-term failure. Future degradation at this location is estimated to be another 2.5-feet. The recommendation of this report is to replace the stones that have been washed downstream a few feet and grout them with gunite. Finished length of the grouted riprap should be a minimum of 20-feet and thickness should be a minimum of 18-inches.

9.8.1.4 Grade Control Structure 140-Feet Upstream of SR-90

This grade control structure provides protection against degradation for a sewer line crossing so periodic monitoring is highly recommended. Field inspection found the structure to be in good condition and long-term degradation is estimated to be only another 0.8-feet. The recommendation of this report is to install a 15- to 20-foot long grouted riprap apron at the toe of the existing drop structure as a low cost measure to mitigate short-term scour. A 3-foot deep concrete toe wall at the downstream end of the riprap apron is also recommended (see Figure 9.30).

FIGURE 9.29 - Existing Grade Control Structure 140-ft Upstream of SR-90

9.8.1.5 Reach between Calle Portal and Lenzner Drive

Very little degradation is evident along this reach so recommendations for structures or modifications to existing structures are not given in this report. However, annual monitoring is recommended to determine if degradation or headcutting begins. Long-term degradation is estimated by the equilibrium slope calculations to be 7- to 8- feet at Coronado Drive and Lenzner Drive due to the long distances to the next downstream grade control. The absence of historical change suggests that predicted change may be overestimated.

9.9 Woodcutters Canyon Wash

Equilibrium channel slope was calculated using Equation 6.26 of the City of Tucson Drainage Design Standards Manual. This methodology estimates the slope needed to balance sediment supply with sediment transport capacity. Long-term degradation is calculated as the difference between existing slope and equilibrium slope, times the reach length.

The existing channel slopes along Woodcutters Canyon Wash are as given in Table 9 below, along with the existing channel elevation change below drop structures, the average existing channel slope, calculated equilibrium slopes and estimated future degradation at the upstream end of each reach.

The equilibrium slope computation spreadsheets for Woodcutters Canyon Wash are provided in Appendix E.4.

Table 9: Long-Term Degradation Estimates for Woodcutters Canyon Wash

Reach Description	Reach Length (feet)	2009 Drop Height (ft)	Existing Slope (ft/ft)	Equilibrium Slope (ft/ft)	Future Long -Term Degradation Estimate (feet)
3rd Street Drainageway Confluence to Fry Blvd.	2130	None*	0.010	0.0062	1.5
Fry Blvd. to Lenzner Avenue	495	<1	0.011	0.0068	2.1
Lenzner Avenue to Wilcox Dr.	600	<1	0.017	0.011	3.9
Wilcox Dr to Busby Dr.	2100	<1	0.013	0.010	6.0
Busby Dr. to drop structure downstream of 7th St.	1750	5.5	0.011	0.0086	4.3
7th St. to Golf Links Rd.	890	None**	0.0075	0.0058	1.5
Golf Links Rd to Buffalo Soldier Trail	1030	None	0.014	0.011	3.2

*No degradation present, 3 grade controls present along this reach.
** no degradation but stabilized drops at Savannah and Golf Links Rd culvert inlets

Future degradation along Woodcutters Wash is estimated to be limited downstream of Lenzner Avenue, probably due to the frequency of street crossings and presence of grade control structures downstream of Fry Blvd. Up to four feet of degradation is predicted at Wilcox Drive so additional grade stabilization may be needed at the culvert outlet. Historical degradation has occurred at this location evidenced by the vertical cut banks along the downstream channel reach.

9.9.1 Busby Dr. to existing Drop Structure Downstream of 7th St.

Significant degradation has been occurring along this reach. A grade control was installed about 850-feet downstream of the culvert at 7th Street; probably to prevent headcutting from propagating up to the concrete channel that is present along the reach from 7th Street to a point about 350-feet downstream thereof. The 2009 drop height is about 5.5 feet, and an additional 4.3 feet of degradation is predicted over long-term.

The existing grade control is beginning to fail so additional measures are needed to stabilize it (see Figure 4.9.1 for photograph of the existing grade control). The recommendations for a new grade control at this location are shown on Figure 9.31. The recommendations include a concrete lined spillway constructed on a 3:1 slope from existing channel invert elevation at the top to the estimated long-term degradation elevation at the toe. Three-foot deep toe walls are needed at both ends to prevent scour from undermining the structure, and a 30-foot long grouted riprap apron is also needed at the downstream end for the same purpose. Gunite lined bank protection should be installed along the full length of the improvements to prevent bank erosion from outflanking the grade control. Five-foot bank protection key-ins should also be provided at the upstream and downstream ends of the bank protection.

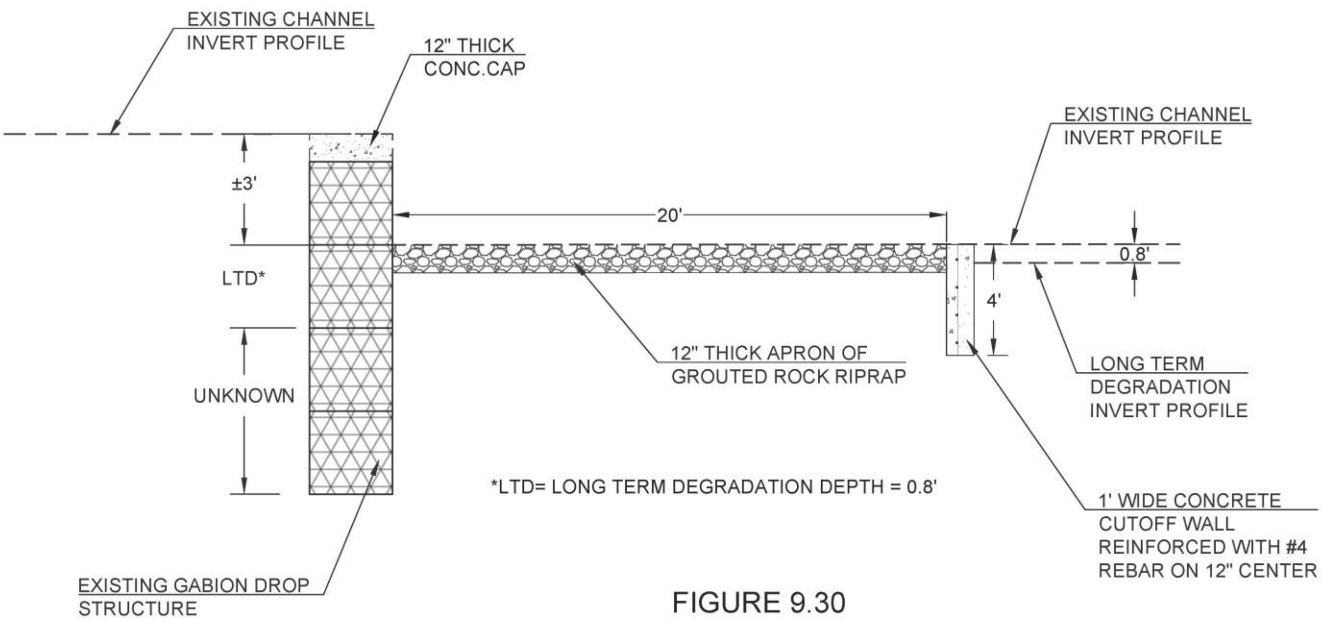


FIGURE 9.30

TYPICAL PROFILE FOR MODIFICATIONS TO THE EXISTING GRADE CONTROL STRUCTURE ALONG KINGS MANOR WASH 140-FT UPSTREAM OF SR-90
N.T.S.

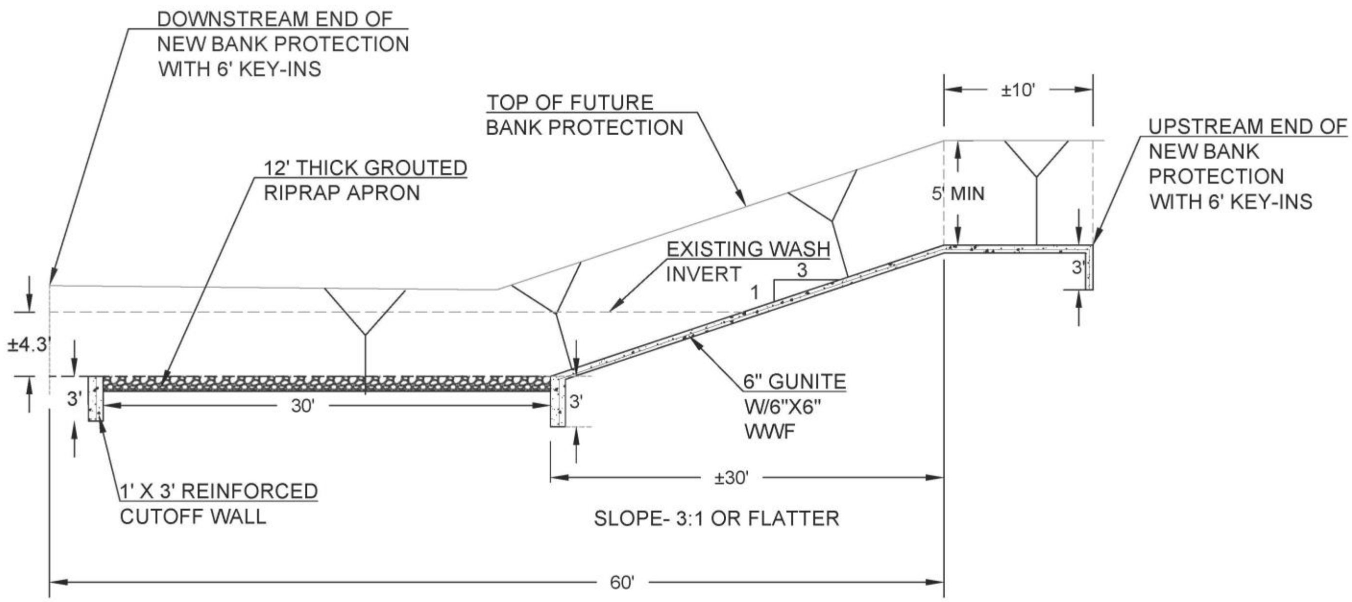


FIGURE 9.31
WOODCUTTERS CANYON WASH GRADE CONTROL STRUCTURE
N.T.S.

9.10 South Garden Wash

Equilibrium channel slope was calculated using Equation 6.26 of the City of Tucson Drainage Design Standards Manual. This methodology estimates the slope needed to balance sediment supply with sediment transport capacity. Long-term degradation is calculated as the difference between existing slope and equilibrium slope, times the reach length.

The existing channel slopes along South Garden Wash are as given in Table 10 below, along with the existing channel elevation change below drop structures, the average existing channel slope, calculated equilibrium slopes and estimated future degradation at the upstream end of each reach.

Table 10: Long-Term Degradation Estimates for South Garden Wash

Reach Description	Reach Length (feet)	2009 Drop Height (feet)	Existing Slope (ft/ft)	Equilibrium Slope (ft/ft)	Future Long - Term Degradation Estimate (feet)
SR-92 to S. Wardell Road	2850	4.5	0.012	0.010	5.0
S. Wardell Road to E. Wardell Road	1410	3	0.013	0.011	2.7
E. Wardell Road to Oakmont Dr.	1820	3	0.017	0.015	4.5
Oakmont Dr. to Avenida Cochise	1040	<1	0.009	0.008	1.4
Avenida Cochise to Winterhaven Dr.	4500	2	0.013	0.011	8.5
Winterhaven Dr. to Buffalo Soldier Trail	990	Unknown	0.012	0.01	1.7

The equilibrium slope computation spreadsheets for South Garden Wash are provided in Appendix E.5.

Degradation of about 4.5-feet has occurred along the reach downstream of the South Wardell Road. A grade control structure has been installed just downstream of this road crossing; field inspections found this structure to be in good condition, however, plans were not available to determine the toe depth for the structure.

Equilibrium slope calculations estimate that another 5-feet of long-term degradation could occur at this location. One possible preventive measure would be to install a 30-foot long by 12-inch thick grouted riprap apron to control scour and degradation below the structure. An 8-foot deep concrete toe wall should also be installed at the downstream end of the apron to account for future degradation of up to 5-feet.

Historical degradation along upstream reaches has been minimal although estimates of future degradation are large along the reaches from Wardell Road to Oakmont Drive and between Avenida Cochise and Winterhaven Drive. Grade control structures will probably be needed at these locations sometime in the future. The recommendation of this report is to install grade control structures at these locations such as the one shown on Figure 9.13 for drop heights of 4-feet or less.

Lateral migration should also be monitored periodically since there is a sewer line running parallel to the banks from Oakmont Drive to a point more than ¼ mile south of Avenida Cochise.

9.11 Summit Drainageway

Summit Drainageway is a tributary to South Garden Wash; the confluence being in the vicinity of Oakmont Drive. Review of the 2009 channel profile in the HEC-RAS model along with field inspections found no evidence of degradation or headcut propagation. As such, no recommendations for monitoring or control measures are given at the current time.

9.12 Pueblo del Sol Drainageway and Tributary Mt. Mesa Drainageway

Pueblo del Sol Drainageway and Tributary Mt. Mesa Drainageway are located toward the south limit of the study area. The Mt. Mesa Drainageway watershed is largely developed but most of the Pueblo del Sol watershed remains undeveloped. East of SR-92, the wash remains in a natural condition and vegetation density is high; which provides for flow dispersion and lower flow velocities.

Review of the 2009 channel profile in the HEC-RAS model along with field inspections found no evidence of degradation or headcut propagation. As such, no recommendations for monitoring or control measures are given at the present time.

9.13 Murray Springs Wash

Murray Springs Wash runs through the Cochise County Wastewater Treatment facility north of SR-90. Treatment Plant facilities encroach into the floodplain area, but the primary channel remains natural.

Review of the 2009 channel profile in the HEC-RAS model along with field inspections found no evidence of degradation or headcut propagation. As such, no recommendations for monitoring or control measures are given.

9.14 Graveyard Gulch Wash

The study reach for Graveyard Gulch Wash begins north of SR-90 for a distance of about 2,900-feet; crossing San Xavier Road and San Juan Capistrano Dr. The only identified concern was headcut erosion at the outlet of the 42-inch culvert at San Xavier Road. The headcut depth at this location is about 4-feet. A concrete apron and gabion baskets were installed supposedly to prevent undercutting of the culvert and pavement. The depth of the toe down is unknown so periodic inspections are needed to monitor change. It is expected that additional headcut mitigation measures such as a concrete cutoff wall will be needed.

9.15 Garden Canyon Wash

Garden Canyon Wash exists in a generally natural condition with the primary exception being in the vicinity of a gravel pit east of SR-92. The impacts of this pit are unknown, but field observation did not find any location where headcut erosion was propagating upstream from the pit boundary.

Channel incisement is significant beginning about 350-feet downstream of SR-92. Concrete has been dumped in the wash at this location, presumably to prevent the headcut from reaching the SR-92 culverts. Periodic inspections should occur to ensure that lateral erosion around the dumped concrete does not outflank it, which would allow the headcut to continue propagating upstream. No other observations or recommendations are offered at the current time.

SECTION 10: Lateral Erosion Assessment

In general, lateral erosion is not a widespread problem throughout this community; it usually occurs locally where the force of flow is directed toward the outside bank at bends in the channel alignment and is associated with block failure of high vertical banks. Some locations where this occurred were cited in previous sections of this report.

Over time, lateral erosion can threaten structures and public infrastructure near the banks. As such, it is the recommendation of this report that the City of Sierra Vista adopt regulations for calculating lateral erosion setback requirements for future construction of habitable structures near the banks of named watercourses within the community. A simplified procedure for calculating erosion setback distances is given in State Standard ADWR SS5-96. This methodology suggests a three-level approach and equations for calculating erosion setback distance based on the square root of the 100-year discharge and degree of channel curvature.

A copy of SS5-96 is provided in Appendix F.

APPENDIX A-SCOPE OF WORK

<u>Scope of Work Outline for City of Sierra Vista Surface Water Master Plan Update</u>

Phase 1

1) Review previous City of Sierra Vista Surface Water Master Plan documents which include:
   a. Surface Water Plan Summary Report prepared by Simons, Li and Associates, 1988,
   b. Technical Support Data Notebook – Flood Study – Volume 1 prepared by Hydro Science Engineering Southwest, Inc., 1998,
   c. Cochise County Flood Control / Urban Runoff Recharge Plan, Appendix A – Hydrology and Flood Control prepared by Stantec, 2006.
   Purpose of the reviews is to familiarize CMG with the previous study results, community goals and drainage issues of the past.
2) Conduct field investigations to examine existing drainageways and structures for washes within the City limits. Purpose of this task is to familiarize CMG with the current physical condition of the drainageways and structures which aids in the identification potential concerns such as bank erosion and degradation which could impact on existing infrastructure. Photo documentation, summary descriptions and survey recommendations will be provided in a GIS format.
3) Conduct review of FEMA's HEC-RAS modeling and provide comments for City's response draft Physical Map Revision. This review will focus on modeling methodologies particularly culverts and lidar registration. The task scope will include attending and providing support to the City of Sierra Vista during meetings with local jurisdictions, development of comment lists, as well as review meetings with FEMA. This task allows up to four 1-hour meetings with two CMG staff.
4) Identify channel reaches that have been experiencing severe degradation and/or lateral migration (bank erosion). Prepare text and graphics that describe sediment transport and erosion concerns along the major watercourses.
5) Locate existing public infrastructure from available information that is not already mapped by the City within the wash environments and identify structures that may be vulnerable to flood related damages. Ground survey data will be collected, as needed and within budget. Specific areas requested for review are listed below. These are to compliment any additional review areas suggested later by the City and or by CMG.
   a) Sulger subdivision drainage.
   b) Fry-town area drainage.
   c) El Camino Real and Summit Wash.
   d) North of Village Meadows Elementary.
   e) Kachina Trail and SR-92.
   f) North of Ramsey Road and Calle Encina
6) Prepare Phase 1 SMP hydraulics report

Phase 2

7) At City staff's direction, conduct preliminary assessment of alternatives such as additional regional flood detention basins, channelization, erosion control structures, land acquisition, and culvert redesign and construction. Detailed hydraulic modeling is not included in this scope; only locational concept plans. Meet with City to discuss concerns and prioritize project needs.
8) Develop concept plan for mitigation and/or minimization of future erosion damages along the major watercourses where significant erosion is ongoing. Develop criteria for lateral erosion setback requirements where structural measures are not present. Develop a short-term, mid-term, and long-term improvement breakdown that are categorized by approximate cost.
9) Prepare Phase 2 SMP report
10) Facilitate workshop meeting with City to discuss concerns and prioritize project needs.

11) Review the City's Development Code related to Floodplain Development and provide recommendations for modifications, if needed. This review will look at the Code relative to other recently updated codes in other communities like Pima County and the Town of Marana.
12) Provide final presentation to City.
13) Optional -Attend meetings to discuss project issues, needs and hold in-house training workshops for City staff. Purpose of the workshops will be to train City staff on the hydrologic and hydraulic models and floodplain management procedures. With regards to staff training sessions, two to three meetings are proposed. Staff training instruction deliverables will address:
    - Use of ArcGIS geographical information systems software to ingest existing data sets de-scribing the land surface of a watershed,
    - Use of ArcGIS software to manipulate data to produce input data necessary to use hydrologic models (i.e., HEC-HMS)
    - Translate a design rainstorm into a hydrograph for a stream

    The instructors will provide:
    - General overview of hydrologic and hydraulic techniques using HEC-HMS, HEC-RAS 1D and other relevant software.

13) Prepare LOMR Application and submit to FEMA for the Avenida Escuela Culvert crossing proposed for the Avenida Escuela extension project.

APPENDIX B-EXCERPTS FROM PREVIOUS SMP'S

Excerpts From Previous Surface Water Plan Reports

It should be noted that the hydrologic results given in the previous study reports described below have been superseded by more recent studies completed by the City and FEMA. Several detention basins located on the Ft. Huachuca Military Reservation have been constructed or enlarged since the date of the reports that precede 2006. Also, some of the recommended flood control solutions identified in the reports have since been constructed.

<u>City of Sierra Vista Surface Water Plan Summary Report – Simons Li & Associates, Inc. January 27, 1988</u>
Purpose was to present a regional approach to the future management of surface water runoff within the study area, while at the same time addressing existing conditions and problems. The intent of the plan was also to provide means for protecting the public against the hazards of flooding and erosion while recognizing that storm water runoff as well as the natural systems are public amenities. The first phase of the study was an investigation of existing hydrologic and hydraulic conditions. Peak discharge rates for the major watercourses were calculated using the Pima County Method.

The second phase was to develop and evaluate alternative surface water management schemes using information developed as a part of Phase 1. The third phase was to identify the preferred alternatives. More details regarding the results can be found in the study reports.

Most of the identified problem areas were related to undersized roadway culverts which could prevent all-weather access during floods or result in flow breaking out of the channel (listed in Table 2.3.2.1 of the SLA report). Undersized channels were Identified as being along portions of Soldiers Creek, Charleston Wash at Coronado Drive, Woodcutters Canyon Wash upstream of Savannah Drive, and other locations. Bank erosion and channel bottom degradation was also identified as a concern and standardized erosion setbacks, based on contributing watershed area, were also recommended.

The third phase of the study developed surface water plan alternatives and a decision making model capable of comparing significant factors for ranking alternatives. The process involved qualitative ranking indices of the alternatives under evaluation. The indices were grouped together based on technical, economic, social and environmental factors. Weighting factors were also used to reflect relative importance. Appendix B includes a copy of tables from this report that summarize high potential flood hazard areas and alternatives for flood mitigation. Some of the recommendations, particularly construction of detention basins, have been implemented since the study date (1988).

<u>Technical Data Support Notebook – Flood Study for the City of Sierra Vista AZ- Hydro-Sciences Southwest, Inc June 1998</u>
Purpose was to compile and augment previous efforts to delineate floodplains in the City and compile into a set of workmaps. The report summarizes results of the review of available hydrologic information. Hydrologic analyses results included information presented in the Surface Water Plan prepared by Simons Li and Associates, Inc., augmented by HEC-1 routing to simulate flood peaks affected by stormwater detention facilities. The study stated that although some of the major watercourses include improved channels, the level of protection they afford is generally inadequate to convey storm water runoff during severe flooding conditions.

HEC-2 hydraulic modeling was conducted to develop to provide 100-year floodplain and floodway delineations for the major watercourses, with the intended purpose being a floodplain management tool for the community. Cross-sections were based on 1985 topography and the interval averaged 200-feet.

The study also identified possible locations for and evaluated several detention basin alternatives. Preliminary detention routing computations (using HEC-1) were also conducted to determine potential peak flow reductions for the downstream channel reaches.

Cochise County Flood Control / Urban Runoff Recharge Plan- Appendix A – Hydrology and Flood Control – Stantec Consulting Inc. April 2006

This report presents the procedures and results of studies conducted jointly by Stantec Consulting, Inc. and Geosystems Analysts, Inc. to evaluate the potential flood control and incidental recharge benefits associated with construction of regional detention basins. The study area included the City of Sierra Vista and the Ft. Huachuca Military Reservation. Hydrologic modeling was performed using the U.S Army Corps of Engineers HEC-HMS model although the report states that the results are considered appropriate for planning purposes but not intended to replace the requisite more detailed study efforts that should be performed for design purposes.

Detailed detention routings were conducted for 38 flood control facilities; 12 of which were in Cochise County, 16 were in the City and 9 were within Ft. Huachuca. The results of the detention modeling found that given the installation of all facilities, the targeted flood control objectives can generally be met. Estimated opinion of probable construction cost for each basin facility were calculated.

