Appendices

For A Stormwater Action Plan for Sierra Vista

15 December 2015

Prepared for the City of Sierra Vista

with funding from the Walton Family Foundation by

Contents

Appendix A – Hydrologic Modeling

Rational Method Assessment of Impervious Surfaces

Within the City of Sierra Vista, a rough calculation of the amount of water available to harvest was performed. In 2009, a LiDAR data collection effort included estimates of the structure rooftop areas and impervious surfaces (driveways, streets, parking lots, etc.) throughout the City and the Fort. A close examination of these GIS shapefiles performed by the Senior Civil Engineer of the City indicated that the area of structures and impervious surfaces was underestimated by roughly 18%. Using the rational method, high and low estimates of runoff were calculated. Rooftop runoff coefficients were assumed to be between 0.8 and 0.95 and impervious surface runoff coefficients were assumed to be between 0.9 and 1. The results are shown in the table below, the high estimates multiply the area by 1.18 to account for the discrepancy observed in the shapefile areas.

Table 1. Rational Method Runoff Volume Calculation

GIS Layer Name	A - Area (Acres)	C - Runoff coefficient		R - Annual Rainfall (ft)	Net Runoff = C*A*R		
		Low	High		Low Est (Ac-Ft)	High Est (Ac-Ft)	Avg.
Structures	1,142.47	0.8	0.95	1.214	1,110	1,555	1,333
Impervious Surface	761.66	0.9	1	1.214	832	1,091	962
						Total	2,295

HEC-HMS and EPA SWMM Hydrologic Models

This section contains the *Hydrologic Modeling Report For Rainwater Harvesting Scenarios City of Sierra Vista, Arizona* prepared by JE Fuller (July 2015). An executive summary and additional analysis supplementing this report are provided as well.

Executive Summary

JE Fuller/Hydrology & Geomorphology, Inc. (JE Fuller) performed rainfall/runoff modeling to assess the effects of rainwater harvestings in the City of Sierra Vista. The rainfall/runoff modeling included modeling the Charleston Wash (aka Woodcutter Wash) Watershed using HEC-HMS and a small neighborhood within that Watershed using EPA SWMM. LID scenarios were developed for each of the two models to compare to existing conditions during the 2-year 6-hour rainfall event. A consecutive model run of back to back 2-year 6-hour rain events was performed for the HEC-HMS model of Charleston Wash Watershed. The report describes the model development and provides model outputs. The report prepared by JE Fuller Hydrology and Geomorphology, Inc titled "Hydrologic Modeling Report for Rainwater Harvesting Scenarios City of Sierra Vista, Arizona" and dated July 10th, 2015 can be provided upon request. A copy of this report has also been provided to the City of Sierra Vista.

The additional analysis section provides further information on the development of LID scenarios for each of the models and interprets the model results. The model of the Charleston Wash Watershed demonstrated that a 10% LID adoption scenario throughout the model area could results in significant local reductions in flow volume and velocity without significantly impacting the amount of water that reaches the San Pedro River. The neighborhood model further quantified the potential for rainwater harvesting to reduce flood volumes and velocities, reduce wash erosion, and capture rainwater in rain barrels to supplement the individual parcel water supply.

Additional Analysis

Developing the LID Scenario for the HEC-HMS Modeling of the Charleston Wash Watershed

The Charleston Wash LID Scenario represents a 10% LID adoption rate of parcels in the watershed. Representative parcels from each of the dominant zoning designations (SFR-6, SFR-8, SFR-18, SFR-36, MHR, MFR, and GC) were selected to implement LID. Water harvesting basins were delineated on these representative parcels where practical. A water harvesting basin depth of 0.3 feet (3.6 inches) was assumed reasonable for all LID basins. In order to account for enhanced infiltration within the basins, an average infiltration rate of 0.25 in/hr above the assigned Green and Ampt infiltration rate was assumed. For a 6 hour modeling period, this meant the volume retained in the water harvesting basins was represented by a depth of 0.425 feet.

Due to the method of implementing LID in the HEC-HMS model, it was necessary to determine the max storage for each parcel identified to adopt LID in the scenario. The average volume of water harvesting basins was divided by the average parcel area for each group of representative parcels to obtain the max storage for each parcel based on zoning designation. 10% of parcels in each zoning designation were randomly selected to implement LID in the scenario. Not all of the representative parcels were selected to be included in the LID scenario. The number of representative parcels, selected parcels and the calculated max storage depth for each zoning designation is shown in the table below. To simplify integration of the LID scenario into the model, an average max storage of 0.053 ft was assigned to all zoning designations except MHR and MFR.

Table 2. LID Scenario Summary for HEC-HMS Modeling

Zoning Designation	Number of Representative Parcels	Max Storage (ft)	Number of Parcels in 10% LID Scenario
SFR-6	5	0.056	66
SFR-8	5	0.049	42
SFR-18 & SFR-36	5	0.059	6
MHR	1	0.022	11
MFR	1	0.010	29
GC	5	0.049	33

Additionally, 10% of the streets in the project area were selected that were best suited for green infrastructure. This was based on their width, proximity to community centers, and likelihood of implementation. Water harvesting basins were delineated on the selected streets and assigned max storage of 0.425 feet. The parcels and street included in the 10% LID scenario are shown in the Figure 1. The LID area of each HEC-HMS sub-basin is shown as a percent of the total sub-basin area in Figure 2.

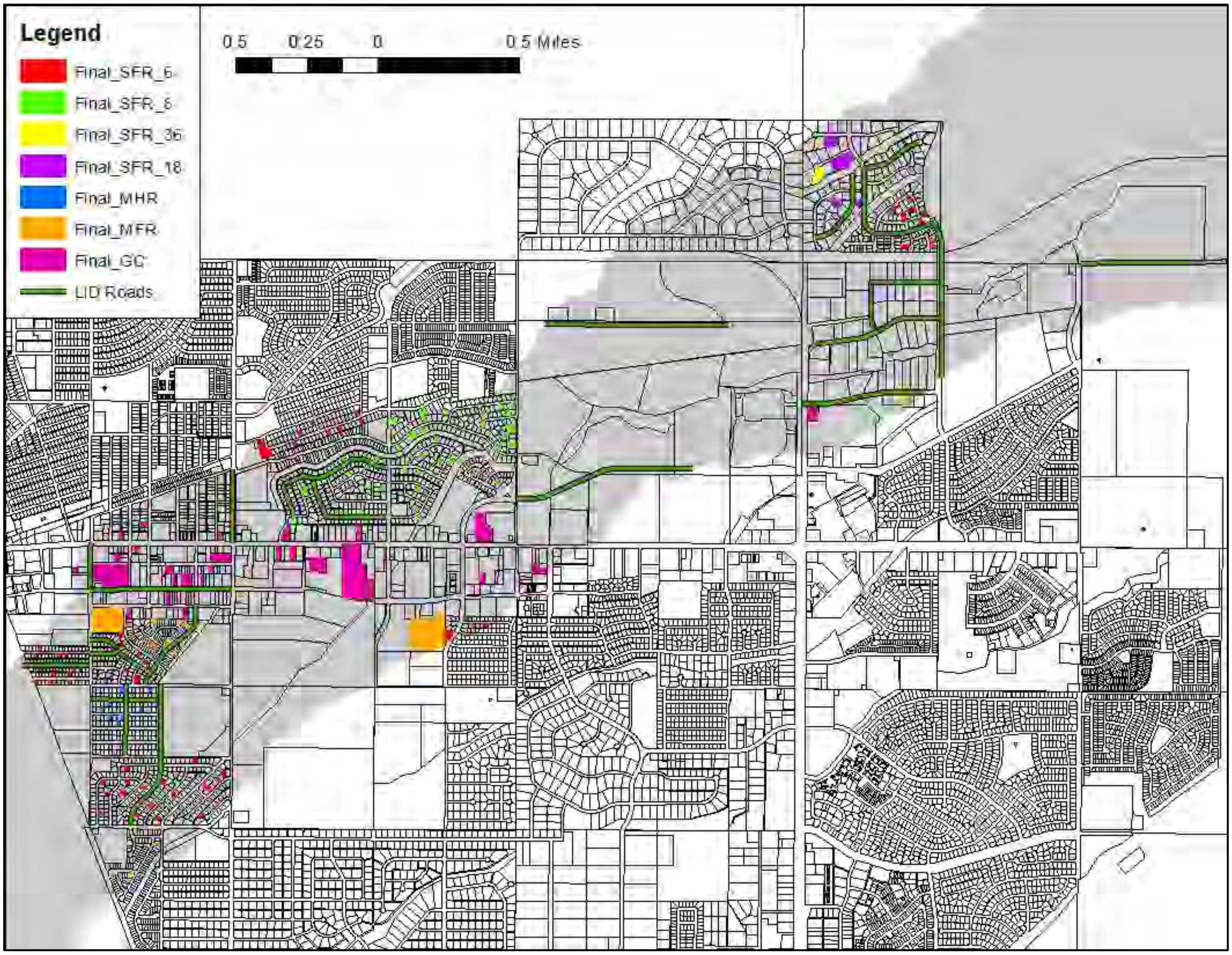

Figure 1. Parcels and Streets included in the 10% LID Scenario

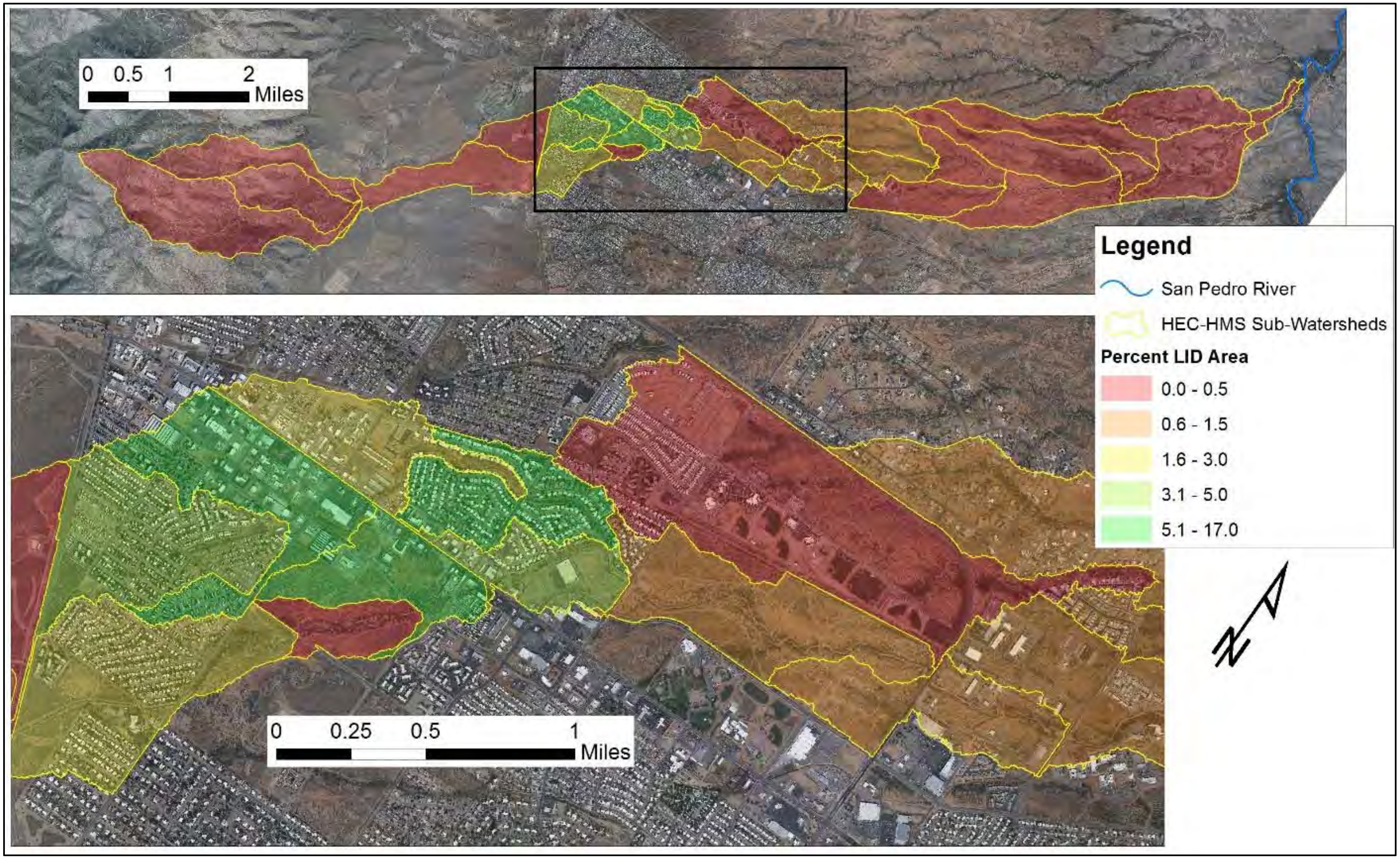

Figure 2. HEC-HMS sub-watershed delineation and percent of drainage area that was converted to LID practices.

Results Analysis

The Woodcutter Wash (a sub-watershed to the larger Charleston Wash watershed) cuts through the middle of the City and was singled out for its visibility and erosion issues within the City limits. This wash drains directly to the San Pedro River after passing through historic sections of the City. The Charleston Wash watershed and HEC-HMS sub-watershed delineation is shown in the Figure 2.

Fourteen of the sub-watersheds in the HEC-HMS model had LID implemented occupying between 0.4 and 16.7% of the drainage area (Figure 2). The reduction in the volume and peak runoff from each sub-watershed with LID implemented was strongly correlated to the percent of drainage area the rainwater harvesting basins occupied (Figure 3). In the sub-watershed with the most LID implemented (16.7%) the reduction in peak flow during the 2-year storm event was 14.3% and the reduction in total volume was 15.7%. For individual sub-basins with LID implemented, the average reduction in peak flow was 3.5%, the average reduction in volume was 3.7%, and the average area occupied by LID in the sub-basins was 3.9%.

The observed volume reductions in each sub-watershed can be directly related to reductions in landscape watering. Based on the shallow basin design essentially only the “irrigation” benefit is retained and infiltrated on-site and so the retained volume is fairly equivalent to the irrigation savings minus a small evaporative loss factor. The model results show that 3.8 Ac-Ft of water would have been passively harvested in 10% adoption of LID practices throughout the Charleston Wash watershed. Scaling this up to a 10% LID adoption throughout the City, the total savings could be 20.5 Ac-Ft.[1] Scaling these results up to a 25% adoption could result in 51.3 Ac-Ft of water savings.

[1] The Woodcutter Wash sub-watershed occupies about 1/5.4 of the developed area within city limits.

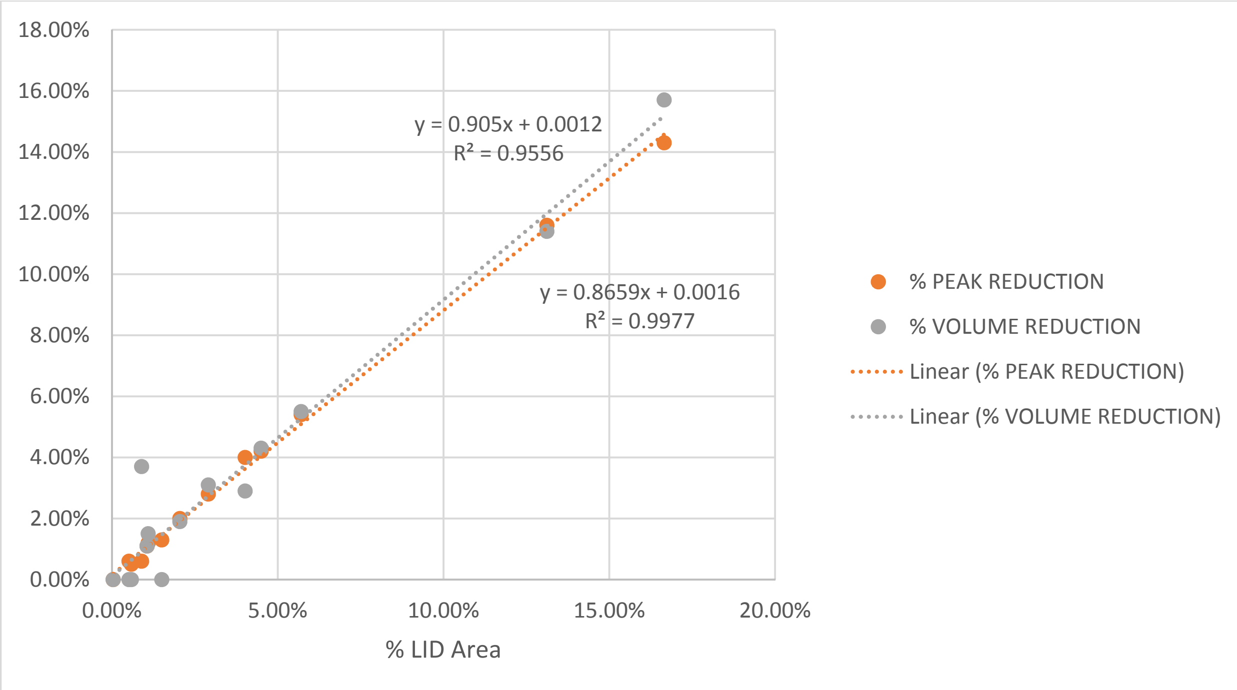

Figure 3. Relationship between % LID area and % reductions in runoff peak and volume.

In addition to water savings from reduced irrigation demand on the aquifer, the local reductions in peak flow and runoff volume can be related to reductions in wash erosion. Reduced flows result in reduced velocities and shear stresses which will entrain less sediment. The peak flow passing under Fry Blvd in the 3rd Street wash was reduced from 134.3 cfs to 122.5 cfs. Assuming a rectangular cross section at the grade control structure downstream of this culvert, the velocities passing over the grade control structure are reduced from 4.7 fps to 4.5 fps (4.3% velocity reduction).[2]

The reduced velocities can affect individual locations where runoff enters a wash and widespread erosion within the wash. These model results provide flow volumes and velocities at junctions along the channel wherever a sub-basin enters a wash or two washes confluence. The results of the HEC-HMS modeling are shown in Figure 4 and Figure 5.

The purpose of implementing LID throughout the City is to preserve the San Pedro River and aquifer. It can be argued that rainwater harvesting practices could negatively impact the San Pedro by removing water from the system that would otherwise make it downstream to the river. The results of this modeling effort show that even though LID led to significant reductions in peak flow and runoff volumes, the total volume reaching the San Pedro River was only reduced by 1.2% during the 2-year storm event (Figure 5). The LID would be overwhelmed in larger events and the changes in flow making to the San Pedro River during such larger events would be minimal.

During the LID model run there was no runoff from the BMP sub-basins in HEC-HMS, meaning that the incident rainfall was all infiltrated or stored in the basins. A simulation of two consecutive 2-year 6-hour

[2] Critical velocities calculated over the grade control structure crest with bottom width of 42 feet.

events was also performed. In this consecutive events model, only one BMP sub-basin had any outflow and the amount of outflow was minimal. The peak flows from the other basins (without BMPs) in the consecutive storm were larger than the initial peak by an average of 61% for both existing conditions and the LID scenario. A hydrograph from the consecutive storm model simulation for the Bella Vista neighborhood draining to the 3rd Street Wash is provided in Figure 6.

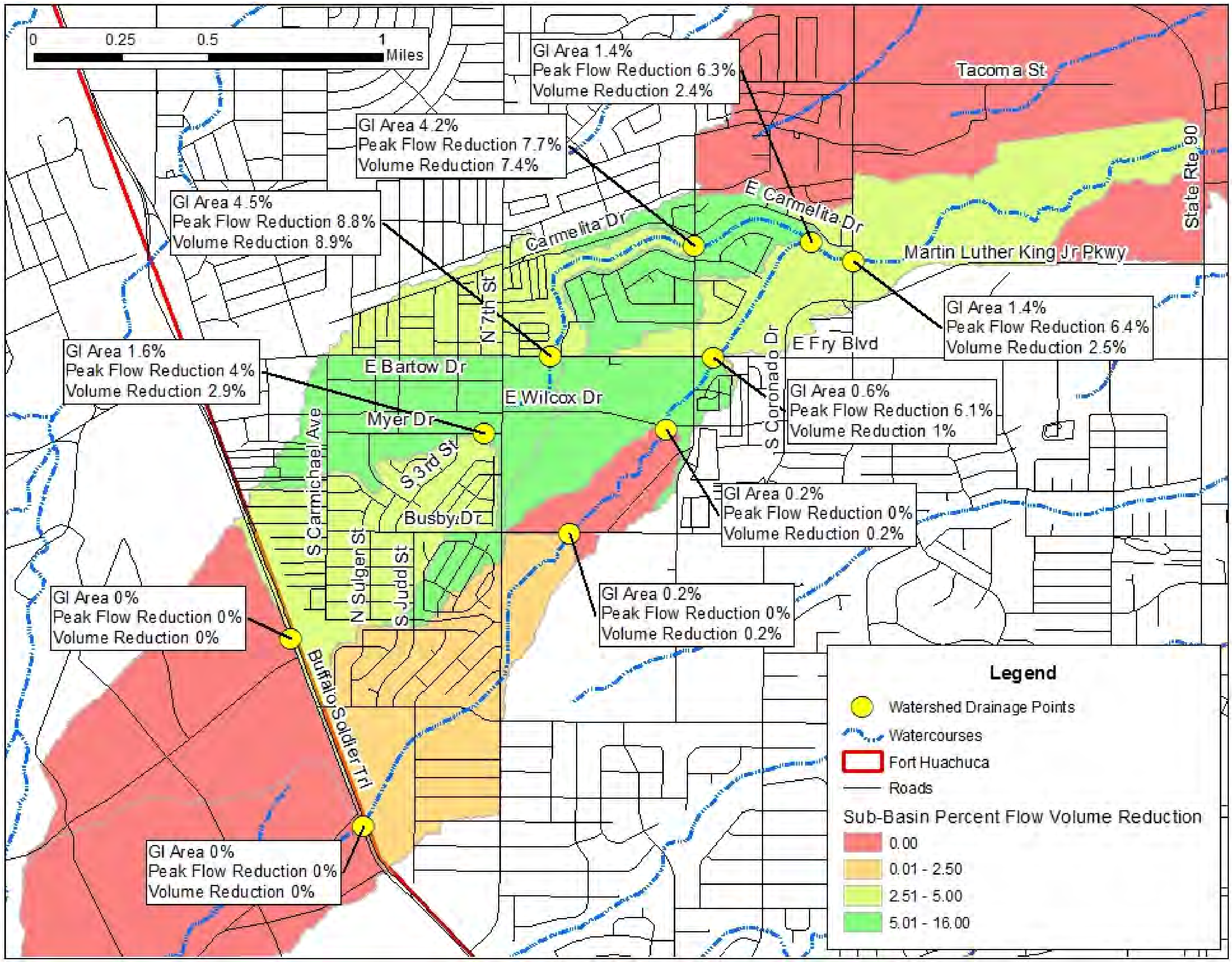

Figure 4. Charleston Wash Modeling Results in the City. The individual sub-basin reductions in flow volume are shown in the sub-basin shading. The cumulative reductions seen at flow junctions are shown in callouts.