APPENDIX C-DETENTION BASIN ROUTING COMPUTATIONS

Carmichael Avenue Basin

PIMA COUNTY REGIONAL FLOOD CONTROL DISTRICT

ROUTING OF A FLOOD HYDROGRAPH THROUGH A STORMWATER DETENTION / RETENTION FACILITY

Worksheet to Input the Inflow Hydrograph, & Automatically Perform the Routing Calculations using the Stage-Volume data, Volume-Outflow data, & SO Working Curve

PIMA COUNTY FLOOD CONTROL

Project Address	**Project Address**
Designer	**Designer**
Wednesday, December 01, 2021	**Run Date**
pc-route-v8-0_CarmichaelAve_Basin.xls	**Program File Name**

Rev. 10/20

GOVERNING EQUATION: *Ref: Applied Hydrology (Ven Te Chow, Editor 1964)*

Mass Conservation: $0.5 * (I_1 + I_2) * \Delta t - 0.5 * (O_1 + O_2) * \Delta t = S_2 - S_1$

Isolate, divide by Δt: $0.5 * (I_1 + I_2) + S_1 / \Delta t - 0.5 * O_1 = S_2 / \Delta t + 0.5 * O_2$

VARIABLES:

- Δt — time interval between hydrograph discharges.
- I_1, I_2 — inflow rate into facility at start and end of time interval from inflow hydrograph
- O_1, O_2 — facility outflow rate at start & end of time interval
- S_1, S_2 — stormwater in storage in the facility at start and end of time interval

Note: Input Δt, target discharges & inflow hydrographs for 3 storm frequencies into blue cells. Outflow hydrographs (yellow) are calculated from specified outlet configuration (**vol-outflow** tab) and facility geometry (**Stage-Vol** tab). To add rows to this worksheet, add them in roughly the center of the range, then unhide all columns and copy hidden equations into the new rows. Zero discharge within and beyond the end of the hydrograph will not affect the routing. **All blue cells in this spreadsheet must either be blank (highlight, right-click, Clear Contents) or must contain a number. In addition, the Stage - Volume data must be entered in numerically ascending order. This spreadsheet does not have a "clear" button to clear all input data in one action; to accomplish this, restart Excel using a blank copy of the spreadsheet.**

RESULTS:

	max inflow		max outflow		total inflow volume				max stage (H) *				target ** discharge	
100-Year	291.0	cfs	51.0	cfs	327958	ft^3	7.529	af	7.29	ft	35	min		cfs
10-Year	167.7	cfs	36.7	cfs	203527	ft^3	4.672	af	4.74	ft	36	min		csf
2-Year	94.6	cfs	23.1	cfs	127052	ft^3	2.917	af	3.09	ft	38	min		cfs

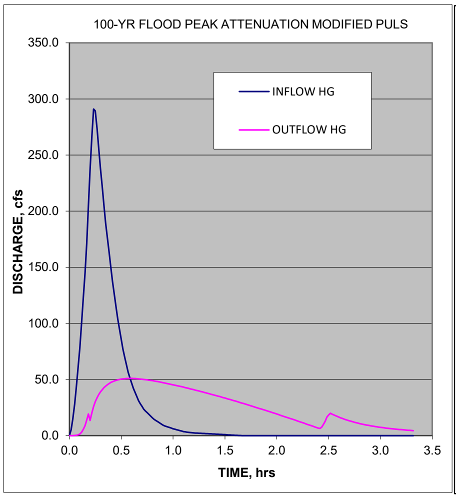
* Max Design Stage = 8.00 ft

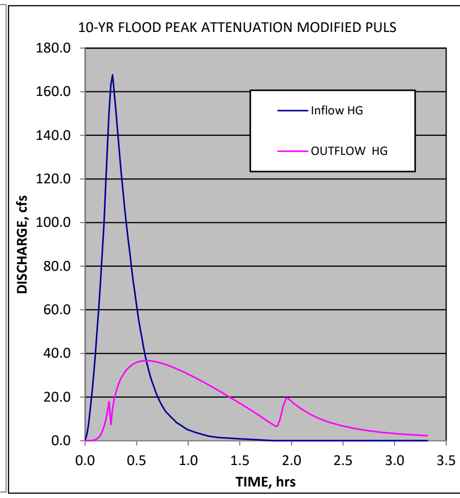
NOTE: IF H > MAX DESIGN STAGE, EXTEND STAGE-VOL DATA TO A HIGHER STAGE

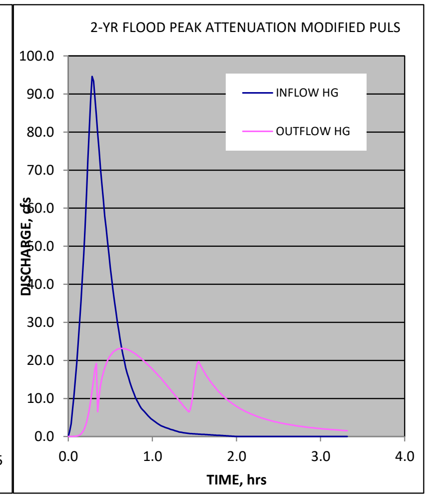
** target discharges not used in calculations; for informational use only

Δt = 1.00 min 0.0167 hr inflow hydrograph time interval

index count	100-Year Inflow I, cfs	time t, hr	100-Year S/Δt+O/2 cfs	10-Year Inflow I, cfs	10-Year S/Δt+O/2 cfs	2-Year Inflow I, cfs	2-Year S/Δt+O/2 cfs	100-Year outflow O, cfs	100-Year Stage H, ft	10-Year outflow O, cfs	10-Year Stage H, ft	2-Year outflow O, cfs	2-Year Stage H, ft
0	0.00	0.0000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	5.21	0.0167	2.61	2.74	1.37	1.39	0.70	0.00	0.01	0.00	0.00	0.00	0.00
2	14.78	0.0333	12.60	7.26	6.37	3.35	3.07	0.02	0.03	0.01	0.02	0.00	0.01
3	28.01	0.0500	33.97	14.05	17.01	6.81	8.14	0.06	0.08	0.03	0.04	0.01	0.02
4	43.23	0.0667	69.53	21.90	34.96	10.60	16.83	0.22	0.17	0.06	0.09	0.03	0.04
5	59.90	0.0833	120.88	30.25	60.97	14.67	29.44	0.61	0.30	0.17	0.15	0.05	0.07
6	78.28	0.1000	189.35	39.34	95.60	19.20	46.32	1.43	0.47	0.39	0.24	0.11	0.12
7	99.76	0.1167	276.94	50.09	139.92	23.92	67.77	2.91	0.68	0.81	0.35	0.21	0.17
8	121.58	0.1333	384.70	61.42	194.87	29.67	94.36	5.35	0.93	1.51	0.48	0.38	0.23
9	145.09	0.1500	512.69	72.92	260.53	35.45	126.53	8.92	1.22	2.59	0.64	0.67	0.31
10	171.16	0.1667	661.89	85.82	337.32	41.31	164.25	13.88	1.56	4.19	0.82	1.09	0.41
11	204.54	0.1833	835.86	99.82	425.94	47.87	207.74	19.29	1.94	6.42	1.03	1.70	0.51
12	236.94	0.2000	1037.31	117.57	528.21	54.84	257.39	13.49	2.37	9.40	1.26	2.53	0.63
13	267.45	0.2167	1276.01	134.58	644.88	63.67	314.11	20.53	2.86	13.29	1.52	3.68	0.77
14	291.02	0.2333	1534.72	150.91	774.33	72.58	378.56	25.92	3.37	17.97	1.80	5.19	0.92
15	289.36	0.2500	1798.98	163.47	913.55	80.89	450.10	30.31	3.87	7.38	2.12	7.09	1.08
16	273.12	0.2667	2049.91	167.74	1071.78	88.31	527.61	33.84	4.33	14.75	2.44	9.38	1.26
17	255.39	0.2833	2280.33	159.41	1220.60	94.61	609.69	36.71	4.74	19.14	2.75	12.07	1.44
18	237.99	0.3000	2490.31	150.26	1356.30	93.36	691.61	39.07	5.11	22.37	3.02	14.93	1.62
19	221.02	0.3167	2680.74	141.01	1479.56	89.13	767.92	41.06	5.43	24.88	3.26	17.73	1.79
20	204.75	0.3333	2852.56	131.92	1591.15	84.39	836.96	42.73	5.72	26.94	3.48	19.17	1.94
21	189.23	0.3500	3006.82	123.21	1691.77	79.68	899.82	44.16	5.97	28.63	3.67	6.50	2.09
22	175.24	0.3667	3144.89	114.73	1782.12	75.05	970.69	45.38	6.19	30.05	3.84	10.64	2.23
23	162.91	0.3833	3268.58	106.62	1862.74	70.57	1032.86	46.43	6.39	31.27	3.99	13.32	2.36
24	149.77	0.4000	3378.50	99.48	1934.53	66.22	1087.94	47.35	6.56	32.28	4.12	15.33	2.48
25	137.25	0.4167	3474.66	93.01	1998.49	62.05	1136.74	48.11	6.71	33.16	4.24	16.83	2.58
26	125.34	0.4333	3557.84	86.11	2054.89	57.98	1179.93	48.76	6.84	33.91	4.34	18.06	2.66
27	114.49	0.4500	3628.99	79.46	2103.76	54.69	1218.21	49.31	6.95	34.55	4.43	19.08	2.74
28	104.10	0.4667	3688.98	73.10	2145.49	51.32	1252.13	49.77	7.04	35.08	4.50	19.94	2.81
29	94.34	0.4833	3738.43	67.34	2180.64	47.81	1281.76	50.13	7.11	35.51	4.57	20.67	2.87
30	85.79	0.5000	3778.37	61.76	2209.68	44.47	1307.23	50.42	7.17	35.87	4.62	21.27	2.92
31	77.24	0.5167	3809.47	56.39	2232.89	41.23	1328.81	50.64	7.22	36.14	4.66	21.77	2.96
32	69.54	0.5333	3832.22	51.71	2250.80	38.30	1346.81	50.81	7.25	36.36	4.69	22.16	3.00
33	63.08	0.5500	3847.72	47.22	2263.90	35.42	1361.50	50.92	7.27	36.51	4.71	22.49	3.03
34	56.61	0.5667	3856.64	42.73	2272.36	32.69	1373.07	50.99	7.29	36.61	4.73	22.74	3.05
35	51.21	0.5833	3859.56	38.96	2276.59	30.15	1381.76	51.01	7.29	36.66	4.73	22.92	3.07
36	46.72	0.6000	3857.52	35.56	2277.19	27.86	1387.85	50.99	7.29	36.67	4.74	23.04	3.08
37	42.24	0.6167	3851.01	32.17	2274.39	25.58	1391.52	50.94	7.28	36.64	4.73	23.12	3.09
38	38.38	0.6333	3840.37	29.34	2268.50	23.29	1392.84	50.87	7.26	36.57	4.72	23.15	3.09
39	34.83	0.6500	3826.11	26.98	2260.09	21.55	1392.11	50.76	7.24	36.47	4.71	23.13	3.09
40	31.29	0.6667	3808.41	24.62	2249.42	19.82	1389.66	50.64	7.22	36.34	4.69	23.08	3.08


100-YR FLOOD PEAK ATTENUATION MODIFIED PULS
INFLOW HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5


10-YR FLOOD PEAK ATTENUATION MODIFIED PULS
Inflow HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5


2-YR FLOOD PEAK ATTENUATION MODIFIED PULS
INFLOW HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs
0.0
10.0
20.0
30.0
40.0
50.0
60.0
70.0
80.0
90.0
100.0
0.0
1.0
2.0
3.0
4.0

Proposed North Ave Basin

PIMA COUNTY REGIONAL FLOOD CONTROL DISTRICT

ROUTING OF A FLOOD HYDROGRAPH THROUGH A STORMWATER DETENTION / RETENTION FACILITY

Worksheet to Input the Inflow Hydrograph, & Automatically Perform the Routing Calculations using the Stage-Volume data, Volume-Outflow data, & SO Working Curve

PIMA COUNTY FLOOD CONTROL

North Ave SV	Project Address
North Ave Prp Basin	Designer
Wednesday, December 01, 2021	Run Date
NorthAve_Basin_Prp_Expansion_pc-route-v8-0.xls	Program File Name

Rev. 10/20

GOVERNING EQUATION: *Ref: Applied Hydrology (Ven Te Chow, Editor 1964)*

Mass Conservation: $0.5 * (I_1 + I_2) * \Delta t - 0.5 * (O_1 + O_2) * \Delta t = S_2 - S_1$

Isolate, divide by Δt: $0.5 * (I_1 + I_2) + S_1 / \Delta t - 0.5 * O_1 = S_2 / \Delta t + 0.5 * O_2$

VARIABLES:

- Δt time interval between hydrograph discharges.
- I_1, I_2 inflow rate into facility at start and end of time interval from inflow hydrograph
- O_1, O_2 facility outflow rate at start & end of time interval
- S_1, S_2 stormwater in storage in the facility at start and end of time interval

Note: Input Δt, target discharges & inflow hydrographs for 3 storm frequencies into blue cells. Outflow hydrographs (yellow) are calculated from specified outlet configuration (**vol-outflow** tab) and facility geometry (**Stage-Vol** tab). To add rows to this worksheet, add them in roughly the center of the range, then unhide all columns and copy hidden equations into the new rows. Zero discharge within and beyond the end of the hydrograph will not affect the routing. **All blue cells in this spreadsheet must either be blank (highlight, right-click, Clear Contents) or must contain a number. In addition, the Stage - Volume data must be entered in numerically ascending order. This spreadsheet does not have a "clear" button to clear all input data in one action; to accomplish this, restart Excel using a blank copy of the spreadsheet.**

RESULTS:

	max inflow		max outflow		total inflow volume				max stage (H) *				target ** discharge	
100-Year	206.9	cfs	135.4	cfs	1514220	ft^3	34.762	af	7.11	ft	330	min		cfs
10-Year	0.0	cfs	0.0	cfs	0	ft^3	0.000	af	0.00	ft	0	min		csf
2-Year	0.0	cfs	0.0	cfs	0	ft^3	0.000	af	0.00	ft	0	min		cfs

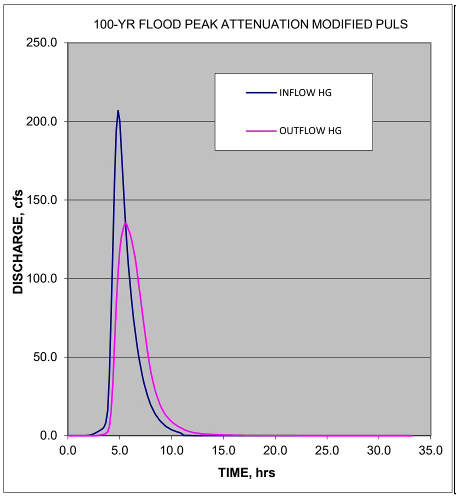
* Max Design Stage = 8.00 ft

NOTE: IF H > MAX DESIGN STAGE, EXTEND STAGE-VOL DATA TO A HIGHER STAGE

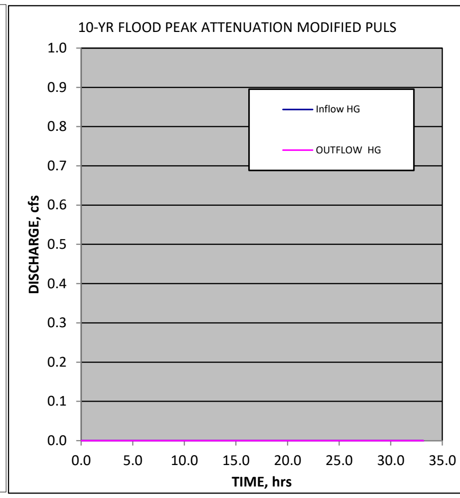
** target discharges not used in calculations; for informational use only

Δt = 10.00 min 0.1667 hr inflow hydrograph time interval

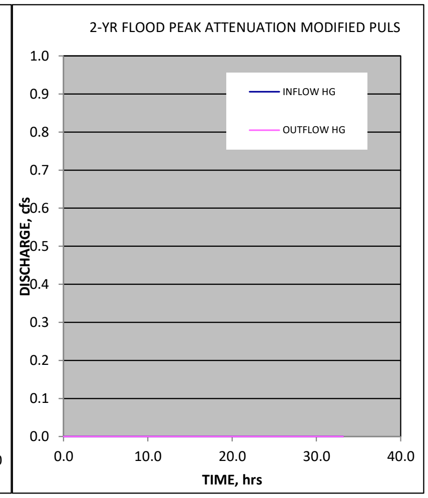
index count	100-Year Inflow I, cfs	100-Year time t, hr	100-Year S/Δt+O/2 cfs	10-Year Inflow I, cfs	10-Year S/Δt+O/2 cfs	2-Year Inflow I, cfs	2-Year S/Δt+O/2 cfs	100-Year outflow O, cfs	100-Year Stage H, ft	10-Year outflow O, cfs	10-Year Stage H, ft	2-Year outflow O, cfs	2-Year Stage H, ft
0	0.00	0.0000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	0.00	0.1667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.00	0.3333	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.5000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	0.00	0.6667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.8333	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	0.00	1.0000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	1.1667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	0.00	1.3333	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	0.00	1.5000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	0.00	1.6667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
11	0.10	1.8333	0.05		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
12	0.20	2.0000	0.20		0.00		0.00	0.00	0.01	0.00	0.00	0.00	0.00
13	0.40	2.1667	0.50		0.00		0.00	0.00	0.01	0.00	0.00	0.00	0.00
14	0.70	2.3333	1.05		0.00		0.00	0.00	0.03	0.00	0.00	0.00	0.00
15	1.20	2.5000	2.00		0.00		0.00	0.00	0.05	0.00	0.00	0.00	0.00
16	1.70	2.6667	3.45		0.00		0.00	0.03	0.09	0.00	0.00	0.00	0.00
17	2.30	2.8333	5.42		0.00		0.00	0.15	0.12	0.00	0.00	0.00	0.00
18	2.90	3.0000	7.88		0.00		0.00	0.30	0.15	0.00	0.00	0.00	0.00
19	3.60	3.1667	10.83		0.00		0.00	0.46	0.19	0.00	0.00	0.00	0.00
20	4.40	3.3333	14.37		0.00		0.00	0.65	0.25	0.00	0.00	0.00	0.00
21	5.70	3.5000	18.76		0.00		0.00	0.95	0.31	0.00	0.00	0.00	0.00
22	8.20	3.6667	24.76		0.00		0.00	1.43	0.40	0.00	0.00	0.00	0.00
23	16.10	3.8333	35.48		0.00		0.00	2.51	0.55	0.00	0.00	0.00	0.00
24	38.00	4.0000	60.03		0.00		0.00	5.96	0.89	0.00	0.00	0.00	0.00
25	75.30	4.1667	110.72		0.00		0.00	15.19	1.50	0.00	0.00	0.00	0.00
26	119.80	4.3333	193.08		0.00		0.00	33.85	2.40	0.00	0.00	0.00	0.00
27	162.70	4.5000	300.48		0.00		0.00	59.07	3.42	0.00	0.00	0.00	0.00
28	193.90	4.6667	419.72		0.00		0.00	83.62	4.46	0.00	0.00	0.00	0.00
29	206.90	4.8333	536.49		0.00		0.00	104.11	5.39	0.00	0.00	0.00	0.00
30	200.50	5.0000	636.08		0.00		0.00	118.30	6.15	0.00	0.00	0.00	0.00
31	181.30	5.1667	708.68		0.00		0.00	126.78	6.67	0.00	0.00	0.00	0.00
32	159.90	5.3333	752.50		0.00		0.00	131.80	6.99	0.00	0.00	0.00	0.00
33	140.80	5.5000	771.06		0.00		0.00	135.37	7.11	0.00	0.00	0.00	0.00
34	123.90	5.6667	768.04		0.00		0.00	134.67	7.09	0.00	0.00	0.00	0.00
35	109.10	5.8333	749.87		0.00		0.00	131.43	6.97	0.00	0.00	0.00	0.00
36	96.00	6.0000	720.99		0.00		0.00	128.16	6.76	0.00	0.00	0.00	0.00
37	84.60	6.1667	683.13		0.00		0.00	123.86	6.49	0.00	0.00	0.00	0.00
38	74.50	6.3333	638.81		0.00		0.00	118.63	6.17	0.00	0.00	0.00	0.00
39	65.50	6.5000	590.18		0.00		0.00	112.36	5.80	0.00	0.00	0.00	0.00
40	57.50	6.6667	539.32		0.00		0.00	104.58	5.41	0.00	0.00	0.00	0.00



100-YR FLOOD PEAK ATTENUATION MODIFIED PULS
INFLOW HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs



10-YR FLOOD PEAK ATTENUATION MODIFIED PULS
Inflow HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs



2-YR FLOOD PEAK ATTENUATION MODIFIED PULS
INFLOW HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs

Existing North Ave Basin

PIMA COUNTY REGIONAL FLOOD CONTROL DISTRICT

ROUTING OF A FLOOD HYDROGRAPH THROUGH A STORMWATER DETENTION / RETENTION FACILITY

Worksheet to Input the Inflow Hydrograph, & Automatically Perform the Routing Calculations using the Stage-Volume data, Volume-Outflow data, & SO Working Curve

North Ave - SV	**Project Address**
Ext North Basin	**Designer**
Wednesday, December 01, 2021	**Run Date**
NorthAve_Basin_Ext_pc-route-v8-0.xls	**Program File Name**

Rev. 10/20

GOVERNING EQUATION: *Ref: Applied Hydrology (Ven Te Chow, Editor 1964)*

Mass Conservation: $0.5 * (I_1 + I_2) * \Delta t - 0.5 * (O_1 + O_2) * \Delta t = S_2 - S_1$

Isolate, divide by Δt: $0.5 * (I_1 + I_2) + S_1 / \Delta t - 0.5 * O_1 = S_2 / \Delta t + 0.5 * O_2$

VARIABLES:

- Δt — time interval between hydrograph discharges.
- I_1, I_2 — inflow rate into facility at start and end of time interval from inflow hydrograph
- O_1, O_2 — facility outflow rate at start & end of time interval
- S_1, S_2 — stormwater in storage in the facility at start and end of time interval

Note: Input Δt, target discharges & inflow hydrographs for 3 storm frequencies into blue cells. Outflow hydrographs (yellow) are calculated from specified outlet configuration (**vol-outflow** tab) and facility geometry (**Stage-Vol** tab). To add rows to this worksheet, add them in roughly the center of the range, then unhide all columns and copy hidden equations into the new rows. Zero discharge within and beyond the end of the hydrograph will not affect the routing. **All blue cells in this spreadsheet must either be blank (highlight, right-click, Clear Contents) or must contain a number. In addition, the Stage - Volume data must be entered in numerically ascending order. This spreadsheet does not have a "clear" button to clear all input data in one action; to accomplish this, restart Excel using a blank copy of the spreadsheet.**

RESULTS:

	max inflow		max outflow		total inflow volume				max stage (H) *				target ** discharge	
100-Year	206.9	cfs	187.9	cfs	1514220	ft^3	34.762	af	7.89	ft	310	min		cfs
10-Year	0.0	cfs	0.0	cfs	0	ft^3	0.000	af	0.00	ft	0	min		csf
2-Year	0.0	cfs	0.0	cfs	0	ft^3	0.000	af	0.00	ft	0	min		cfs

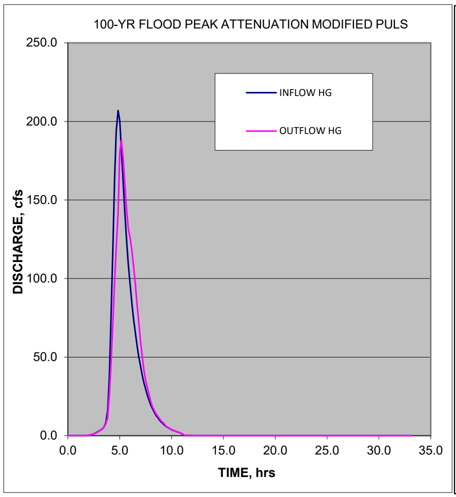
* Max Design Stage = 8.00 ft

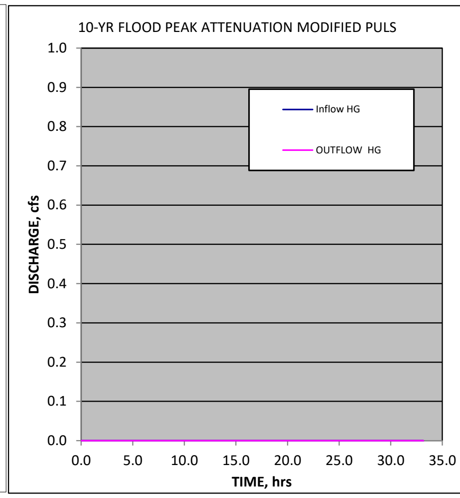
NOTE: IF H > MAX DESIGN STAGE, EXTEND STAGE-VOL DATA TO A HIGHER STAGE

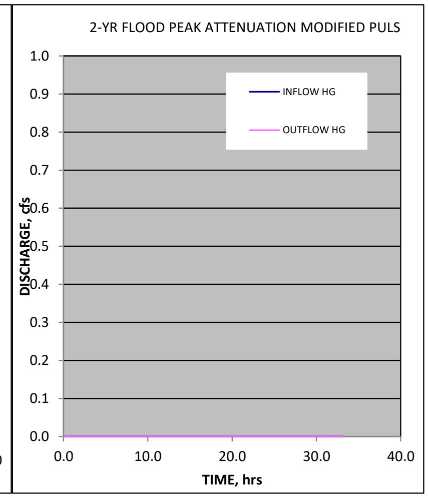
** target discharges not used in calculations; for informational use only

Δt = 10.00 min 0.1667 hr inflow hydrograph time interval

	100-Year			10-Year		2-Year		100-Year		10-Year		2-Year	
index count	Inflow I, cfs	time t, hr	S/Δt+O/2 cfs	Inflow I, cfs	S/Δt+O/2 cfs	Inflow I, cfs	S/Δt+O/2 cfs	outflow O, cfs	Stage H, ft	outflow O, cfs	Stage H, ft	outflow O, cfs	Stage H, ft
0	0.00	0.0000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	0.00	0.1667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.00	0.3333	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	0.00	0.5000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	0.00	0.6667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.00	0.8333	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	0.00	1.0000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	0.00	1.1667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	0.00	1.3333	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	0.00	1.5000	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	0.00	1.6667	0.00		0.00		0.00	0.00	0.00	0.00	0.00	0.00	0.00
11	0.10	1.8333	0.05		0.00		0.00	0.00	0.08	0.00	0.00	0.00	0.00
12	0.20	2.0000	0.20		0.00		0.00	0.17	0.12	0.00	0.00	0.00	0.00
13	0.40	2.1667	0.33		0.00		0.00	0.32	0.16	0.00	0.00	0.00	0.00
14	0.70	2.3333	0.56		0.00		0.00	0.57	0.22	0.00	0.00	0.00	0.00
15	1.20	2.5000	0.94		0.00		0.00	1.02	0.32	0.00	0.00	0.00	0.00
16	1.70	2.6667	1.37		0.00		0.00	1.58	0.42	0.00	0.00	0.00	0.00
17	2.30	2.8333	1.79		0.00		0.00	2.16	0.51	0.00	0.00	0.00	0.00
18	2.90	3.0000	2.23		0.00		0.00	2.80	0.58	0.00	0.00	0.00	0.00
19	3.60	3.1667	2.68		0.00		0.00	3.47	0.66	0.00	0.00	0.00	0.00
20	4.40	3.3333	3.22		0.00		0.00	4.27	0.74	0.00	0.00	0.00	0.00
21	5.70	3.5000	3.99		0.00		0.00	5.48	0.85	0.00	0.00	0.00	0.00
22	8.20	3.6667	5.46		0.00		0.00	7.48	1.01	0.00	0.00	0.00	0.00
23	16.10	3.8333	10.13		0.00		0.00	11.60	1.29	0.00	0.00	0.00	0.00
24	38.00	4.0000	25.57		0.00		0.00	26.14	2.05	0.00	0.00	0.00	0.00
25	75.30	4.1667	56.08		0.00		0.00	45.92	2.90	0.00	0.00	0.00	0.00
26	119.80	4.3333	107.72		0.00		0.00	72.31	3.95	0.00	0.00	0.00	0.00
27	162.70	4.5000	176.66		0.00		0.00	97.46	5.09	0.00	0.00	0.00	0.00
28	193.90	4.6667	257.50		0.00		0.00	119.78	6.24	0.00	0.00	0.00	0.00
29	206.90	4.8333	338.12		0.00		0.00	141.53	7.24	0.00	0.00	0.00	0.00
30	200.50	5.0000	400.29		0.00		0.00	180.30	7.80	0.00	0.00	0.00	0.00
31	181.30	5.1667	410.89		0.00		0.00	187.86	7.89	0.00	0.00	0.00	0.00
32	159.90	5.3333	393.63		0.00		0.00	175.66	7.74	0.00	0.00	0.00	0.00
33	140.80	5.5000	368.32		0.00		0.00	158.91	7.52	0.00	0.00	0.00	0.00
34	123.90	5.6667	341.76		0.00		0.00	143.39	7.28	0.00	0.00	0.00	0.00
35	109.10	5.8333	314.87		0.00		0.00	131.96	7.00	0.00	0.00	0.00	0.00
36	96.00	6.0000	285.45		0.00		0.00	125.77	6.61	0.00	0.00	0.00	0.00
37	84.60	6.1667	249.98		0.00		0.00	118.11	6.14	0.00	0.00	0.00	0.00
38	74.50	6.3333	211.41		0.00		0.00	108.37	5.58	0.00	0.00	0.00	0.00
39	65.50	6.5000	173.05		0.00		0.00	96.32	5.04	0.00	0.00	0.00	0.00
40	57.50	6.6667	138.23		0.00		0.00	83.79	4.46	0.00	0.00	0.00	0.00