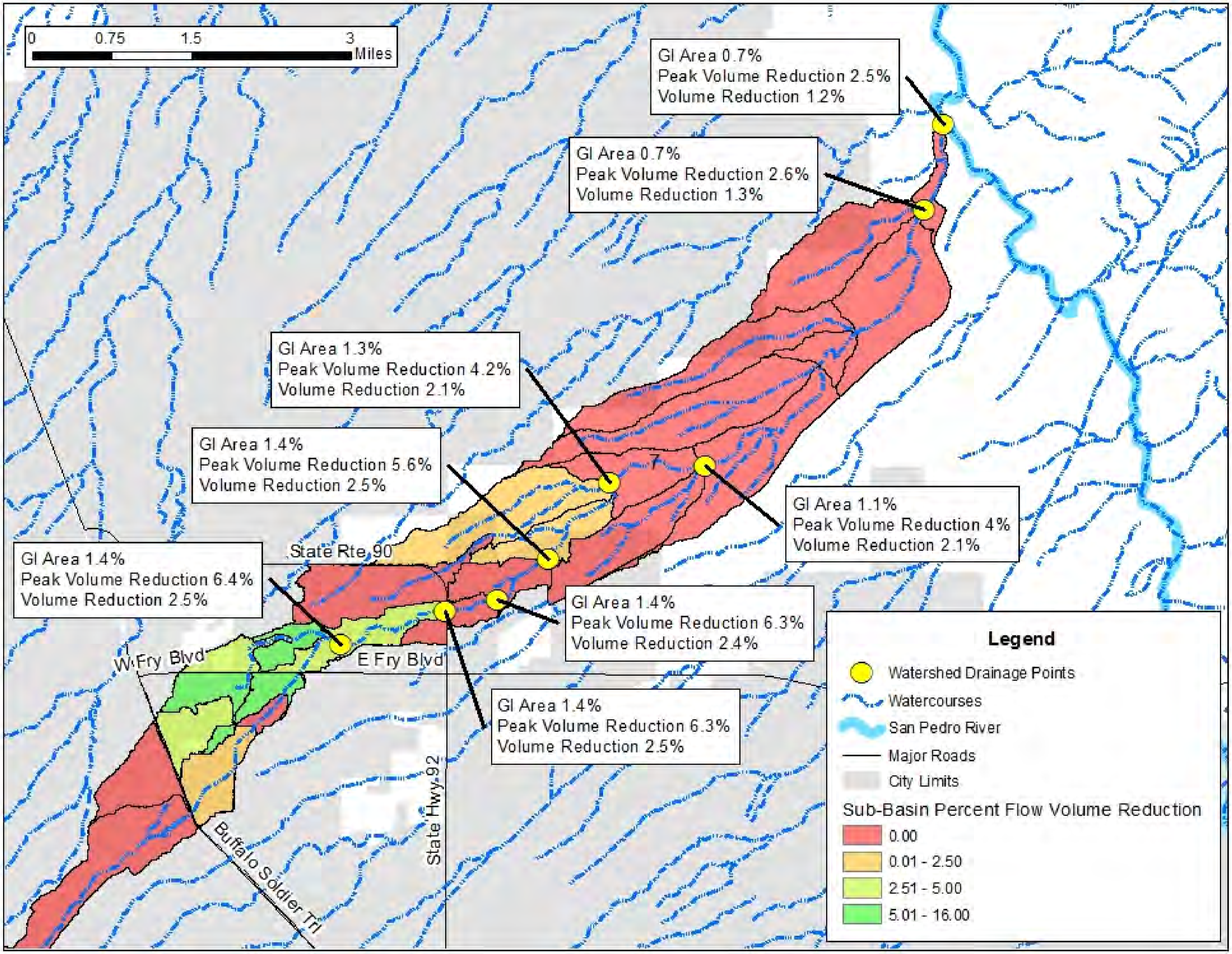

Figure 5. Charleston Wash Modeling Results to confluence with the San Pedro River. The individual sub-basin reductions in flow volume are shown in the sub-basin shading. The cumulative reductions seen at flow junctions are shown in callouts.

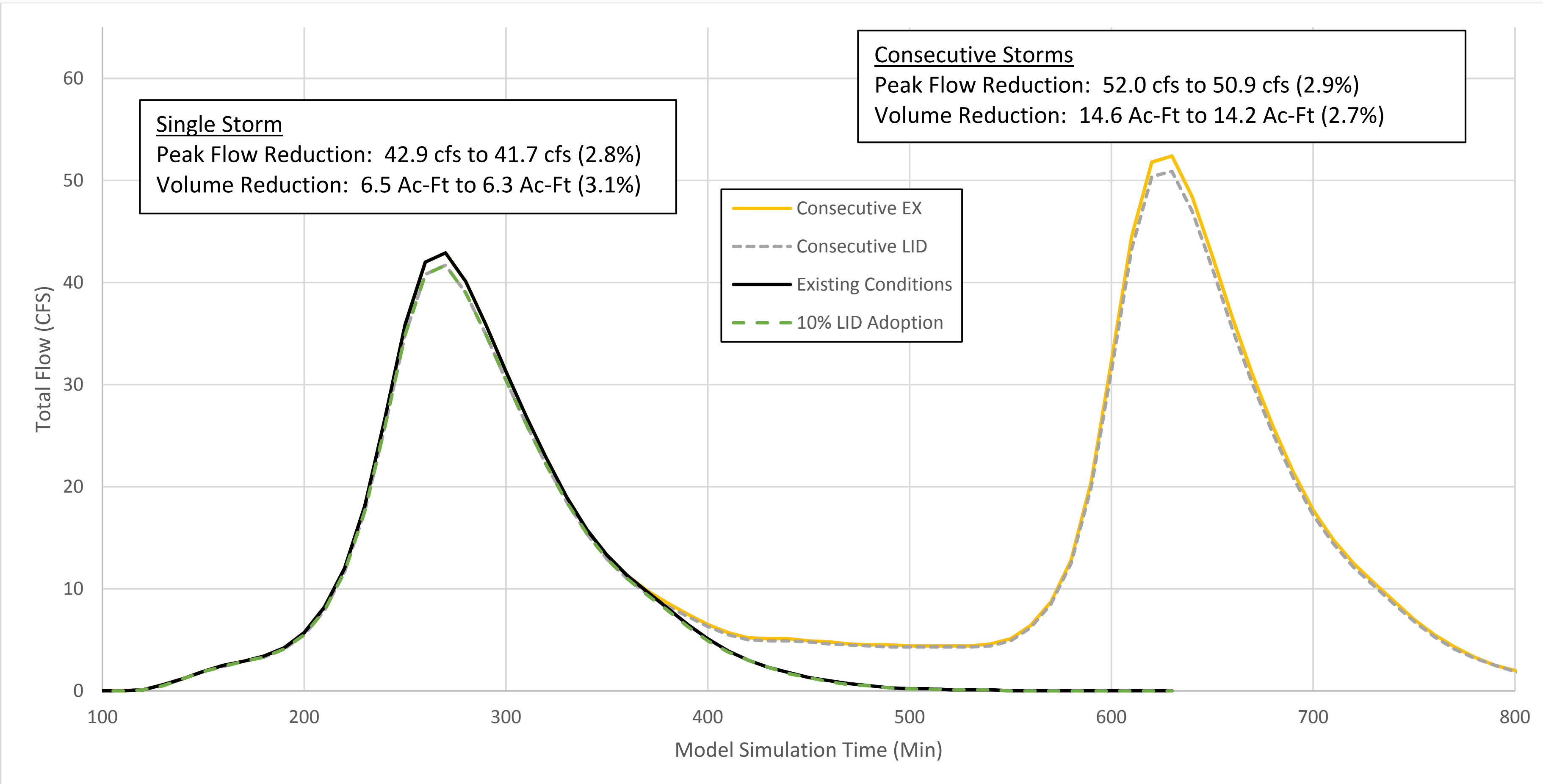

Figure 6. Hydrographs for the runoff from the Bella Vista Neighborhood

EPA SWMM Modeling of the Bella Vista Neighborhood

The Bella Vista neighborhood (Figure 7) is located within the Charleston Wash watershed modeled in the previous section. This neighborhood drains east directly to the 3rd Street Wash which was identified by the City for potential stream stability improvements. The neighborhood is historic but is highly impervious.

Figure 7. Bella Vista Neighborhood (Google Earth imagery dated January 2015)

Developing the LID Scenario

Retention basins (i.e. rain gardens) and rain barrels were added to residential, commercial and a school parcel within the neighborhood shown in Figure 7. Rain barrels were sized based on the roof area observed in aerial imagery and rain gardens were sized based on opportunities in the parcel landscape. The hydraulic conductivity for all retention basins was assumed to be 0.85 in/hr. A study in 2004 found the field derived, in-situ K values for the 3rd Street Basins (located 0.9 km upstream of the Fry Ave bridge) averaged 0.85 in/hr (Appendix B). The hydraulic conductivity used as a default for pervious surfaces in the model is 0.4 in/hr.

Twenty five residential parcels on the north side of S Denman Ave that drain toward 7th Street were fitted with both rain barrels and rain gardens. The roof area of a few constructed houses along Denman Ave were measured from aerial imagery to be between 1,200 to 1,500 SF. While 865 gallon rain barrels would be preferable, not everyone will want such a large barrel on their property. It is assumed that about half of the houses (12 parcels) can be talked into the 865 gallon rain barrels. The remaining 13 parcels were assigned 420 gallon barrels. There is limited space for rain gardens on the new residential parcels. It was estimated that rain gardens can occupy 210 SF in the front yards of these parcels, sufficient for 1-2 trees. Standard depth for the rain gardens allows for 4”-6” of ponding, therefore 5” was used for this scenario. All 25 parcels were assigned a rain garden in this scenario.

Three parcels on the south side of S Denman Ave in between N 2^{nd} St and 5^{th} St were included in the GI scenario. Imagery shows that all three parcels were occupied by manufactured homes as recently as 2009, one has already been developed as a gated community while the other two have been cleared of the manufactured homes that were present in the 2009 imagery. The percent of impervious surface in the developed parcel was estimated at 65%. It was determined that detaining the first flush (0.5 in.) volume of water was practical within the parcel assuming rain garden with a 4 inch depth. Similar conditions were assumed for the development of the other two parcels: 65% impervious and able to capture the first flush volume from impervious surfaces in retention basins within the landscape.

The school parcel is 7 acres and approximately 52% impervious. Discounting the baseball field, there are 4.3 acres of land available for landscaping. It was assumed that 1/3 of this land can be transformed into rain gardens with 4" ponding depth, creating a total of 62,370 SF of rain garden. This is sufficient to capture more than just the first flush of rainfall. There are 3 buildings on the school property, a 2,825 gallon rain barrel was assigned to each building to collect roof runoff. This dimension of rain barrel is sufficient to capture half of the first flush runoff from the roofs of the 2 smaller buildings and 1/3 of the first flush runoff from the largest building.

The majority of business parcels along 7^{th} St and Fry Blvd are almost entirely impervious surfaces. Note that some of these parcels fall under county zoning rather than city zoning. To treat these parcels, it was assumed that these business and their associated parking lots would be renovated in the near future and that retention basins would be installed to intercept as much runoff as possible. Commercial parcels that are less than 0.5 Acres are assumed to accommodate a 400 SF rain garden, parcels greater than 0.5 Acres are assumed to accommodate an 800 SF rain garden. The large commercial parcel in the north east corner of Fry Blvd and 7^{th} St (9.2 Acres) drains directly to the corner of the grade control structure in the 3^{rd} street wash that is actively eroding and endangering a sewer line. This parcel was assigned a rain garden area of 1,200 SF.

Results Analysis

While results are available from every parcel in the model, six junctions were placed in the model to collect cumulative results where flows converge or points of interest. These junction locations are shown in Figure 8. Across the street from the apartment complex between N 2^{nd} St and 4^{th} St a large wash collects the majority of the runoff from the model area. The wash runs due east from 4^{th} street, beginning about 250 feet south of the intersection with S Denman Ave (J1 West). This junction collects all of the runoff coming from parcels west of 4^{th} St. The wash terminates on the west side of N 7^{th} Street and conveys runoff through a culvert under N 7^{th} Street to a large catch basin (J2 West). There is a cross street drain in an alley off of N 7^{th} St, approximately 200 feet north of Fry Blvd (J3 West), that drains to this catch basin (J2 West) as well. From this catch basin, one large storm drain conveys the runoff to the 3^{rd} Street wash.

The neighborhood east of N 7^{th} St is isolated from the remaining watershed and most of these parcels drain independently to the 3^{rd} Street wash. This area was modeled separately in EPA SWMM and junctions were placed at locations of interest. The commercial parcels on the northeast corner of Fry and 7^{th} St drain to junction J1 East where there is active erosion on the west side of the grade control structure that is endangering a sanitary sewer line. There is also active erosion in a little wash collecting runoff from Carmelita Drive (J2 East). The drainage from school parcel drains to junction J3 East.

Figure 8. Bella Vista Neighborhood. Imagery from 2009 LiDAR data collection.

The hydrographs at each junction during the 2-year 6-hour storm event are shown in the figures below. The prefix PRE indicates the existing conditions model and POST indicates the model results with LID implemented.

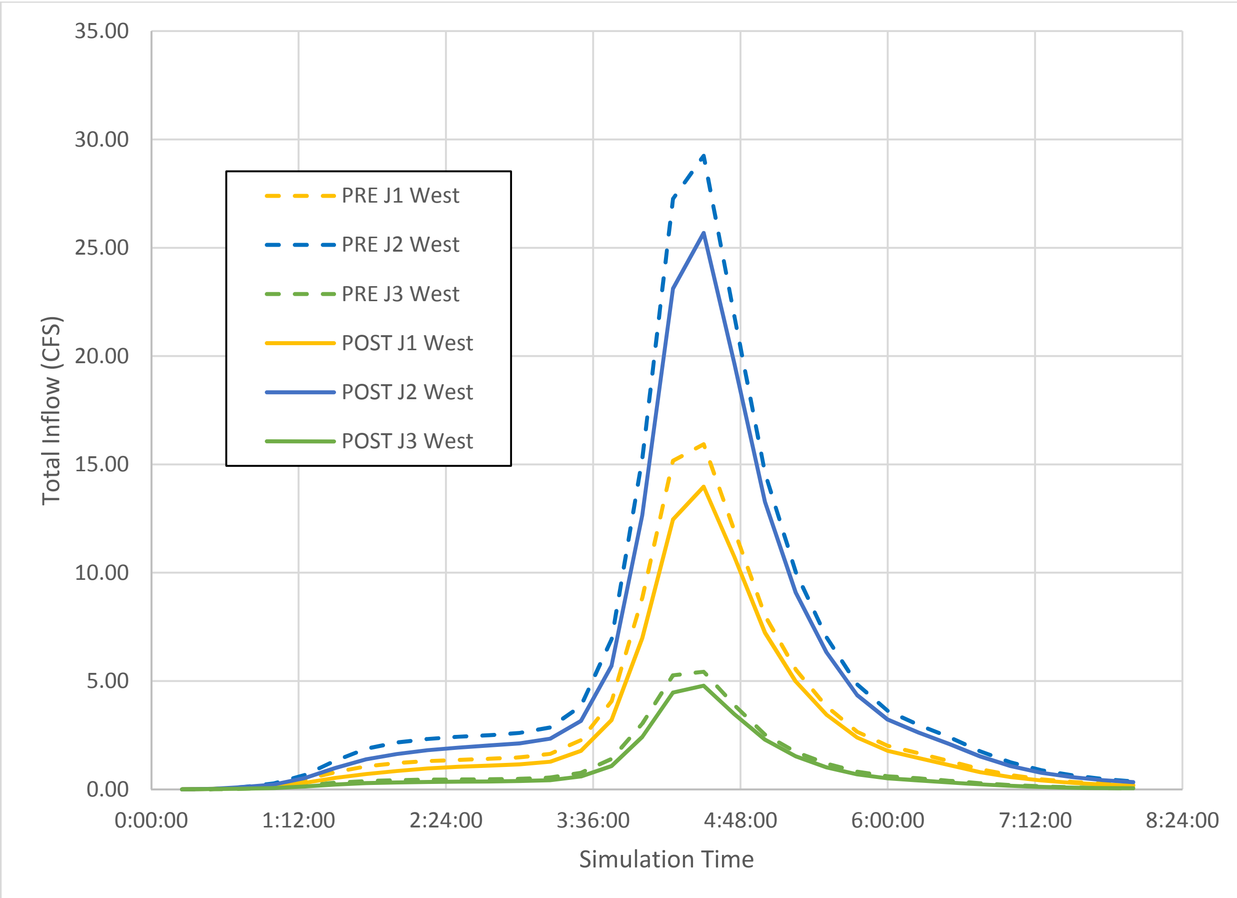

Figure 9. Hydrographs from junctions in the model west of 7th St.

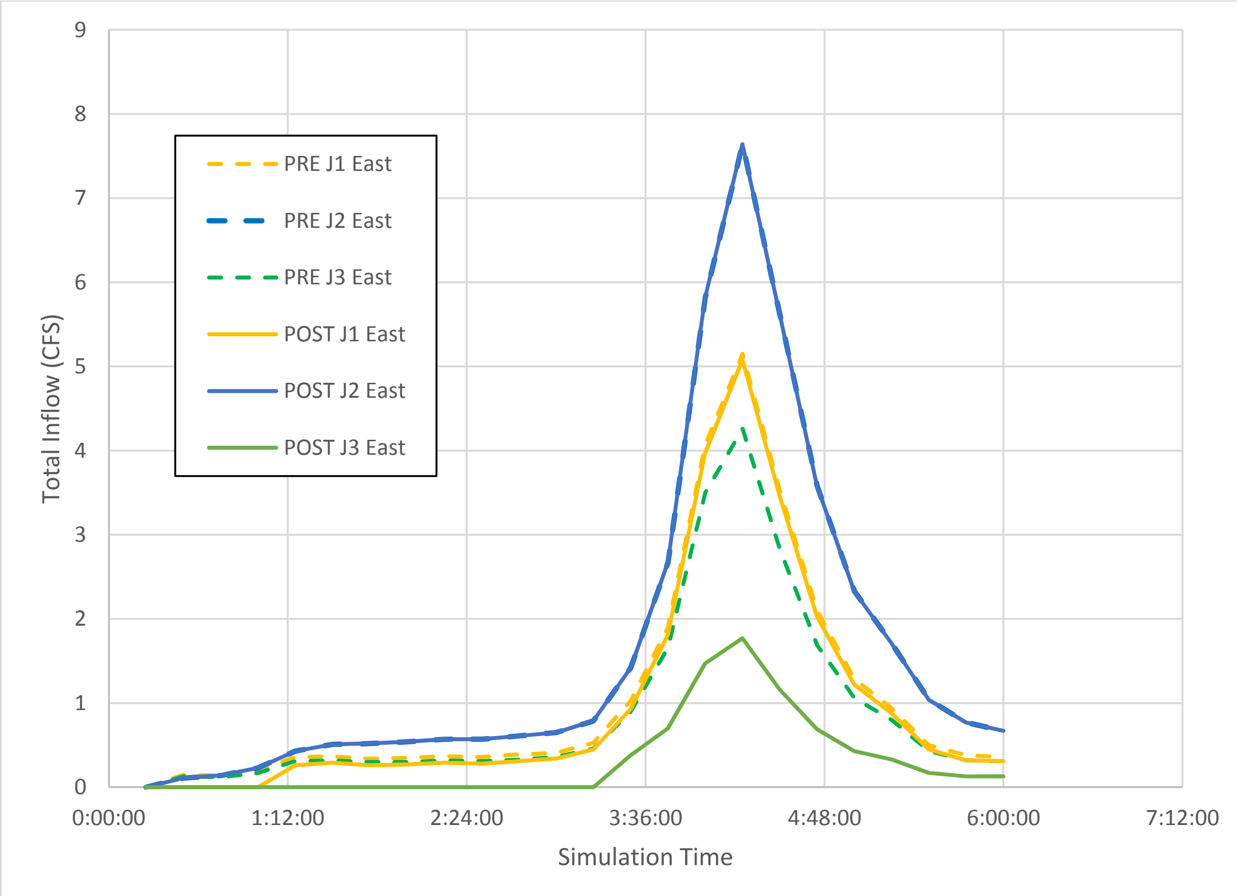

Figure 10. Hydrographs from junctions in the model east of 7th St.

The peak flow during the 2-year 6-hour storm event at each junction are summarized in Table 3. Throughout the west portion of the model area there is consistently a 12% decrease in peak flow at each location. Results east of 7th Street are mixed, the rain gardens in the commercial parcels on the northeast corner of Fry Blvd and 7th St did not have a significant impact on the flow eroding the corner of the grade control structure while the rain barrels and rain garden proposed at the school significantly reduced the flow from this parcel.

Table 3. Peak Flows at Junctions in Bella Vista Neighborhood

	West			**East**		
	Pre (CFS)	**Post (CFS)**	**% Decrease**	**Pre (CFS)**	**Post (CFS)**	**% Decrease**
J1	15.93	13.97	12.3%	5.15	5.08	1.4%
J2	29.25	25.68	12.2%	7.64	7.64	0.0%
J3	5.43	4.79	11.8%	4.26	1.77	58.5%

The reductions in total runoff volume at each junction are shown in Table 4.

Table 4. Total Volume at Junctions in Bella Vista Neighborhood

	West			East		
	Pre (Gal)	**Post (Gal)**	**% Decrease**	**Pre (Gal)**	**Post (Gal)**	**% Decrease**
J1	654,000	552,000	15.6%	17,000	15,000	11.8%
J2	1,170,000	1,010,000	13.7%	26,000	26,000	0.0%
J3	217,000	183,000	15.7%	14,000	5,000	64.3%

Of the runoff retained in the model, 26,500 gallons (0.08 Ac-Ft) were stored in rain barrels during this single storm. This is sufficient water to provide 27 people 140 GPCD for seven days. 303,600 gallons (0.93 Ac-Ft) of water was retained in rain gardens, watering native vegetation and infiltrating the shallow aquifer. Water infiltrating in rain gardens is treated by bio-filtration processes in the soils, reducing stormwater pollution.