100-YR FLOOD PEAK ATTENUATION MODIFIED PULS
INFLOW HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs
250.0
200.0
150.0
100.0
50.0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0


10-YR FLOOD PEAK ATTENUATION MODIFIED PULS
Inflow HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs


2-YR FLOOD PEAK ATTENUATION MODIFIED PULS
INFLOW HG
OUTFLOW HG
DISCHARGE, cfs
TIME, hrs

APPENDIX D-COST ESTIMATE SPREADSHEET

D.1

Cost Estimate for Increasing Size of North Ave Basin

Cost Estimate for Fab Avenue Drainageway - North Avenue Detention Basin Expansion						
Item #	Description	Unit	Quantity	Unit Price	Amount	Amount
1	Clearing and Grubbing	ACRE	1.5	$2,000.00	$ 3,000.00	$ 3,000.00
2	Removal of Structures & Obstructions (fencing)	L.S.	1	$2,000.00	$ 2,000.00	$ 2,000.00
3	Detention Basin Excavation	C.Y.	11,150	$8.00	$ 89,200.00	$ 89,200.00
4	Excess Spoils Export	C.Y.	11,150	$10.00	$ 111,500.00	$ 111,500.00
5	Asphalt Removal (600'x24'x0.5') and disposal	C.Y.	270	$20.00	$ 5,400.00	$ 5,400.00
6	AZPDES/NPDES	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
7	Mobilization (at 5% of cost sub-total)	L.S.	1	$ 20,000.00	$ 20,000.00	$ 20,000.00
8	Construction Survey and Layout	L.S.	1	$12,000.00	$ 12,000.00	$ 12,000.00
9	Access Gate (16') (Type I)	EACH	5	$2,500.00	$ 12,500.00	$ 12,500.00
			Subtotal		$ 270,600.00	$ 270,600.00
			Contingencies (10%)		$ 27,060.00	$ 27,060.00
			Total Construction Cost		$ 297,660.00	$ 297,660.00

D.2

Cost Estimate for Carmichael Ave Basin

Cost Estimate for Fab Avenue Drainageway - Carmichael Avenue Detention Basin						
Item #	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**	**Amount**
1	Clearing and Grubbing	ACRE	1	$2,000.00	$ 2,000.00	$ 2,000.00
2	Removal of Structures & Obstructions	L.S.	1	$60,000.00	$ 60,000.00	$ 60,000.00
3	Detention Basin Excavation	C.Y.	9,592	$8.00	$ 76,736.00	$ 76,736.00
4	Excess Spoils Export	C.Y.	9,592	$10.00	$ 95,920.00	$ 95,920.00
5	Asphalt Removal (600'x24'x0.5') and disposal	C.Y.	270	$20.00	$ 5,400.00	$ 5,400.00
6	Pipe, Reinforced Concrete, Class II, 36"	L.F.	80	$105.00	$ 8,400.00	$ 8,400.00
7	Concrete Cutoff Wall (1ft x 5 ft)	L.F.	60	$65.00	$ 3,900.00	$ 3,900.00
8	Basin Outlet Spillway	S.F.	1,880	$7.00	$ 13,160.00	$ 13,160.00
9	Concrete Ramps (Basin Access - 8in thick)	S.Y.	20	$80.00	$ 1,600.00	$ 1,600.00
10	AZPDES/NPDES	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
11	Basin Inlet Spillway	S.Y.	120	$80.00	$ 9,600.00	$ 9,600.00
12	Mobilization (at 5% of cost sub-total)	L.S.	1	$ 20,000.00	$ 20,000.00	$ 20,000.00
13	Riprap (Dumped) D_{50}=18"	C.Y.	70	$100.00	$ 7,000.00	$ 7,000.00
14	Construction Survey and Layout	L.S.	1	$12,000.00	$ 12,000.00	$ 12,000.00
15	Subgrade Material and Placement	C.Y.	135	$55.00	$ 7,425.00	$ 7,425.00
16	Asphalt Replacement	C.Y.	270	$120.00	$ 32,400.00	$ 32,400.00
17	Street Curb	L.F.	600	$15.00	$ 9,000.00	$ 9,000.00
18	Access Gate (16') (Type I)	EACH	5	$2,500.00	$ 12,500.00	$ 12,500.00
19	Post Barricades (Type B) (PC/COT Std. Dtl. 106)	EACH	10	$400.00	$ 4,000.00	$ 4,000.00
			Subtotal		$ 396,041.00	$ 396,041.00
			Contingencies (10%)		$ 39,604.10	$ 39,604.00
			Total Construction Cost		$ 435,645.10	$ 435,645.00

D.3

Cost Estimate for Grade Control Structure at Coyote Wash Sewer Line Crossing

Cost Estimate for Coyote Wash Grade Control Structure - Soil Cement Option 1						
Item #	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**	**Amount**
1	Clearing and Grubbing	ACRE	1	$2,000.00	$ 2,000.00	$ 2,000.00
2	Removal of Structures & Obstructions (broken concrete)	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
3	Soil Cement for Grade Control Structure	C.Y.	1,330	$60.00	$ 79,800.00	$ 79,800.00
4	Soil cement for Bank protection	C.Y.	1,880	$60.00	$ 112,800.00	$ 112,800.00
5	Channel Excavation for Bank Protection Toe Downs	C.Y.	4,480	$8.00	$ 35,840.00	$ 35,840.00
6	Riprap (Grouted) Thickness=18"	L.F.	420	$130.00	$ 54,600.00	$ 54,600.00
7	Concrete Cutoff Wall	L.F.	125	$70.00	$ 8,750.00	$ 8,750.00
8	Section 404 permit	L.S.	1	$40,000.00	$ 40,000.00	$ 40,000.00
10	AZPDES/NPDES	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
12	Mobilization (at 5% of cost sub-total)	L.S.	1	$ 20,000.00	$ 20,000.00	$ 20,000.00
14	Construction Survey and Layout	L.S.	1	$12,000.00	$ 12,000.00	$ 12,000.00
			Subtotal		$ 395,790.00	$ 395,790.00
			Contingencies (10%)		$ 39,579.00	$ 39,579.00
			Total Construction Cost		$ 435,369.00	$ 435,369.00

Cost Estimate for Coyote Wash Grade Control Structure - Multiple Vertical Concrete Cutoff Walls Option 2						
Item #	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**	**Amount**
1	Clearing and Grubbing	ACRE	1	$2,000.00	$ 2,000.00	$ 2,000.00
2	Removal of Structures & Obstructions (broken concrete)	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
3	Reinforced Concrete Cutoff Walls for Grade Control Structure	L.F.	500	$70.00	$ 35,000.00	$ 35,000.00
	Reinforced Concrete Cutoff Walls for Bank protection	L.F.	200	$70.00	$ 14,000.00	$ 14,000.00
4	Gunite for Bank protection	S.Y.	400	$200.00	$ 80,000.00	$ 80,000.00
5	Channel Excavation for Bank Protection Toe Downs	C.Y.	200	$8.00	$ 1,600.00	$ 1,600.00
6	Riprap (Grouted) Thickness=18"	L.F.	400	$130.00	$ 52,000.00	$ 52,000.00
8	Section 404 permit	L.S.	1	$40,000.00	$ 40,000.00	$ 40,000.00
10	AZPDES/NPDES	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
12	Mobilization (at 5% of cost sub-total)	L.S.	1	$ 20,000.00	$ 20,000.00	$ 20,000.00
14	Construction Survey and Layout	L.S.	1	$12,000.00	$ 12,000.00	$ 12,000.00
			Subtotal		$ 286,600.00	$ 286,600.00
			Contingencies (10%)		$ 28,660.00	$ 28,660.00
			Total Construction Cost		$ 315,260.00	$ 315,260.00

D.4
Cost Estimate for Sulger Subdivision Storm Drains

Cost Estimate for Sulger Subdivision Storm Drain System						
Item #	**Description**	**Unit**	**Quantity**	**Unit Price**	**Amount**	**Amount**
1	Clearing and Grubbing	ACRE	1	$2,000.00	$ 2,000.00	$ 2,000.00
2	Storm Drain Excavation	C.Y.	2,980	$8.00	$ 23,840.00	$ 23,840.00
3	Excess Spoils Export	C.Y.	1,500	$10.00	$ 15,000.00	$ 15,000.00
4	Asphalt Removal (3300'x12'x0.5') and disposal	C.Y.	740	$20.00	$ 14,800.00	$ 14,800.00
5	30-inch dia. CMP Pipe	L.F.	580	$60.00	$ 34,800.00	$ 34,800.00
6	36-inch dia CMP Pipe	L.F.	170	$65.00	$ 11,050.00	$ 11,050.00
7	42-inch dia CMP Pipe	L.F.	630	$84.00	$ 52,920.00	$ 52,920.00
8	48-inch dia CMP Pipe	L.F.	900	$96.00	$ 86,400.00	$ 86,400.00
9	54-inch dia CMP Pipe	L.F.	250	$108.00	$ 27,000.00	$ 27,000.00
10	60-inch dia CMP Pipe	L.F.	990	$120.00	$ 118,800.00	$ 118,800.00
11	Storm Drain Catch Basins	L.S.	10	$6,000.00	$ 60,000.00	$ 60,000.00
12	Strom Drain Manholes	L.S.	10	$3,500.00	$ 35,000.00	$ 35,000.00
13	AZPDES/NPDES	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
14	Mobilization (at 5% of cost sub-total)	L.S.	1	$ 20,000.00	$ 20,000.00	$ 20,000.00
15	Riprap (Dumped) D_{50}=18"	C.Y.	10	$100.00	$ 1,000.00	$ 1,000.00
16	Construction Survey and Layout	L.S.	1	$20,000.00	$ 20,000.00	$ 20,000.00
17	Utility Relocations	L.S.	1	$200,000.00	$ 200,000.00	$ 200,000.00
18	Subgrade Material and Placement	C.Y.	740	$55.00	$ 40,700.00	$ 40,700.00
19	Asphalt Replacement	C.Y.	740	$120.00	$ 88,800.00	$ 88,800.00
20	Street Curb	L.F.	6,600	$15.00	$ 99,000.00	$ 99,000.00
			Subtotal		$ 966,110.00	$ 966,110.00
			Contingencies (10%)		$ 96,611.00	$ 96,611.00
			Total Construction Cost		$ 1,062,721.00	$ 1,062,721.00

D.5

Cost Estimate for Soldier Creek Channelization

Cost Estimate for Soldiers Canyon Wash Channelization						
Item #	Description	Unit	Quantity	Unit Price	Amount	Amount
1	Clearing and Grubbing	ACRE	3	$2,000.00	$ 5,000.00	$ 5,000.00
2	Removal of Structures & Obstructions (building demolition)	L.S.	3	$20,000.00	$ 60,000.00	$ 60,000.00
	Removal of bank protection	L.S.	550	$10.00	$ 5,500.00	$ 5,500.00
3	Property Acquisition	L.S.	3	$150,000.00	$ 450,000.00	$ 450,000.00
4	Gunite for Bank Protection	S.Y.	350	$200.00	$ 70,000.00	$ 70,000.00
5	Channel Excavation	C.Y.	12,000	$8.00	$ 96,000.00	$ 96,000.00
6	Spoils Removal (Export)	C.Y.	8,000	$10.00	$ 80,000.00	$ 80,000.00
7	Concrete Cutoff Walls for Bank Protection Toe Downs	L.F.	1,000	$70.00	$ 70,000.00	$ 70,000.00
8	Section 404 permit	L.S.	1	$40,000.00	$ 40,000.00	$ 40,000.00
10	AZPDES/NPDES	L.S.	1	$15,000.00	$ 15,000.00	$ 15,000.00
12	Mobilization (at 5% of cost sub-total)	L.S.	1	$ 20,000.00	$ 20,000.00	$ 20,000.00
14	Construction Survey and Layout	L.S.	1	$12,000.00	$ 12,000.00	$ 12,000.00
			Subtotal		$ 923,500.00	$ 923,500.00
			Contingencies (10%)		$ 92,350.00	$ 92,350.00
			Total Construction Cost		$ 1,015,850.00	$ 1,015,850.00

APPENDIX E-EQUILIBRIUM SLOPE SPREADSHEETS

Equation 6.26

$$S_{eq} = \left(\left[\frac{n_u}{n_n} \right]^2 \left[\frac{Q_{u,10}}{Q_{n,10}} \right]^{-1.1} \left[\frac{b_u}{b_n} \right]^{0.4} (1-R_s)^{0.7} \right) S_n \qquad \textbf{(6.26)}$$

Where:

n_u	=	Manning's roughness coefficient for an urban channel;
n_n	=	Manning's roughness coefficient for a natural or existing channel;
$Q_{u,10}$	=	Ten-year discharge, under urbanized conditions, in cubic feet per second;
$Q_{n,10}$	=	Ten-year or bank-full discharge (whichever is less), under natural conditions, in cubic feet per second;
b_u	=	Bottom width of channel, under urbanized conditions, in feet;
b_n	=	Bottom width of channel, under natural conditions, in feet;
R_s	=	Reduction factor for sediment supply. This factor is usually assumed to be equal to the ratio of the impervious area to the total area of the upstream watershed (i.e., $0.0 \leq R_s \leq 1.0$); and,
S_n	=	Natural or existing channel slope, in feet per foot.

E.1

Coyote Wash

CMG DRAINAGE ENGINEERING, INC.
3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash -Camino Real to Camino Rancho

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1180
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.010
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0062
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0062
Natural Channel Slope * L_h (ft):	11.80
Design Equilibrium Slope * L_h (ft):	7.26
Long Term Aggradation/Degradation (ft):	4.54

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3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash - Coronado Dr. to Town and Country Dr.

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	630
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	20
Urbanized Channel Bottom Width, b_n (ft):	20
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.010
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0010
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0062
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0062
Natural Channel Slope * L_h (ft):	6.30
Design Equilibrium Slope * L_h (ft):	3.88
Long Term Aggradation/Degradation (ft):	2.42

CMG DRAINAGE ENGINEERING, INC.

3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash -Foothills Dr to SR-92

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1300
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.009
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0057
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0057
Natural Channel Slope * L_h (ft):	11.96
Design Equilibrium Slope * L_h (ft):	7.36
Long Term Aggradation/Degradation (ft):	4.60

CMG DRAINAGE ENGINEERING, INC.

3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash - Kings Manor Confluence to SR-90

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	5300
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.011
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0086
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0086
Natural Channel Slope * L_h (ft):	58.30
Design Equilibrium Slope * L_h (ft):	45.42
Long Term Aggradation/Degradation (ft):	12.88

CMG DRAINAGE ENGINEERING, INC.

3555 N. Mountain Ave. Tucson, Arizona 85719
Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash -Sewer Line Grade Control to Foothills Dr

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	3010
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.007
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0046
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0046
Natural Channel Slope * L_h (ft):	22.27
Design Equilibrium Slope * L_h (ft):	13.71
Long Term Aggradation/Degradation (ft):	8.56

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Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash - SR-90 to Avenida del Sol

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1980
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.009
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0069
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0069
Natural Channel Slope * L_h (ft):	17.42
Design Equilibrium Slope * L_h (ft):	13.57
Long Term Aggradation/Degradation (ft):	3.85

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Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash -SR-92 to Camino Real

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	3460
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.012
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0074
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0074
Natural Channel Slope * L_h (ft):	41.52
Design Equilibrium Slope * L_h (ft):	25.56
Long Term Aggradation/Degradation (ft):	15.96

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Design Scour Depth C.O.T. EQTN 6.3

Coyote Wash -Town and Country Dr to Buffalo Soldiers Trail

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	3170
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.019
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0117
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0117
Natural Channel Slope * L_h (ft):	60.23
Design Equilibrium Slope * L_h (ft):	37.08
Long Term Aggradation/Degradation (ft):	23.15

E.2

Charleston Wash

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Design Scour Depth C.O.T. EQTN 6.3

Charleston Wash Wash - Avenida del Sol to Sewer Line Grade Control

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	800
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0085
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0052
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0052
Natural Channel Slope * L_h (ft):	6.80
Design Equilibrium Slope * L_h (ft):	4.19
Long Term Aggradation/Degradation (ft):	2.61

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Design Scour Depth C.O.T. EQTN 6.3

Charleston Wash Wash - Columbo Ave to SR-90

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2300
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0125
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0077
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0077
Natural Channel Slope * L_h (ft):	28.75
Design Equilibrium Slope * L_h (ft):	17.70
Long Term Aggradation/Degradation (ft):	11.05

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Design Scour Depth C.O.T. EQTN 6.3

Charleston Wash Wash - Avenida Escuela to Sewer Line Crossing

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	900
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0085
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0052
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0052
Natural Channel Slope * L_h (ft):	7.65
Design Equilibrium Slope * L_h (ft):	4.71
Long Term Aggradation/Degradation (ft):	2.94

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Charleston Wash Wash - SR-90 to Avenida Escuela

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1330
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0085
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0052
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0052
Natural Channel Slope * L_h (ft):	11.31
Design Equilibrium Slope * L_h (ft):	6.96
Long Term Aggradation/Degradation (ft):	4.35

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Design Scour Depth C.O.T. EQTN 6.3

Charleston Wash Wash - Sewer Line Grade Control to Coronado Drive

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	3915
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0070
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0043
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0043
Natural Channel Slope * L_h (ft):	27.41
Design Equilibrium Slope * L_h (ft):	16.87
Long Term Aggradation/Degradation (ft):	10.54

E.3

3rd Street Drainageway

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Design Scour Depth C.O.T. EQTN 6.3

3rd Street drainageway Coronado Drive Grade Control above Woodcutters Wash Confluence

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	800
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0070
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0043
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0043
Natural Channel Slope * L_h (ft):	5.60
Design Equilibrium Slope * L_h (ft):	3.45
Long Term Aggradation/Degradation (ft):	2.15

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Design Scour Depth C.O.T. EQTN 6.3

3rd Street drainageway - Grade Control above Woodcutters Wash Confluence to Lenzner Dr.

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1660
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0077
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0047
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0047
Natural Channel Slope * L_h (ft):	12.78
Design Equilibrium Slope * L_h (ft):	7.87
Long Term Aggradation/Degradation (ft):	4.91

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Design Scour Depth C.O.T. EQTN 6.3

3rd Street drainageway - Lenzner Dr. to Grade control Downstream of Fry Blvd.

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2955
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0073
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0045
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0045
Natural Channel Slope * L_h (ft):	21.57
Design Equilibrium Slope * L_h (ft):	13.28
Long Term Aggradation/Degradation (ft):	8.29

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Design Scour Depth C.O.T. EQTN 6.3

3rd Street drainageway - Grade control Downstream of Fry Blvd. to Fry Blvd

Client: City of Sierra Vista
Project #: 21-001

Date 2023.04.12
By: cmg

INPUTS	
Length to Hinge Point, (ft):	460
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.0008
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0005
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0012
Natural Channel Slope * L_h (ft):	0.37
Design Equilibrium Slope * L_h (ft):	0.53
Long Term Aggradation/Degradation (ft):	-0.17

E.4

Montebello/Kings Manor Wash

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash 140-ft Upstream of SR-90

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	140
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.025
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0195
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0195
Natural Channel Slope * L_h (ft):	3.50
Design Equilibrium Slope * L_h (ft):	2.73
Long Term Aggradation/Degradation (ft):	0.77

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at Avenida Escuela

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	550
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.011
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0086
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0086
Natural Channel Slope * L_h (ft):	6.05
Design Equilibrium Slope * L_h (ft):	4.71
Long Term Aggradation/Degradation (ft):	1.34

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at Calle Portal

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1250
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.013
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0101
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0101
Natural Channel Slope * L_h (ft):	16.25
Design Equilibrium Slope * L_h (ft):	12.66
Long Term Aggradation/Degradation (ft):	3.59

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Kings Manor Wash Coyote Wash at Camino Real

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1690
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.009
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0071
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0071
Natural Channel Slope * L_h (ft):	15.38
Design Equilibrium Slope * L_h (ft):	11.98
Long Term Aggradation/Degradation (ft):	3.40

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Kings Manor Wash Coyote Wash at Columbo Ave

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	760
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.015
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0117
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0117
Natural Channel Slope * L_h (ft):	11.40
Design Equilibrium Slope * L_h (ft):	8.88
Long Term Aggradation/Degradation (ft):	2.52

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Kings Manor Wash Coyote Wash at Coronado Drive

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2500
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.013
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0101
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0101
Natural Channel Slope * L_h (ft):	32.50
Design Equilibrium Slope * L_h (ft):	25.32
Long Term Aggradation/Degradation (ft):	7.18

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash Confluence to Giulio Cesare Ave

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	5400
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.009
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0073
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0073
Natural Channel Slope * L_h (ft):	50.76
Design Equilibrium Slope * L_h (ft):	39.54
Long Term Aggradation/Degradation (ft):	11.22

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at Lenzner Ave

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2520
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.014
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0109
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0109
Natural Channel Slope * L_h (ft):	35.28
Design Equilibrium Slope * L_h (ft):	27.49
Long Term Aggradation/Degradation (ft):	7.79

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Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash Confluence to Giulio Cesare Ave

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	5400
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.009
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0073
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0073
Natural Channel Slope * L_h (ft):	50.76
Design Equilibrium Slope * L_h (ft):	39.54
Long Term Aggradation/Degradation (ft):	11.22

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at Raffaelle

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1160
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.008
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0059
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0059
Natural Channel Slope * L_h (ft):	8.82
Design Equilibrium Slope * L_h (ft):	6.87
Long Term Aggradation/Degradation (ft):	1.95

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at Savannah Springs Apt

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	490
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.021
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0164
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0164
Natural Channel Slope * L_h (ft):	10.29
Design Equilibrium Slope * L_h (ft):	8.02
Long Term Aggradation/Degradation (ft):	2.27

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at SR-90

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	890
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.010
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0075
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0075
Natural Channel Slope * L_h (ft):	8.54
Design Equilibrium Slope * L_h (ft):	6.66
Long Term Aggradation/Degradation (ft):	1.89

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Design Scour Depth C.O.T. EQTN 6.3

Kings Manor Wash Coyote Wash at SR-92

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	830
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.010
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0078
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0078
Natural Channel Slope * L_h (ft):	8.30
Design Equilibrium Slope * L_h (ft):	6.47
Long Term Aggradation/Degradation (ft):	1.83

E.5
Woodcutters Canyon Wash

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at Buffalo Soldiers Trail

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1030
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.014
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0109
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0109
Natural Channel Slope * L_h (ft):	14.42
Design Equilibrium Slope * L_h (ft):	11.23
Long Term Aggradation/Degradation (ft):	3.19

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at Busby Dr.

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2100
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.013
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0101
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0101
Natural Channel Slope * L_h (ft):	27.30
Design Equilibrium Slope * L_h (ft):	21.27
Long Term Aggradation/Degradation (ft):	6.03

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at drop structure downstream of 7th St.

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1750
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.011
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0086
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0086
Natural Channel Slope * L_h (ft):	19.25
Design Equilibrium Slope * L_h (ft):	15.00
Long Term Aggradation/Degradation (ft):	4.25

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at Fry Blvd.

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	395
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.010
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0062
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0062
Natural Channel Slope * L_h (ft):	3.95
Design Equilibrium Slope * L_h (ft):	2.43
Long Term Aggradation/Degradation (ft):	1.52

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at Golf Links Rd

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	890
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.008
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0058
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0058
Natural Channel Slope * L_h (ft):	6.68
Design Equilibrium Slope * L_h (ft):	5.20
Long Term Aggradation/Degradation (ft):	1.47

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at Lenzner Ave

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	495
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.011
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0068
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0068
Natural Channel Slope * L_h (ft):	5.45
Design Equilibrium Slope * L_h (ft):	3.35
Long Term Aggradation/Degradation (ft):	2.09

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Design Scour Depth C.O.T. EQTN 6.3

Woodcutters Canyon Wash at Wilcox Dr.

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	600
10-Year Natural Discharge, Q_n (cfs):	850
10-Year Urbanized Discharge, Q_u (cfs):	850
Natural Channel Bottom Width, b_n (ft):	45
Urbanized Channel Bottom Width, b_n (ft):	45
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.017
Reduction Factor for Sediment Supply, R_s:	0.50

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0105
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0105
Natural Channel Slope * L_h (ft):	10.20
Design Equilibrium Slope * L_h (ft):	6.28
Long Term Aggradation/Degradation (ft):	3.92

E.6

South Garden Wash

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Design Scour Depth C.O.T. EQTN 6.3

South Garden Wash at Avenida Cochise

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1040
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.009
Reduction Factor for Sediment Supply, R_s:	0.20

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0077
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0077
Natural Channel Slope * L_h (ft):	9.36
Design Equilibrium Slope * L_h (ft):	8.01
Long Term Aggradation/Degradation (ft):	1.35

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Design Scour Depth C.O.T. EQTN 6.3

South Garden Wash at Buffalo Soldiers Trail

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	990
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.012
Reduction Factor for Sediment Supply, R_s:	0.20

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0103
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0103
Natural Channel Slope * L_h (ft):	11.88
Design Equilibrium Slope * L_h (ft):	10.16
Long Term Aggradation/Degradation (ft):	1.72

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Design Scour Depth C.O.T. EQTN 6.3

South Garden Wash at E. Wardell Road

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1410
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.013
Reduction Factor for Sediment Supply, R_s:	0.20

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0111
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0111
Natural Channel Slope * L_h (ft):	18.33
Design Equilibrium Slope * L_h (ft):	15.68
Long Term Aggradation/Degradation (ft):	2.65

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Design Scour Depth C.O.T. EQTN 6.3

South Garden Wash at Oakmont Dr

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	1820
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.017
Reduction Factor for Sediment Supply, R_s:	0.20

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0145
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0145
Natural Channel Slope * L_h (ft):	30.94
Design Equilibrium Slope * L_h (ft):	26.47
Long Term Aggradation/Degradation (ft):	4.47

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Design Scour Depth C.O.T. EQTN 6.3

South Garden Wash at S. Wardell Road

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2850
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.012
Reduction Factor for Sediment Supply, R_s:	0.20

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0103
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0103
Natural Channel Slope * L_h (ft):	34.20
Design Equilibrium Slope * L_h (ft):	29.25
Long Term Aggradation/Degradation (ft):	4.95

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Design Scour Depth C.O.T. EQTN 6.3

South Garden Wash at Winterhaven Dr.