The reductions in flow volume and peak discharge values relate to reduced erosion in the receiving washes and locations downstream. The entire west model drains to junction J2 West, a catch basin on the east side of 7th street. The catch basin has a 4 ft outlet pipe that runs parallel to a channel that collects runoff from 7th street. The 4 ft pipe terminates at another catch basin where the flow from the channel joins it via three 30 in storm drains. Flow from this catch basin travels through a 66 in (80 L.F. at 0.625%) storm drain that spills directly into the 3rd Street Wash. The 12.2% decrease in peak flow seen at J2 West would reduce the velocities coming out of the pipe by 5% (3.9 fps reduced to 3.7 fps). Runoff from the school was reduced by 58.5% which mostly drains to an 18 in CMP drain pipe that runs from the parking lot directly to the 3rd Street Wash. Flows in this pipe would be reduced from 5.5 fps to 4.1 fps (26% reduction). As flow velocity and wash erosion are related, these decreases in flow velocities entering the wash will reduce localized erosion. The reduced flows and volumes reaching the wash will reduce widespread wash erosion.

Comparing the Two Models

The two hydrologic models were developed to assess the impact GI/LID can have on beneficial use of stormwater runoff. A comparison of the results found by the two models is provided in this section. The model of Bella Vista neighborhood using EPA SWMM was more detailed as the modeling software has integrated tools specifically to assess GI practices. The following items are important when comparing the costs and model results:

- The HMS scenario randomly selected 10% of parcels in the Charleston Wash watershed while the SWMM scenario maximized the LID adoption likely to occur in the Bella Vista neighborhood.
- HMS only considered rain gardens while SWMM had rain barrels in addition to rain gardens.
- Rain garden in HMS were modeled by removing their area from the sub-watershed and creating a separate BMP sub-basin with a higher surface storage.

The Bu3ST_02 basin in the HEC-HMS model covers the Bella Vista neighborhood. The results from this sub-basin in the HEC-HMS model are compared to the EPA SWMM model results (Table 5). The reductions in peak flow and volume are significantly higher in the SWMM model than in the HMS model at a similar cost as a result of the higher cost per feature of HMS scenario for retrofits that require asphalt or concrete removal. For a detailed description of Cost Benefit Analysis results and methods see Appendix C.

Table 5. Model Results Comparison

	SWMM	**HMS**
Passive Harvesting (Ac-Ft)	0.93	0.2
Active Harvesting (Ac-Ft)	0.08	0.0
% Volume Reduction	15.5%	2.74%
% Peak Flow Reduction	Typ. 12% Range: 0 – 58.5%	6.28% Range: 0 – 14.3%
Total Cost	$1,040,000	$1,000,200
Annual Benefits	$139,000	$26,000

Appendix B – Opportunities for Enhanced Channel Recharge

Executive Summary

Two technical memorandums were prepared by Hassayampa Associates (M.T. Murphy) to support the development of stormwater management recommendations. The first report "Buena #3 Geomorphic and Geologic Assessment" dated March 10th, 2015 analyzes a tributary of Woodcutter Wash for use of alternative channel treatments to prevent erosion and increase infiltration in or adjacent to the channel. A conceptual sketch developed for the critical area highlights both upland features to intercept and make use of stormwater to improve the xeroriparian corridor prior to being discharged into the Woodcutter wash tributary and channel features to enhance natural channel dynamics to further stabilize both lateral and downcutting forces.

The second report "Opportunities for recharge using grade-control structures" dated May 6th, 2015 contains a literature review of spatially distributed hydrogeological data to identify where grade control structures might be located in the Sierra Vista area to enhance recharge. Hydraulic conductivity estimates from the City based on NRCS soil maps were determined to be the best general picture of the spatial distribution of recharge potential. However, the presence of cemented calcium carbonate layers (caliche) at unmapped depths could prevent deep recharge. A qualified geologist familiar with calcic soils may be able to use existing studies to map regions within the City where calcic horizons near the surface are likely. Hydrogeological site investigations are recommended at grade control structure sites to determine site specific recharge potential.

A copy of these reports can be provided upon request and have been also provided to the City of Sierra Vista.

Additional Analysis

Following the findings of the memorandum, this section provides more detail on the NRCS soil mapping in the City and the weighted hydraulic conductivity for each soil type provided by Alan Humphrey, the Senior Civil Engineer for the City. An inventory of stormwater infrastructure is in progress within the City and data has been collected on all washes. There are currently 73 existing grade control structures (GCS) mapped in the City. The locations of these and the underlying soil data is shown in Figure 11. The weighted hydraulic conductivity based on vertical infiltration of soil layers during 1-hr, 6-hr and 24-hr storm events are provided in the following table.

Table 6. Weighted hydraulic conductivity by soil mapping unit.

Soil Map Unit Symbol	1-Hr XKSAT (in/hr)	6-Hr XKSAT (in/hr)	24-Hr XKSAT (in/hr)	# Existing GCS
127	3.01	3.01	3.01	1
144	0.22	0.22	0.21	36
149	0.40	0.40	0.29	-
24	0.40	0.40	0.40	-
32	1.20	1.20	0.89	-
40	0.40	0.40	0.18	32
60	0.40	0.13	0.07	-
71	0.40	0.40	0.21	2
76	0.40	0.40	0.45	-

87	1.62	1.62	1.41	2
97	0.40	0.12	0.03	-

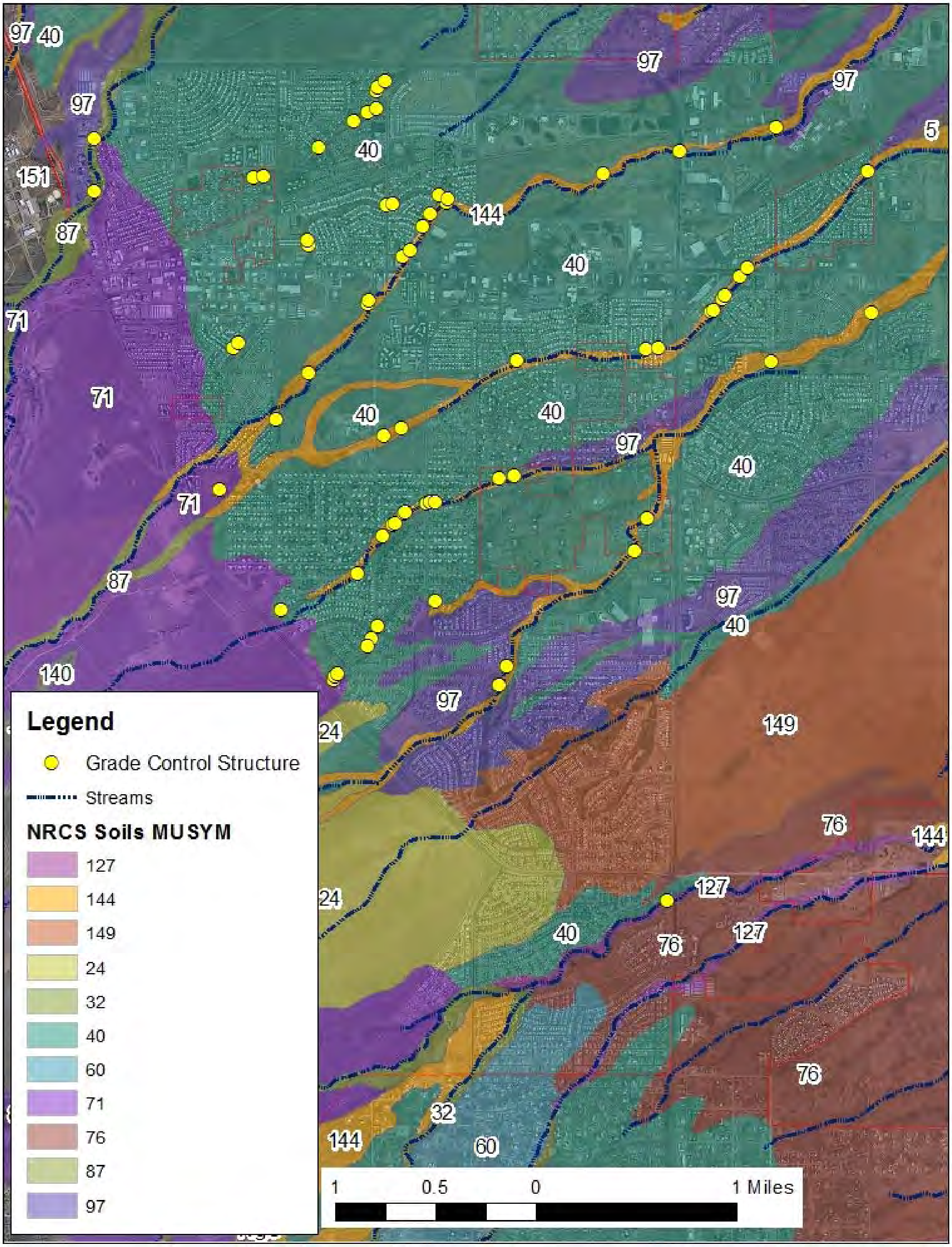

Figure 11. Existing grade control structures and associated surface soil map units.

From these data, it is evident that soils identified as map unit symbol 127 have the highest recharge potential. Soils 87 and 32 both have fairly high infiltration rates even though their infiltration rate decreases at deeper horizons. Therefore the highest recharge potential exists in portions of the wash at the northwest corner of the Figure above and the washes at the southern border of the City (see Figure 12).

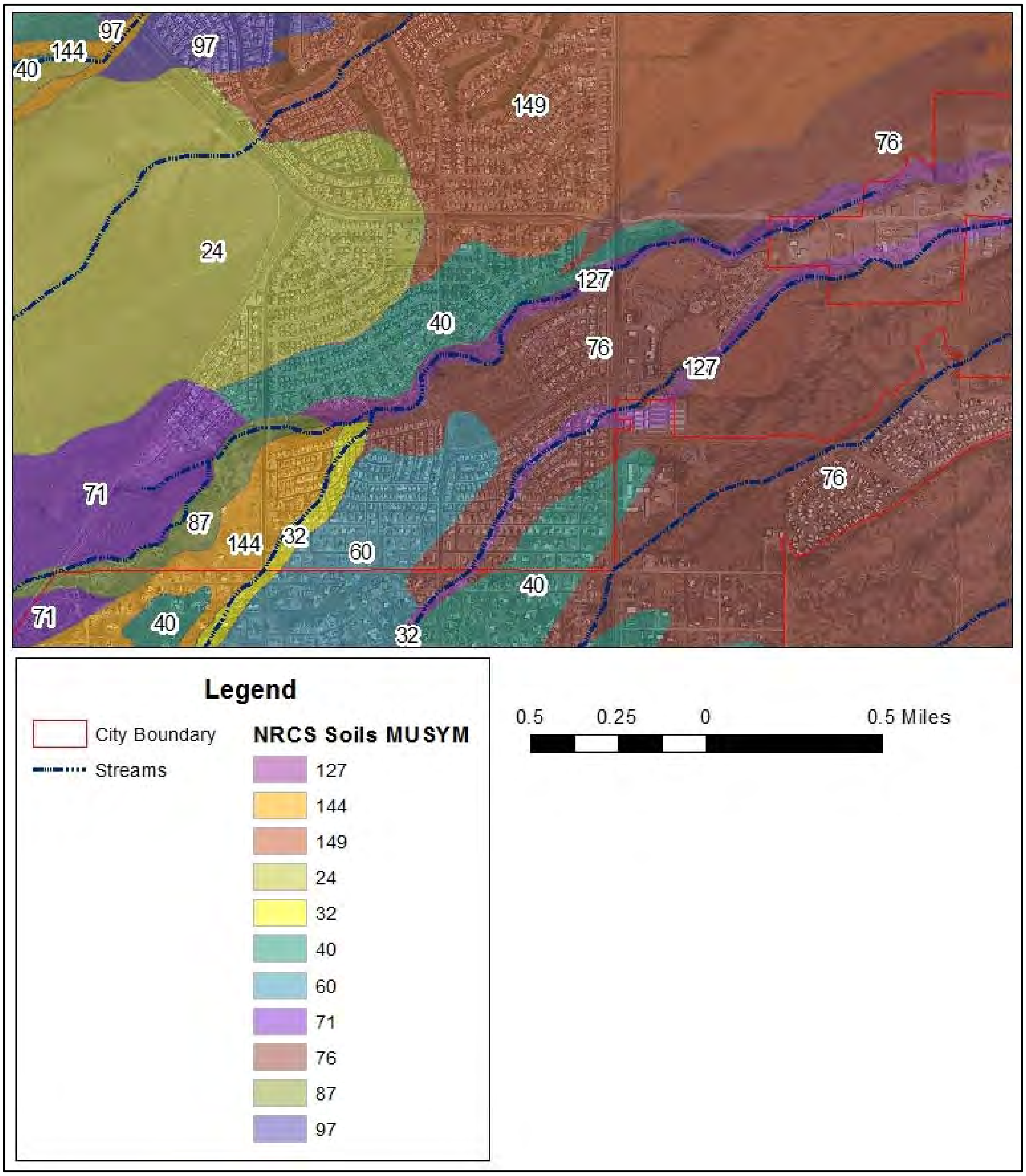

Figure 12. Detail of channels and associated surface soil mapping units.

In the heart of the city, soils 144 and 40 dominate and most existing GCS are located on these soils. Insight into the recharge potential of GCS or facilities is provided by hydraulic conductivity measurements at the 3rd Street Wash and Fry Basin detention facilities. The former is located on a tributary to Woodcutter Wash overlaying map unit symbol 40 soils. This site measured hydraulic conductivities ranged from 0.1 – 0.85 in/hr (median = 0.48 in/hr). While the Fry Basin site is located over soil map unit symbols 40 and 71 the measured hydraulic conductivities were as high as 6.65 in/hr.

Appendix C – Cost Benefit Analysis Details

Detailed Benefit Description

Below is a detailed summary of economic values calculated for LID features. For additional details on the research that supports these costs see the footnotes, i-Tree references, and the Business Case Evaluator (BCE) report 'Evaluation of GI/LID Benefits in the Pima County Environment'.[3] Regression analyses, cost data based on WMG's experience and outputs from these tools were summarized in a Microsoft Excel spreadsheet in order to develop a method to quickly calculate watershed scale results.

Direct Benefits

Water Conservation – Water demand during establishment for rain gardens was based on low water use plants[4] for two native trees, 4 grasses, 2 groundcover plants and 2 shrubs for every 100 square feet of basin area. It is assumed that LID has similar irrigation efficiencies as flood irrigation. Flood irrigation efficiencies range from 40-85%[5]. A conservative value of 50% is used here. Irrigation demand is reduced by 50% for commercial properties as used by City of Tucson Commercial development requirements. It is assumed that irrigation to LID is discontinued after the first 3 years of plant establishment.

Air Quality Improvement – The BCE tool was used to define a relationship between number of trees planted and benefit value. Air quality benefits are calculated in the BCE as the sum of reduced emissions of air pollutants from power-generating plants, and the value of pollutant uptake from trees.

Energy Savings – The i-Tree Streets tool was used to define a relationship between number of trees planted and benefit value. Energy savings are based on reduction in air conditioning and heating bills as a result of tree shade. This value is calculated using the method described by McPherson et al.,[6] which uses a typical single family residence to model energy simulations.

Reduced Street Maintenance – Shade created by trees installed with LID extends the life of asphalt pavement, reducing the maintenance required. As shown by McPherson & Muchnick,[7] significant financial savings can occur from pavement shading.

Property Values – Property value increases occur as a result of local environmental attributes. Studies show that each large front yard tree is associated with a 0.88% increase in property value.[8] The BCE[9] uses local property values and applies research values to estimate the increase in property value due to Low Impact Development (LID) projects. The BCE tool was used to define a relationship between water harvesting basin dimensions, number of trees planted and benefit value.

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[3] Impact Infrastructure, Stantec, 2014. Evaluation of GSI/LID Benefits in the Pima County Environment.
[4] Tucson Water. Harvesting Rainwater: Guide to Water-Efficient Landscaping.
[5] Howell, T.A. Irrigation Efficiency. Encyclopedia of Water Science. United States Department of Agriculture.
[6] McPherson, E.G., J.R. Simpson, P.J. Peper, S.E. Maco, Q. Xiao, and E. Mulrean. Desert Southwest Tree Guide: Benefits, Costs, and Strategic Planting. Arizona Community Tree Council: Phoenix, Ariz., 2004.
[7] McPherson, E.G., and Muchnick, J. Effects of Street Tree Shade on Asphalt Concrete Pavement Performance. Journal of Arboriculture, Vol. 31, No. 6, November 2005.
[8] Anderson, L.M. and Cordell, H.K. Influence of Trees on Residential Property Values in Athens, Georgia (U.S.A.): A Survey based on Actual Sales Prices. Landscape and Urban Planning, Vol. 15, 1988.
[9] Parker, J. and Meyers, R. (2015). Business Case Evaluator A Value and Risk Based Enhancement to Envision User & Documentation Manual. Impact Infrastructure.

Avoided Grey Infrastructure – Potential savings of large scale flood mitigation infrastructure if LID retention is taken into account in sizing detention basins, storm drains or culverts. Values used in the CBA are based on the cost estimates of proposed grey alternatives for the Airport Wash area and conservative estimates of savings. Additionally included in this benefit is the amount of water intercepted by trees planted, estimated using the i-Tree Streets tool. McPherson et al. used Glendale, Arizona's cost for retention/detention basins to determine the value of water collected and stored by trees.

Indirect Benefits

Social Value of Water Conservation - The indirect cost of water is determined by the cost of water extraction and purification from alternative water sources. This represents the cost to provide water if limitations on pumping from the SPRNCA were enacted and new sources of water are accessed. The cost of water from alternative sources was found in "Augmentation Alternatives for the Sierra Vista Sub-watershed, Arizona."[10] The alternatives presented in this report were summarized in another USGS report[11] and implementation of the cheapest alternative was assumed.

Greenhouse Gas Emissions Reduction – The carbon reduction value from the BCE was calculated by subtracting the carbon emissions emitted during construction from the total benefits of decreased energy use in lifetime maintenance for the project and the carbon sequestration as a result of tree plantings. The average value for carbon emissions utilized based on BCE research is $50/metric ton.

Flood Risk Reduction – The flood risk reduction value is based on water that is retained by water harvesting basins. The BCE [12] models rainfall in Tucson (nearest city included in BCE tool to Sierra Vista) over the next 100 years to determine a rainfall model that is used to determine flood damages that are mitigated by the reduced runoff volume associated with active and passive rainwater harvesting.

Groundwater Pumping – The average energy used to source and treat one gallon of groundwater in Arizona is 0.0013 kWh.[13] The cost for 1 kWh of energy is $0.106.[14] This number was used to determine the value of water harvesting features that require no additional energy input.

Stormwater Pollution Reduction – Water harvesting basins and tree plantings provide the service of removing pollutants and heavy metals from runoff and treating them through natural filtration. There are regions in the southwest that have a stormwater utility fee that provides incentives for LID implementation to meet stormwater management needs. Property owners who implement LID have reduced utility fees as a means to incentivize LID. These fee reductions for LID represent the best local approximation of the economic benefits of stormwater pollution reduction from LID. Costs are based on Oro Valley's fee structure of $2.90 per equivalent residential unit. [15]

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10 Bureau of Reclamation, 2007, Appraisal report—Augmentation alternatives for the Sierra Vista sub-watershed, Arizona—Lower Colorado Region: Denver, Colo., U.S.

11 Bagstad, K.J., Semmens, Darius, Winthrop, Rob, Jaworski, Delilah, and Larson, Joel, 2012, Ecosystem services valuation to support decisionmaking on public lands—A case study of the San Pedro River watershed, Arizona: U.S. Geological Survey Scientific Investigations Report 2012–5251, 93 p.

12 Business Case Evaluator. http://www.impactinfrastructure.com/businesscaseevaluator/

13 Energy Costs of Water (ECW) – U.S. Units: http://www.harvestingrainwater.com/water-energy-carbon-nexus/#Energy Costs of Water

14 Cost adjusted for inflation. http://www.swenergy.org/publications/factsheets/az-factsheet.pdf

15 Oro Valley Storm Water Utility Service Fee Proposal: https://wrrc.arizona.edu/publications/water-

Urban Heat Island – The urban heat island effect occurs in urban areas where temperature is often higher than that of surrounding rural areas. This is due to the density of impervious surfaces and lack of trees in urban areas, which allows heat to be stored and slowly released, keeping the surrounding air hotter for longer. This value was calculated based on the mitigation of deaths associated with heat stress related illnesses as calculated in the BCE. A benefit number can be calculated to express the value of LID in the urban landscape based on the estimated value of a statistical life.[16]

Traffic Calming – Traffic calming techniques such as roundabouts, curb extensions and changes to road environment (such as trees and shrubs) have been shown to reduce the frequency and severity of accidents.[17] Following the procedure used by the Autocase tool,[18] published Arizona crash rates and associated costs were combined with annual average daily traffic values for Sierra Vista to estimate the potential benefit.

Results for the City of Sierra Vista

Costs and benefits for LID features were analyzed within the Woodcutter Wash with a conservative 10% adoption scenario over a 10 to 20 year timeframe. This watershed was utilized as an example for what the potential for LID would be throughout the entire City of Sierra Vista. The area was selected based on input from City staff. Results were scaled up to expand across the entire city based on the relative portion of the Woodcutter Wash area to the area of the city, a factor of 5.4. Net present values (NPV) are calculated to account for costs and benefits that occur over the 40 year life of the LID feature in order to completely assess their net value.

Table 711. City of Sierra Vista LID cost benefit results

Scenario	On-site Rain Garden	Residential ROW Rain Garden	Non-Residential ROW Rain Gardens
LID Area (Sq Ft)	1,057,653	453,539	808,402
# features	5,288	630	8,084
NPV Costs	($8,757,098)	($6,885,247)	($11,513,404)
NPV Benefits	$58,709,284	$20,790,870	$67,899,965
Benefit/Cost Ratio	$6.70	$3.02	$5.90
Total Net Benefit	**$49,952,186**	**$13,905,622**	**$56,386,561**
Annualized Net Benefit	$1,248,805	$347,641	$1,409,664

On-site rain gardens are utilized on residential or commercial property where there is existing landscape area. Residential ROW rain gardens are located in rights-of-ways on residential streets and feature curb cuts to allow stormwater flowing along the curb to enter the landscape. Non-residential ROW rain gardens are located adjacent to arterial and connector streets and utilize curb cuts.

harvesting/oro-valley-storm-water-utility-service-fee-proposal

16 Impact Infrastructure, Stantec, 2014. Evaluation of GI/LID Benefits in the Pima County Environment, p. 29.

17 Retting, R.A., S.A. Ferguson, & A.T. McCartt (2003). A Review of Evidence-Based Traffic Engineering Measures Designed to Reduce Pedestrian-Motor Vehicle Crashes. American Journal of Public Health, 93:9.

18 Impact Infrastructure , LLC & Stantec (2014). AutoCASE Beta Testing Project Evaluation of GI/LID in Pima County Environment.

Appendix D – Sierra Vista Development Code

A review of the City of Sierra Vista's Development Code highlighted several opportunities to better promote adoption of LID features and reduce barriers to effective LID implementation. A summary of both short-term and longer-term issues which need to be addressed are listed as follows. Detailed edits of the following sections can be provided upon request.

- Section 151.04 General Regulations.
    - 151.04.010 – Overly restrictive clear-vision distances are set for both street intersections and driveways which impede the ability to plant shade trees along roadways. Trees along roadways are critical for myriad benefits including traffic calming effects, shading of asphalt to both reduce heat island effects and prolong lifespan of asphalt, and to improve air quality and finally to improve overall community aesthetics.
    - 151.04.015 – Suggested language to ensure that all roof and foundation drains be discharged to landscaped areas rather than storm drains to provide an irrigation benefit
- Section 151.08 Public Improvement Standards.
    - Section 151.08.003.L – Include language to allow use of curb inlets to landscape areas and provide city-approved specifications.
    - 151.08.004 – Existing road width requirements are excessive and restrict right-of-way width for integrating LID features to manage stormwater. In addition the wide roadways promote heat absorption leading to greater heat-island effects and generate greater quantities of stormwater impacting stormwater infrastructure and downstream channel health.
    - Section 151.08.004 – To enable opportunities for LID features along principal and minor arterial and collector rights-of-way opportunities exist to adjust the typical roadway profiles. For example, a 10-ft shared-use path could be maintained on one side of the roadway only. In addition the shared use path could be shifted further to the side and overlay the utilities to allow vegetated LID features to buffer pathway users from the roadway while also handling roadway runoff. The roadway still maintains two, three foot bike lanes on either side of street. Residential connector streets could have a travel width reduction to 11feet (accepted in other municipalities) which opens up 2 feet additional for LID features. Utility corridors should be shifted to coincide with the sidewalk or roadway areas to ensure functional vegetative LID features can be implemented.
    - Section 151.08.004.A.6 – Cul-de-sacs hinder neighborhood connectivity and typically provide only a vast expanse of asphalt baking in the sun. If a cul-de-sac is to be used it could include an inset LID water harvesting feature.
    - Section 151.08.008.C – It is recommended that to aid in the review of LID features proposed for the development that additional summary information is included in the required drainage report. These additional items include: informational pairing of collection hardscape areas with receiving landscape areas, ratio of catchment to landscape area (recommended to not be less than 2:1), and flow direction arrows of all impervious surfaces in relation to proposed receiving landscape LID practices.

- Section 151.08.008.E – Inserting language to ensure that grading of commercial and residential lots direct runoff from impervious surfaces to landscape LID features prior to discharging to public right-of-way or other constructed stormwater infrastructure.
- Section 151.08.008.E (*continued*) – Inserting language to ensure that streets be designed so that runoff in the public right-of-way enters a landscaped LID feature prior to discharge to a storm drain or collection system.
- Section 151.08.008.E (*continued*) – Preference should be stated to accept geomorphic-based channel features (i.e. natural channel design features) made from local rock and aggregate material, wherever possible, to allow for streambed infiltration and bank storage. Channel bed slopes should mimic existing grates of reference stable local streams. Preference for armoring that has a natural appearance and utilizes vegetation for long-term stabilization should be made wherever possible. Lastly, all grade-control structures shall be designed to promote incidental recharge in natural channels when possible.
- Section 151.08.008.E (*continued*) – Storm water basins development should include retention of the site's first flush (0.5 inches of rainfall) within distributed landscaped areas of the development. Larger retention or detention basins are to include landscaping plants (irrigated by stormwater runoff). See Pima County's Guidance *Design Standards for Stormwater Detention and Retention*.[19]
- Section 151.08.008.E (*continued*) – Remove restrictions for use of harvested rooftop rainwater from code. Rooftop rainwater is viable for a variety of site uses including drinking water.

- Section 151.09 Off-Street Parking and Loading
  - Section 151.09.005.A – Include wording to ensure that every parking lot is graded to drain first to an adjacent landscaped area prior to draining to a stormwater conveyance feature and to allow permeable pavement within parking stalls if needed to meet stormwater management retention goals.
  - Section 151.09.005.C – An incentive/waiver to avoid an oil and grease separator may be worked into this paragraph if stormwater from the parking stalls is directed to a distributed landscaped bio-retention basin.
  - Section 151.09.005 (adding a point "S") – To avoid redundancy of parking lots and foster best economic use of urban land it is suggested to add language to encourage shared use and/or multiple uses of parking areas.
- Section 151.15 Landscaping, Walls, Screening and Buffering
  - Section 151.15.004.A.1 – To promote sustainable, healthy landscaping it is suggested to insert the allowance of organic mulch as a surface material. Organic mulch is less costly both financially and environmentally than rock mulch and assists with soil health and infiltration.
  - Section 151.15.004.A.4 – Again, include a statement that landscaping shall promote water harvesting by receiving stormwater runoff from adjacent impervious surfaces.

[19] Pima County Flood Control, Feb 2015. Design Standards for Stormwater Detention and Retention. http://webcms.pima.gov/cms/one.aspx?portalId=169&pageId=65527

This water can then irrigate associated trees. Tree spacing should be reduced to 25ft to enable greater tree density and received benefit from cooling and shading.

Section 151.15.004.A.8 – Parking planters should not be raised and curbed. This defeats the opportunity for trees and vegetation to be irrigated by stormwater flowing off the hardscapes.

Appendix E – A Water Balance Scenario for Sierra Vista Sub-watershed

The following assumptions were made in the water balance model:

- The Sierra Vista Sub-watershed water supply was based on values obtained from: http://www.sierravistaaz.gov/water/content.php?fDD=22-247
- The enhanced urban stream recharge was based on the 2014 value of 2300 AF/Y. Increases are assumed to come from future channel enhancements that promote recharge in the City of Sierra Vista (1.7% of the total stream length in the Sierra Vista Sub-watershed based on streams derived from a DEM that resulted in 23miles out of 1,323 miles total). It was assumed that the average hydraulic conductivity rate would increase 0.45 inch per hour (from 0.12 in/hr to 0.57 in/hr) due to geomorphic channel restoration treatments.
- The population estimates was provided by Cado Daily based on a 2014 annual report by Cochise County Cooperative Extension. Source: personal communication, 13 August 2015
- Assumed 2.4 persons per home based on 2009-2013 census. Source: http://quickfacts.census.gov/qfd/states/04/0466820.html;
- Sierra Vista starting GPCD provided by http://www.sierravistaaz.gov/water/division.php?fDD=22-110;
- Other community per capita demand starting value was assumed to be the difference between total demand and calculated Sierra Vista's demand.
- Demand reduction scenarios were based on recent trends with Sierra Vista making a targeted effort to reduce demand. Demand reduction for other areas was based on recent regional trends.
- Population growth was based on recent trends for both Sierra Vista and other areas within the sub-watershed.

HYDROLOGIC MODELING
REPORT

FOR

RAINWATER HARVESTING SCENARIOS
CITY OF SIERRA VISTA, ARIZONA

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INTRODUCTION AND PURPOSE

JE Fuller/Hydrology & Geomorphology, Inc. (JE Fuller), under contract with Watershed Management Group (WMG) is hereby submitting this Report to summarize modeling done to assess the effects of rainwater harvesting scenarios formulated for the City of Sierra Vista, Arizona (City). The overall scope of this project is to inform public policy decisions regarding the implementation of rainwater harvesting and other green infrastructure/low impact development (GI/LID) requirements for future development.

The project area is within the limits of the City, and more specifically, within the Charleston Wash Watershed. The project study area is located within the following PLSS sections: Sections 34, 35, and 36 of Township 21 South, Range 20 East; Sections 29, 30, and 31 of Township 21 South, Range 21 East; and Sections 2, 3, and 10 of Township 22 South, Range 20 East of the Gila & Salt River Meridian. Refer to Figure 1: Location Map, on the next page.

The hydrologic modeling efforts detailed within this Report consisted of modifying regulatory HEC-HMS rainfall/runoff modeling prepared by the City of Sierra Vista in order to assess the impacts on flood peak hydrographs caused by implementation of conceptual GI/LID practices dispersed throughout an approximate three square mile area within the City limits. Generally speaking, the GI/LID practices consisted of retrofits to existing development areas, where conceptual rainwater harvesting basins or cisterns were added to parcels with existing residential or commercial structures, and along paved streets.

Also included in the project efforts was scenario rainfall/runoff modeling for a smaller sub-set of the HEC-HMS modeling area, using EPA SWMM 5.1, reflective of a separate set of scenarios using input data generated for the project.

It should be noted that the results presented within this Report reflect modeling performed on a broad scale, for strictly conceptual scenarios, using methodologies that deviate from standard regulatory design guidelines. The model results should not be used for design or construction of any feature, as site-specific data analyses should be performed prior to implementation of any scenario. The values presented within this Report may not be construed as absolute, but were generated to estimate percent-change resulting from the implementation of the scenarios.

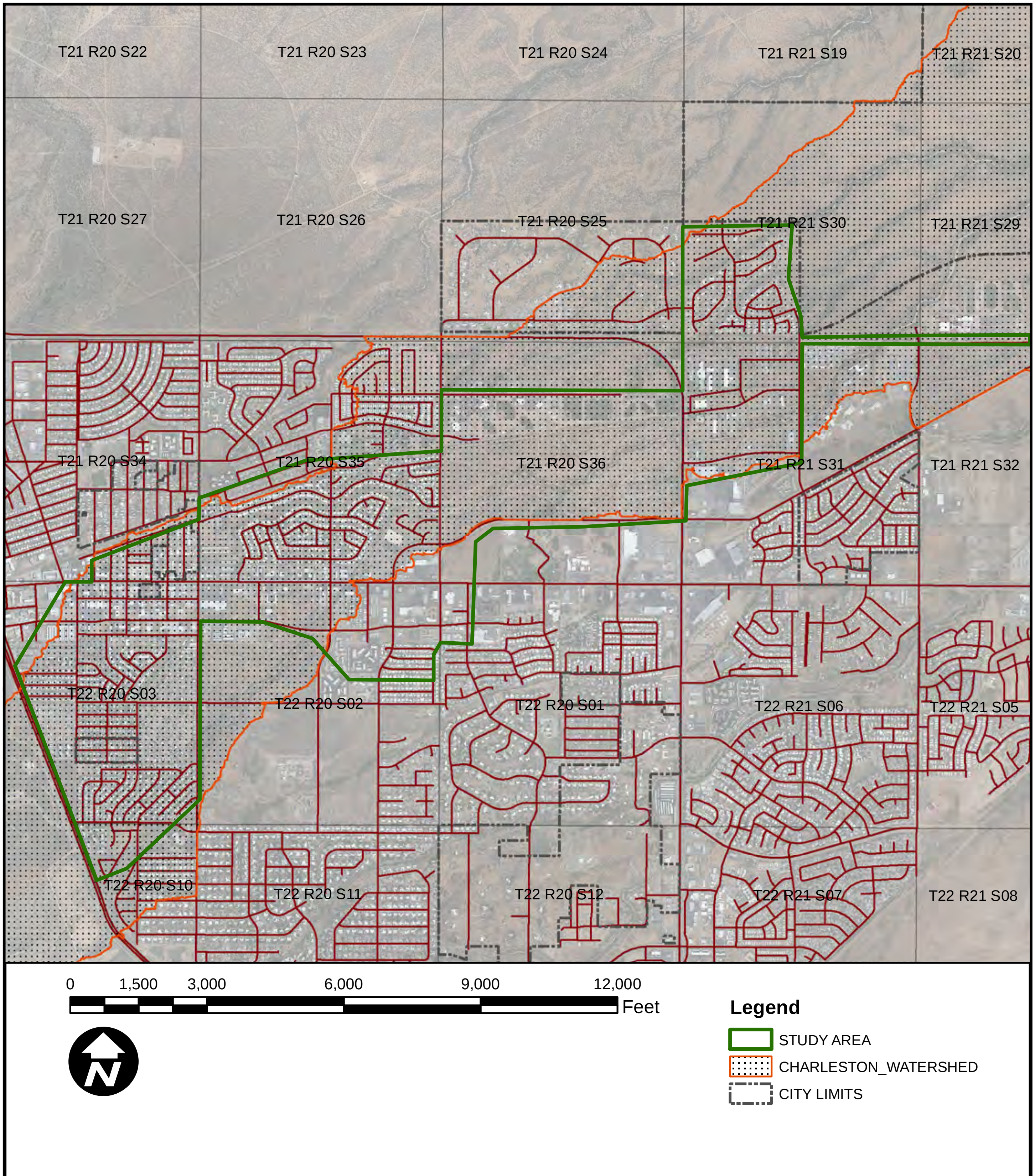

FIGURE 1
STUDY LOCATION MAP

EXISTING RAINFALL/RUNOFF HYDROLOGY

HEC-HMS Modeling

The rainfall/runoff hydrology modeling efforts began with HEC-HMS modeling prepared by the City for floodplain regulation and drainage design purposes, intended to define rainfall/runoff characteristics for selected watersheds and concentration points throughout the City, for the 1-percent chance flood (commonly referred to as the 100-year storm). The project scenario sites are located within the Charleston Wash watershed. The HEC-HMS model files were obtained in early February, 2015, and do not reflect the current regulatory models as those have been subsequently updated and revised by the City since they were obtained for this project.

One notable revision to the City HEC-HMS modeling since the beginning of this project is the addition of a subwatershed (CH-05 in the updated model) to cover an area near the Fry Boulevard and Coronado Drive intersection. This revision (among others) was made during the project as a result of JE Fuller's technical review of the City HEC-HMS model. Another notable subsequent revision to the City HEC-HMS modeling is the implementation of rainfall areal reduction factors.

The City HEC-HMS model was adapted to consider the 50 percent chance design storm (i.e. 2-year storm), maintaining the 6-hour storm duration from the City model. The 2-year 6-hour precipitation depth for the project sites and considered in the adapted HEC-HMS modeling was 1.46 inches, corresponding to the value published in NOAA Atlas 14. As the City model also considered a separate rainfall depth for the mountain subareas of the model region, the 2-year 6-hour depth input for the mountain subwatersheds was defined as 2.08 inches. The 'Pattern 3' temporal rainfall distribution was also maintained, and incremental (time step) values were generated for the revised rainfall depths. Refer to Table 1 for the rainfall data generated for this project.

As part of the project analysis efforts, the modeling examined the results of consecutive 2-year 6-hour design storms. This was performed by appending the rainfall input data with an additional set of rainfall values, lengthening the period of rainfall data input by another 6 hours.

The following table shows the hyetographs input to the model for the 2-year 6-hour design storm:

Table 1
HEC-HMS Tabular Hyetographs

	City Subwatersheds		Mountain Subwatersheds	
	2-Year	100-Year	**2-Year**	100-Year
Time (hrs.)	**Incremental Depth (in.)**	Incremental Depth (in.)	**Incremental Depth (in.)**	Incremental Depth (in.)
0.00	**0.00**	0.00	**0.00**	0.00
0.25	**0.02**	0.05	**0.03**	0.07
0.50	**0.01**	0.02	**0.01**	0.02
0.75	**0.01**	0.03	**0.02**	0.05
1.00	**0.03**	0.06	**0.04**	0.09
1.25	**0.02**	0.05	**0.03**	0.07
1.50	**0.02**	0.04	**0.03**	0.06
1.75	**0.02**	0.05	**0.03**	0.07
2.00	**0.02**	0.05	**0.03**	0.07
2.25	**0.02**	0.05	**0.03**	0.07
2.50	**0.02**	0.05	**0.03**	0.08
2.75	**0.02**	0.06	**0.04**	0.08
3.00	**0.03**	0.08	**0.05**	0.11
3.25	**0.07**	0.16	**0.10**	0.23
3.50	**0.12**	0.27	**0.17**	0.40
3.75	**0.25**	0.56	**0.35**	0.82
4.00	**0.29**	0.66	**0.41**	0.97
4.25	**0.18**	0.42	**0.26**	0.62
4.50	**0.11**	0.24	**0.15**	0.35
4.75	**0.06**	0.15	**0.09**	0.22
5.00	**0.05**	0.11	**0.07**	0.17
5.25	**0.02**	0.05	**0.03**	0.07
5.50	**0.02**	0.04	**0.03**	0.06
5.75	**0.02**	0.05	**0.03**	0.07
6.00	**0.02**	0.04	**0.03**	0.06

The existing conditions HEC-HMS file is named *CHARLESTON_WMG* and the existing conditions simulation run is named *2-Yr 6-Hr Pattern 3*, representing the 2-year precipitation data. The existing conditions consecutive-storm simulation run is named *2-Yr 6-Hr Cons*.

Refer to Table 2 below for tabulation of the pertinent results of the existing conditions HEC-HMS analysis. Refer to Appendix A for additional output data, and refer to Appendix B for the HEC-HMS files.

EPA SWMM 5.1 Modeling

Following the HEC-HMS modeling of proposed GI/LID features on a City-wide basis (discussed below), the focus of the hydrologic modeling turned to examining similar placements on a smaller area, and modeling those using Storm Water Management Model (SWMM) version 5.1, produced by the U.S. Environmental Protection Agency (EPA).

The SWMM modeling covered an area roughly bounded by East Fry Boulevard to the south, North Canyon Drive to the west, North Railroad to the north, and the 3rd Street/Buena Number 3 Drainageway to the east, an area generally referred to as the Fry Townsite. The SWMM modeling area contains a mix of City and Cochise County land. The SWMM model area is within the boundaries of the HEC-HMS model subwatershed *Bu3ST_02*.

The SWMM modeling area was split along North 7th Street, and 2 model sets were developed for the areas east of North 7th Street and west of North 7th Street. For those two separate regions, existing conditions models were developed, and then modified to reflect the implementation of GI/LID features, based on concept designs provided by WMG (discussed below).

Using a shapefile dataset of the parcels located within the study area, each parcel was assigned a unique number identifier to facilitate SWMM model data input. Approximately 180 parcels were input into SWMM as individual subcatchments (named S* with * representing the unique identifier). It was assumed that each parcel discharges to one point. As the streets, alleys, and other inter-parcel areas generally convey much of the runoff from the parcels, these areas were also input into SWMM as individual subcatchments (named R-*). Generally speaking, the runoff from the parcels was conveyed to the street subcatchments, which were then routed downstream to other street subcatchments and ultimately to junctions. Junctions in the SWMM models were used to combine runoff flows. Conduits were used to convey the runoff collected at junctions to the downstream junction when applicable.

Refer to Tables 3A and 3B below for tabulation of the pertinent results of the existing conditions SWMM analysis. Refer to Appendix A for additional output data, and refer to Appendix B for the SWMM files.

WITH-SCENARIO HYDROLOGIC MODELING

HEC-HMS Modeling

The adapted City HEC-HMS model was revised to consider implementation of the GI/LID scenario data provided by WMG in January, 2015. The data provided by WMG was delivered to JE Fuller in the form of a polygon shapefile, delineating the following features: parcels intended to receive stormwater harvesting basins; and, individual street-side basin footprints. These polygons were intersected with the subwatershed polygons from the HEC-HMS modeling, to arrive at a total area of GI/LID footprint in each subwatershed proposed to receive the features. Not all subwatersheds in the model were proposed to receive GI/LID features.

For each HEC-HMS subwatershed proposed to receive GI/LID practices, the sum total areas of the feature polygons within each subwatershed were subtracted from the original subwatershed area, a separate subwatershed was created (named with a *_BMP suffix), assigned the area of the sum of the feature polygon areas within the original subwatershed retaining the original subwatershed input parameters, and the runoff from the created *_BMP subwatershed was conveyed to the downstream point of the original subwatershed such that the hydrographs combine at the original point in the model. Deviations from the original subwatershed input parameters for the created subwatersheds were as follows: ‘Area’ was defined as described above, ‘Max Storage (IN)’ was set to 3.6 (inches), and ‘Impervious (%)’ was set to 0. Essentially, this method calculates the hydrographs from the GI/LID features along with the runoff from the subwatersheds in which the features are located, then combines runoff hydrographs from the two.

The modeling approach described above is represented by the HEC-HMS file *CHARLESTON_BMP*, and the scenario run names match the original model names. One aspect of this modeling approach that did not align with the intent of the WMG scenario layout is that for the parcels selected for GI/LID treatment, the entire parcel area was considered to have the depression depth of 3.6 inches or 0.3 feet. With this modeling approach, cisterns for the impervious areas would be needed, and sized to retain at least 0.3’ of rainfall from the contributory drainage area, and the entire parcel would require the ability to retain 0.3’ of runoff.

In an attempt to better approximate the intent of the WMG scenario designs, additional input data was provided by WMG, and various depths associated with each City zoning classification were assigned to the feature polygons and the with-scenario input data creation process was repeated. The created subwatersheds (with *_BMP) retained the original areas that were assigned, but the input parameter ‘Max Storage (IN)’ was set to the results of averaging the depths across the subwatershed (net reduction in available surface storage). The HEC-HMS file *CHARLESTON_BMP2* reflects the revised with-scenario modeling relating to this refined approach.

Refer to Table 2 below for tabulation of the pertinent results of the with-scenario conditions HEC-HMS analysis. The results of the refined with-scenario modeling approach matched the results of the first for the 2-year 6-hour storm; the values in the table below reflect both sets of results. Refer to Appendix A for additional output data, and refer to Appendix B for the HEC-HMS files.

Table 2
Summary Table for Scenario Modeling With HEC-HMS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	BMP DRAINAGE AREA (MI.^2)	**% AREA REDUCTION**	**% PEAK FLOW REDUCTION**	**% VOLUME REDUCTION**
B3N_09	0.6238	0.00023	**0.0%**	**0.0%**	**0.0%**
Bu3ST_01	0.1608	0.00918	**5.4%**	**5.4%**	**5.5%**
Bu3ST_02	0.1769	0.00515	**2.8%**	**2.8%**	**3.1%**
Bu3ST_03	0.2369	0.03108	**11.6%**	**11.6%**	**11.4%**
Bu3ST_04	0.2692	0.01082	**3.9%**	**4.0%**	**2.9%**
CH_01	0.3551	0.00389	**1.1%**	**1.2%**	**1.5%**
CH_02	0.2943	0.00173	**0.6%**	**0.5%**	**0.0%**
CH_03	0.202	0.00304	**1.5%**	**1.3%**	**0.0%**
CH_04	0.2676	0.0024	**0.9%**	**0.6%**	**3.7%**
CTY_01	0.6964	0.0036	**0.5%**	**0.6%**	**0.0%**
RCE_01	0.8203	0.00872	**1.1%**	**1.1%**	**1.1%**
WC_01	0.1139	0.00513	**4.3%**	**4.2%**	**4.3%**
WC_02	0.1312	0.02185	**14.3%**	**14.3%**	**15.7%**
WC_04	0.3439	0.00705	**2.0%**	**2.0%**	**1.9%**

While the modeling approaches used to reflect the GI/LID practices are approximations of the physical processes involved in the scenarios and their impacts on the rainfall/runoff characteristics of the subwatersheds, they are not exact methods to evaluate the placement of any feature or permit any construction. Site-level study would be required to assess the effects of any proposed feature.