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	4500
10-Year Natural Discharge, Q_n (cfs):	800
10-Year Urbanized Discharge, Q_u (cfs):	800
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.013
Reduction Factor for Sediment Supply, R_s:	0.20

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0013
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0111
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0111
Natural Channel Slope * L_h (ft):	58.50
Design Equilibrium Slope * L_h (ft):	50.04
Long Term Aggradation/Degradation (ft):	8.46

E.7

Vista Village Wash

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Phone (520) 882-4244 Fax (520) 888-1421

Design Scour Depth C.O.T. EQTN 6.3

Vista Village Drainageway - Tacoma St Alignment to Catalina St

Client: City of Sierra Vista
Project #: 21-001

Date 04.12.2023
By: cmg

INPUTS	
Length to Hinge Point, (ft):	2140
10-Year Natural Discharge, Q_n (cfs):	1500
10-Year Urbanized Discharge, Q_u (cfs):	1500
Natural Channel Bottom Width, b_n (ft):	40
Urbanized Channel Bottom Width, b_n (ft):	40
Manning's "n" Natural Channel:	0.035
Manning's "n" Urbanized Channel:	0.035
Natural Channel Slope, S_n (ft/ft):	0.014
Reduction Factor for Sediment Supply, R_s:	0.30

Results	
Equilibrium Slope after urbanization, S_{eq} (EQ 6.25):	0.0012
Equilibrium Slope after urbanization, S_{eq} (EQ 6.26):	0.0109
Design Equilibrium Slope (Steepest of 6.26 & 6.25)	0.0109
Natural Channel Slope * L_h (ft):	29.96
Design Equilibrium Slope * L_h (ft):	23.34
Long Term Aggradation/Degradation (ft):	6.62

APPENDIX F – ADWR SS5-96

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

State Standard

for

Watercourse System Sediment Balance

Under authority of ARS 48-3605(a), the Director of the Arizona Department of Water Resources establishes the following standard for identification of and development within erosion hazard areas and areas affected by a net system sediment deficit or surplus in Arizona:

The guidelines outlined in State Standard Attachment 5-96 entitled "Watercourse System Sediment Balance" or by an alternative procedure reviewed and accepted by the Director will be used in the identification of, or regulation of development within erosion hazard areas, and watercourses affected by a net system sediment deficit or surplus in Arizona for fulfilling the requirements of Flood Insurance Studies, and local community and county flood damage prevention ordinances.

For the purpose of application of these guidelines, erosion hazard area and watercourse system sediment balance standards will apply to all watercourses identified by the Federal Emergency Management Agency as part of the National Flood Insurance Program, all watercourses which have been identified by the local floodplain administrator as having significant potential flood hazards and all watercourses with drainage areas more than 1/4 square mile or a 100-year discharge estimate of more than 500 cubic feet per second. Application of these guidelines will not be necessary if the local community or county has in effect a drainage, grading or stormwater ordinance which, in the opinion of the Department, results in the same or greater level of flood protection as application of these guidelines would ensure.

This requirement is effective October 1, 1996. Copies of this State Standard and State Standard Attachment 5-96 can be obtained by contacting the Department's Flood Warning and Dam Safety Section at (602) 417-2445.

NOTICE

This document is available in alternative formats. Contact the Department of Water Resources, Flood Warning and Dam Safety Section at (602) 417-2445 or (602) 417-2455 (TDD).

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD WARNING AND DAM SAFETY SECTION

Watercourse System Sediment Balance

500 North Third Street
Phoenix, Arizona 85004

(602) 417-2445

STATE STANDARD ATTACHMENT
SSA 5-96

SEPTEMBER 1996

Contents

Introduction

The National Flood Insurance Program regulations 44 CFR 60.5 require local communities to review permits for development with regard to erosion hazards in "flood-related erosion-prone areas". Specifically 44 CFR 60.5.a.2 states "...Require review of each permit application to determine whether the proposed site alterations and improvements will be reasonably safe from flood-related erosion and will not cause flood-related erosion hazards or otherwise aggravate the existing flood-related erosion hazard....".

This document contains three guidelines for identification of, and development within erosion hazard areas, watercourses with a net sediment deficit, and watercourses with a net sediment surplus. The three guidelines in this document each contain their own table of contents relevant to its particular subject. These guidelines are:

Guideline 1: Lateral Migration Setback Allowance for Riverine Floodplains in Arizona

Guideline 2: Channel Degradation Estimation for Alluvial Channels in Arizona

Guideline 3: Evaluation of River Stability Impacts associated with Sand and Gravel Mining

Guideline 1 presents procedures for estimating the size of buffer (setback distance) that shall be provided along watercourses to allow for the lateral migration that may occur during future floods. Three methods of setback evaluation are discussed -- a first level procedure to be applied in normal conditions, a second level procedure for use in demonstrating the erosion resistance of existing channel materials, and a third level procedure which may be applied in unusual circumstances, or where more definite dimensioning of lateral migration potential is desired.

Guideline 2 presents procedures that may be used for estimation of channel degradation in unlined watercourses within Arizona. Three levels of procedures are provided, with data requirements, procedural complexity, and accuracy of results all increasing as the analysis level is incremented. The Level I approach provides an initial estimate of local channel degradation potential for generally stable, natural channel conditions. The resulting initial estimate may be reduced through use of the more rigorous Level II methodologies. Level III procedures are outlined for situations that warrant more detailed channel degradation determination.

Guideline 3 presents general guidelines that have been developed for determination of the adequacy of buffer areas between proposed mining operations and active river channels, and procedures that are available for analysis of the effects of instream activities.

A large part of Arizona has a "Basin and Range" topography which consists of mountain "blocks" of hard rock areas and adjoining basins that are filled with sediments which have been deposited by water (alluvium). The mountain areas do not have a problem with channel migration due to the stability of bed rock and large fragment rock found there. Basin areas, or the valley and low land areas containing alluvium are characterized by sediments that are erodible. The many variables associated with channel lateral migration, sediment balance, river mechanics, and hydraulic engineering preclude the development of a comprehensive design manual in this short document: therefore, these guidelines are intended to be utilized with good engineering judgement and common sense.

Within this document the following acronyms will be used:

ADWR	Arizona Department of Water Resources
FEMA	Federal Emergency Management Agency
NFIP	National Flood Insurance Program

GUIDELINE 1

Lateral Migration Setback Allowance for Riverine Floodplains in Arizona

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Introduction

The floodplain boundaries associated with a given watercourse are not fixed features if the channel shifts and migrates over the course of time. Lateral migration of river channels is commonly observed in the arid southwest, where the flows are predominantly ephemeral and the bed and banks tend to be erodible. The migration relocates the channel banks and redefines the location of the river for the current and subsequent flow events.

This document presents procedures for estimating the size of buffer (setback distance) that shall be provided along watercourses to allow for the lateral migration that may occur during future floods. Three methods of setback evaluation are discussed -- a first level procedure to be applied in normal conditions, a second level procedure for use in demonstrating the erosion resistance of existing channel materials, and a third level procedure which may be applied in unusual circumstances, or where more definite dimensioning of lateral migration potential is desired.

Procedure

General

Three levels of analysis procedures are presented for determination of recommended setback distances for development in areas adjacent to watercourses. The Level I procedure provides a reasonable estimate of safe setback distance under normal conditions, with minimal channel geometry and hydrologic information required in its application. The higher level procedures, Level II and Level III, are more rigorous means of determining lateral migration potential, requiring knowledge of site specific hydraulic and channel material characteristics. The Level II procedure is provided as a straightforward means of demonstrating the stability of channel banks, in cases where a developer or floodplain manager seeks to apply a lesser setback than may be computed through application of the Level I equations. A flowchart outlining the procedure is provided on the following page. The Level III approaches referenced may be used for this purpose as well, or may be required by the local regulating agency for analysis of areas of particular concern, such as the following situations where the Level I allowances or Level II evaluations may not fully demonstrate the lateral migration potential:

(i) areas where massive shifting of the river channel has been observed in the past;
(ii) areas undergoing channel filling (aggradation) to a significant degree;
or, (iii) areas where local river mining, channelization, or other modifications could result in flow redirection unanticipated in the development of the Level I or Level II approaches.

Level I

This level of analysis requires the following information:

Drainage area. The area of the watershed contributing to the site of interest. Drainage areas should be estimated conservatively to account for all possible sources of runoff. USGS topographic quadrangle maps usually provide sufficient detail for delineating watershed areas.

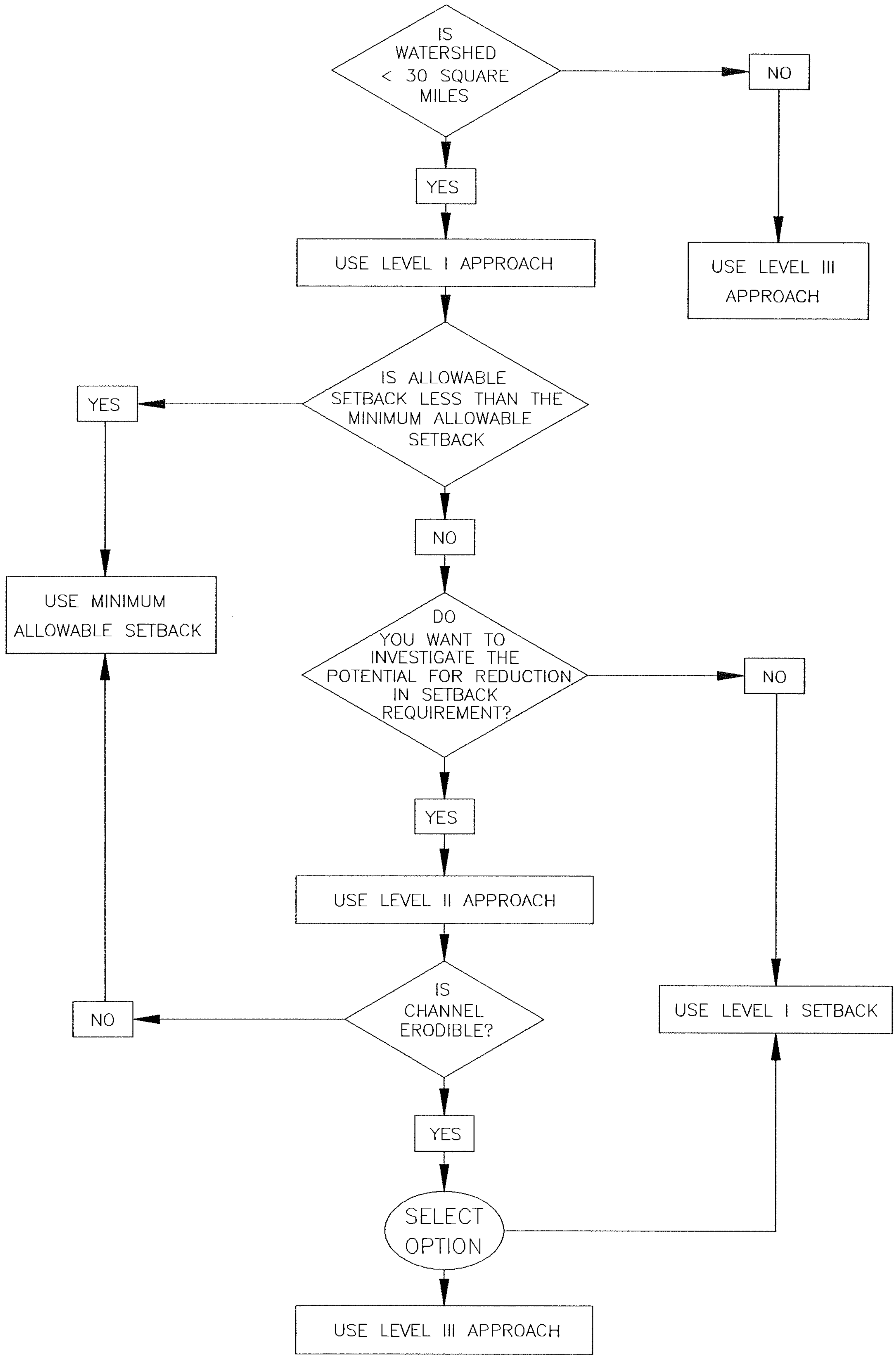
Peak discharge associated with the 100-year flood (Q_{100}). May be estimated using simplified methodologies such as ADWR State Standard #2 (SS 2-96), USGS regression equations, or other similar approximate method.

A Level I or Level II analysis should not be used on watercourses which have drainage areas greater than 30 square miles. If the watercourse has a drainage area greater than 30 square miles, a Level III analysis shall be performed.

For watercourses which have drainage areas of less than 30 square miles, the recommended setback allowances are as follows:

for straight channel reaches or reaches with minor curvature:	setback $= 1.0(Q_{100})^{0.5}$
for channels with obvious curvature or channel bend:	setback $= 2.5(Q_{100})^{0.5}$

where setback is in feet and Q_{100} is in cubic feet per second.



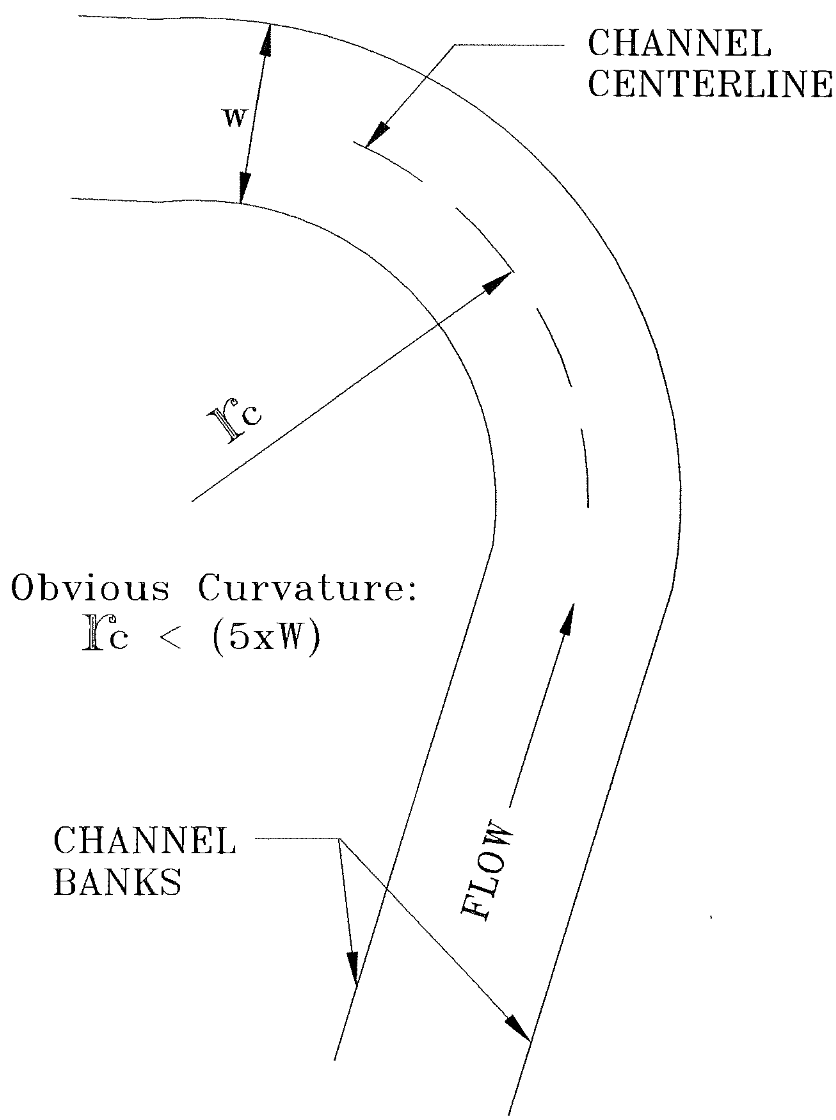
IS WATERSHED < 30 SQUARE MILES
NO
YES
USE LEVEL I APPROACH
USE LEVEL III APPROACH
IS ALLOWABLE SETBACK LESS THAN THE MINIMUM ALLOWABLE SETBACK
YES
NO
USE MINIMUM ALLOWABLE SETBACK
DO YOU WANT TO INVESTIGATE THE POTENTIAL FOR REDUCTION IN SETBACK REQUIREMENT?
NO
YES
USE LEVEL II APPROACH
IS CHANNEL ERODIBLE?
NO
USE LEVEL I SETBACK
YES
SELECT OPTION
USE LEVEL III APPROACH

FLOWCHART

In all cases for the Level I analysis, the minimum setback shall be 20 feet for straight channel reaches and 50 feet for channels with obvious curvature. Obvious curvature is defined as a channel centerline with a radius of curvature less than 5 times the channel top width.

The setback allowance is to be measured outward from the 100-year floodway or the top of the channel bank, whichever is greater. The above equations provide a larger setback allowance in areas with relatively tight channel bends. This larger setback allowance is to applied in areas adjacent to the outside bend of the channel.

A sketch is provided below to help differentiate between minor curvature and obvious curvature.



CHANNEL CURVATURE

Level II

This approach may be applied to demonstrate the stability of the channel material under 100-year flood conditions, and to justify a lesser setback requirement than that computed using the Level I equations. Setback allowances for conditions which pass one or more of the following channel stability approaches, and which are not located in areas of specific concern (i.e. areas adjacent to river mining sites, highly aggradational areas, or areas with artificial flow redirection) should be based on normal building safety criteria rather than the Level I equations presented above, since the bank limits would not be expected to change during the course of a 100-year design event.

Allowable velocity analysis

Under this approach, the velocity of the 100-year peak flow within the watercourse adjacent to the site under consideration is compared to an "allowable" velocity -- the velocity at and below which erosion is not expected to occur.

The basic maximum allowable velocity for unprotected earthen channels is determined from a relationship developed by the USDA Soil Conservation Service, shown in the attached **Figure 1**. In order to use this figure, flow must be classified as either sediment free or sediment laden. Sediment free flow is defined as flow in which fine material in suspension is at concentrations so low that it has negligible effect upon channel stability. Sediment free flows generally have sediment concentrations of less than 1,000 parts per million (ppm) by weight. Sediment-laden flows are classified as flows carrying sediments in concentrations equal to or exceeding 20,000 ppm, by weight.

Typical natural channel flows within Arizona can be characterized as sediment-laden when flow occurs. The sediment-free curve in Figure 1 should be used only under unusual circumstances, such as for runoff which emanates from a totally impervious watershed.

Use of Figure 1 requires that the D_{75} particle size (the size for which 75% of the sediment, by weight, is finer) be known for the soil forming the channel banks. This information can be obtained from a sieve analysis or alternate means should there be large fragmented rock present.

The basic allowable maximum velocity obtained from Figure 1 must normally be modified to account for variations in channel design. This is done by the use of correction factors for channel alignment, bank slope, and depth of flow. The equation for allowable velocity, V_a, in an unprotected earthen channel then becomes:

$$V_a = V_b \times C_a \times C_b \times C_d$$

where

V_a = Maximum allowable flow velocity, in feet per second;
V_b = Basic maximum allowable flow velocity obtained from Figure 1, in feet per second; and,
C_a, C_b, C_d = Correction factors for channel alignment, bank slope, and flow depth, respectively (see Figure 2 through 4).

Tractive stress analysis

Flowing water exerts a tangential boundary pull on the wetted perimeter of the channel boundary. The total force exerted on the boundary by the flow of water is called the tractive force. The tractive stress is the tractive force per unit area of the boundary. Tractive force and tractive stress are equal to the friction forces resisting the flow of water. Tractive stress can therefore be used as a method of determining the erodibility of an earthen channel. To accomplish this, the tractive stress is compared to an allowable tractive stress for the bed material.

Case 1: 0.25 inches $< D_{75} <$ 5.0 inches

The tractive stress acting on the soil grains in an infinitely wide channel can be computed from:

$$\tau_\infty = \gamma_w Y \, [\, D_{75}^{1/6} / 39n \,]^2 \, S_e$$

where

τ_∞	=	Tractive stress for an infinitely wide channel, in lbs/ft^2;
γ_w	=	Unit weight of water = 62.4 lbs/ft^3;
D_{75}	=	Diameter of soil particle for which 75 percent of the total soil consists of smaller particles, in inches;
n	=	Manning's roughness coefficient for the channel;
Se	=	Energy slope of flowing water, in feet per foot; and,
Y	=	Depth of flow, in feet.

Once the tractive force for an infinitely wide channel is determined, it must be modified for a narrower trapezoidal channel. **Figures 5 through 7** give correction factors for tractive stresses in trapezoidal and curved channels. The correction factors taken from these figures are multiplied by the tractive stress computed from the above equation to obtain the actual tractive stress.

The definitions of the symbols shown in Figures 5 through 7 are as follows:

τ_s	=	Actual maximum tractive stress on sides of straight trapezoidal channels, in pounds per square foot;
τ_{sc}	=	Actual maximum tractive stress on sides of trapezoidal channels within a curved reach, in pounds per square foot;
τ_{st}	=	Actual maximum tractive stress on sides of trapezoidal channels in straight reaches immediately downstream from curved reaches, in pounds per square foot;
Z	=	Channel side slope (horizontal/vertical), in feet per foot;
b	=	Channel bottom width, in feet;
y	=	Flow depth, in feet;
r_c	=	Radius of curvature of channel centerline, in feet; and,
L_c	=	Length of curve, in feet.

The actual tractive stress is compared to an allowable tractive stress to determine the propensity of the soil to erode under the expected hydraulic conditions. The allowable tractive stress is calculated by:

$$\tau_{ls} = 0.4 \, [\, (Z^2 - \mathrm{Cot}^2 \, \phi R) \, / \, (1+Z^2) \,]^{1/2} \, D_{75}$$

where

τ_{ls}	=	Allowable tractive stress, in lb/ft^2; and,
ϕR	=	Angle of repose of soil, in degrees (see **Figure 8**).

Case 2: $D_{75} \leq 0.25$ inches

Under these conditions, a reference tractive stress as determined from **Figures 9 and 10** is used, following the steps listed below:

1. Determine the velocity (V), kinematic viscosity (ν), and the energy slope (S_e) for the channel.

2. Enter Figure 9 or 10, from the top, with a value computed from the expression:

$$V^3 \, / \, (g \nu S_e)$$

Find the point of intersection of the above value and the value of:

$$V / (gk_sS_e)^{1/2}$$

where

k_s = Equivalent roughness height = D_{65}, in feet (the size for which 65% of the sediment, by weight, is finer).

3. Move horizontally along the figure to read the numerical value for:

$$V / (\tau/\rho)^{1/2}$$

where

τ = Reference tractive stress, in pounds per square foot;
V = Flow velocity, in feet per second; and,
ρ = Density of water = 1.94 slugs per cubic foot.

The value for τ can be found by equating the numeric value read from Figure 9 or 10 to this expression.

The maximum tractive stress on the sides of the channel, τ_s, can be computed from the reference tractive stress and a correction factor obtained from **Figure 11**. Figures 6 and 7 may be used to further modify the reference tractive stress for curved channel reaches. The adjusted reference tractive stress is then compared to the allowable tractive stress determined from **Figure 12**.

Curve number 1 in Figure 12 is to be used when the flow is expected to have a high sediment content. A high sediment content is considered to be 20,000 ppm, by weight, or more of sediment. Curve number 2 is to be used for watercourses with low sediment contents of no more than 2,000 ppm, by weight. This curve should only be used in association with areas of high impervious cover (> 50%) and/or downstream of urban area detention basins. Interpolate between curves 1 and 2 for water courses with known sediment content between 2,000 ppm and 20,000 ppm. Curve number 3 is to be used for watercourses conveying clear water, and should not be used unless unusual circumstances exist (e.g., runoff which emanates from a totally impervious watershed).

Tractive power analysis

Tractive power is defined as the product of the mean velocity of flow and the tractive stress. The tractive power analysis takes into consideration the effects of cementation, partial lithification, and other long-term processes that can affect the ability of the channel to withstand erosion. Neither the velocity analysis nor the tractive stress analysis account for the effects of these long-term processes. With the tractive power approach, the stability of saturated soils comprising the channel banks is first assessed by the use of an unconfined compression test. The unconfined compressive strength (UCS) of these saturated embankment soils is then reduced by at least a factor of two, for design purposes, and compared to the tractive power of the flow by use of **Figure 13**. Conditions falling above the S-line in this figure are considered to be erosive, and those falling below the S-line are considered to be non-erosive. The method has some limitations due to variability and stratification of material along natural channels, and the limited data available to develop Figure 13.

Bank Lining Adequacy Analysis

Bank lining of some form may be proposed or already in place which may act to limit the lateral migration potential of the watercourse of concern. In some areas within Arizona, procedures are in place for assessment of the adequacy of the bank protection measures. For areas without standardized procedures, two references are recommended which detail evaluation procedures:

Design Manual for Engineering Analysis of Fluvial Systems, Arizona Department of Water Resources, 1985.

Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, City of Tucson Department of Transportation, Engineering Division, 1989.

Level III

This level of analysis involves modeling the hydraulic and sediment transport characteristics of the local watercourse in order to simulate the erosion/sedimentation and channel deformation processes which are expected to occur in the area of concern. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for establishment of setback be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles;
and, (3) steady state hydraulic and sediment transport analysis.

Works Cited

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Requirement for Floodplain Delineation in Riverine Environments - State Standard 2-96", July 1996.

Arizona Department of Water Resources, "Design Manual for Engineering Analysis of Fluvial Systems", March 1985.

City of Tucson Department of Transportation, Engineering Division. "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona. December 1989.

Thomas, B.E., H.W. Hjalmarson, and S.D. Waltemeyer. "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States." USGS Open-File Report. 1994. 93-419

Example Application

Example 1: Development Adjacent to a Watercourse

- **Problem Statement.** Single lot development proposed on 1-acre parcel bordered on one side by a small, earthen channel. The contributing watershed upstream of the site is 700 acres in area.