EPA SWMM 5.1 Modeling

The recent modifications to SWMM allow for direct input of GI/LID feature elements as part of the subcatchment input data set, and therefore facilitate higher accuracy and greater confidence in the results of the modeling. The GI/LID designs provided by WMG consisted of the following: shallow retention basins, rainwater harvesting cisterns, or a combination of the two. The map/table in Appendix A shows the designs provided to JE Fuller by WMG.

Refer to Tables 3A and 3B below for tabulation of the pertinent results of the with-scenario conditions SWMM analysis. Refer to Appendix A for additional output data, and refer to Appendix B for the SWMM files.

Table 3A
Summary Table for Scenario Modeling With SWMM
East of North 7th Avenue

	WITH-SCENARIO			EXISTING		
	J1	J2	J3	J1	J2	J3
	Total Inflow POST	Total Inflow POST	Total Inflow POST	Total Inflow PRE	Total Inflow PRE	Total Inflow PRE
Hours	(CFS)	(CFS)	(CFS)	(CFS)	(CFS)	(CFS)
0:15:00	0	0	0	0	0	0
0:30:00	0	0.11	0	0.15	0.11	0.13
0:45:00	0	0.14	0	0.14	0.14	0.12
1:00:00	0	0.23	0	0.2	0.23	0.17
1:15:00	0.26	0.43	0	0.36	0.43	0.31
1:30:00	0.29	0.51	0	0.37	0.51	0.32
1:45:00	0.26	0.52	0	0.34	0.52	0.3
2:00:00	0.27	0.54	0	0.35	0.54	0.3
2:15:00	0.29	0.57	0	0.37	0.57	0.32
2:30:00	0.28	0.57	0	0.36	0.57	0.31
2:45:00	0.31	0.61	0	0.39	0.61	0.33
3:00:00	0.34	0.65	0	0.41	0.65	0.36
3:15:00	0.45	0.79	0	0.53	0.79	0.46
3:30:00	0.93	1.42	0.38	1.03	1.42	0.9
3:45:00	1.81	2.67	0.7	1.9	2.67	1.66
4:00:00	3.97	5.82	1.47	4.07	5.82	3.5
4:15:00	5.08	7.64	1.77	5.15	7.64	4.26
4:30:00	3.46	5.62	1.16	3.53	5.62	2.83
4:45:00	2.03	3.58	0.69	2.1	3.58	1.69
5:00:00	1.22	2.33	0.43	1.29	2.33	1.06
5:15:00	0.88	1.71	0.33	0.94	1.71	0.8
5:30:00	0.45	1.04	0.17	0.5	1.04	0.43
5:45:00	0.32	0.77	0.13	0.38	0.77	0.32
6:00:00	0.31	0.67	0.13	0.36	0.67	0.31

Table 3B
Summary Table for Scenario Modeling With SWMM
West of North 7th Avenue

	WITH-SCENARIO			EXISTING		
	J1	J2	J3	J1	J2	J3
	Total Inflow POST	Total Inflow POST	Total Inflow POST	Total Inflow PRE	Total Inflow PRE	Total Inflow PRE
Hours	(CFS)	(CFS)	(CFS)	(CFS)	(CFS)	(CFS)
0:15:00	0	0	0	0	0	0
0:30:00	0.02	0.02	0.01	0.02	0.02	0.01
0:45:00	0.06	0.1	0.03	0.08	0.11	0.03
1:00:00	0.13	0.23	0.06	0.2	0.28	0.08
1:15:00	0.3	0.52	0.13	0.45	0.67	0.18
1:30:00	0.53	0.99	0.22	0.8	1.33	0.31
1:45:00	0.71	1.39	0.29	1.07	1.88	0.39
2:00:00	0.85	1.63	0.32	1.22	2.17	0.43
2:15:00	0.97	1.81	0.34	1.31	2.33	0.45
2:30:00	1.04	1.93	0.35	1.36	2.43	0.46
2:45:00	1.09	2.02	0.36	1.41	2.5	0.47
3:00:00	1.15	2.12	0.38	1.48	2.61	0.49
3:15:00	1.28	2.34	0.42	1.64	2.86	0.55
3:30:00	1.78	3.16	0.6	2.28	3.85	0.77
3:45:00	3.2	5.7	1.08	4.08	6.95	1.41
4:00:00	6.98	12.65	2.42	8.86	15.3	3.05
4:15:00	12.46	23.11	4.47	15.17	27.26	5.27
4:30:00	13.97	25.68	4.79	15.93	29.25	5.43
4:45:00	10.73	19.65	3.47	11.94	21.84	3.89
5:00:00	7.23	13.27	2.3	8.01	14.59	2.53
5:15:00	4.98	9.09	1.53	5.53	10.02	1.73
5:30:00	3.45	6.33	1.02	3.84	7.03	1.19
5:45:00	2.39	4.35	0.69	2.66	4.85	0.82
6:00:00	1.78	3.22	0.51	2.01	3.62	0.61
6:15:00	1.45	2.62	0.42	1.67	2.98	0.51
6:30:00	1.12	2.1	0.32	1.31	2.42	0.4
6:45:00	0.8	1.53	0.23	0.94	1.77	0.29
7:00:00	0.57	1.08	0.16	0.66	1.25	0.2
7:15:00	0.41	0.77	0.11	0.47	0.89	0.14
7:30:00	0.3	0.57	0.08	0.35	0.65	0.1
7:45:00	0.22	0.43	0.06	0.26	0.48	0.07
8:00:00	0.17	0.33	0.05	0.2	0.37	0.05

Similar to the HEC-HMS modeling, the SWMM modeling approach used to reflect the GI/LID practices is an approximation of the physical processes involved in the scenarios and their impacts on the rainfall/runoff characteristics of the subwatersheds, it is not an exact method to evaluate the placement of any feature or permit any construction. Site-level study would be required to assess the effects of any proposed feature.

RESULTS AND DISCUSSION

This Memo serves to summarize the results of rainfall/runoff hydrology modeling done to reflect conceptual implementation of green infrastructure/low impact development (GI/LID) practices within the City of Sierra Vista, Arizona. The modeling performed was generally for a wide area, and as a result, simplifying assumptions were made to arrive at the results. The HEC-HMS modeling approach attempted to simulate the effects of GI/LID features distributed throughout the subwatersheds being modeled using lumped-parameter model input. The SWMM modeling allowed for greater precision as the individual feature geometry can be input on a parcel basis, and the results of the GI/LID practices can be inspected on in greater detail. Refer to Appendix B for the digital input files associated with each model.

While this Report is not meant to expound the benefits of GI/LID practices, selected benefits can be presumed from the scenarios modeled as part of this project, as follows: increased vegetation canopy; reduced cooling costs for structures and vehicles due to the additional vegetation canopy; reduced groundwater pumping for landscape irrigation; and, reductions to urban runoff and therefore to nuisance flooding.

JE Fuller is pleased to present this Report to Watershed Management Group, and eagerly anticipates assisting WMG with further investigations into the potential effects of implementation of GI/LID practices in the City Sierra Vista and elsewhere.

REFERENCES

1. NOAA Atlas 14, Precipitation Frequency Atlas of the United States; U.S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA), National Weather Service; 2004
2. Soil Survey of Cochise County, Arizona, Douglas-Tombstone Part (AZ 671); Natural Resources Conservation Service (NRCS), 2008

FIGURES

Legend

WMG GI/LID Features

Charleston Subwatersheds

0 500 1,000 2,000 3,000 4,000 Feet

APPROXIMATE SCALE: 1" = 1,000 FT.

FIGURE 2
WATERSHED NETWORK MAP
FOR HEC-HMS MODELING

0 50 100 200 300 400
Feet
APPROXIMATE SCALE: 1" = 100 FT.

FIGURE 3
WATERSHED NETWORK MAP
FOR SWMM MODELING

APPENDICES

Supporting Information, Calculations and Data

APPENDIX A
Hydrology Data/Calculations

NOAA Atlas 14, Volume 1, Version 5
Location name: Sierra Vista, Arizona, US*
Latitude: 31.5553°, Longitude: -110.2847°
Elevation: 4578 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.272 (0.240–0.309)	0.349 (0.307–0.397)	0.455 (0.398–0.516)	0.533 (0.466–0.604)	0.639 (0.554–0.725)	0.719 (0.618–0.818)	0.800 (0.679–0.914)	0.881 (0.739–1.01)	0.988 (0.812–1.15)	1.07 (0.865–1.25)
10-min	0.414 (0.365–0.471)	0.531 (0.467–0.604)	0.692 (0.607–0.786)	0.811 (0.710–0.920)	0.972 (0.844–1.10)	1.10 (0.941–1.25)	1.22 (1.03–1.39)	1.34 (1.13–1.54)	1.50 (1.24–1.74)	1.63 (1.32–1.91)
15-min	0.513 (0.453–0.584)	0.658 (0.579–0.749)	0.858 (0.752–0.974)	1.01 (0.880–1.14)	1.21 (1.05–1.37)	1.36 (1.17–1.54)	1.51 (1.28–1.73)	1.66 (1.40–1.91)	1.86 (1.53–2.16)	2.02 (1.63–2.36)
30-min	0.691 (0.609–0.787)	0.886 (0.780–1.01)	1.16 (1.01–1.31)	1.36 (1.19–1.54)	1.62 (1.41–1.84)	1.83 (1.57–2.08)	2.03 (1.73–2.32)	2.24 (1.88–2.57)	2.51 (2.06–2.91)	2.72 (2.20–3.18)
60-min	0.855 (0.754–0.974)	1.10 (0.966–1.25)	1.43 (1.25–1.62)	1.68 (1.47–1.90)	2.01 (1.74–2.28)	2.26 (1.94–2.57)	2.52 (2.14–2.88)	2.77 (2.33–3.18)	3.11 (2.55–3.60)	3.36 (2.72–3.94)
2-hr	0.975 (0.864–1.11)	1.24 (1.10–1.40)	1.60 (1.41–1.80)	1.88 (1.65–2.11)	2.26 (1.98–2.55)	2.57 (2.22–2.90)	2.89 (2.47–3.28)	3.21 (2.70–3.68)	3.66 (3.01–4.23)	4.02 (3.25–4.68)
3-hr	1.03 (0.912–1.16)	1.29 (1.15–1.46)	1.65 (1.46–1.86)	1.94 (1.71–2.18)	2.34 (2.05–2.63)	2.66 (2.30–2.99)	2.99 (2.56–3.39)	3.35 (2.82–3.82)	3.83 (3.15–4.43)	4.23 (3.40–4.94)
6-hr	1.17 (1.04–1.33)	1.46 (1.30–1.66)	1.85 (1.64–2.09)	2.17 (1.92–2.45)	2.63 (2.30–2.96)	2.99 (2.59–3.38)	3.39 (2.88–3.85)	3.80 (3.18–4.34)	4.39 (3.58–5.06)	4.86 (3.89–5.68)
12-hr	1.31 (1.17–1.47)	1.64 (1.46–1.84)	2.05 (1.82–2.30)	2.39 (2.12–2.68)	2.86 (2.51–3.21)	3.24 (2.82–3.64)	3.64 (3.12–4.11)	4.06 (3.42–4.62)	4.64 (3.83–5.35)	5.12 (4.14–5.95)
24-hr	1.51 (1.41–1.63)	1.88 (1.76–2.03)	2.34 (2.18–2.52)	2.69 (2.50–2.90)	3.18 (2.94–3.43)	3.56 (3.28–3.83)	3.94 (3.62–4.25)	4.33 (3.95–4.68)	4.85 (4.38–5.40)	5.24 (4.71–6.01)
2-day	1.67 (1.56–1.80)	2.08 (1.94–2.24)	2.56 (2.39–2.76)	2.97 (2.76–3.19)	3.52 (3.25–3.77)	3.95 (3.63–4.26)	4.41 (4.02–4.75)	4.88 (4.42–5.28)	5.52 (4.93–6.03)	6.03 (5.33–6.60)
3-day	1.85 (1.72–1.99)	2.29 (2.14–2.48)	2.83 (2.64–3.05)	3.28 (3.05–3.53)	3.90 (3.60–4.19)	4.39 (4.04–4.74)	4.91 (4.48–5.30)	5.44 (4.92–5.91)	6.18 (5.51–6.75)	6.76 (5.97–7.43)
4-day	2.02 (1.88–2.17)	2.51 (2.34–2.71)	3.10 (2.89–3.34)	3.59 (3.33–3.87)	4.28 (3.95–4.61)	4.84 (4.44–5.22)	5.41 (4.93–5.85)	6.01 (5.42–6.53)	6.84 (6.09–7.48)	7.50 (6.61–8.26)
7-day	2.44 (2.26–2.64)	3.04 (2.81–3.30)	3.77 (3.49–4.08)	4.36 (4.03–4.72)	5.16 (4.74–5.58)	5.80 (5.30–6.26)	6.44 (5.86–6.98)	7.11 (6.40–7.72)	8.01 (7.13–8.74)	8.71 (7.69–9.58)
10-day	2.81 (2.61–3.03)	3.49 (3.25–3.77)	4.32 (4.02–4.65)	4.96 (4.61–5.34)	5.82 (5.37–6.26)	6.47 (5.95–6.97)	7.13 (6.52–7.70)	7.79 (7.08–8.43)	8.67 (7.79–9.43)	9.34 (8.33–10.2)
20-day	3.89 (3.60–4.20)	4.85 (4.50–5.25)	5.94 (5.50–6.41)	6.75 (6.25–7.29)	7.80 (7.20–8.41)	8.57 (7.90–9.25)	9.33 (8.54–10.1)	10.1 (9.18–10.9)	11.0 (9.95–12.0)	11.6 (10.5–12.7)
30-day	4.82 (4.46–5.20)	6.00 (5.56–6.48)	7.28 (6.75–7.86)	8.23 (7.63–8.88)	9.40 (8.69–10.1)	10.2 (9.43–11.0)	11.0 (10.2–11.9)	11.8 (10.8–12.8)	12.7 (11.6–13.9)	13.4 (12.1–14.6)
45-day	5.88 (5.46–6.31)	7.31 (6.80–7.85)	8.79 (8.17–9.42)	9.85 (9.14–10.6)	11.1 (10.3–11.9)	12.0 (11.1–12.9)	12.9 (11.9–13.9)	13.6 (12.6–14.7)	14.6 (13.4–15.8)	15.2 (13.9–16.5)
60-day	6.80 (6.34–7.28)	8.44 (7.87–9.05)	10.1 (9.42–10.8)	11.3 (10.5–12.1)	12.7 (11.8–13.6)	13.7 (12.7–14.7)	14.6 (13.5–15.7)	15.4 (14.3–16.6)	16.4 (15.1–17.7)	17.1 (15.7–18.5)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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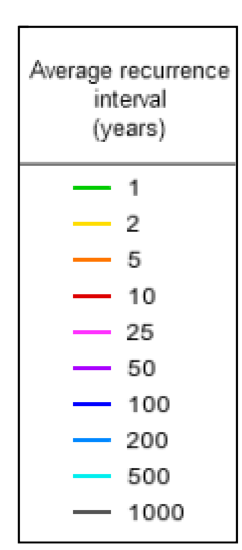
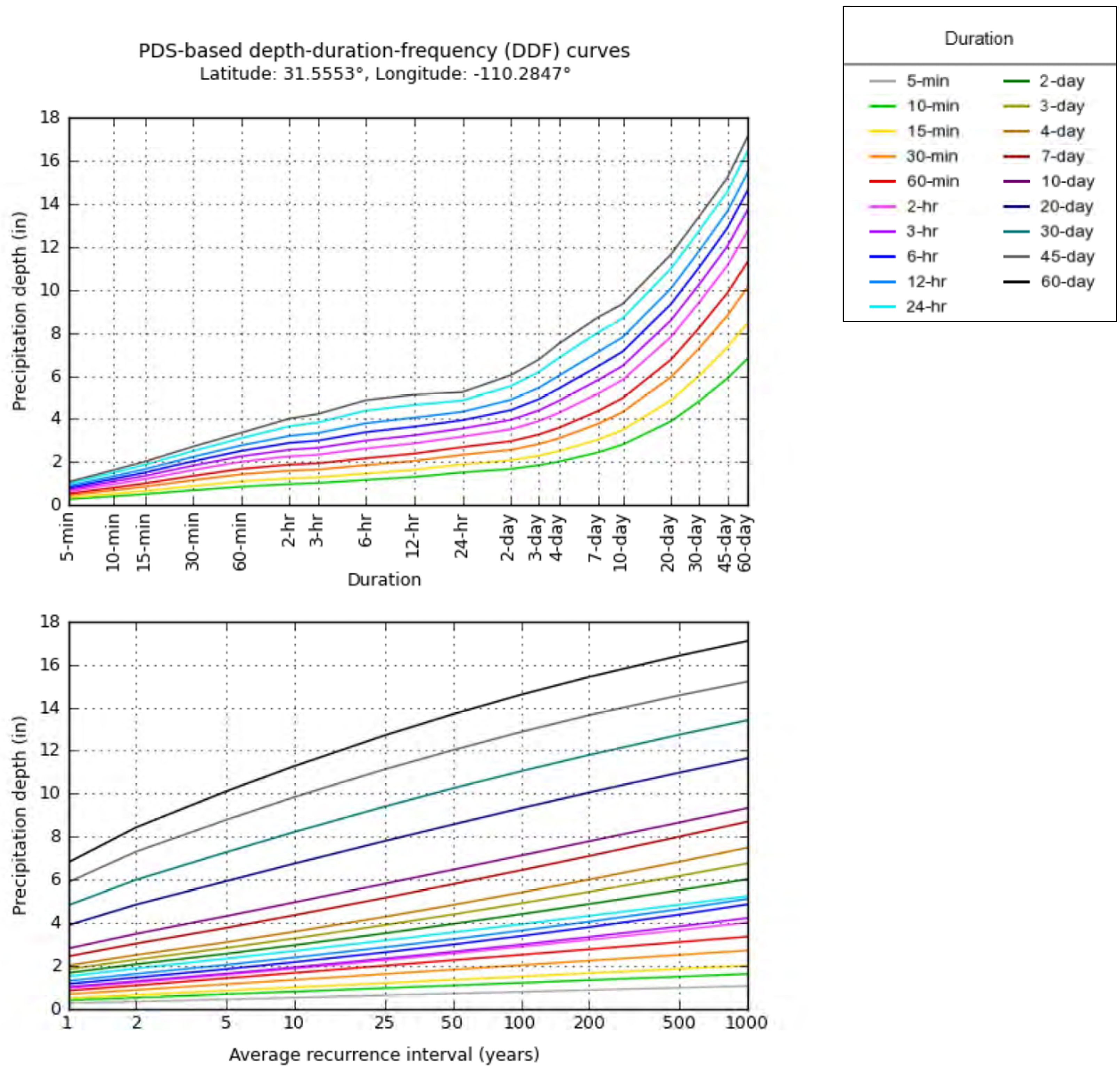
PF graphical





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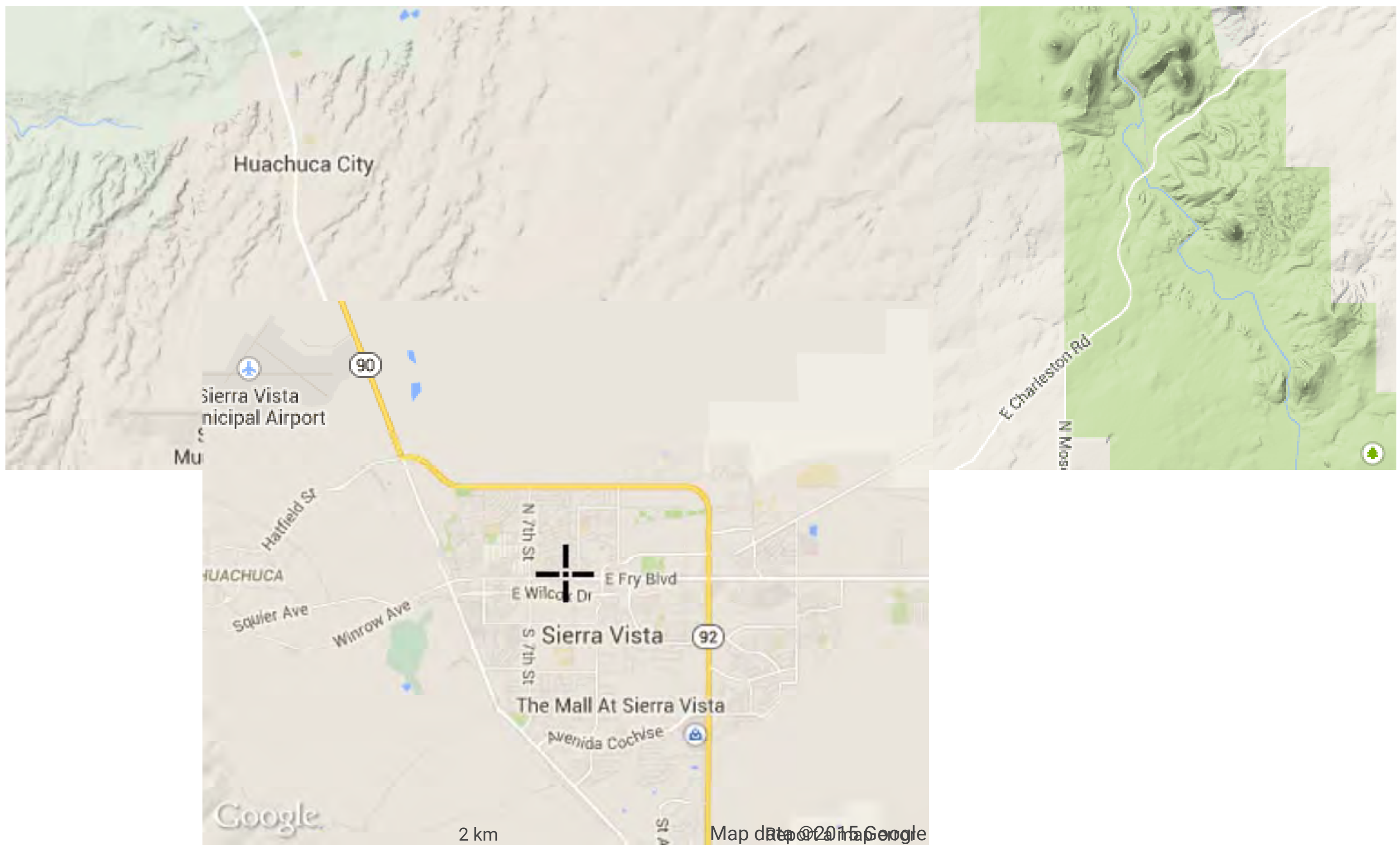
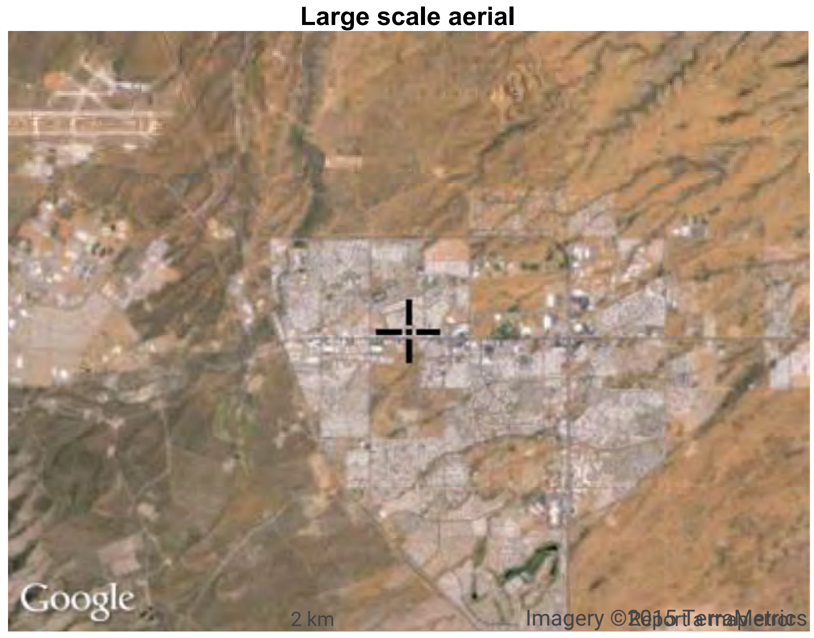
Maps & aerials