- **Objective.** Determine setback allowance from top of channel bank.

Level I Analysis

A 100-year peak discharge value of 530 cfs was determined from local hydrology methodology. The width of the 100-year floodplain in the site vicinity is 35 feet. The site is adjacent to the outside of a mild bend (i.e., radius of curvature greater than 5 times topwidth) in the channel.

Calculations:

A = 700 acres x (1 sq. mile/640 acres) = 1.09 sq. miles $<$ 30 square miles

setback = $1.0\ (530)^{0.5}$ = 23 feet

Since the calculated setback is greater than the minimum recommended setback of 20 feet, use a 23 foot setback. The setback is measured from the top of the channel bank or the 100-year floodway limit, whichever is greater.

Level II Analysis

The developer would like to minimize the setback as much as possible without having to provide bank lining. Accordingly, the site specific hydraulic and grain size information is collected to check if erosion of the channel would be naturally limited. Local geometry for the channel is obtained using site measurements:

Bottom Width = 15 feet
Side Slope = 2 horizontal to 1 vertical
Channel Slope = Energy Slope = 0.01 feet/foot
Radius of curvature = 500 feet

The Manning n value for the channel is estimated at 0.030.

Using normal depth procedures, the hydraulic characteristics of the local channel under 100-year flood conditions are determined:

Flow Depth = 3.0 feet
Flow Velocity = 8.4 feet/second

Results of a sieve analysis of a local channel material sample yields the following information:

D_{75} = 4 mm = 0.013 ft = 0.16 inches
D_{65} = 1.2 mm = 0.0039 ft = 0.05 inches
D_{50} = 0.6 mm = 0.002 ft = 0.024 inches

Calculations:

(1) Allowable velocity approach, assuming sediment laden flow

Entering Figure 1 with D_{75} = 4 mm yields a basic velocity of 4.0 ft/sec.
Entering Figure 2 with r/w = 18.5 yields C_a = 1.0
Entering Figure 3 with Z = 2 yields C_b = 0.72
Entering Figure 4 with Depth = 3.0 feet yields C_c = 1.0

Maximum allowable velocity = (4.0)(1.0)(0.72)(1.0) = 2.9 ft/sec

Since the computed velocity of 8.4 ft/sec exceeds the maximum allowable velocity, erosion may be expected to occur.

(2) Tractive stress approach

Since D_{75} is less than 0.25 inches, the reference tractive stress method is used;

Assuming a water temperature of 60° F, the kinematic viscosity (ν) = 0.0000121 ft²/sec, and the density (ρ) = 1.94 slugs/ft³

Compute $V^3/(g\nu S_e) = 1.52 \times 10^8$

Compute $V/[(gD_{65}S_e)^{1/2}] = 237$

From Figure 9, $V/(\tau/\rho)^{1/2} = 19.0$

Solving the above equation yields $\tau = 0.38$ lb/ft².

From Figure 11, with bottom width over flow depth (b/Y) = 15/3 = 5, $\tau_s = (1.03)\tau = 0.39 \text{ lb/ft}^2$.

From Figure 6, with radius of curvature over bottom width (r/b) = 500/15 = 33, $\tau_{sc} = 1.0\ \tau_s = 0.39 \text{ lb/ft}^2$.
[Note that radius of curvature over bottom width is used in this procedure while radius of curvature over top width of flow is used in the allowable velocity approach.]

From Figure 12, Curve 1 (for high sediment content), the allowable tractive force is 0.083 lb/ft^2. Since 0.083 is less than 0.39, the channel is erosive.

(3) Tractive power approach

An unconfined compressive strength (UCS) test of the saturated embankment soils is performed, yielding a strength of 1000 lb/ft3.

Assuming half of this strength for design purposes, UCS_{design} = 500 lb/ft^3.

Compute tractive power = $V\tau_{sc} = 3.3$

From Figure 13, the condition falls above the S-Line, indicating that the channel is erosive.

All three approaches indicate that the channel is erosive. Therefore, the 23 foot setback allowance determined by Level I procedures can not be reduced unless the channel banks are armored or the channel is obviously in bedrock.

Level III Analysis

The conclusions derived from the Level II analysis and the small size of the development indicate that the Level III analysis would probably not be applied in this case. However, should the developer wish to proceed with the setback allowance investigation, a registered engineer with experience in sediment transport modeling could be employed for this purpose. The engineer would be expected to collect available historic information, document the historic planform changes to the watercourse under events of varying frequency, apply steady state hydraulic and sediment transport calculation procedures to determine the erosion/sedimentation characteristics of the local reach of channel, and, potentially apply a moveable boundary river simulation model to quantify the changes likely along the study reach under design event conditions.

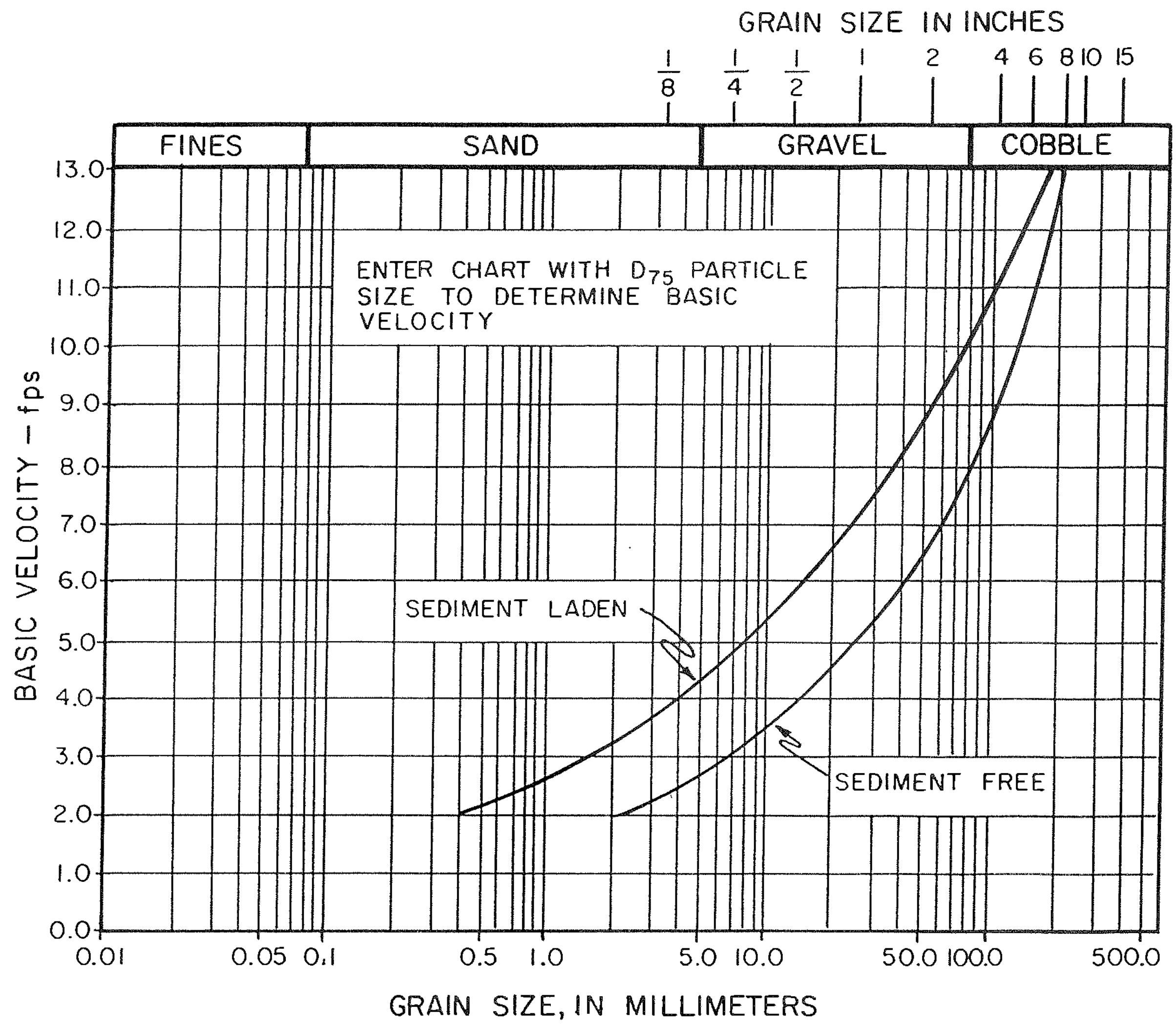
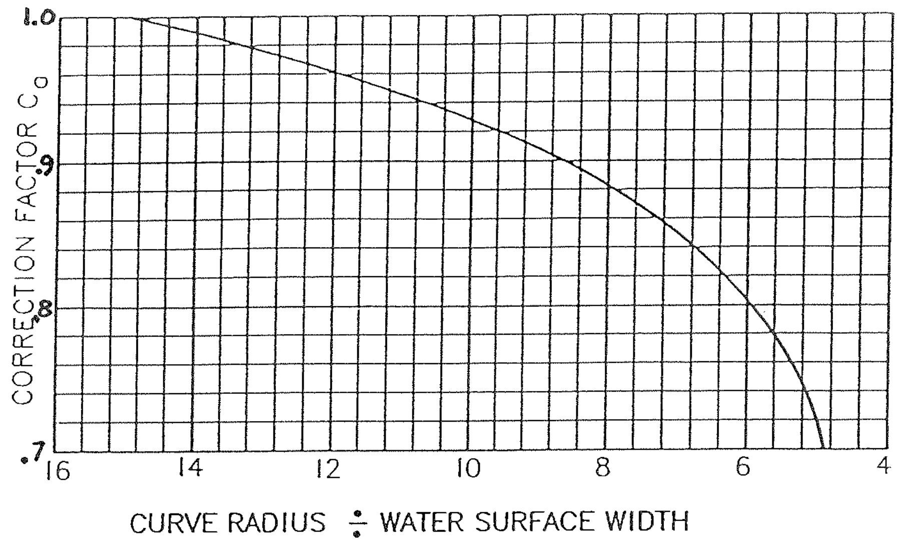


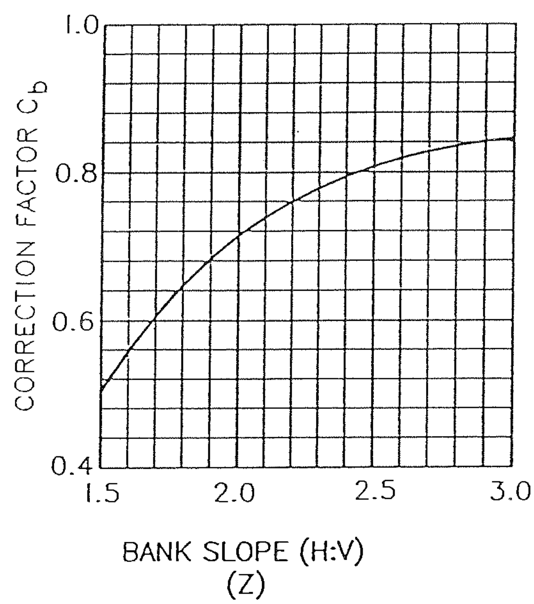
FIGURE 1

BASIC ALLOWABLE VELOCITY FOR EARTHEN CHANNELS



<u>FIGURE 2</u>

CORRECTION FACTOR C_a FOR CHANNEL ALIGNMENT



<u>FIGURE 3</u>

CORRECTION FACTOR C_b FOR BANK SLOPE

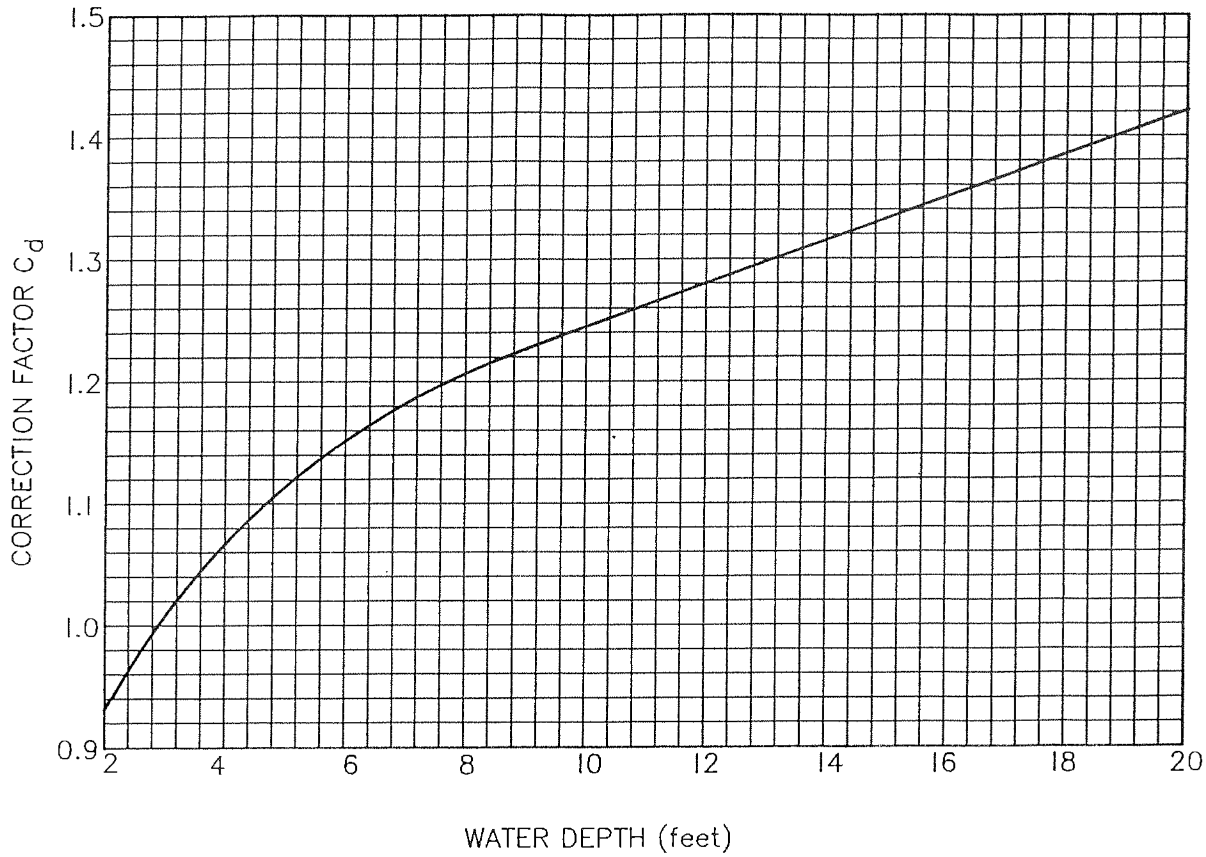


FIGURE 4
CORRECTION FACTOR C_d FOR DEPTH OF FLOW

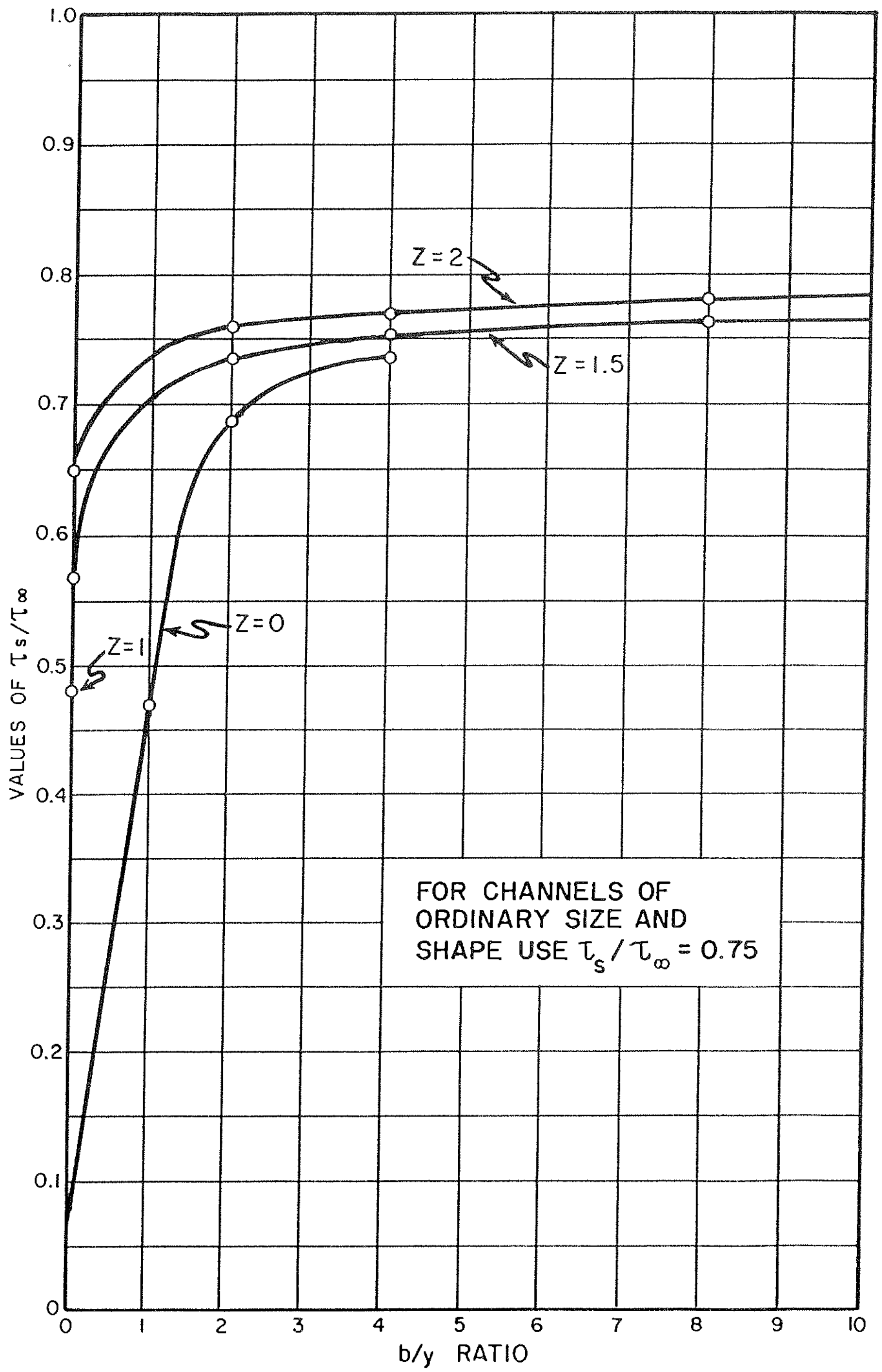


FIGURE 5

ACTUAL MAXIMUM TRACTIVE STRESS, τ_s, ON SIDES OF STRAIGHT TRAPEZOIDAL CHANNELS

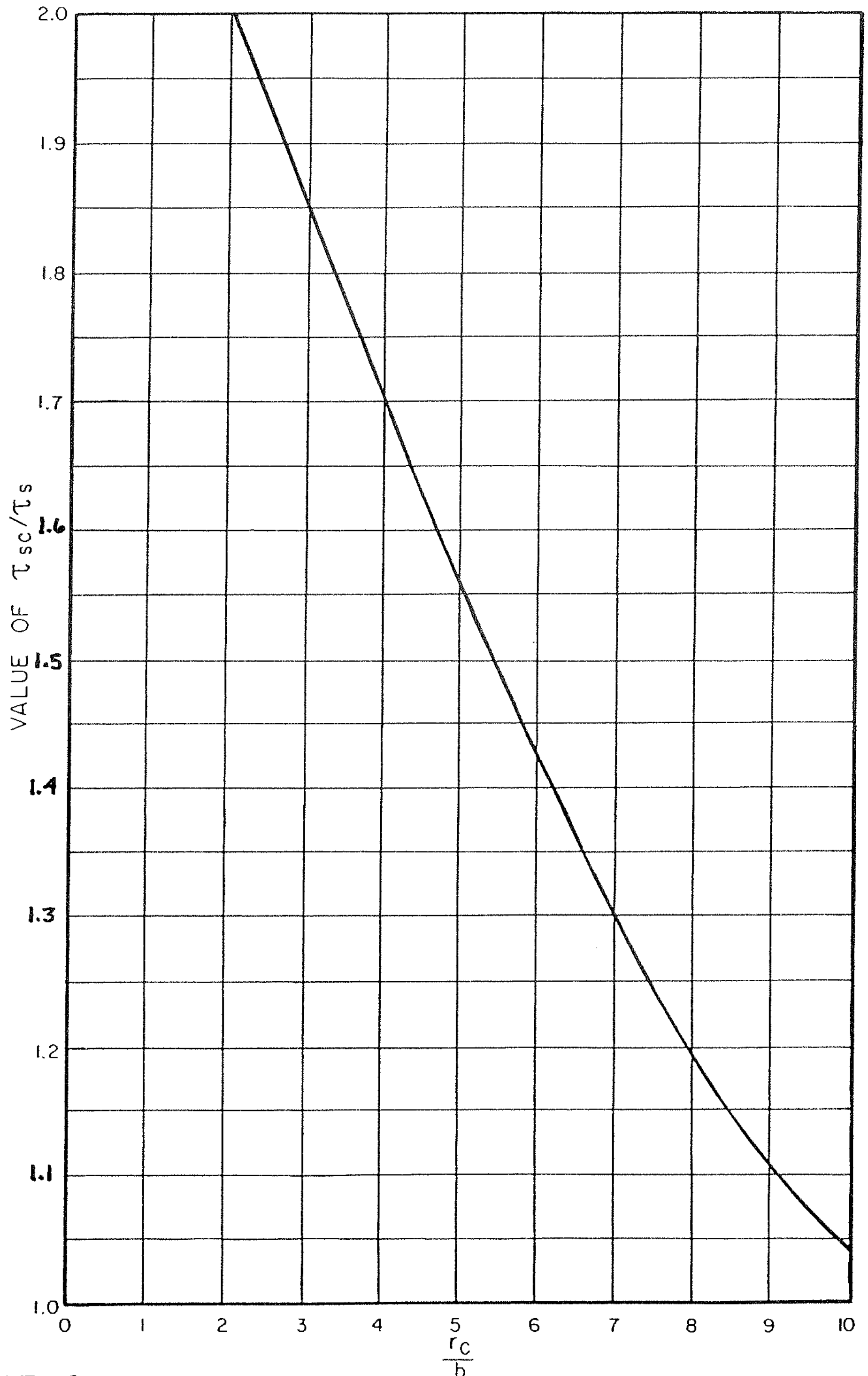


FIGURE 6

ACTUAL MAXIMUM TRACTIVE STRESS, τ_{sc}, ON SIDES OF TRAPEZOIDAL CHANNELS WITHIN A CURVED REACH

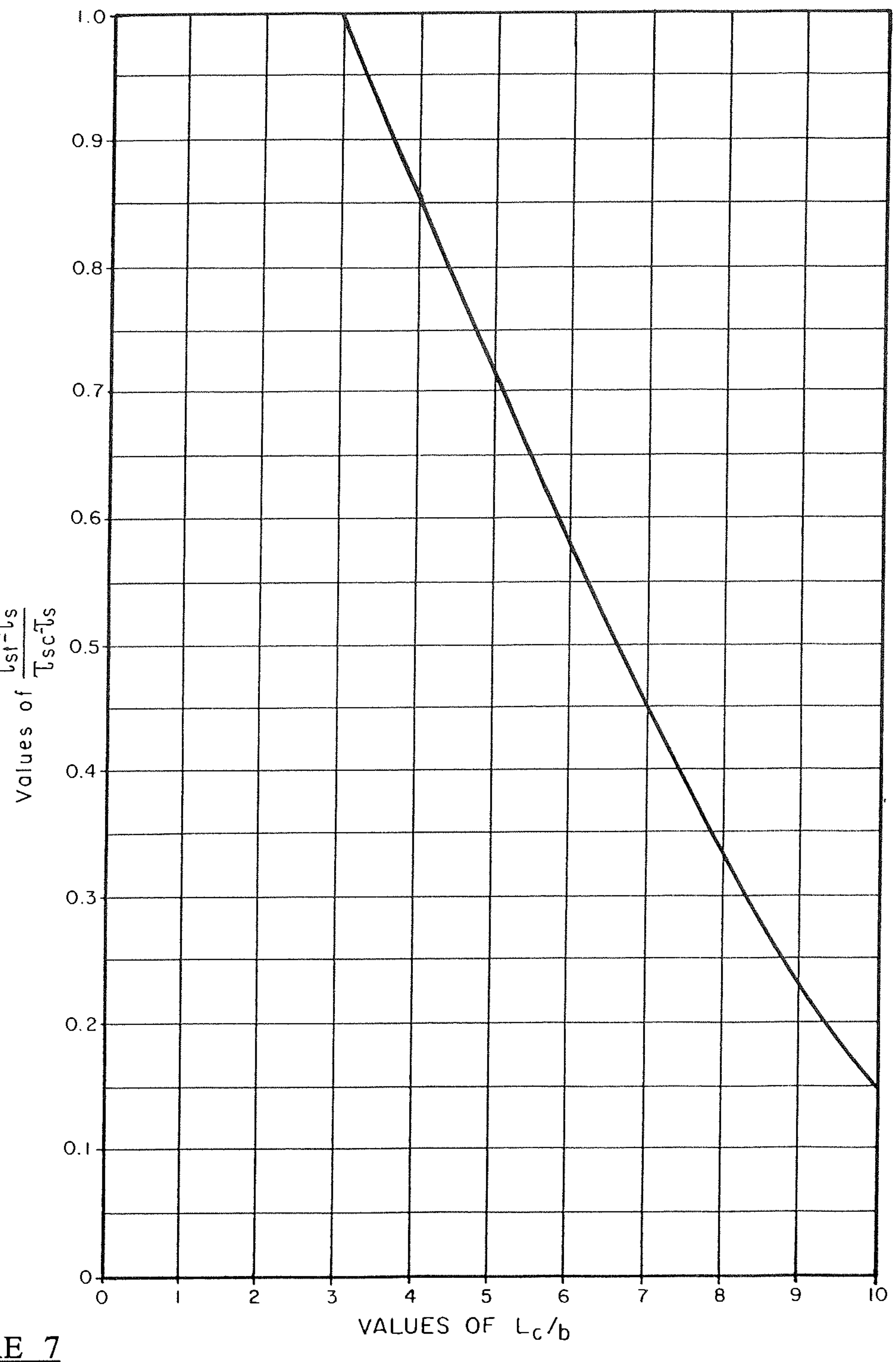


FIGURE 7

ACTUAL MAXIMUM TRACTIVE STRESS, τ_{st}, ON SIDES OF TRAPEZOIDAL CHANNELS IN STRAIGHT REACHES IMMEDIATELY DOWNSTREAM FROM CURVED REACHES