Small scale terrain

Large scale terrain



Large scale aerial



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Disclaimer

NOAA Atlas 14, Volume 1, Version 5
Location name: Hereford, Arizona, US*
Latitude: 31.4466°, Longitude: -110.3506°
Elevation: 7700 ft*
* source: Google Maps

POINT PRECIPITATION FREQUENCY ESTIMATES

Sanja Perica, Sarah Dietz, Sarah Heim, Lillian Hiner, Kazungu Maitaria, Deborah Martin, Sandra Pavlovic, Ishani Roy, Carl Trypaluk, Dale Unruh, Fenglin Yan, Michael Yekta, Tan Zhao, Geoffrey Bonnin, Daniel Brewer, Li-Chuan Chen, Tye Parzybok, John Yarchoan

NOAA, National Weather Service, Silver Spring, Maryland

PF_tabular | PF_graphical | Maps_&_aerials

PF tabular

PDS-based point precipitation frequency estimates with 90% confidence intervals (in inches)[1]

Duration	Average recurrence interval (years)									
	1	2	5	10	25	50	100	200	500	1000
5-min	0.391 (0.346-0.444)	0.502 (0.443-0.569)	0.650 (0.571-0.733)	0.755 (0.662-0.850)	0.892 (0.777-1.00)	0.990 (0.857-1.12)	1.09 (0.932-1.23)	1.18 (1.00-1.34)	1.30 (1.09-1.49)	1.39 (1.15-1.62)
10-min	0.595 (0.527-0.676)	0.764 (0.674-0.867)	0.989 (0.869-1.12)	1.15 (1.01-1.30)	1.36 (1.18-1.53)	1.51 (1.30-1.70)	1.65 (1.42-1.87)	1.79 (1.53-2.04)	1.97 (1.65-2.27)	2.11 (1.75-2.47)
15-min	0.737 (0.653-0.838)	0.948 (0.836-1.07)	1.23 (1.08-1.38)	1.43 (1.25-1.61)	1.68 (1.47-1.89)	1.87 (1.62-2.11)	2.05 (1.76-2.32)	2.22 (1.89-2.53)	2.45 (2.05-2.82)	2.62 (2.16-3.06)
30-min	0.993 (0.880-1.13)	1.28 (1.13-1.45)	1.65 (1.45-1.86)	1.92 (1.68-2.16)	2.27 (1.97-2.55)	2.52 (2.18-2.84)	2.76 (2.37-3.13)	2.99 (2.55-3.41)	3.29 (2.76-3.79)	3.53 (2.91-4.12)
60-min	1.23 (1.09-1.40)	1.58 (1.39-1.79)	2.04 (1.80-2.31)	2.38 (2.08-2.68)	2.80 (2.44-3.16)	3.11 (2.70-3.51)	3.41 (2.93-3.87)	3.71 (3.16-4.22)	4.08 (3.42-4.69)	4.37 (3.61-5.10)
2-hr	1.38 (1.22-1.56)	1.75 (1.55-1.98)	2.25 (1.98-2.53)	2.62 (2.31-2.95)	3.14 (2.75-3.53)	3.54 (3.08-3.99)	3.95 (3.39-4.47)	4.36 (3.70-4.96)	4.91 (4.09-5.63)	5.35 (4.39-6.20)
3-hr	1.46 (1.30-1.64)	1.83 (1.64-2.06)	2.32 (2.06-2.60)	2.71 (2.40-3.04)	3.25 (2.86-3.65)	3.68 (3.21-4.14)	4.12 (3.55-4.65)	4.58 (3.89-5.19)	5.19 (4.31-5.95)	5.68 (4.63-6.60)
6-hr	1.66 (1.48-1.88)	2.08 (1.85-2.35)	2.61 (2.31-2.93)	3.04 (2.69-3.42)	3.66 (3.21-4.12)	4.16 (3.60-4.68)	4.68 (4.01-5.30)	5.23 (4.40-5.94)	5.96 (4.91-6.85)	6.56 (5.29-7.61)
12-hr	1.87 (1.67-2.10)	2.34 (2.09-2.63)	2.92 (2.59-3.28)	3.40 (3.01-3.82)	4.07 (3.57-4.57)	4.61 (4.00-5.19)	5.17 (4.43-5.86)	5.76 (4.86-6.57)	6.57 (5.43-7.58)	7.20 (5.85-8.39)
24-hr	2.01 (1.85-2.19)	2.51 (2.31-2.74)	3.13 (2.87-3.42)	3.61 (3.31-3.95)	4.28 (3.90-4.68)	4.79 (4.35-5.24)	5.32 (4.81-5.92)	5.86 (5.26-6.63)	6.60 (5.85-7.65)	7.27 (6.30-8.48)
2-day	2.37 (2.17-2.59)	2.96 (2.71-3.24)	3.68 (3.37-4.03)	4.29 (3.91-4.70)	5.13 (4.65-5.62)	5.81 (5.23-6.37)	6.52 (5.83-7.17)	7.26 (6.44-8.02)	8.29 (7.25-9.21)	9.12 (7.88-10.2)
3-day	2.55 (2.34-2.79)	3.19 (2.93-3.49)	3.98 (3.65-4.36)	4.64 (4.24-5.08)	5.56 (5.05-6.09)	6.31 (5.68-6.92)	7.09 (6.35-7.79)	7.90 (7.01-8.72)	9.04 (7.91-10.0)	9.96 (8.61-11.1)
4-day	2.73 (2.52-2.99)	3.42 (3.14-3.74)	4.27 (3.92-4.68)	4.99 (4.56-5.47)	5.99 (5.45-6.57)	6.80 (6.14-7.47)	7.65 (6.86-8.42)	8.54 (7.59-9.43)	9.79 (8.57-10.9)	10.8 (9.34-12.1)
7-day	3.26 (3.00-3.55)	4.08 (3.76-4.45)	5.12 (4.70-5.57)	5.95 (5.45-6.49)	7.11 (6.47-7.75)	8.03 (7.27-8.77)	8.98 (8.08-9.84)	9.98 (8.90-11.0)	11.3 (9.98-12.5)	12.4 (10.8-13.8)
10-day	3.81 (3.52-4.11)	4.76 (4.40-5.16)	5.92 (5.47-6.41)	6.85 (6.31-7.41)	8.09 (7.41-8.76)	9.05 (8.25-9.82)	10.0 (9.09-10.9)	11.0 (9.93-12.1)	12.4 (11.0-13.6)	13.4 (11.8-14.8)
20-day	5.23 (4.84-5.65)	6.55 (6.06-7.09)	8.05 (7.44-8.71)	9.19 (8.48-9.94)	10.7 (9.79-11.5)	11.8 (10.8-12.7)	12.8 (11.7-14.0)	13.9 (12.6-15.1)	15.3 (13.7-16.7)	16.3 (14.5-17.9)
30-day	6.75 (6.25-7.28)	8.44 (7.81-9.11)	10.3 (9.52-11.1)	11.7 (10.8-12.6)	13.4 (12.4-14.5)	14.7 (13.5-15.9)	15.9 (14.6-17.3)	17.1 (15.6-18.6)	18.6 (16.8-20.3)	19.7 (17.7-21.6)
45-day	8.35 (7.74-8.97)	10.4 (9.66-11.2)	12.6 (11.7-13.6)	14.2 (13.1-15.3)	16.1 (14.9-17.4)	17.5 (16.1-19.0)	18.9 (17.3-20.5)	20.2 (18.4-22.0)	21.9 (19.8-23.9)	23.1 (20.8-25.4)
60-day	9.72 (9.03-10.4)	12.1 (11.3-13.0)	14.6 (13.6-15.7)	16.4 (15.2-17.6)	18.5 (17.1-20.0)	20.0 (18.5-21.7)	21.5 (19.8-23.3)	22.9 (20.9-24.9)	24.6 (22.4-26.9)	25.9 (23.4-28.4)

[1] Precipitation frequency (PF) estimates in this table are based on frequency analysis of partial duration series (PDS).

Numbers in parenthesis are PF estimates at lower and upper bounds of the 90% confidence interval. The probability that precipitation frequency estimates (for a given duration and average recurrence interval) will be greater than the upper bound (or less than the lower bound) is 5%. Estimates at upper bounds are not checked against probable maximum precipitation (PMP) estimates and may be higher than currently valid PMP values.

Please refer to NOAA Atlas 14 document for more information.

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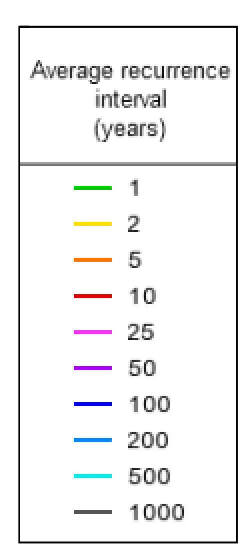
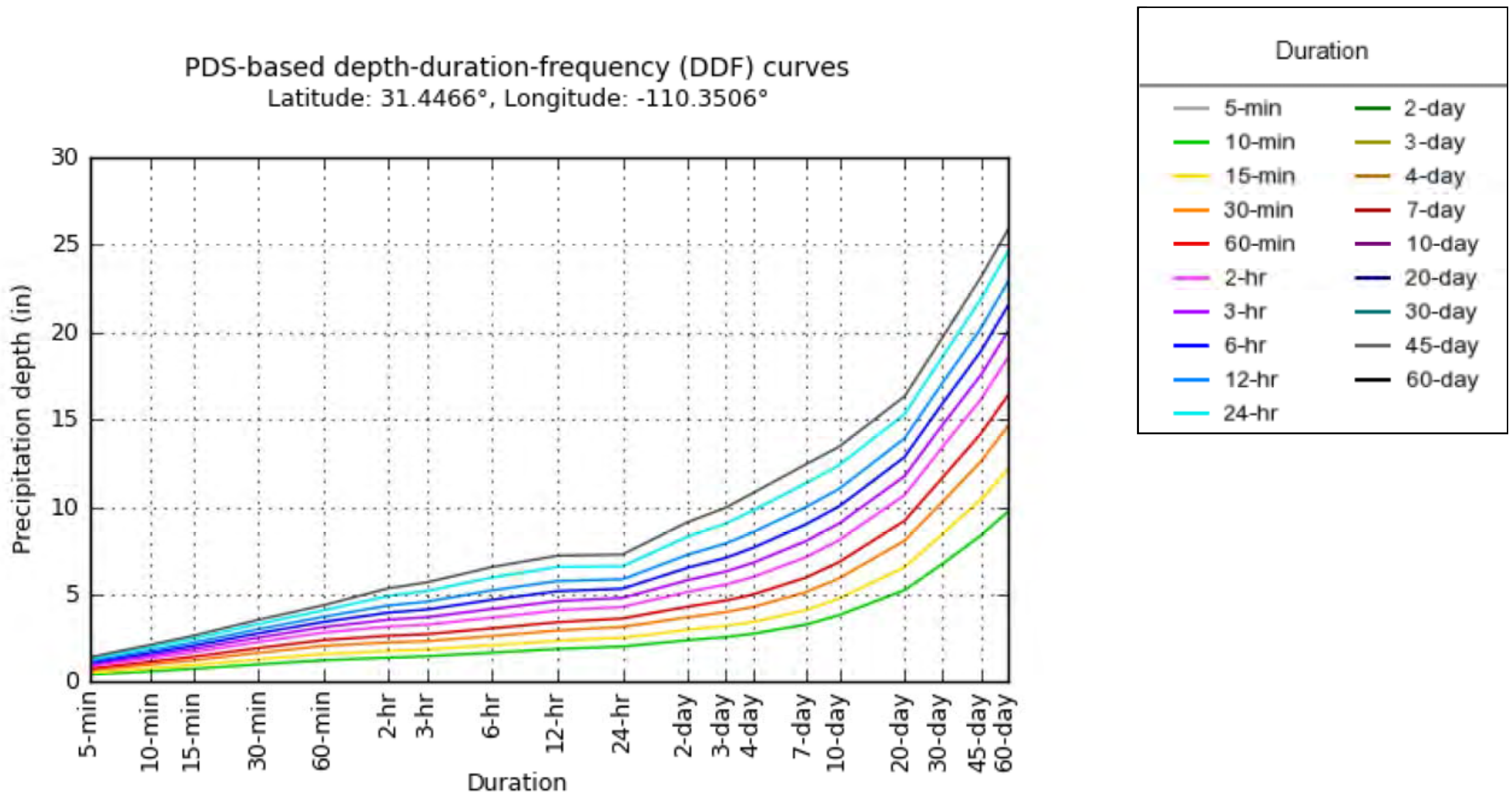
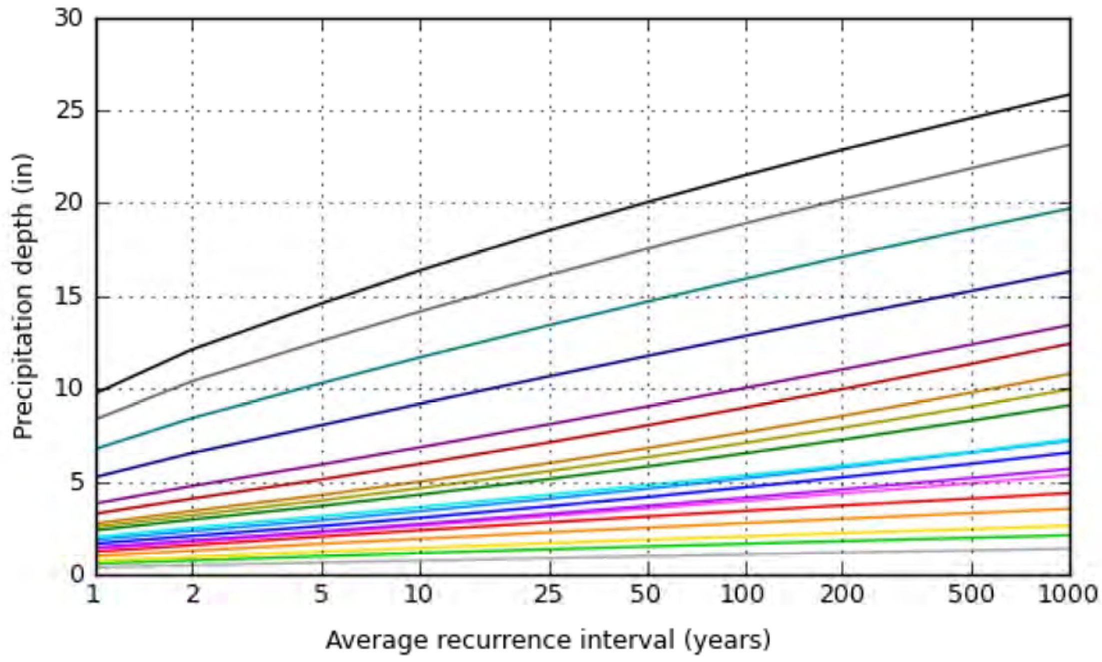
PF graphical







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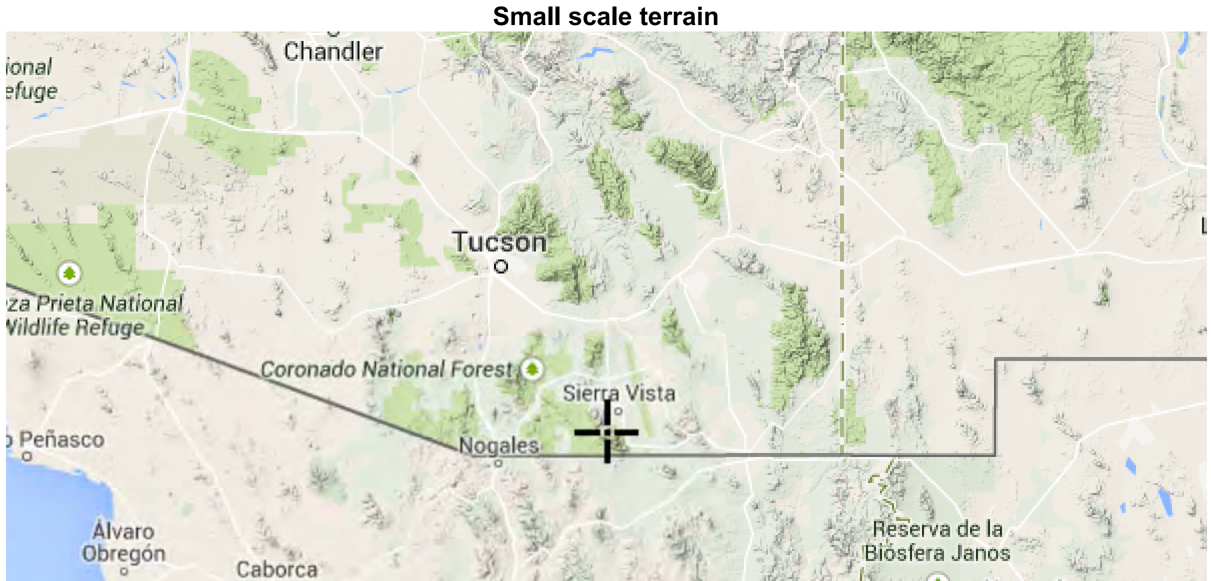
Maps & aerials

Small scale terrain



Large scale aerial

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HEC-HMS OUTPUT

Project: WMG/HYDROLOGIC MODELING REPORT-COSV
Detail: OUTPUT FROM HEC-HMS MODELING
2-YEAR 6-HOUR PRECIPITATION EVENT

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

EX CONDITIONS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	PEAK DISCHARGE (CFS)	TIME OF PEAK	VOLUME (AC.-FT.)
WC_08	1.925	223.4	01Jan2000, 06:20	48.8
WC_07	0.801	116.8	01Jan2000, 05:50	18.8
WC_06	0.732	158.7	01Jan2000, 05:40	17.7
WC_J_6_7_8	3.458	438.2	01Jan2000, 05:50	85.4
WC_R_5	3.458	436	01Jan2000, 06:10	85.3
WC_05	0.892	10.5	01Jan2000, 06:10	2
WC_J_DB	4.35	446.5	01Jan2000, 06:10	87.3
WC_Det_Bas	4.35	99.2	01Jan2000, 08:50	80.2
WC_R_4	4.35	99.1	01Jan2000, 09:00	79.9
WC_04	0.351	89.9	01Jan2000, 05:20	10.7
WC_J_4	4.701	99.2	01Jan2000, 09:00	90.6
WC_R_3	4.701	99.2	01Jan2000, 09:10	90.4
WC_03	0.076	5.8	01Jan2000, 05:30	0.5
WC_J_3	4.777	99.2	01Jan2000, 09:10	91
WC_R_2	4.777	99.2	01Jan2000, 09:10	90.9
WC_02	0.153	48.8	01Jan2000, 05:20	5.1
WC_J_2	4.93	137.9	01Jan2000, 05:30	96
WC_R_1	4.93	134.2	01Jan2000, 05:40	95.8
Bu3ST_05	0.381	20.9	01Jan2000, 05:40	2.4
Busby_Det_Bas	0.381	0	01Jan2000, 01:00	0
Bu3ST_R_4	0.381	0	01Jan2000, 01:00	0
Bu3ST_04	0.28	47.6	01Jan2000, 05:30	6.9
Bu3ST_J_4	0.661	47.6	01Jan2000, 05:30	6.9
Bu3ST_R_3	0.661	46.8	01Jan2000, 05:40	6.9
Bu3ST_03	0.268	91	01Jan2000, 05:20	12.3
Bu3ST_J_3	0.929	134.3	01Jan2000, 05:30	19.2
Bu3ST_R_2	0.929	133	01Jan2000, 05:30	19.2
Bu3ST_02	0.182	42.9	01Jan2000, 05:30	6.5
Bu3ST_J_2	1.111	175.8	01Jan2000, 05:30	25.7
Bu3ST_R_1	1.111	173.1	01Jan2000, 05:40	25.7
Bu3ST_01	0.17	40.6	01Jan2000, 05:20	5.5
WC_J_Bu3ST	6.211	342.4	01Jan2000, 05:40	127
CH_R_WC_1	6.211	339.3	01Jan2000, 05:40	126.8
WC_01	0.119	33	01Jan2000, 05:20	4.6
CH_J_WC_01	6.33	368.2	01Jan2000, 05:40	131.4
CH_R_4	6.33	364.9	01Jan2000, 05:50	131.1
CH_04	0.27	15.9	01Jan2000, 05:50	2.7
CH_J_4	6.6	380.8	01Jan2000, 05:50	133.8
CH_R_3	6.6	374.1	01Jan2000, 05:50	133.6
CH_03	0.205	23.3	01Jan2000, 05:30	2.8
CH_J_3	6.805	392.5	01Jan2000, 05:50	136.4
CH_R_2	6.805	389.5	01Jan2000, 06:00	136.2
CH_02	0.296	41.3	01Jan2000, 05:40	5.2
CH_J_2	7.101	421.5	01Jan2000, 06:00	141.5