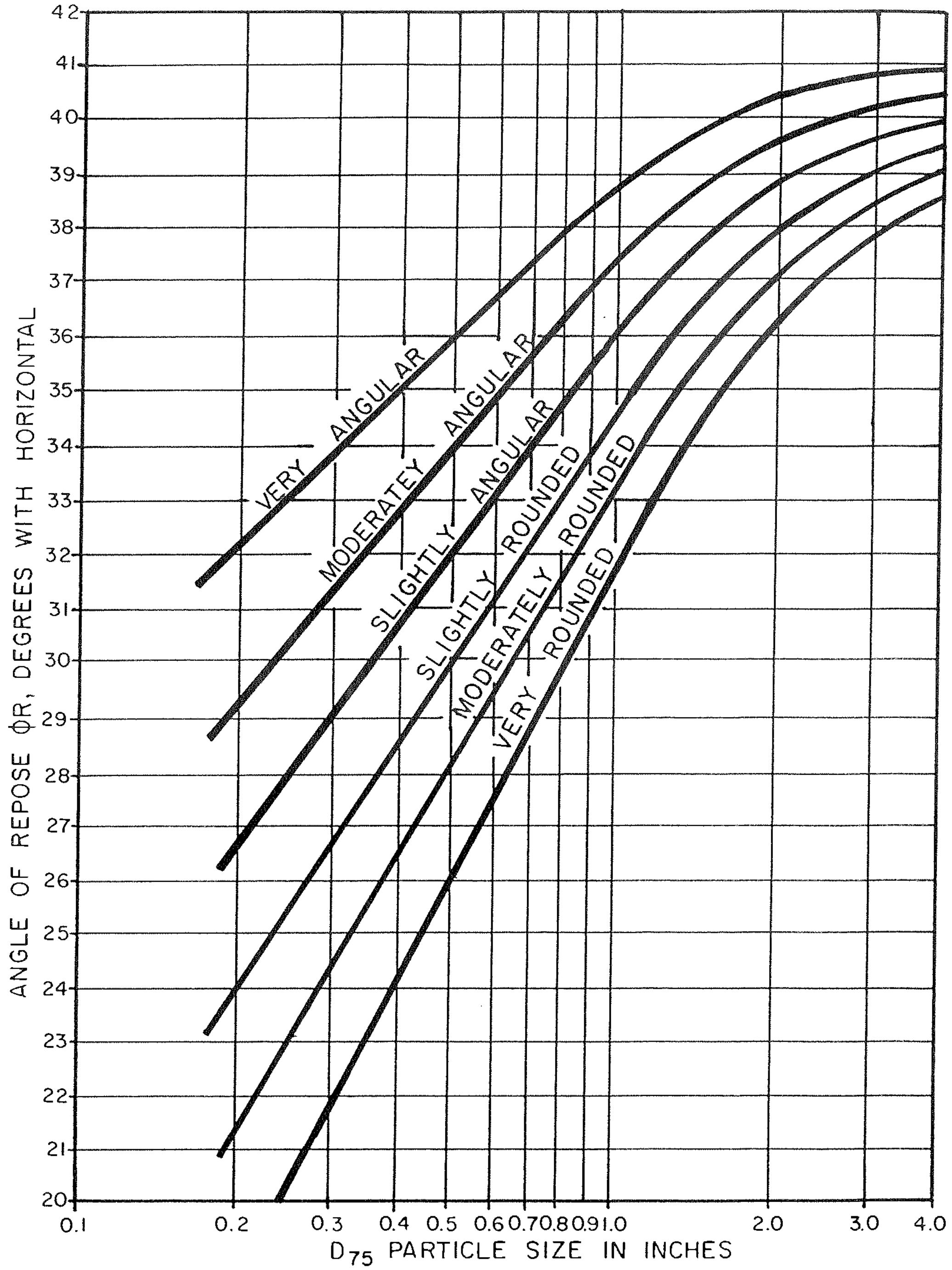


FIGURE 8

ANGLE OF REPOSE, ϕR, FOR NON-COHESIVE MATERIALS

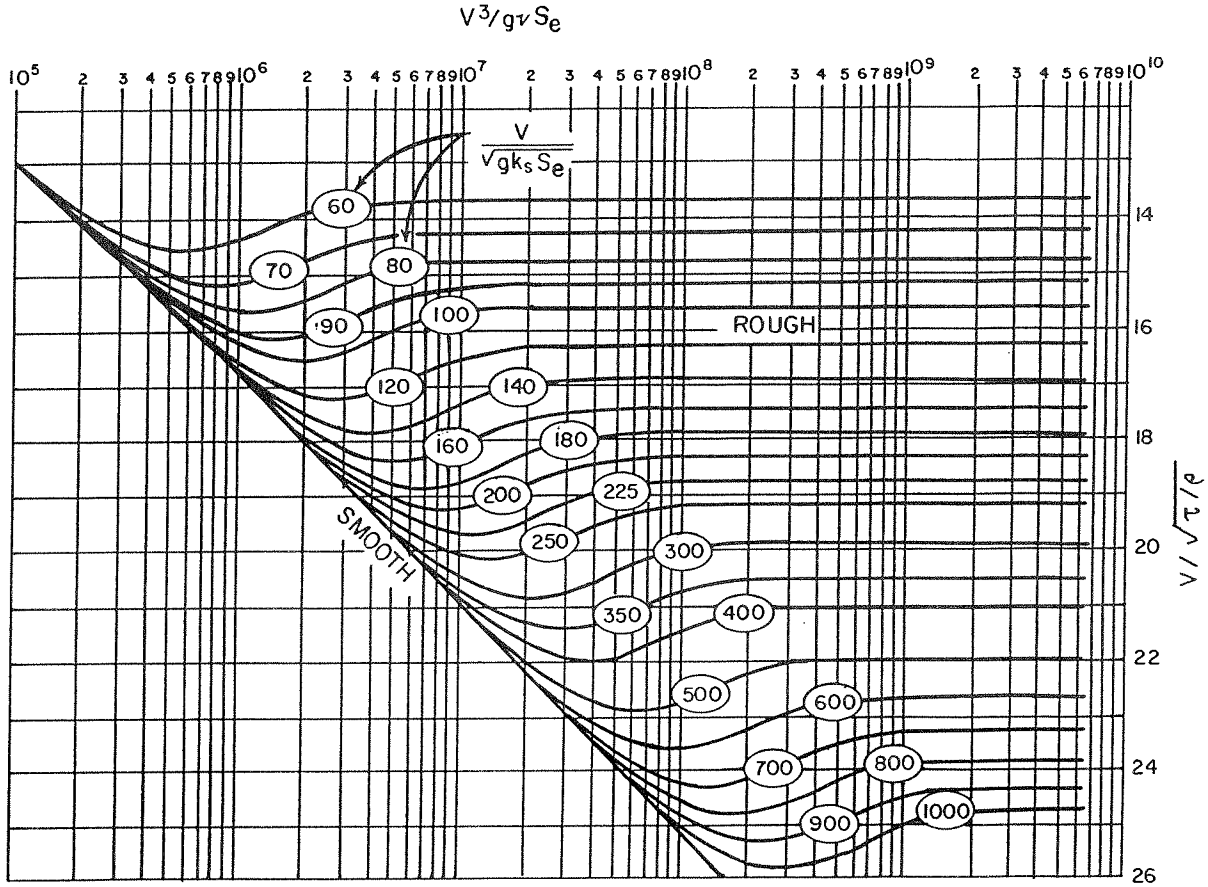


FIGURE 9

GRAPHIC SOLUTION OF REFERENCE TRACTIVE STRESS

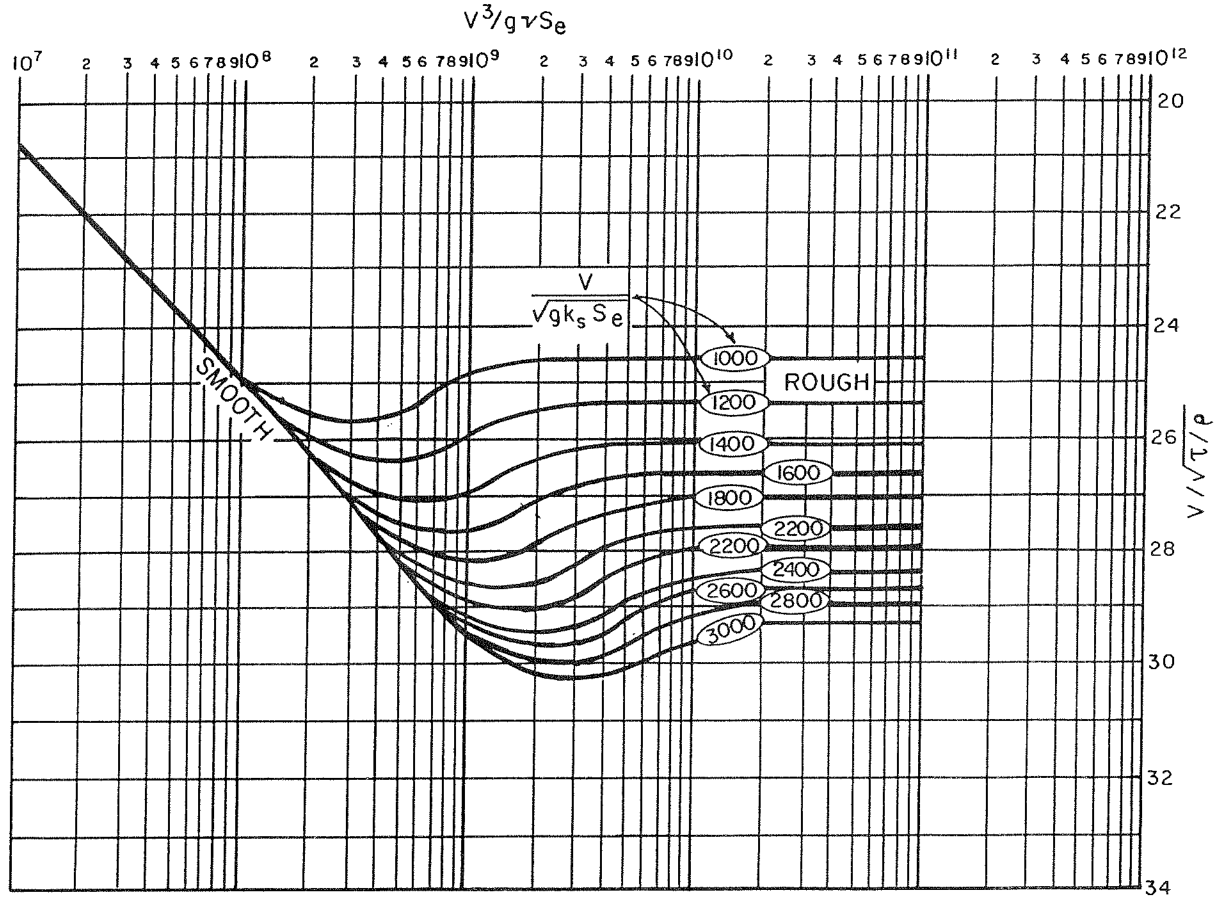


FIGURE 10

GRAPHIC SOLUTION OF REFERENCE TRACTIVE STRESS (CONTINUED)

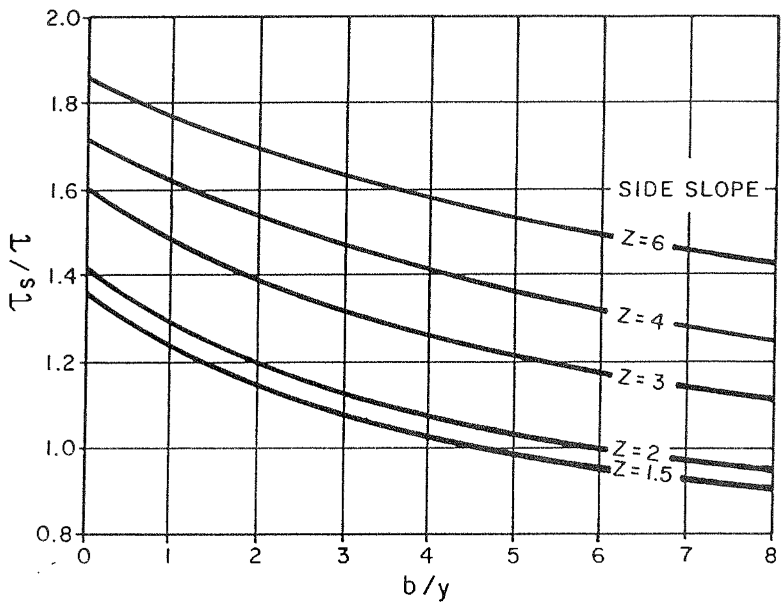


FIGURE 11

APPLIED MAXIMUM TRACTIVE STRESSES, τ_s, ON SIDES OF STRAIGHT TRAPEZOIDAL CHANNELS

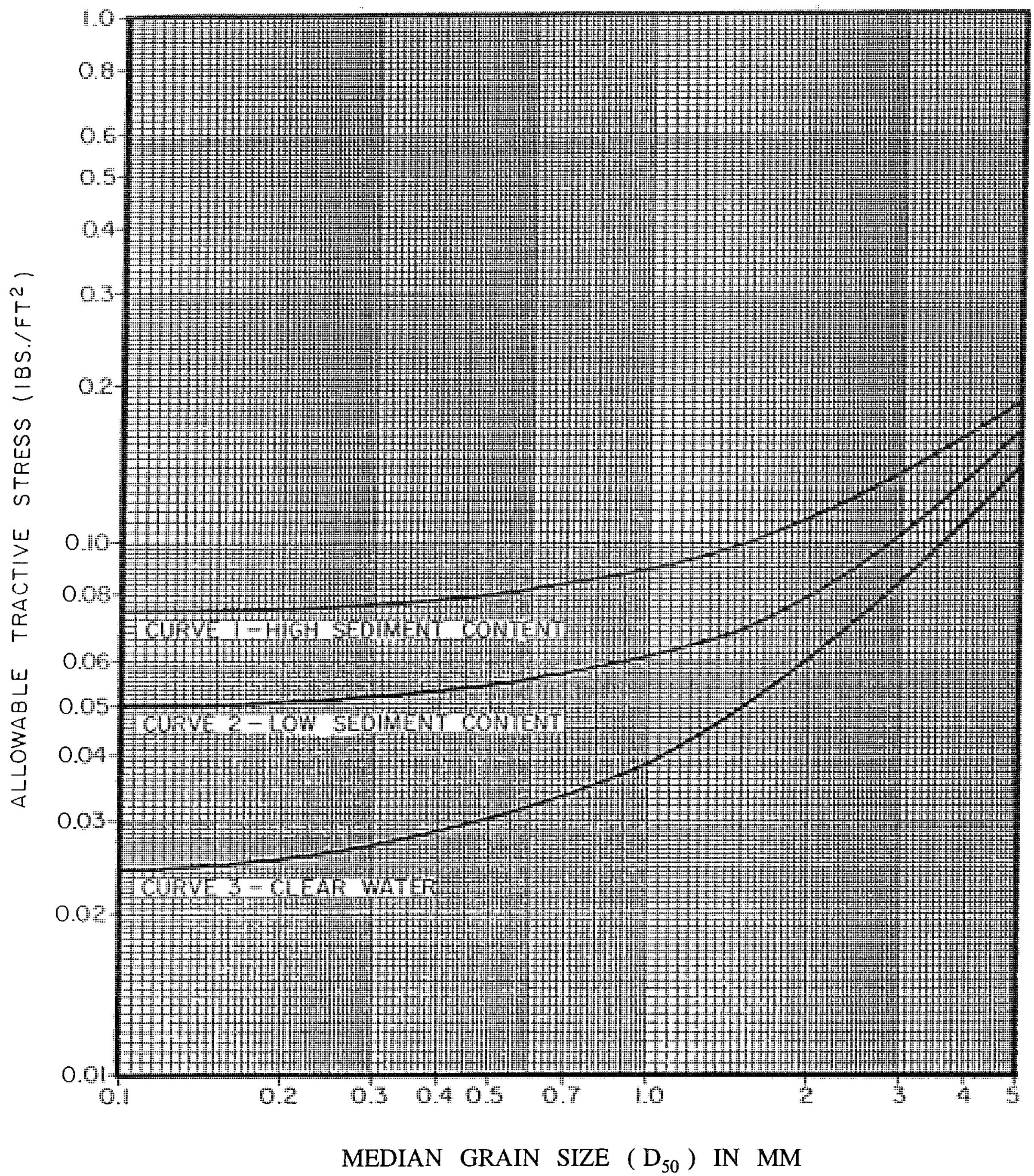


FIGURE 12

MAXIMUM ALLOWABLE TRACTIVE STRESS FOR NON-COHESIVE SOILS, $D_{75} < 0.25''$

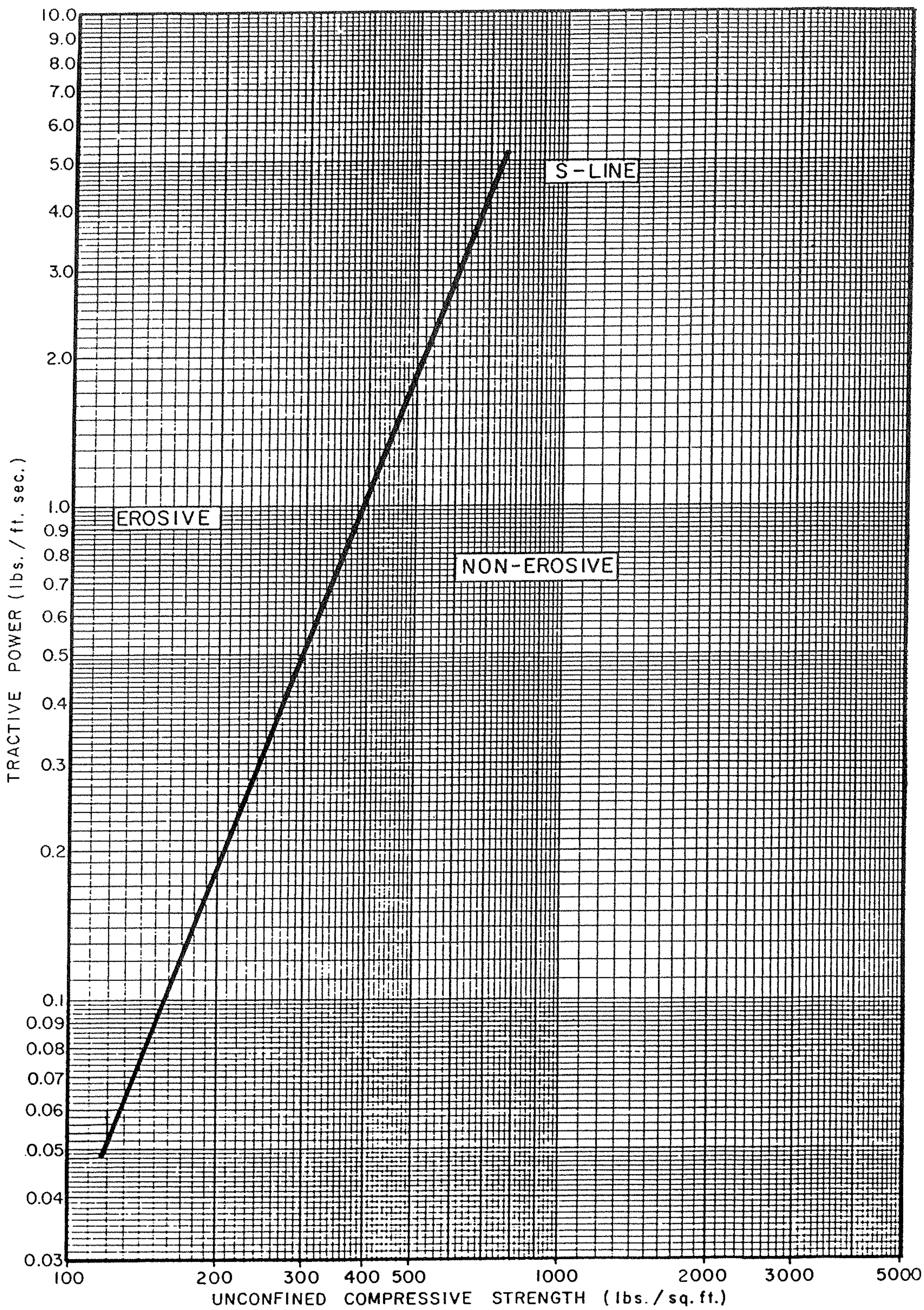


FIGURE 13

UNCONFINED COMPRESSIVE STRENGTH AND TRACTIVE POWER AS RELATED TO CHANNEL STABILITY

GUIDELINE 2

Channel Degradation Estimation
for Alluvial Channels
in Arizona

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Introduction

Channel degradation occurs within watercourses composed of erodible material, where local or general differentials in sediment transport capacity exist. Numerous factors control the short and long term degradation potential of channel reaches, including the size and cohesiveness of the material of which the channel is composed, the vegetation type and density in the channel, the hydraulic characteristics generated within the channel under flood events, and the existence of flow redirection or concentration structures within the channel. A key factor, however, is the amount of variation in channel properties from reach to reach. A channel reach attempts to adjust to conditions imposed on it by factors occurring up- and downstream; thus, the more uniform the channel is along the system under study, the less the potential exists for channel degradation to be a significant factor. Natural and man-made discontinuities along the system can create local increases in sediment transport potential, which often result in local degradation of the channel. System-wide disturbances, such as those associated with urbanization of the watershed or dam construction, have more far reaching impact, as the entire channel is forced to adjust to a change in sediment supply.

This document presents procedures that may be used for estimation of channel degradation in unlined watercourses within Arizona. Three levels of procedures are provided, with data requirements, procedural complexity, and accuracy of results all increasing as the analysis level is incremented. The Level I approach provides an initial estimate of local channel degradation potential for generally stable, natural channel conditions. The resulting initial estimate may be reduced through use of the more rigorous Level II methodologies. Level III procedures are outlined for situations that warrant more detailed channel degradation determination.

Procedure

General

Three levels of procedures for estimation of channel degradation depth are described in the following paragraphs. The first level of analysis provides an initial estimate of the potential scour depth to consider for design of structures to be placed near a streambed or along the banks of a channel. This first level of analysis is recommended only for channel reaches that are expected to be in general balance with the surrounding system -- i.e. no major disturbances (dams, bridges, encroachments, etc..) are evident in the site vicinity -- and where the desire is to establish a "safe" scour depth to allow for the concentration of flows that can naturally occur within channels composed of erodible material. The Level II procedures provided are methods for demonstrating the site specific limits to erosion potential, involving computations which require local hydraulic information and sediment size distributions, or historical evidence of channel performance. The third level of procedures outlined will provide more definitive determination of channel stability in the reaches under study. This level of analysis is recommended in areas where local flow characteristics are complex, where the channel has been redirected or otherwise modified by acts of man, or where the safety of local paralleling or crossing structures is of high concern.

Level I

This level of analysis requires the following information :

> Peak discharge associated with the 100-year flood (Q_{100}). May be estimated using simplified methodologies such as ADWR State Standard #2 (SS 2-96), USGS regression equations, or other appropriate local or more detailed methods.

The total scour depth, d_s, is the combination of general degradation and long term degradation and can be computed as follows:

$$d_s = d_{gs} + d_{lts}$$

where:

d_s = Total scour depth, in feet
d_{gs} = General degradation, in feet
d_{lts} = Long term degradation, in feet

General degradation can be computed as follows:

$$d_{gs} = 0.157(Q_{100})^{0.4} \quad \text{for straight channel reaches.}$$

and

$$d_{gs} = 0.219(Q_{100})^{0.4} \quad \text{for channel reaches with curvature.}$$

The second equation will give the worst-case scour for channel curvature, and is not recommended unless significant curvature is evident along the channel reach.

Long term degradation can be computed as follows:

$$d_{lts} = 0.02(Q_{100})^{0.6}$$

This equation for long term degradation should only be used when no downstream controls exist within the channel system.

The total scour depth, d_s, should be applied to the lowest point in the local cross section for determination of the elevation to which scour will occur.

For Level I, the minimum total scour depth, d_s, shall be 3 feet.

Level II

The Level II approaches presented below may be used to demonstrate the ability of the existing channel system to resist degradation, and to justify a lesser burial requirement than that computed using the Level I equations.

Erodibility evaluation

Three procedures for determination of the erodibility of local channel material under computed hydraulic conditions are presented in the ADWR's State Standard for Lateral Migration Setback Allowance for Riverine Floodplains in Arizona. These procedures are: (1) the allowable velocity approach; (2) the tractive stress approach; and, (3) the tractive power approach. One or more of these procedures can be used to demonstrate the adequacy of the material of which the channel is composed to resist the erosive action of the flow under 100 year flow conditions.

Armoring potential evaluation.

An evaluation of relative channel stability can be made by evaluating incipient motion parameters and determining armoring potential. The definition of incipient motion is based on the critical or threshold condition where hydrodynamic forces acting on a grain of sediment have reached a value that, if increased even slightly, will move the grain. Under critical conditions, or at the point of incipient motion, the hydrodynamic forces acting on the grain are just balanced by the resisting forces of the particle. For given hydrodynamic forces, or equivalently for a given discharge, incipient motion conditions will exist for a single particle size. Particles smaller than this will be transported downstream and particles equal to or larger than this will remain in place.

The Shields diagram (Figure 1) may be used to evaluate the particle size at incipient motion for a given discharge. The Shields diagram was developed through measurements of bed-load transport for various values of the Shields parameter (y axis of Figure 1) at least twice as large as the critical value, and extrapolated to the point of vanishing bed load. In the turbulent range, where most flows of practical engineering interest occur, this diagram suggests that the Shields parameter is independent of flow conditions and the following relationship is established:

$$D_c = \tau_p / [0.047 (\gamma_s - \gamma)]$$

where D_c is the diameter of the sediment particle for conditions of incipient motion, τ_p is the boundary shear stress acting on the particle, γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient. Any consistent set of units may be used with this equation. Typical values for γ_s and γ in English units are 165 lb/ft^3 and 62.4 lb/ft^3, respectively.

For computation of shear stress on the boundary particles, the following relations are recommended:

$$\tau_p = \tfrac{1}{8} f \rho V^2$$

$$f = 116.5\, n^2 / R^{1/3}$$

$$n = D_{90}^{1/6} / 26$$

where

f	=	friction factor (dimensionless)
ρ	=	density of the water
V	=	flow velocity
n	=	Manning resistance value
R	=	hydraulic radius of the channel
D_{90}	=	particle size which is larger than 90 percent of all sizes

The units of the above are as follows: τ is in lb/ft^2; ρ is in $slugs/ft^3$ (typically 1.94 $slugs/ft^3$); V is in feet per second; and R is in feet. The relation presented above relating the Manning n value to the D_{90} of the local bed material yields the resistance factor associated with the particle roughness only, and assumes D_{90} is in meters.

The shear stress computed from the above equation should be increased in areas of channel curvature using Figure 2.

The armoring process begins as the non-moving coarser particles segregate from the finer material in transport. The coarser particles are gradually worked down into the bed, where they accumulate in a sublayer. Fine bed material is leached up through this coarse sublayer to augment the material in transport. As movement continues and degradation progresses, and increasing number of non-moving particles accumulate in the sublayer. This accumulation interferes with the leaching of fine material so that the rate of transport over the sublayer is not maintained at its former intensity. Eventually, enough coarse particles accumulate to shield, or "armor," the entire bed surface. When fines can no longer be leached from the underlying bed, degradation is arrested.

Potential for development of an armor layer can be assessed using Shields' criteria for incipient motion and a representative bed-material composition. In this case a representative bed material composition is that which is typical of the depth of anticipated degradation. Using the equation presented above, the incipient-motion particle size can be computed for a given set of hydraulic conditions. If no sediment of the computed size or larger is present in significant quantities in the bed, armoring will not occur. Armoring is probable when the particle size computed from the above equation is equal to or smaller than the D_{90} size.

After determination of the percentage of the bed material equal to or larger than the armor particle size (D_c), the depth of scour necessary to establish an armor layer (ΔZ_a) can be calculated from the following equation:

$$\Delta Z_a = y_a\,[(1/P_c) - 1]$$

where y_a is the thickness of the armoring layer and P_c is the decimal fraction of material coarser than the armoring size. The thickness of the armoring layer (y_a) ranges from one to three times the armor particle size (D_c), depending on the value of D_c. Field observations suggest that a relatively stable armoring conditions requires a minimum of two layers of armoring particles.

<u>Channel profile history comparison</u>

This procedure, applicable where sufficient data is available, relies on the historical record for indication of the degradation potential of the local channel reach. This procedure should be used to demonstrate the stable or aggrading tendency of the reach in question, rather than to estimate potential degradation depths. Given a reach of channel with successive record of channel profile changes, associated with hydrologic information for the events occurring between surveys, the reviewer can determine the trend of the channel changes and assess the likelihood of trend continuation for the future. Where the stable or aggradational trend is obvious, and no changes are anticipated in the channel system to alter the on-going trend, a lesser degradation allowance than that provided under the Level I guidelines would be reasonable.

<u>Grade stabilization measures adequacy analysis</u>

Grade stabilization measures of some form may be proposed or already in place which may act to limit the degradation potential of the watercourse of concern. In some areas within Arizona, procedures are in place for assessment of the adequacy of channel stabilization measures. For areas without standardized procedures, two references are recommended which detail evaluation procedures:

> Design Manual for Engineering Analysis of Fluvial Systems, Arizona Department of Water Resources, 1985.
>
> Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona, City of Tucson Department of Transportation, Engineering Division, 1989.

Level III

This level of analysis involves modeling the hydraulic and sediment transport characteristics of the local watercourse in order to simulate the erosion/sedimentation and channel deformation processes which are expected to occur in the area of concern. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for establishment of degradation potential be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles;
and, (3) steady state hydraulic and sediment transport analysis.

Works Cited

Arizona Department of Water Resources, Flood Warning and Dam Safety Section, "Requirement for Floodplain Delineation in Riverine Environments - State Standard 2-96", July 1996.

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Lacey, G., "Stable Channels in Alluvium," in Proceedings of the Institution of Civil Engineers, Vol. 229, 1930.

Thomas, B.E., H.W. Hjalmarson, and S.D. Waltemeyer. "Methods for Estimating Magnitude and Frequency of Floods in the Southwestern United States." USGS Open-File Report. 1994. 93-419

Example Application

Example 1: Proposed Siphon Crossing of an Earthen Channel

- **Problem Statement.** A natural earthen channel traverses a site where an irrigation channel is being constructed. The watershed contributing to the earthen channel upstream of the site is 700 acres in area. A siphon is proposed to convey irrigation water across the channel.

- **Objective.** Determine the burial depth for the proposed siphon.

Level I Analysis

A 100-year peak discharge value of 530 cfs was determined from local hydrology methodology. The channel in the site vicinity has 2:1 side slopes and a bottom width of 15 feet. The proposed crossing site is at a mild bend in the channel. A sieve analysis of the local bed material yields a median grain size D_{50} = 1.0 mm = 0.0033 feet.

Calculations:

General degradation, $d_{gs} = 0.157(530)^{0.4} = 1.93$ feet

Long term degradation, $d_{lts} = 0.02(530)^{0.6} = 0.86$ feet

Total scour, d_s = 1.93 feet + 0.86 feet = 2.79 feet

Since the total scour calculated is less than the recommended minimum of 3 feet, use a total scour depth of 3.0 feet.

Level II Analysis

Further evaluation is desired to investigate the potential for reducing the burial depth indicated through application of the Level I procedure. Although no historical data is available for determination of the local aggradation/degradation trends of the earthen channel, the erodibility and armoring potential of the existing channel material can be checked using the recommended Level II procedures. The site specific hydraulic and grain size information is collected to check if erosion of the channel would be naturally limited. The channel slope in the site vicinity is estimated from USGS quadrangle maps at 0.010 feet/foot, and the Manning n value for total channel resistance is estimated at 0.030.

Using normal depth procedures, the hydraulic characteristics of the local channel under 100-year flood conditions are determined:

Flow Depth = 3.0 feet
Flow Velocity = 8.4 feet/second

The sieve analysis of the local channel material sample yields the following information:

D_{90} = 55 mm = 0.180 ft = 0.217 inches
D_{75} = 4 mm = 0.013 ft = 0.16 inches
D_{65} = 1.9 mm = 0.0062 ft = 0.07 inches

Calculations:

Erodibility Evaluation (using procedures and figures provided in Attachment 1 to this State Standard)

(1) Allowable velocity approach, assuming sediment laden flow

Entering Figure 1 with D_{75} = 4 mm yields a basic velocity of 4.0 ft/sec.

In this case, we are concerned with erosion of the channel invert in a reach containing only a mild bend, so the correction factors for channel curvature reduces to 1.0. The correction factor for side slope, which must be considered for evaluating the erodibility of the channel banks, is not applied in this case.

Entering Figure 4 with Depth = 3.0 feet yields C_c = 1.01

Maximum allowable velocity = (4.0)(1.0)(1.01) = 4.0 ft/sec

Since the computed velocity of 8.4 ft/sec exceeds the maximum allowable velocity, erosion may be expected to occur.

(2) Tractive stress approach

Since D_{75} is less than 0.25 inches, the reference tractive stress method is used;

Assuming a water temperature of 60° F, the kinematic viscosity (ν) = 0.0000121 ft^2/sec, and the density (ρ) = 1.94 slugs/ft^3

Compute $V^3/(g\nu S_e) = 1.52 \times 10^8$

Compute $V/(gD_{65}S_e)^{1/2} = 188$

From Figure 9, $V/(\tau/\rho)^{1/2} = 18.2$

Solving the above equation yields $\tau = 0.41$ lb/ft^2.

No correction factor for side slope is applied, and the correction factor for channel curvature reduces to 1.0 for a mild bend.

From Figure 12, Curve 1 (for high sediment content), the allowable tractive force is 0.09 lb/ft^2. Since 0.09 is less than 0.41, the channel is erosive.

(3) Tractive power approach

An unconfined compressive strength (UCS) test of the saturated channel soils is performed, yielding a strength of 800 lb/ft^3.

Assuming half of this strength for design purposes, UCS_{design} = 400 lb/ft^3.

Compute tractive power = $V\tau_s = 3.44$

From Figure 13, the condition falls above the S-Line, indicating that the channel is erosive.

Armoring potential evaluation

Manning's n related to particle roughness = $[55/1000]^{1/6} / 26 = 0.024$

Channel flow area = $[15+2(3.0)](3.0) = 63.0$ square feet

Channel wetted perimeter = $15 + 2(3.0)(5)^{1/2} = 28.4$ feet

Hydraulic Radius = $63.0/28.4 = 2.22$ feet

Friction factor = $f = 116.5\ (0.024)^2 / (2.22)^{1/3} = 0.051$

Particle shear stress = $\tau_p = \frac{1}{8}\ (0.051)(1.94)(8.4)^2 = 0.87$ lb/ft^2

Critical particle size = $D_c = .87/[0.047(165\text{-}62.4)] = 0.18$ feet = 54.9 mm

Since the critical particle size is essentially equal to D_{50}, armoring is a possibility.

Therefore, the percent of material greater than D_c = 54.9 mm is 10%

Armor thickness = $y_a = 2D_c$ = 0.36 feet

Depth of degradation required for armoring to form:

$$\Delta Z_a = y_a\,[(1/P_c) - 1] = 0.36[(1/0.10) - 1] = 3.24 \text{ feet}$$

Since the depth required for armoring to occur exceeds the Level I burial depth, armoring will not control, and the recommended burial depth is the minimum allowable value of 3.0 feet.

Level III Analysis

The conclusions derived from the Level II analysis and the nature of the problem indicate that the Level III analysis would probably not be applied in this case. However, should the designer wish to proceed with the degradation investigation, a registered engineer with experience in sediment transport modeling could be employed for this purpose. The engineer would be expected to collect available historic information, document the historic planform changes to the watercourse under events of varying frequency, apply steady state hydraulic and sediment transport calculation procedures to determine the erosion/sedimentation characteristics of the local reach of channel, and, potentially apply a moveable boundary river simulation model to quantify the changes likely along the study reach under design event conditions.

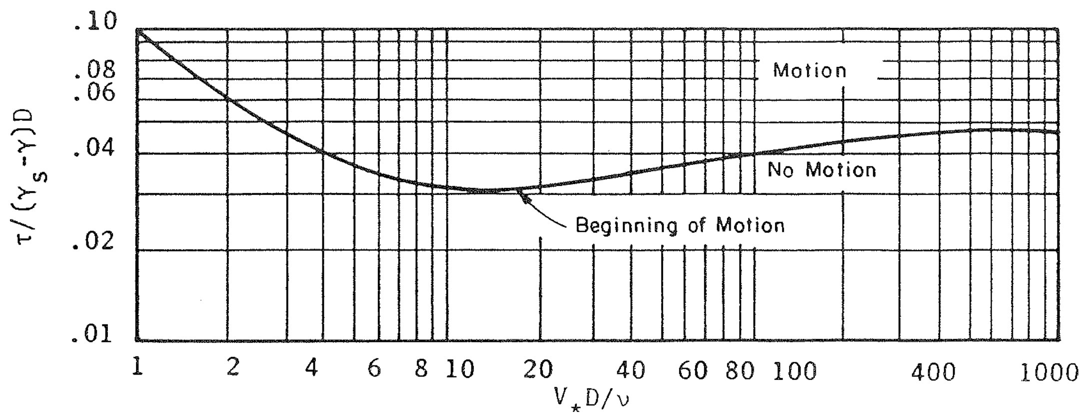
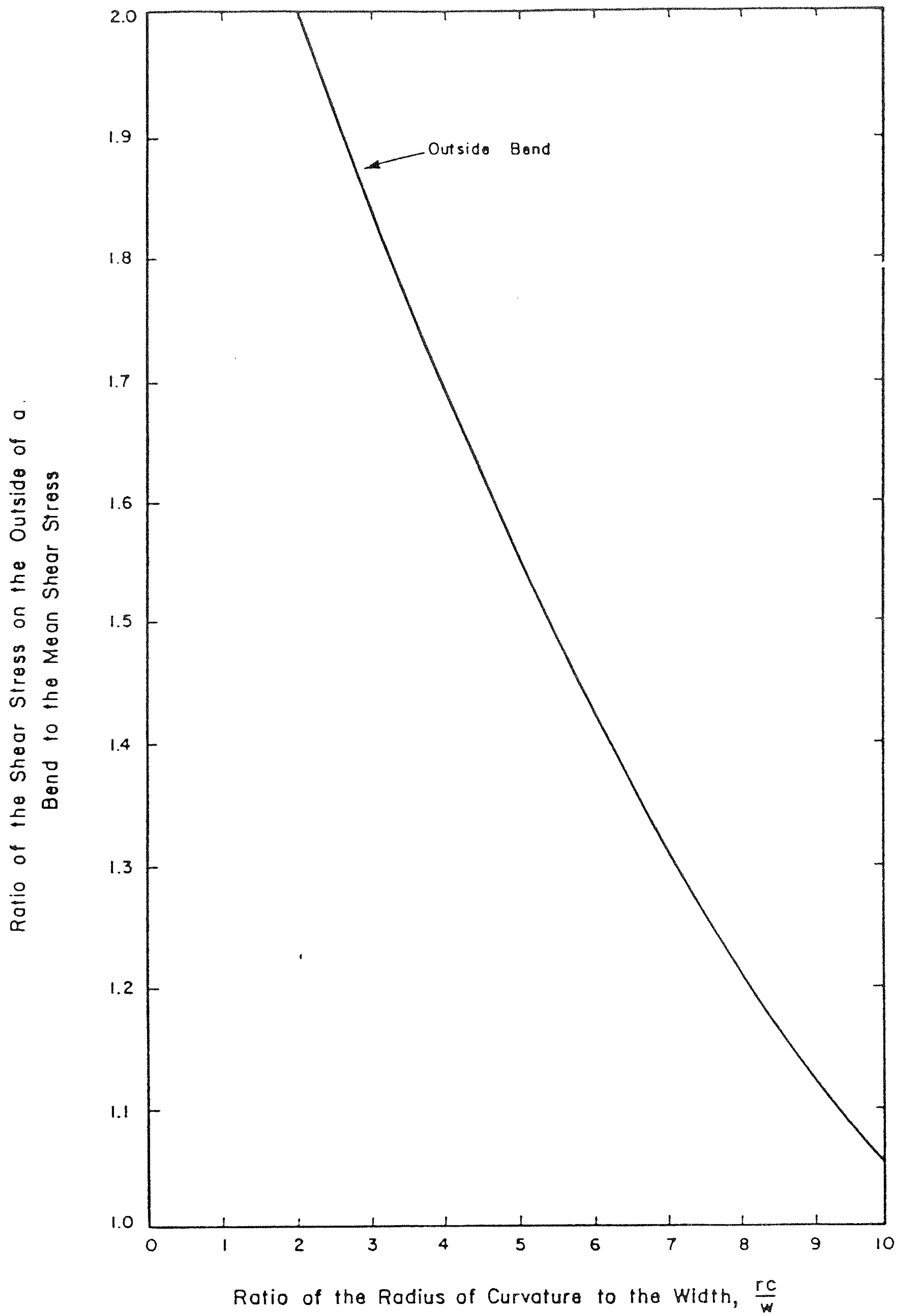


FIGURE 1

SHIELD'S RELATION FOR BEGINNING OF MOTION



<u>FIGURE 2</u>
EFFECT OF BEND ON BOUNDARY SHEAR STRESS

GUIDELINE 3

Evaluation of River Stability Impacts associated with Sand and Gravel Mining

TABLE OF CONTENTS

Introduction

The river stability impacts associated with instream or near-stream sand and gravel operations depend on the local watershed and river characteristics, and on the mining and management practices followed. Excessive sand and gravel removal from a river channel can endanger the stability of the river system by inducing general scour and headcutting. These processes can undermine the burial and/or support materials for facilities that cross or parallel the watercourse, increasing the likelihood of structure failure. These processes can also increase the rate of erosion of a dike or buffer zone designed to separate a near-river pit from an active river channel. A headcut and erosion through such a buffer zone could alter local river channel characteristics and transport rates, and impact both upstream and downstream reaches. If the channel reach adjacent to a floodplain mining pit is geomorphically active (e.g., migrating laterally), the same result might occur if protective measures or an adequate buffer zone are not provided during site development.

The scour and deposition problems associated with sand and gravel mining are very complicated. The dominant physical processes include water runoff, sediment transport, sediment routing, degradation, aggradation, and breaking and forming of the armor layer. These processes are unsteady and complicated in nature. Furthermore, each situation is unique and requires independent analysis. No standard equation or formula can be adopted which is universally applicable to all gravel mining evaluations. However, general guidelines have been developed for determination of the adequacy of buffer areas between proposed mining operations and active river channels, and procedures are available for analysis of the effects of instream activities.

Procedure

General

This document presents three levels of procedures that may be applied for evaluation of sand and gravel operations in areas adjacent to or within watercourses. The first level procedure may be applied to estimate the size of an adequate erosion buffer area between an active river channel and a near-stream operation. The second level procedure may be used to investigate the erosion resistance of buffer materials, in cases where the applicant desires to reduce the buffer area developed using the Level 1 procedures. A third level procedure is presented to enable more definitive determination of the erosional/depositional tendencies of a channel adjacent to a near-stream mining site, or to determine the potential impacts of instream mining operations.

The aggradation/degradation trends of river reach that contains or is adjacent to a sand and gravel mining operation are governed by the same processes that act on an unmined reach -- differentials in sediment transport capacities and sediment supply result in degradation in areas of deficit and aggradation in areas of surplus. The potential hazard associated with sand and gravel mining operations in the vicinity of watercourses may be evaluated using the same procedures as those described in the Channel Degradation and Lateral Migration portions of this State Standard. The mining area is analyzed either as a particular portion of the river (for the case of an instream site), or as an off-channel development (for an operation established adjacent to a river's banks).

For mining operations that are to be established outside of the floodplain, the Level I, II, or III techniques detailed in the Lateral Migration guideline would apply. Instream operations, however, require the application of more rigorous procedures. The mining area is separated into subreaches of similar geometry and hydraulics (i.e., (1) the reach upstream of the mining area, (2) the upstream slope down into the pit, (3) the pit itself, and (4) the reach immediately downstream of the pit), and analyzed using river modeling procedures.

The recommended approaches for evaluation of sand and gravel mining operations in the vicinity of watercourses are summarized below:

Level I

Estimate of the required buffer distance between a near-stream site and the active channel.

Setback the top of the proposed mining pit a distance from the floodplain given by the Level I setback criteria (as detailed in the Lateral Migration Guidelines).

Level II

Evaluation of the erodibility of the buffer materials for minimization of near-stream site setback requirements.

Require a smaller setback from the floodplain boundary if justified by application of the Level II setback criteria (as detailed in the Lateral Migration Guidelines).

Level III

Mathematical modeling of the river channel to better determine the adequacy of the buffer provided for a near-stream operation or to quantify the river stability impacts associated with an instream operation.

Use steady state or movable boundary sediment transport analysis (backed up by qualitative analysis and historical evidence) to determine the short and long term impact of proposed mining operation, including headcut impacts and downstream impacts due to sediment deficit. For this level of analysis, Level III hydrology shall be performed to generate required hydrographs. Level III analyses should be performed by persons with knowledge and experience in the fields of sediment transport and river geomorphology. It is recommended that any movable boundary river modeling used for determination of lateral channel stability or for evaluation of instream mining impacts be the culmination of a thorough analysis consisting of:

(1) evaluation of historical trends;
(2) qualitative analysis based on field evaluation and application of geomorphic principles;
and, (3) steady state hydraulic and sediment transport analysis.

APPENDIX G-FAB AVENUE / FRY BLVD PROPERTY ASSESSMENT REPORT

Memorandum

Date : April 22, 2021

To : Jing Lou P.E. , City Engineer, City of Sierra Vista

From : Clinton Glass P.E., CMG Drainage Engineering, Inc.

Regarding: Fab Avenue Property Redevelopment Drainage Design

This memorandum discusses drainage design options for a 1.25-acre parcel of vacant land owned by the City of Sierra Vista (City) located at the southeast corner of Fry Blvd and Fab Avenue. The City seeks to redevelop the property, primarily as a community center that includes event plazas, open spaces such as lawns and landscaping; and possibly allocating a portion of the site as a live/work area.

Presently, most of the site is within the 100-year floodplain associated with storm water flows that drain south to north along Fab Avenue. This floodplain must be mitigated in order to allow development of the property. Onsite detention storage will also be needed to prevent an increase in downstream flooding along the Fab Avenue and Vista Village Drainageways. Opportunities for providing excess detention storage that could be purchased by future nearby development is also explored.

<u>Existing Surface drainage Conditions</u>
The Fab Avenue drainage occurs as street flow south of Fry Blvd. Currently, storm water runoff collects in Fab Avenue and drains to a 2 cell, 8' x 3' box culvert beneath Fry Blvd. located about 100-feet north of the intersection with Fab Avenue. The 100-year discharge as determined by the City's HEC-RAS model is 207 cubic feet per second (cfs).

Hydraulic modeling conducted by CMG has determined that the street capacity is inadequate to contain flow, partly being due to the absence of curb along the east side of the street and adjoining grades being only a foot or less higher than the street. Hydraulic modeling (HEC-RAS) determined that the City's property is inundated to a depth of 6-inches to one foot during the 100-year flood peak. Preliminary FEMA floodplain mapping extends only a couple hundred feet south of Fry Blvd. but those results are the same. Exhibit 1 in Appendix A shows the property location and existing conditions floodplain mapping results from the CMG HEC-RAS modeling. Appendix B contains the HEC-RAS model output.

Presently, flows within Fab Avenue and shallow flow crossing the City's property drain to the 2 – 8' x 3' culvert beneath Fry Blvd., however, Fry Blvd. is higher than Fab Avenue and the land surface just south of the intersection is very flat, so storm water ponds until flow can trickle into the culvert. Elevation change between Fab Avenue and the culvert inlet is negligible which causes flow to the culvert to be very inefficient. Hydraulic computations conducted by CMG determined that the 2 – 8' x 3' culvert has adequate capacity to convey the 100-year discharge of 207 cfs at an inlet ponding depth approximately equal to the elevation of Fry Blvd. so overtopping of the road can be mitigated if the efficiency of conveying flow to the culvert can be improved.

In addition to the storm water flows draining south the north along Fab Avenue, there is about 8.9 acres of land along the east and south sides of the property that drain towards the site (see Exhibit 5 in Appendix A). Storm water runoff from the area arrives at three concentration points (see Exhibit 5)) primarily as sheet flow. Total peak flow for the 100-year storm is about 74 cfs. Hydrologic computation sheets for determining the 100-year discharge rates at concentration points along the east boundary are provided in Appendix C.

<u>Project Goals</u>
The City contracted with Stantec Consulting to develop three alternative redevelopment plans. All of these alternatives include significant open space and landscape areas that could potentially be used for storm water storage. Concept plans for these alternatives are provided on Exhibits 2, 3 and 4 in Appendix A of this report.

There are three primary goals for the Fab Avenue drainage project:

1. To mitigate flooding of the City's property and prevention of periodic flows over Fab Avenue, and,
2. To facilitate redevelopment of the City's property, and,
3. Provide sufficient onsite detention storage to serve redevelopment of the City's property and to provide additional detention storage for redevelopment of nearby parcels, to the extent possible.

A discussion on the possible methods of accomplishing these goals is provided below.

<u>Mitigation of Site Flooding and Fry Blvd. Overtopping</u>
CMG conducted hydraulic modeling for an assumed condition of flow containment within the Fab Avenue pavement limits. This analysis determined flow depths within the street section to average about 1.2 feet and the increase in depths when compared to existing conditions range from about 0.1 to 0.4 feet depending on locations. These increases could potentially be mitigated by widening the street, if needed, although no analysis of this has been conducted thus far. Another possible means of mitigating the increases would be to lower the street profile by 0.5 feet or less. Ground elevations on the west side of Fab Avenue are higher than on the east side so increases in flow depths are unlikely to cause damages to existing structures, but further analysis of this is needed.

The second requirement for removing most, if not all of the City property from the floodplain is to construct a channel between the low point along Fab Avenue to the inlet of the culvert under Fry Blvd. This channel is envisioned to be earthen, landscaped to be visually compatible with the proposed alternatives and would be aligned in a southwest to northeast direction. The depth of the channel would be no more than two feet having mild sideslopes at 3:1 or flatter and a topwidth in the range of 25- to 30-feet. All three of the Stantec alternatives have landscaped open space in the vicinity of the northwest corner to accommodate the channel as shown on Exhibits 2, 3 and 4. Pedestrian crossings of the channel would likely occur as small bridges.

It should also be mentioned that the design should avoid offsite storm water from comingling with onsite storm water to minimize trash and oils from entering the project. Whether or not this can be accomplished is dependent onsite grading. A goal for the project design should be, if possible, to raise the site above the inlet headwater elevation of the 2 – 8' x 3' RCBC beneath Fry Blvd, and to raise the remainder of the site above the water surface elevations in Fab Avenue.; keeping in mind though that the east and south edges of the project site must remain at or below existing grade so that storm water runoff that drains to the three concentration points shown on Exhibit 5 can be accepted and drained north to the Fry Blvd. culvert. This could possibly be accomplished in the alley or by a landscaped swale along the east side of the project.

Redevelopment of City Property

Mitigation of site flooding as described above could facilitate redevelopment of the City property. This would be accomplished by increasing the capacity of Fab Avenue to convey more storm water and by raising the site grades adjoining Fab Avenue by approximately one foot. Habitable structures may need to be raised more but the alternative plans indicate that type of land use to occupy only a limited area of the property. Areas of the property that will be used for lawn, open space or landscaping do not need to be raised provided positive drainage can be accomplished; that being the ability to drain through the project to the Fry Blvd. culvert. Detention storage water depths within the open spaces in excess of 6- to 9-inches is not recommended so as to avoid vector concerns, and soil infiltration capacity rates sufficient to “dry” the basin in 24 hours or less should also be demonstrated.

Detention Storage

Detention storage is needed to avoid increases in downstream peak flows along Fab Avenue and Vista Village Drainageways, and to meet to requirements of the City’s floodplain management policies. The first step of this analysis was to determine the onsite detention storage volumes for each alternative. The required volume varies from one alternative to the others because the area of impervious surfaces are different. The table below lists the estimated detention volumes for each alternative to comply with City requirements.