Project: WMG/HYDROLOGIC MODELING REPORT-COSV
Detail: OUTPUT FROM HEC-HMS MODELING
2-YEAR 6-HOUR PRECIPITATION EVENT

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

EX CONDITIONS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	PEAK DISCHARGE (CFS)	TIME OF PEAK	VOLUME (AC.-FT.)
CH_R_1	7.101	417.5	01Jan2000, 06:10	141
CTY_01	0.7	71.4	01Jan2000, 05:50	12.4
CTY_01_J	0.7	71.4	01Jan2000, 05:50	12.4
RCE_R_1	0.7	70.7	01Jan2000, 06:10	12.5
RCE_01	0.829	150.9	01Jan2000, 05:40	18.8
CH_01	0.359	41.7	01Jan2000, 05:50	6.6
CH_J_RCE_1	8.989	622.2	01Jan2000, 06:10	179
B3N_R_8	8.989	613.5	01Jan2000, 06:20	178.6
B3N_09	0.624	91.1	01Jan2000, 05:50	13.4
B3N_08	0.446	53.9	01Jan2000, 05:40	6.4
B3N_J_8_9	10.059	719.2	01Jan2000, 06:10	198.4
B3N_R_7	10.059	715.2	01Jan2000, 06:30	197.9
B3N_06	0.96	102.6	01Jan2000, 06:00	17.9
B3N_07	0.849	76.3	01Jan2000, 06:00	13.3
B3N_J_6_7	11.868	853	01Jan2000, 06:20	229.2
B3N_R_5	11.868	852.5	01Jan2000, 06:30	229.1
B3N_05	1.91	305.4	01Jan2000, 05:30	31.1
B3N_04	0.826	79.8	01Jan2000, 06:00	15
B3N_J_4_5	14.604	1024	01Jan2000, 06:20	275.2
B3N_R_3	14.604	1013.9	01Jan2000, 06:30	274.8
B3N_03	0.949	108	01Jan2000, 05:40	12.7
B3N_02	0.797	115.9	01Jan2000, 05:40	15.1
B3N_J_2_3	16.35	1123.9	01Jan2000, 06:30	302.6
B3N_R_1	16.35	1118.4	01Jan2000, 06:40	302.4
B3N_01	0.146	13.2	01Jan2000, 05:40	2.1
SPR	16.496	1125.6	01Jan2000, 06:40	304.4

Project: WMG/HYDROLOGIC MODELING REPORT-COSV
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2-YEAR 6-HOUR PRECIPITATION EVENT

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

BMP CONDITIONS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	PEAK DISCHARGE (CFS)	TIME OF PEAK	VOLUME (AC.-FT.)
WC_08	1.925	223.4	01Jan2000, 06:20	48.8
WC_07	0.801	116.8	01Jan2000, 05:50	18.8
WC_06	0.732	158.7	01Jan2000, 05:40	17.7
WC_J_6_7_8	3.458	438.2	01Jan2000, 05:50	85.4
WC_R_5	3.458	436	01Jan2000, 06:10	85.3
WC_05	0.892	10.5	01Jan2000, 06:10	2
WC_J_DB	4.35	446.5	01Jan2000, 06:10	87.3
WC_Det_Bas	4.35	99.2	01Jan2000, 08:50	80.2
WC_R_4	4.35	99.1	01Jan2000, 09:00	79.9
WC_04	0.3439	88.1	01Jan2000, 05:20	10.5
WC_04_BMP	0.00705	0	01Jan2000, 01:00	0
WC_J_4	4.70095	99.2	01Jan2000, 09:00	90.4
WC_R_3	4.70095	99.2	01Jan2000, 09:10	90.2
WC_03	0.076	5.8	01Jan2000, 05:30	0.5
WC_J_3	4.77695	99.2	01Jan2000, 09:10	90.8
WC_R_2	4.77695	99.2	01Jan2000, 09:10	90.7
WC_02	0.1312	41.8	01Jan2000, 05:20	4.3
WC_02_BMP	0.02185	0	01Jan2000, 01:00	0
WC_J_2	4.93	129.5	01Jan2000, 05:30	95
WC_R_1	4.93	127.4	01Jan2000, 05:40	94.9
Bu3ST_05	0.381	20.9	01Jan2000, 05:40	2.4
Busby_Det_Bas	0.381	0	01Jan2000, 01:00	0
Bu3ST_R_4	0.381	0	01Jan2000, 01:00	0
Bu3ST_04	0.2692	45.7	01Jan2000, 05:30	6.7
Bu3ST_04_BMP	0.01082	0	01Jan2000, 01:00	0
Bu3ST_J_4	0.66102	45.7	01Jan2000, 05:30	6.7
Bu3ST_R_3	0.66102	45.3	01Jan2000, 05:40	6.7
Bu3ST_03	0.2369	80.4	01Jan2000, 05:20	10.9
Bu3ST_03_BMP	0.03108	0	01Jan2000, 01:00	0
Bu3ST_J_3	0.929	122.5	01Jan2000, 05:30	17.5
Bu3ST_R_2	0.929	120.6	01Jan2000, 05:30	17.6
Bu3ST_02	0.1769	41.7	01Jan2000, 05:30	6.3
Bu3ST_02_BMP	0.00515	0	01Jan2000, 01:00	0
Bu3ST_J_2	1.11105	162.2	01Jan2000, 05:30	23.8
Bu3ST_R_1	1.11105	160.3	01Jan2000, 05:40	23.8
Bu3ST_01	0.1608	38.4	01Jan2000, 05:20	5.2
Bu3ST_01_BMP	0.00918	0	01Jan2000, 01:00	0
WC_J_Bu3ST	6.21103	320.9	01Jan2000, 05:40	123.9
CH_R_WC_1	6.21103	317	01Jan2000, 05:40	123.7
WC_01	0.1139	31.6	01Jan2000, 05:20	4.4
WC_01_BMP	0.00513	0	01Jan2000, 01:00	0
CH_J_WC_01	6.33006	344.7	01Jan2000, 05:40	128.1
CH_R_4	6.33006	341.2	01Jan2000, 05:50	127.8
CH_04	0.2676	15.8	01Jan2000, 05:50	2.6

Project: WMG/HYDROLOGIC MODELING REPORT-COSV
Detail: OUTPUT FROM HEC-HMS MODELING
2-YEAR 6-HOUR PRECIPITATION EVENT

JE FULLER
HYDROLOGY & GEOMORPHOLOGY, INC.

BMP CONDITIONS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	PEAK DISCHARGE (CFS)	TIME OF PEAK	VOLUME (AC.-FT.)
CH_04_BMP	0.0024	0	01Jan2000, 01:00	0
CH_R_3	6.60006	348.6	01Jan2000, 05:50	130.3
CH_03	0.202	23	01Jan2000, 05:30	2.8
CH_03_BMP	0.00304	0	01Jan2000, 01:00	0
CH_R_2	6.8051	366.1	01Jan2000, 06:00	132.9
CH_02	0.2943	41.1	01Jan2000, 05:40	5.2
CH_02_BMP	0.00173	0	01Jan2000, 01:00	0
CH_J_4	6.60006	357	01Jan2000, 05:50	130.4
CH_R_1	7.10113	392.9	01Jan2000, 06:10	137.6
RCE_01	0.8203	149.3	01Jan2000, 05:40	18.6
RCE_01_BMP	0.00872	0	01Jan2000, 01:00	0
CTY_01	0.6964	71	01Jan2000, 05:50	12.4
CTY_01_BMP	0.0036	0	01Jan2000, 01:00	0
CTY_01_J	0.7	71	01Jan2000, 05:50	12.4
RCE_R_1	0.7	70.3	01Jan2000, 06:10	12.4
CH_01	0.3551	41.2	01Jan2000, 05:50	6.5
CH_01_BMP	0.00389	0	01Jan2000, 01:00	0
CH_J_3	6.8051	366.7	01Jan2000, 05:50	133.1
CH_J_2	7.10113	398	01Jan2000, 06:00	138
B3N_R_8	8.98914	588	01Jan2000, 06:20	174.8
B3N_09	0.6238	91.1	01Jan2000, 05:50	13.4
B3N_09_BMP	0.00023	0	01Jan2000, 01:00	0
B3N_08	0.446	53.9	01Jan2000, 05:40	6.4
CH_J_RCE_1	8.98914	595.8	01Jan2000, 06:10	175.2
B3N_J_8_9	10.05917	690.7	01Jan2000, 06:10	194.6
B3N_R_7	10.05917	688.6	01Jan2000, 06:30	194.2
B3N_06	0.96	102.6	01Jan2000, 06:00	17.9
B3N_07	0.849	76.3	01Jan2000, 06:00	13.3
B3N_J_6_7	11.86817	825.7	01Jan2000, 06:30	225.4
B3N_R_5	11.86817	825	01Jan2000, 06:30	225.3
B3N_05	1.91	305.4	01Jan2000, 05:30	31.1
B3N_04	0.826	79.8	01Jan2000, 06:00	15
B3N_J_4_5	14.60417	994.9	01Jan2000, 06:20	271.4
B3N_R_3	14.60417	985.1	01Jan2000, 06:30	271.1
B3N_03	0.949	108	01Jan2000, 05:40	12.7
B3N_02	0.797	115.9	01Jan2000, 05:40	15.1
B3N_J_2_3	16.35017	1095	01Jan2000, 06:30	298.8
B3N_R_1	16.35017	1090.2	01Jan2000, 06:40	298.7
B3N_01	0.146	13.2	01Jan2000, 05:40	2.1
SPR	16.49617	1097.4	01Jan2000, 06:40	300.8

Project: WMG/HYDROLOGIC MODELING REPORT-COSV
Detail: OUTPUT FROM HEC-HMS MODELING
2-YEAR 6-HOUR PRECIPITATION EVENT

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

BMP2 CONDITIONS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	PEAK DISCHARGE (CFS)	TIME OF PEAK	VOLUME (AC.-FT.)
WC_08	1.925	223.4	01Jan2000, 06:20	48.8
WC_07	0.801	116.8	01Jan2000, 05:50	18.8
WC_06	0.732	158.7	01Jan2000, 05:40	17.7
WC_J_6_7_8	3.458	438.2	01Jan2000, 05:50	85.4
WC_R_5	3.458	436	01Jan2000, 06:10	85.3
WC_05	0.892	10.5	01Jan2000, 06:10	2
WC_J_DB	4.35	446.5	01Jan2000, 06:10	87.3
WC_Det_Bas	4.35	99.2	01Jan2000, 08:50	80.2
WC_R_4	4.35	99.1	01Jan2000, 09:00	79.9
WC_04	0.3439	88.1	01Jan2000, 05:20	10.5
WC_04_BMP	0.00705	0	01Jan2000, 01:00	0
WC_J_4	4.70095	99.2	01Jan2000, 09:00	90.4
WC_R_3	4.70095	99.2	01Jan2000, 09:10	90.2
WC_03	0.076	5.8	01Jan2000, 05:30	0.5
WC_J_3	4.77695	99.2	01Jan2000, 09:10	90.8
WC_R_2	4.77695	99.2	01Jan2000, 09:10	90.7
WC_02	0.1312	41.8	01Jan2000, 05:20	4.3
WC_02_BMP	0.02185	0	01Jan2000, 01:00	0
WC_J_2	4.93	129.5	01Jan2000, 05:30	95
WC_R_1	4.93	127.4	01Jan2000, 05:40	94.9
Bu3ST_05	0.381	20.9	01Jan2000, 05:40	2.4
Busby_Det_Bas	0.381	0	01Jan2000, 01:00	0
Bu3ST_R_4	0.381	0	01Jan2000, 01:00	0
Bu3ST_04	0.2692	45.7	01Jan2000, 05:30	6.7
Bu3ST_04_BMP	0.01082	0	01Jan2000, 01:00	0
Bu3ST_J_4	0.66102	45.7	01Jan2000, 05:30	6.7
Bu3ST_R_3	0.66102	45.3	01Jan2000, 05:40	6.7
Bu3ST_03	0.2369	80.4	01Jan2000, 05:20	10.9
Bu3ST_03_BMP	0.03108	0	01Jan2000, 01:00	0
Bu3ST_J_3	0.929	122.5	01Jan2000, 05:30	17.5
Bu3ST_R_2	0.929	120.6	01Jan2000, 05:30	17.6
Bu3ST_02	0.1769	41.7	01Jan2000, 05:30	6.3
Bu3ST_02_BMP	0.00515	0	01Jan2000, 01:00	0
Bu3ST_J_2	1.11105	162.2	01Jan2000, 05:30	23.8
Bu3ST_R_1	1.11105	160.3	01Jan2000, 05:40	23.8
Bu3ST_01	0.1608	38.4	01Jan2000, 05:20	5.2
Bu3ST_01_BMP	0.00918	0	01Jan2000, 01:00	0
WC_J_Bu3ST	6.21103	320.9	01Jan2000, 05:40	123.9
CH_R_WC_1	6.21103	317	01Jan2000, 05:40	123.7
WC_01	0.1139	31.6	01Jan2000, 05:20	4.4
WC_01_BMP	0.00513	0	01Jan2000, 01:00	0
CH_J_WC_01	6.33006	344.7	01Jan2000, 05:40	128.1
CH_R_4	6.33006	341.2	01Jan2000, 05:50	127.8
CH_04	0.2676	15.8	01Jan2000, 05:50	2.6

Project: WMG/HYDROLOGIC MODELING REPORT-COSV
Detail: OUTPUT FROM HEC-HMS MODELING
2-YEAR 6-HOUR PRECIPITATION EVENT

JE FULLER HYDROLOGY & GEOMORPHOLOGY, INC.