Required Detention Storage for each Development Concept

Alternative	Impervious Cover Estimate	Required Detention Volume (Cubic Feet)
Concept A1	35 %	1720
Concept B1	28 %	1325
Concept C1	22 %	940

The next step of the analysis was to estimate the potential storage volume offered by each alternative. This assessment looked at the larger open space areas offered by each alternative (shown on the concept plans as lawn), and the landscaped areas noted as LID. The LID areas are all very small so CMG combined them into one in the table below. This table lists potential storage volumes for ponding depths ranging from 0.5 feet to 2.0 feet.

Potential Detention Volumes by Concept and Storage Depth

CONCEPT ID*	**Sub-Area ID**	**AREA (ft^2)**	**Volume (ft^3)**			
			DEPTH (FT) 0.5	**DEPTH (FT) 1.0**	**DEPTH (FT) 1.5**	**DEPTH (FT) 2.0**
A1	1	9888	4944	9888	14832	19776
	LID AREAS	9365	4683	9365	14048	18731
Totals		**19254**	**9627**	**19254**	**28880**	**38507**
B1	1	9148	4574	9148	13721	18295
	2	5750	2875	5750	8625	11500

CONCEPT ID*	Sub-Area ID	AREA (ft^2)	Volume (ft^3)			
			DEPTH (FT) 0.5	DEPTH (FT) 1.0	DEPTH (FT) 1.5	DEPTH (FT) 2.0
	LID AREAS	16683	8342	16683	25025	33367
Totals		**31581**	**15791**	**31581**	**47372**	**63162**
C1	1	12632	6316	12632	18949	25265
	2	10454	5227	10454	15682	20909
	LID AREAS	10890	5445	10890	16335	21780
Totals		**33977**	**16988**	**33977**	**50965**	**67954**

* Note Sub-Area ID's are shown on the Alternative Concepts

Comparison of these tables shows that substantially more detention storage capacity is available than require just for site development. For instance, the required storage volume for Concept A1 is 1720 cubic feet; at a storage depth of 0.5 feet, up to 9627 cubic feet are available within lawn and LID area. A total of 38,507 cubic feet of detention could be achieved if the storage depth is 2-feet.

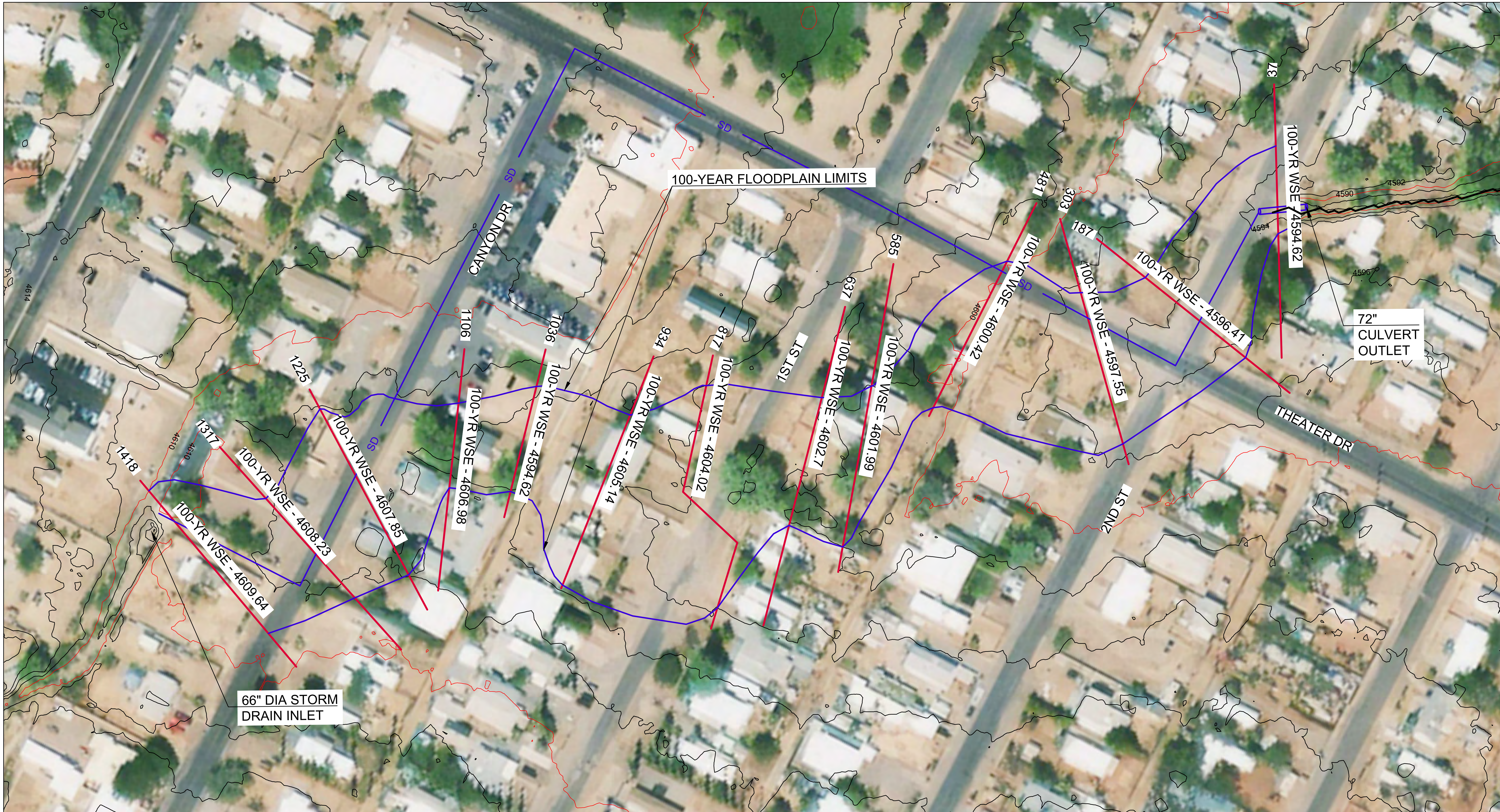
To arrive at an estimate of how many acres of offsite property could “buy” detention storage credits from the Fab Avenue project, CMG conducted a calculation of a typical one acre site having a pre- and post-development impervious cover of 0% and 90%. This calculation determined the required detention storage volume to be 4160 cubic feet per acre. Again, by example, if the design storage depth for Concept A1 is 0.5 feet, then 1.9 acres of offsite development could “buy” 7907 cubic feet of detention storage from the project. This is calculated as: (9627 – 1720)/ 4160 = 1.9 acres.

<u>Summary</u>

All of the alternative Concepts offer opportunity for offsite development to “buy’ detention storage credits which would waive the onsite storage requirement. To determine the storage volume available for offsite development, the City needs to identify the preferred Concept, what areas of that project will provide detention storage, and at what depths. More detailed analyses for the project drainage design must be conducted once the City provides direction on these drainage design parameters.

Another point to consider is the sources of storm water that can be delivered to potential storage areas within the Concept alternatives. It is estimated that 3-inches of direct rainfall on the project area (1.25 acres) would create approximately 13,600 cubic feet of storm water that could be stored within the lawn and LID areas. This would provide 11,880 cubic feet of capacity available for offsite developments, which translates to 2.9 acres (13,600-1,720)/4160 = 2.9 acres of offsite development.

Appendix A Exhibits



LEGEND

Blue line	100-YEAR FLOODPLAIN LIMIT
Red line	HEC-RAS CROSS SECTION LOCATION & ID

N

0' 100'

SCALE: 1" = 100'

CONTOUR INTERVAL - 2-FT

CMG DRAINAGE ENGINEERING, INC.

PROJECT NO.:	21-001.1
DESIGN:	CMG
CHECKED:	CMG
DRAWN:	BJK
DATE:	08/09/2021
REV. DATE:	
REV. DATE:	
REV. DATE:	

EXHIBIT 1:
EXISTING CONDTIONS
FLOODPLAIN MAP

FAB AVE WASH NORTH

SHEET 1 OF 1

Z:\PROJECTS\2021\21-001_CityofSierraVista_SWMP_Update\21-001.1 FAB Ave\CADD\21-001.1_cmgbase.dwg (Fab Ave Wash North) Plotted May 16, 2023 at 9:20am by Brenda

CONCEPT ID	Sub-Area ID	AREA (ft^2)	Volume (ft^3)			
			DEPTH (FT) 0.5	DEPTH (FT) 1.0	DEPTH (FT) 1.5	DEPTH (FT) 2.0
A1	1	9888	4944	9888	14832	19776
	LID AREAS	9365	4683	9365	14048	18731
Totals		19254	**9627**	**19254**	**28880**	**38507**

LEGEND

1 LAWN AREA ID

CMG DRAINAGE ENGINEERING, INC.

PROJECT NO.: 20-001.1
DESIGN:
CHECKED:
DRAWN:
DATE: 04/22/2021
REV. DATE:
REV. DATE:
REV. DATE:

EXHIBIT 2:
4 S. FAB AVENUE
REUSE PLANNING
CONCEPT A1
DETENTION STORAGE OPPORTUNITIES
SHEET 1 OF 1

Z:\PROJECTS\2021\21-001_CityofSierraVista_SWMP_Update\21-001.1 FAB Ave\CADD\21-001.1_concepts.dwg (Exh 2-A1) Plotted Apr 22, 2021 at 3:10pm by Brenda

CONCEPT ID	Sub-Area ID	AREA (ft^2)	Volume (ft^3) DEPTH (FT) 0.5	DEPTH (FT) 1.0	DEPTH (FT) 1.5	DEPTH (FT) 2.0
B1	1	9148	4574	9148	13721	18295
	2	5750	2875	5750	8625	11500
	LID AREAS	16683	8342	16683	25025	33367
Totals		31581	**15791**	**31581**	**47372**	**63162**

LEGEND

1 LAWN AREA ID

CMG DRAINAGE ENGINEERING, INC.

PROJECT NO.: 20-001.1
DESIGN:
CHECKED:
DRAWN:
DATE: 04/22/2021
REV. DATE:
REV. DATE:
REV. DATE:

EXHIBIT 3:
4 S. FAB AVENUE
REUSE PLANNING
CONCEPT B1
DETENTION STORAGE OPPORTUNITIES

SHEET 1 OF 1

Z:\PROJECTS\2021\21-001_CityofSierraVista_SWMP_Update\21-001.1 FAB Ave\CADD\21-001.1_concepts.dwg (Exh 3 B1) Plotted Apr 22, 2021 at 3:08pm by Brenda

CONCEPT ID	Sub-Area ID	AREA (ft^2)	Volume (ft^3) DEPTH (FT) 0.5	DEPTH (FT) 1.0	DEPTH (FT) 1.5	DEPTH (FT) 2.0
C1	1	12632	6316	12632	18949	25265
	2	10454	5227	10454	15682	20909
	LID AREAS	10890	5445	10890	16335	21780
Totals		**33977**	**16988**	**33977**	**50965**	**67954**

LEGEND

1 LAWN AREA ID

CMG DRAINAGE ENGINEERING, INC.

PROJECT NO.: 20-001.1
DESIGN:
CHECKED:
DRAWN:
DATE: 04/22/2021
REV. DATE:
REV. DATE:
REV. DATE:

EXHIBIT 4:
4 S. FAB AVENUE
REUSE PLANNING
CONCEPT C1
DETENTION STORAGE OPPORTUNITIES

SHEET 1 OF 1

Z:\PROJECTS\2021\21-001_CityofSierraVista_SWMP_Update\21-001.1 FAB Ave\CADD\21-001.1_concepts.dwg (Exh 4 C1) Plotted Apr 22, 2021 at 3:08pm by Brenda

Appendix B – HEC-RAS Model for Fab Avenue

HEC-RAS Plan: Plan 01   River: River 1   Reach: Reach 1   Profile: PF 1

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Reach 1	1418	PF 1	207.00	4608.83	4609.64	4609.54	4609.74	0.019889	2.58	80.34	195.14	0.71
Reach 1	1317	PF 1	207.00	4607.05	4608.23		4608.32	0.010361	2.41	85.81	141.07	0.55
Reach 1	1225	PF 1	207.00	4606.22	4607.85		4607.89	0.002536	1.51	137.50	159.52	0.29
Reach 1	1106	PF 1	207.00	4605.15	4606.98		4607.19	0.023295	3.68	56.28	90.08	0.82
Reach 1	1039	PF 1	207.00	4604.65	4606.45	4606.00	4606.52	0.004999	2.11	98.32	114.50	0.40
Reach 1	934	PF 1	207.00	4604.03	4605.14	4605.14	4605.35	0.039980	3.66	56.53	136.64	1.00
Reach 1	817	PF 1	207.00	4602.31	4604.02		4604.05	0.003301	1.39	149.37	239.09	0.31
Reach 1	637	PF 1	207.00	4601.59	4602.70		4602.85	0.019432	3.15	65.81	116.34	0.74
Reach 1	585	PF 1	207.00	4600.84	4601.99		4602.08	0.011385	2.28	90.60	173.26	0.56
Reach 1	481	PF 1	207.00	4599.57	4600.42		4600.55	0.019959	2.81	73.70	157.64	0.72
Reach 1	303	PF 1	207.00	4596.60	4597.55	4597.39	4597.65	0.013517	2.45	84.59	166.15	0.60
Reach 1	187	PF 1	207.00	4594.91	4596.41		4596.48	0.007578	2.23	92.84	135.80	0.48
Reach 1	37	PF 1	207.00	4593.35	4594.62	4594.45	4594.79	0.018012	3.36	61.52	92.94	0.73

APPENDIX H-EXISTING CULVERTS CAPACITY ESTIMATES

WASH Name	Culvert ID & Location	Culvert Type & Size	Culvert Length (ft)	US/DS Invert	Q100 (cfs)	Q100 Overtopping (cfs)	Overtopping
VISTA VILLAGE WSHD							
Vista Village Drainageway	River sta: 4915 N. 7th St.	5-36" RCP	78.5'	US-4574.49 DS-4573.55	469	133.32	Y
Vista Village Drainageway	River sta: 4130 E. Tacoma St.	30" Ellipse Conc	90'	US-4564.94 DS-4563.86	494	183.22	Y
Vista Village Drainageway	River sta: 5318 N. 6th St.	33' X 3' Slab Bridge	32' X 29'	US-4575.25 DS-4574.87	469	N/A	N
Vista Village Drainageway	River sta: 6150 N. 3rd St.	29' X 3' Slab Bridge	33' X 39'	US - 4583.58 DS-4583.18	394	115.22	Y
Fab Ave Wash	River sta: 9800 Fry Blvd & S Fab Ave.	2-8' X 3' RCBC	90'	US-4632.8 DS-4632.4	345	16.41	Y
Fab Ave Wash	River sta: 9119 Fry Blvd & North Ave.	2-48" CMP	58.9	US-4623.59 DS-4622.88	345	82.87	Y
Fab Ave Wash	River sta: 8630 East of North Ave.	1-48" CMP	32.2	US-4617.97 DS-4617.58	345	195.17	Y
COYOTE WASH WSHD							
Coyote Wash	River sta: 32497.95 COY600: Private Bridge	2-10' X 5' ConArch	17.5	US - 4377.3 DS-4377.09	3062	1485	Y
Coyote Wash	River sta: 33736.95 COY700: SR90	4-10' X 7' RCBC	128.3'	US - 4389.24 DS-4387.7	3040	N/A	N
Coyote Wash	River sta: 35970.95 COY800: Ave Del Sol	4-10' X 7' CMPA	104.5	US - 4410.48 DS-4409.23	3011	213.36	Y
Coyote Wash	River sta: 41190.95 COY900: Foothills Dr.	6-84" CMP	87.5'	US - 4464.18 DS-4463.13	3011	386.87	Y
Coyote Wash	River sta: 42599.95 COY1000: SR92	6-10' X 8' RCBC	120'	US - 4477.65 DS-4476.21	3011	N/A	N
Coyote Wash	River sta: 46352.79 COY1300: El Camino Real	3-30" RCP	29'	US - 4525.96 DS-4525.62	1093	919.11	Y
Coyote Wash	River sta: 49962.79 COY1900: Coronado Dr.	3-6.5' X 4.6' CMPA	75'	US -4583.97 DS-4582.53	616	48.78	Y
Coyote Wash	River sta: 52021.79 COY2300: Town & Country Dr.	3-5.8' X 3.5' CMPA	56.5'	US -4617.11 DS-4616.43	616	162.85	Y
Summit SW Drainageway	River sta: 180 SUM100: Oakmont	2-8' X 8' RCBC	86.1	US -4543.52 DS-4542.49	392	N/A	N
Summit SW Drainageway	River sta: 904 SUM200: El Camino Real	3-24" CMP	35.5'	US -4551.74 DS-4551.31	392	313.86	Y
Summit SW Drainageway	River sta: 5566 SUM300: Coronado Dr.	4-10' X 3' RCBC	85.9'	US -4604.64 DS-4603.61	392	N/A	N
Summit SW Drainageway	River sta: 7295 SUM400: Avenida Cochise	3-10' X 3' RCBC	98.6'	US -4629.42 DS-4628.24	392	N/A	N
Summit SW Drainageway	River sta: 7735 SUM500: WildFlower	3-10' X 3' RCBC	51.3	US -4634.99 DS-4634.37	392	N/A	N
South Garden SG Drainageway	River sta: 3635 SOU100: S & E Wardle Rd.	6-36" CMP	40.2'	US -4523.15 DS-4522.67	1662	1339.53	Y
South Garden SG Drainageway	River sta: 5574.6 SOU200: Oakmont Dr.	3-8' X 8' RCBC	112.1'	US -4547 DS-4545.95	1068	N/A	N
South Garden SG Drainageway	River sta: 6718.6 SOU300: Avenida Cochise	6-10' X 6' RCBC	159.4'	US -4555.58 DS-4553.67	1068	N/A	N
South Garden SG Drainageway	River sta: 9864 SOU600: Golf Course Bridge	Golf Course Bridge			1068	N/A	N
South Garden SG Drainageway	River sta: 11550.6 SOU700: Winterhaven Dr.	4-66" CMP	85.8'	US -4618.92 DS-4617.89	1068	N/A	N
Mountain Pueblo MM Mesa DW	River sta: 5270 MOU400: SR90	3-10' X 7' RCBC	140'	US -4333.89 DS-4332.71	2066	N/A	N
Mountain Pueblo MM Mesa DW	River sta: 6536 MOU500: Nature Way	2-8' X 4' RCBC	38.5'	US -4350.65 DS-4350.19	2066	1373.24	Y
Mountain Pueblo MM Mesa DW	River sta: 15038 MOU600: Ave Del Sol	5-10' X 4' RCBC	86'	US -4449.27 DS-4448.24	1592	5.22	Y
Mountain Pueblo CC Mesa DW	River sta: 22466.45 COU100: SR92	2-10' X 4' RCBC	122.7'	US -4548.2 DS-4546.75	598	N/A	N
Mountain Pueblo CC Mesa DW	River sta: 22764.45 COU200: Player Ave	7-3.2' X 2.5' CMPA	50.9'	US -4548.52 DS-4547.91	598	479.5	Y
Mountain Pueblo CC Mesa DW	River sta: 27238.45 COU300: Glen View Dr	8-30" CMP	48.7'	US -4610.4 DS-4609.82	245	N/A	N
Mountain Pueblo CC Mesa DW	River sta: 29005.45 COU400: Winterhaven Dr.	2-10' X 5' RCBC	115.5'	US -4634.09 DS-4632.7	245	N/A	N
Montebello Drainageway	River sta: 5854.69 MON100: Giulio Cesare Ave	5-10' X 4' RCBC	108'	US -4380.72 DS-4379.42	1230	N/A	N
Montebello Drainageway	River sta: 9191 MON400: Colombo Ave	2-10' X 3' RCBC	79'	US -4421.88 DS-4420.93	1245	516.37	Y
Montebello Drainageway	River sta: 10310.31 MON500: SR90	2-10' X 7' RCBC	145'	US -4436.24 DS-4434.5	1245	N/A	N
Montebello Drainageway	River sta: 13114 TOW100: SR92	2-10' X 5' RCBC	133'	US -4476.9 DS-4475.3	1245	N/A	N
Montebello Drainageway	River sta: 14478 TOW200: Avenida De Escuela	2-10' X 4' RCBC	39.7'	US -4491.98 DS-4491.5	853	117.14	Y
Montebello Drainageway	River sta: 17649 TOW400: El Camino Real	3-6' X 4' RCBC	71.4'	US -4538.6 DS-4537.74	624	192.52	Y
Montebello Drainageway	River sta: 20273 TOW500: Coronado Dr	3-48" CMP	75'	US -4577.65 DS-4576.75	201	N/A	N
Montebello Drainageway	River sta: 21790 tow600: Lenzner Ave	3-48" CMP	35.3'	US -4603.26 DS-4602.84	201	N/A	N

CHARLESTON WASH WSHD							
3rd St Buena #3	River sta: 57335.57 THI500 Myer Dr	3-36" CMP	59.9	US -4602.24 DS-4601.58	419	281.72	Y
3rd St Buena #3	River sta: 57771.5 THI400 Wilcox\7th St Int	3-30" CMP	227.8	US -4599.24 DS-4596.73	419	273.48	Y
3rd St Buena #3	River sta: 57771.5 THI200 Fry Blvd	6-10'x7' RCBC	90.3	US -4585.02 DS-4584.03	833	N/A	N
3rd St Buena #3	River sta: 52260.95 THI100 Lenzner Ave	4-10'x5 RCBC	64.2	US -4542.66 DS-4541.95	1044	N/A	N
Charleston Wash	River sta: 49526.61 CHA400 Coronado Dr	6-10'x7.5' RCBC	101.7	US -4515.37 DS-4514.25	1880	N/A	N
Charleston Wash	River sta: 42980.75 CHA300 Hwy 90	3-10'x7' RCBC	106.5	US -4453.8 DS-4452.63	2284	N/A	N
Charleston Wash	River sta: 40148.75 CHA200 Colombo Ave	6-12'x6' RCBC	106.6	US -4419.31 DS-4418.14	2290	N/A	N
Woodcutters Canyon Wash	River sta: 2400 WOO200: Fry Blvd.	1-10' X 5' RCBC	126.5'	US - 4558.83 DS-4557.44	1612	880.1	Y
Woodcutters Canyon Wash	River sta: 3060 WOO300: Lenzer Ave	3-10' X 6' RCBC	81'	US - 4562.31 DS-4561.42	1612	N/A	N
Woodcutters Canyon Wash	River sta: 3775 WOO400: Wilcox Dr	3-10' X 6' RCBC	81'	US - 4572.99 DS-4572.1	1630	N/A	N
Woodcutters Canyon Wash	River sta: 6075 WOO500: Busby Dr	2-36" CMP	76.5'	US - 4603.57 DS-4602.73	1791	1659.46	Y
Woodcutters Canyon Wash	River sta: 8750 7th St.	3-10' X 6' RCBC	111.8'	US - 4641.31 DS-4640.33	1791	57.69	Y
Woodcutters Canyon Wash	River sta: 9450 WOO800: Golf Links Rd	3-10' X 6' RCBC	84'	US - 4650.47 DS-4649.55	1791	80.18	Y
Woodcutters Canyon Wash	River sta: 10350 WOO900: Savannah Dr.	4-54" CMP	81'	US - 4660.98 DS-4660.09	1791	1062.25	Y
GARDEN CANYON WSHD							
Garden Canyon Wash	River sta: 23133.21 NWA_200: PRIVATE DR	PRIVATE DR			9940	1429.77	Y
Garden Canyon Wash	River sta: 26115.22 NWA_300: Moson Rd.	8-4.83' X 3' CMPA	65'	US -4296.17 DS-4295.17	10351	9730.8	Y
Garden Canyon Wash	River sta: 51311.88 GAR300: SR92	90' X 5' Bridge		US - DS-	11080	6266.33	N
Garden Canyon Wash	River sta: 55637.29 GAR400: St. Andrews Dr.	4-12' X 9.66' BOX	64'	US -4633.59 DS-4630.8	11080	4647.5	Y
Garden Canyon Wash	River sta: 58291.81 GAR500: Cherokee Ave	5-12' X 10' RCBC	68.5'	US -4667.14 DS-4666.14	11080	1936.66	Y
							N
Garden Canyon Wash (Stream K)	River sta: 9836.71 NWA_400: AZ SR92	3-8' X 4' C RCBC	122'	US -4618.42 DS4617.92	747	N/A	Y
Garden Canyon Wash (Stream K)	River sta: 8825.25 NWA_300: Canyon De Flores	1-36' X 6.38' ConArch	90	US -4604.78 DS 4603.78	958	N/A	Y
OUTSIDE OF ANY WSHD							
Graveyard Gulch	River sta: 666.49 GRA_200: San Juan Capistrano	2-60" CMP	114.3	US -4441.29 DS-4440.29	555	7.4	Y
Graveyard Gulch	River sta: 3194.56 GRA_300: San Xavier Rd	2-42" CMP	76.5'	US -4475.58 DS-4474.58	555	376.34	Y
SOLDIER CREEK WSHD							
Soldier Creek	River sta: 293 SOL_100: AZ HY90	5-10' X 8' RCBC	168	US -4556.24 DS-4555.74	4277	N/A	N
Soldier Creek	River sta: 1968.51 SOL_300: Kayetan Dr	2-6' X 4' RCBC	30.5	US -4571.48 DS-4571	4334	3859.45	Y
Soldier Creek	River sta: 4766.58 SOL_400: Garden Ave	4-12' X 11' RCBC	61.8'	US -4606.03 DS-4605.5	4334	N/A	N
Soldier Creek	River sta: 5260.98 SOL_500: N Buffalo Soldier Tr	5-12' X 11' RCBC	115.8'	US -4607.03 DS-4606.5	4688	N/A	N
MURRAY SPRINGS WSHD							N
Murray Springs	Reach 1 River sta: 28384.21 VistaPoint Dr	3-42" CMP	31.1	US -4362.9 DS-4362.9	239	N/A	N
Murray Springs	Reach 2 River sta: 23407.53 AZ90	2-8' X 7' RCBC	118.7	US -4295.25 DS-4295.25	658	N/A	N
Murray Springs	Reach 2 River sta: 16235.94 N Morson Rd	2-36" CMP	102.6	US -4222.81 DS-4222.81	911	813.39	Y