BMP2 CONDITIONS

HYDROLOGIC ELEMENT	DRAINAGE AREA (MI.^2)	PEAK DISCHARGE (CFS)	TIME OF PEAK	VOLUME (AC.-FT.)
CH_04_BMP	0.0024	0	01Jan2000, 01:00	0
CH_R_3	6.60006	348.6	01Jan2000, 05:50	130.3
CH_03	0.202	23	01Jan2000, 05:30	2.8
CH_03_BMP	0.00304	0	01Jan2000, 01:00	0
CH_R_2	6.8051	366.1	01Jan2000, 06:00	132.9
CH_02	0.2943	41.1	01Jan2000, 05:40	5.2
CH_02_BMP	0.00173	0	01Jan2000, 01:00	0
CH_J_4	6.60006	357	01Jan2000, 05:50	130.4
CH_R_1	7.10113	392.9	01Jan2000, 06:10	137.6
RCE_01	0.8203	149.3	01Jan2000, 05:40	18.6
RCE_01_BMP	0.00872	0	01Jan2000, 01:00	0
CTY_01	0.6964	71	01Jan2000, 05:50	12.4
CTY_01_BMP	0.0036	0	01Jan2000, 01:00	0
CTY_01_J	0.7	71	01Jan2000, 05:50	12.4
RCE_R_1	0.7	70.3	01Jan2000, 06:10	12.4
CH_01	0.3551	41.2	01Jan2000, 05:50	6.5
CH_01_BMP	0.00389	0	01Jan2000, 01:00	0
CH_J_3	6.8051	366.7	01Jan2000, 05:50	133.1
CH_J_2	7.10113	398	01Jan2000, 06:00	138
B3N_R_8	8.98914	588	01Jan2000, 06:20	174.8
B3N_09	0.6238	91.1	01Jan2000, 05:50	13.4
B3N_09_BMP	0.00023	0	01Jan2000, 01:00	0
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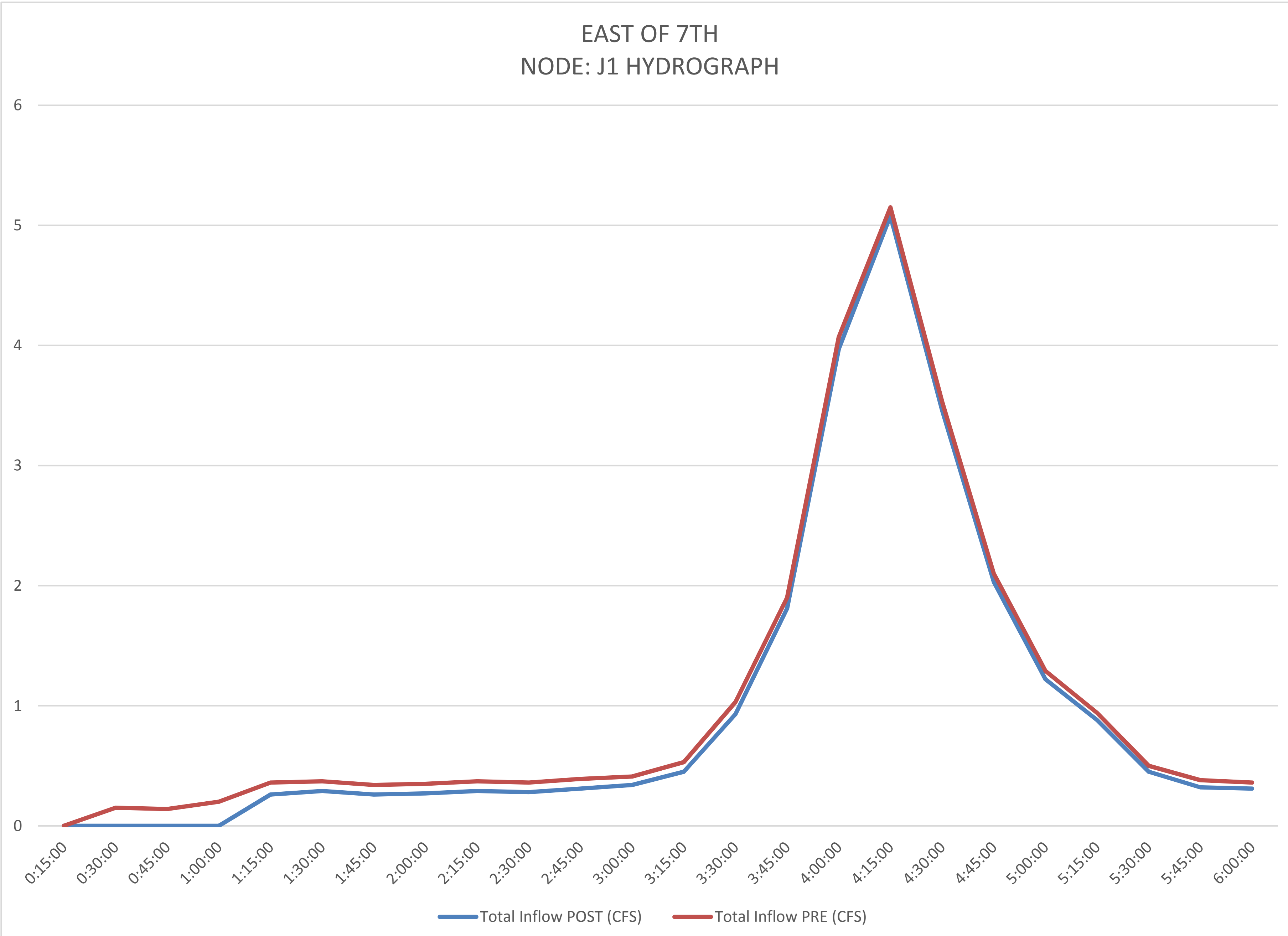
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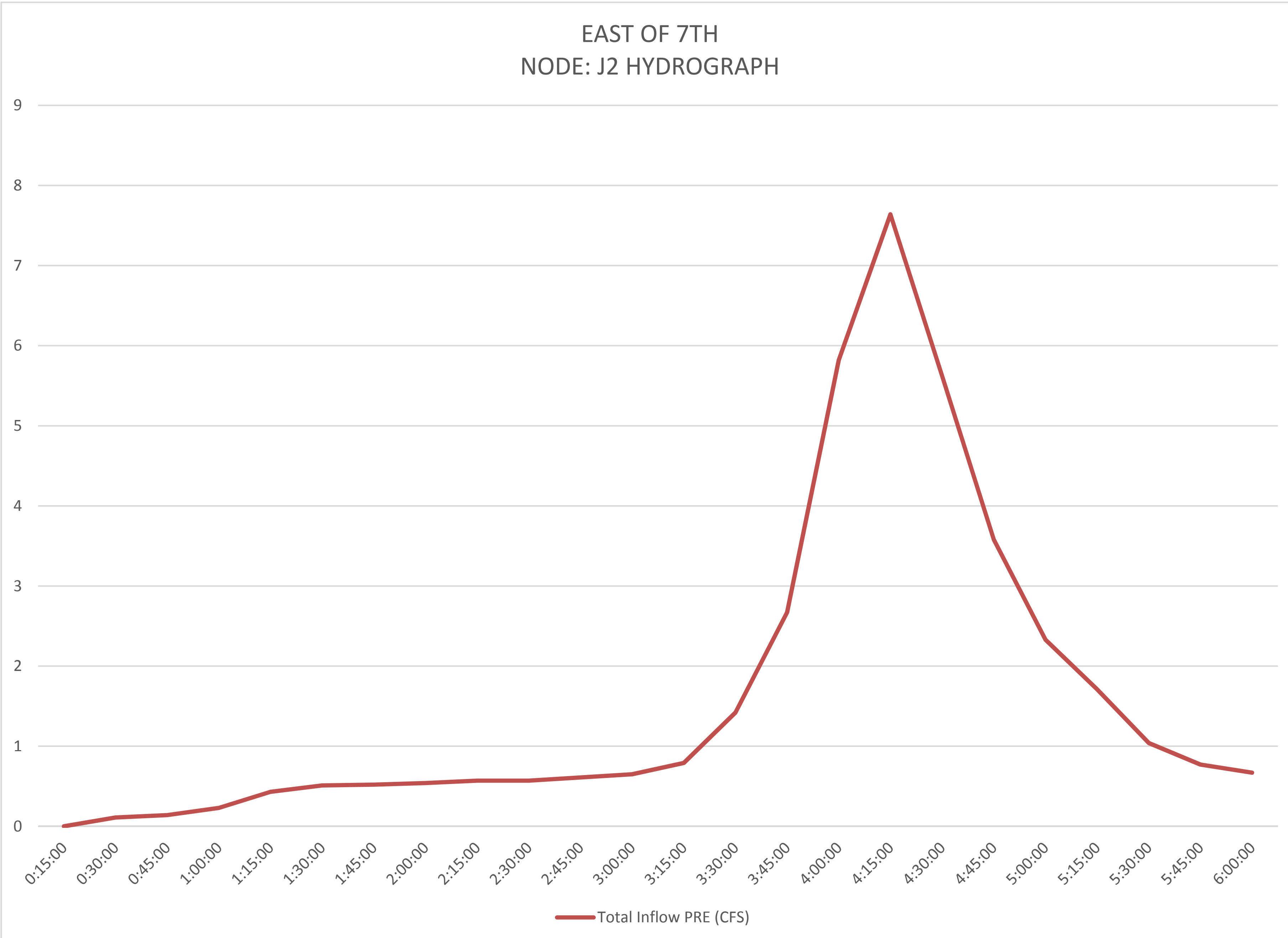
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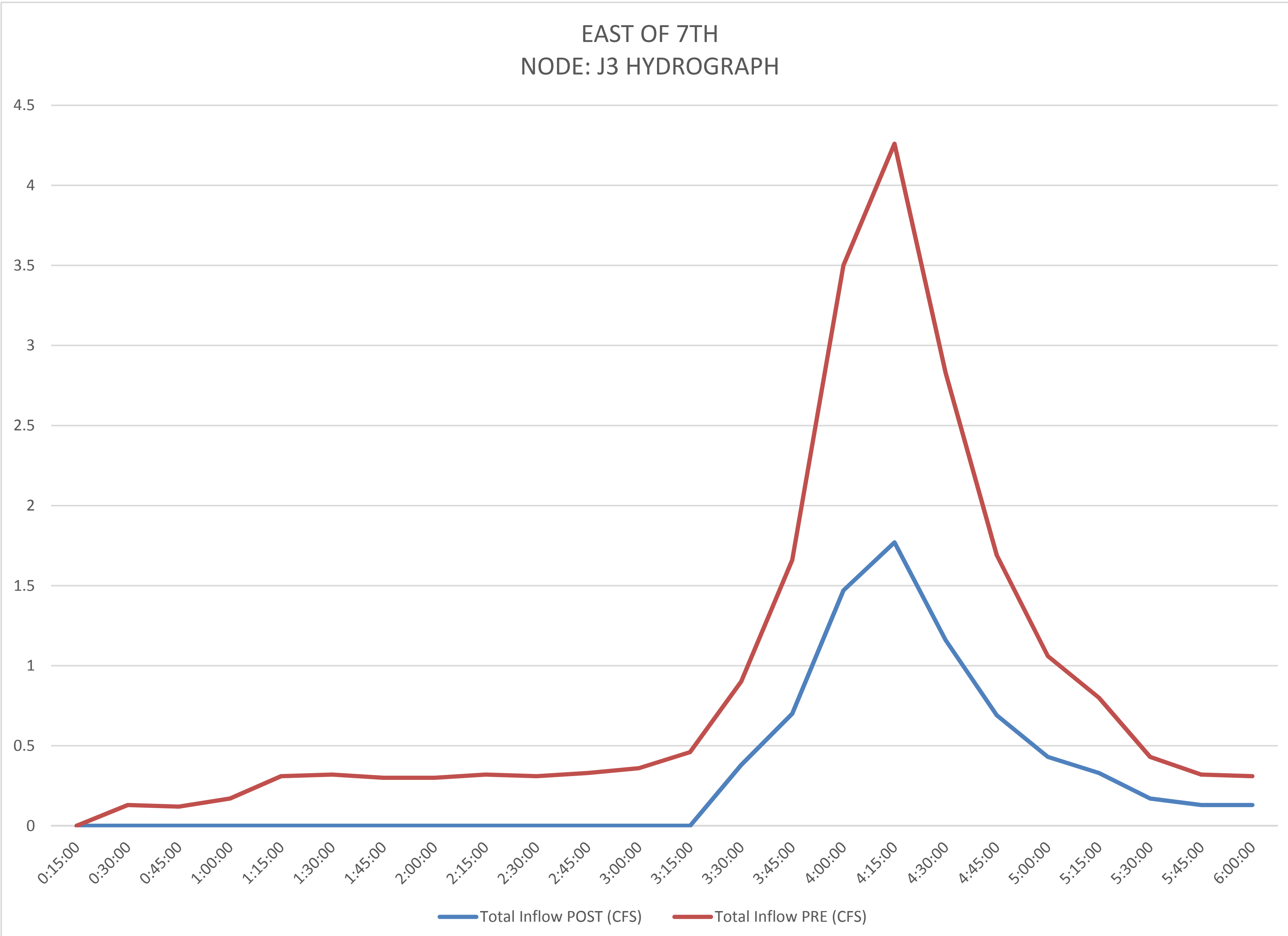
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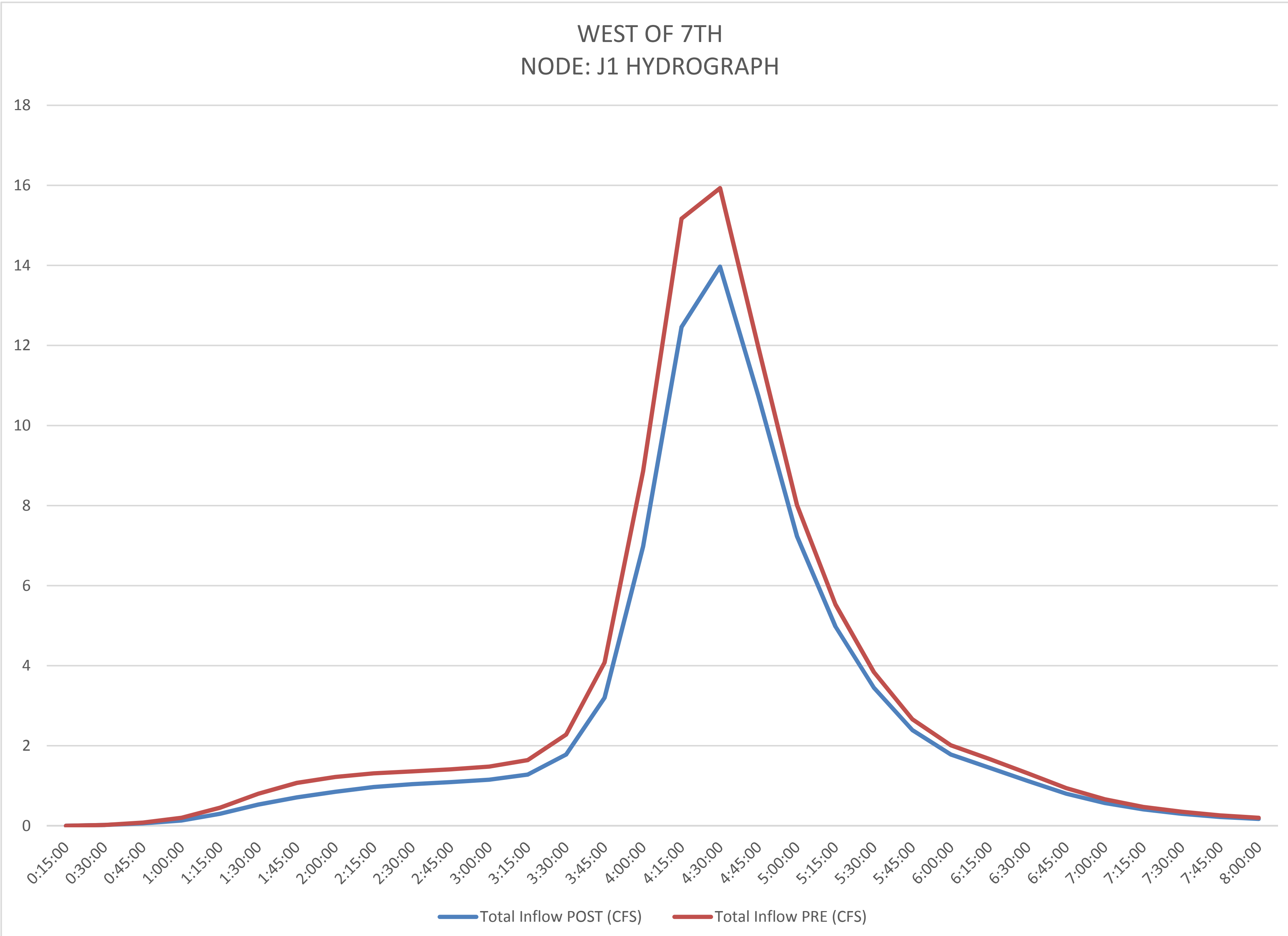
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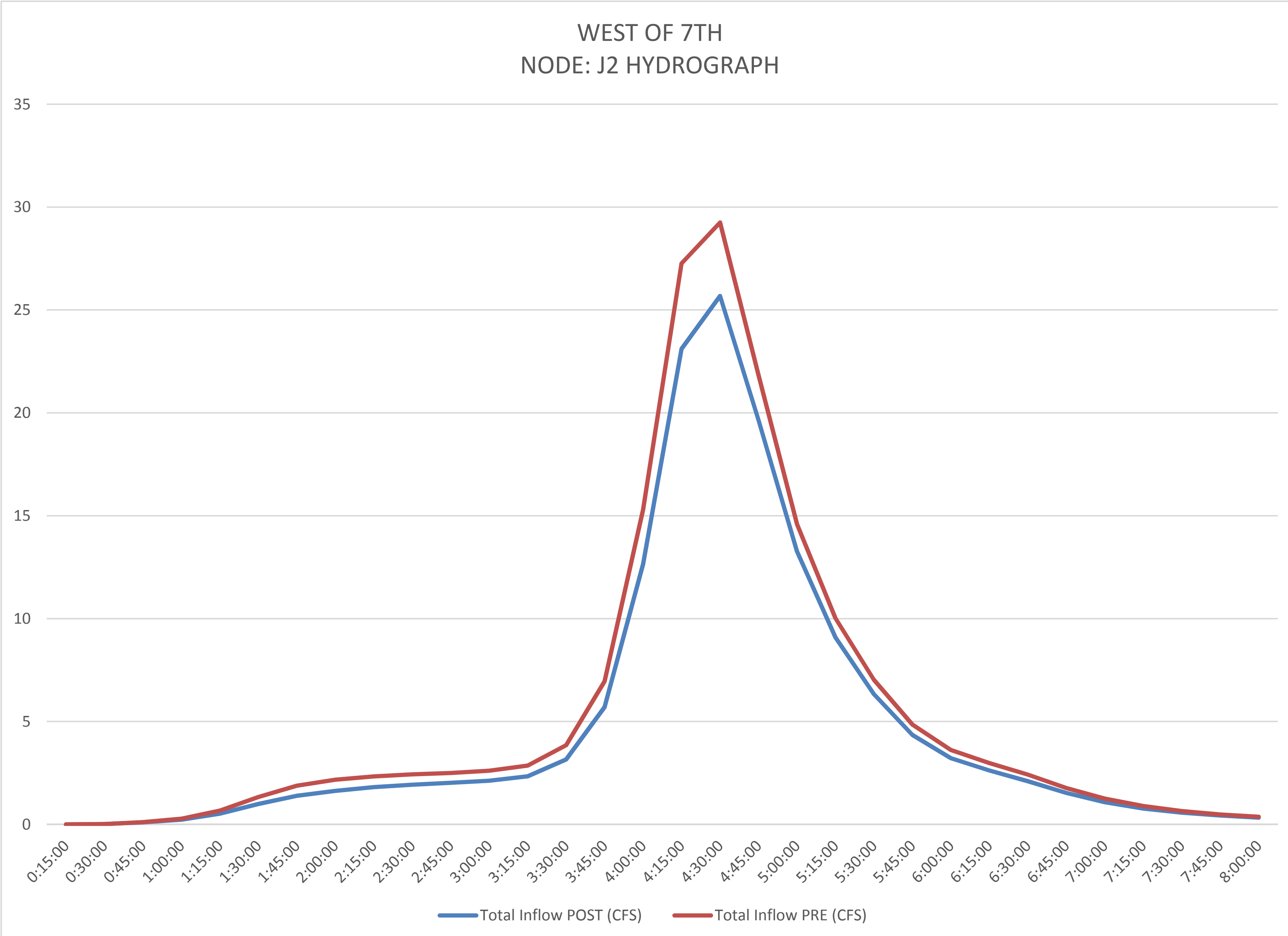
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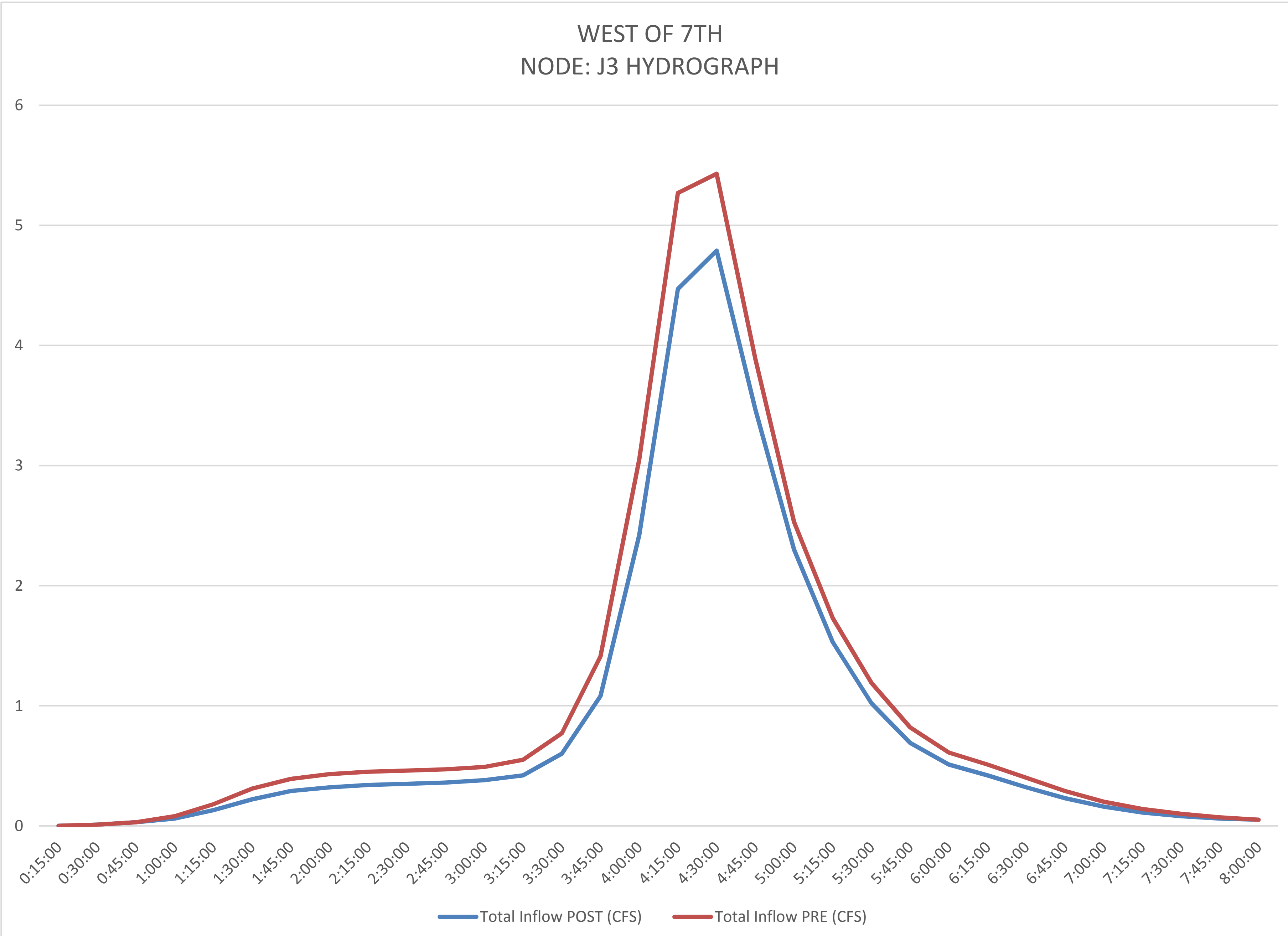
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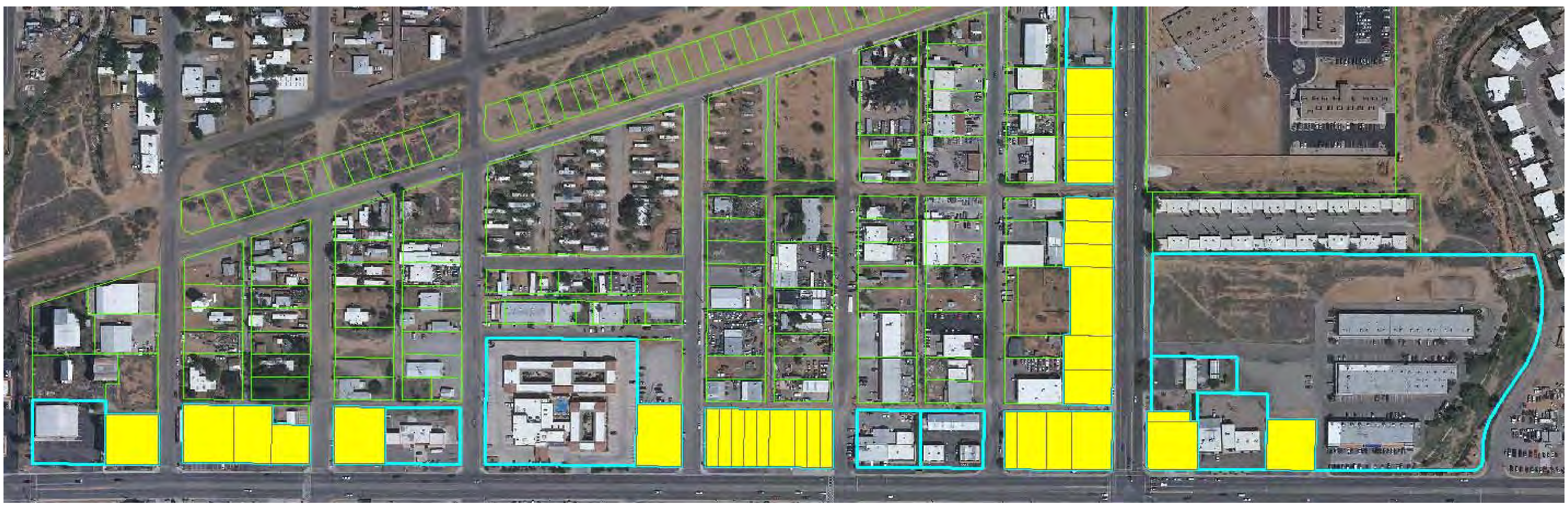
Design #	Retention Basin				Rainwater Harvesting	Permeable Pavement			Parcels
	L (ft)	W (ft)	D (ft)	K (in/hr)	Gallons	L (ft)	W (ft)	K (in/hr)	
1	15	14	0.42	0.85	420	-	-	-	About Half of the Residential Parcels North of Denman Ave
2	15	14	0.42	0.85	865	-	-	-	About Half of the Residential Parcels North of Denman Ave
3	394.2	31.5	0.33	0.85	-	-	-	-	MFR West -->
4	40	68	0.33	0.85	-	-	-	-	MFR Middle -->
5	40	84.1	0.33	0.85	-	-	-	-	MFR East -->
6	462	135	0.33	0.85	8475	-	-	-	School
7	-	-	-	-	-	Whole Surface		0.5 in / Model Time (hr)	Fry Blvd & 7th St
8	60	20	0.33	0.85	-	-	-	-	Large commercial parcel at northeast corner of Fry and 7th
9	20	20	0.33	0.85	-	-	-	-	Commercial parcels < 0.5Ac along Fry & 7th (Figure below)
10	40	20	0.33	0.85	-	-	-	-	The rest of the commercial parcels along Fry & 7th
11	-	-	-	-	-	Whole Surface		0.85	Wash off Carmelita

Commercial parcels along Fry and 7th are selected while those under 0.5 AC are highlighted.



205gallon

420gallon

660gallon

865gallon

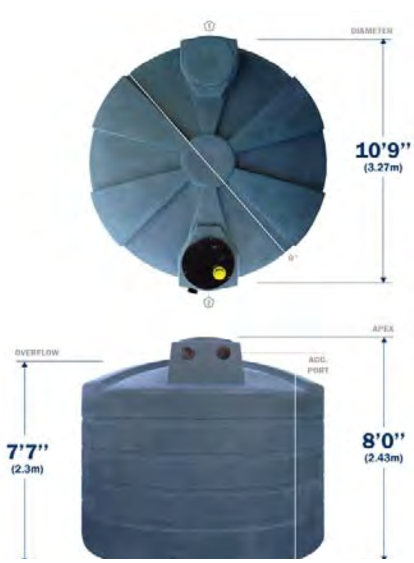
1110gallon

1320gallon

2825gallon

4050gallon

5050gallon



130gallon

265gallon

530gallon

Various above ground tanks

Various below ground tanks

Infiltrator Chamber, 48”x34”x16”

APPENDIX B
Digital Files

Hassayampa Associates
"the science of clean water"

TECHNICAL MEMORANDUM

To: Catlow Shipek
Watershed Management Group

From: Dr. Mark T. Murphy, Ph.D., R.G.
Hassayampa Associates

Date: March 10, 2015

Subject: Buena #3 Geomorphic and Geologic Assessment

Project Number: 13WMG001

In August of 2014, Hassayampa Associates (Hassayampa) was contracted by Watershed Management Group (WMG) to supply geomorphological and hydrogeological expertise in support of a WMG grant from the Walton Family Foundation for applied research in rainwater harvesting in Sierra Vista, Arizona. The scope of work included an analysis of Buena #3 Drainage Channel (Buena #3), a semi-natural tributary of Woodcutter Wash, a southern Arizona stream that flows into the San Pedro River near Charleston, Arizona. Specifically, WMG proposed to examine the use of rainwater harvesting methods and natural channel design (Zeedyk and Clothier, 2009) to prevent erosion and increase infiltration in or adjacent to the channel.

1.0 BACKGROUND

Restoration of the base flow of the San Pedro River in Cochise County, Arizona has been the goal of extensive federal, state and local efforts (Stromberg and Tellman, 2009). Reduction of groundwater pumping and enhanced recharge are two of the keystone remedial strategies to achieve this goal (Lacher et al., 2014). The efforts described in this technical memorandum were undertaken to see if enhanced channel recharge of stormwater could be integrated into remedial work on the Buena #3 channel.

Woodcutter Wash (aka Charleston Wash) is a drainage completely bisecting the of the City of Sierra Vista (City) (Figure 1). The stream headwaters are in the Huachuca Mountains, west of town and runs in a generally northeasterly direction approximately 25 river kilometers (km)(15.5 miles) at an average slope of 1.39%. The wash enters the San Pedro River approximately 0.6 river km (0.4 miles) downstream from the East Charleston Road Bridge, draining an area of approximately 37.5 square kilometers (km^2)(Coes and Pool, 2005). Woodcutter Wash is an ephemeral stream as it flows through the City and most of its length (Callegary et al., 2007; Stewart et al., 2012). The stream is entrenched into its flood plain, similar to most San Pedro River tributaries (Hereford, 1993; Hereford and Betancourt, 2009).

At about the 10 river km (6.2 miles) point, Woodcutter Wash is met by Buena #3 at Buena Vista Neighborhood Park (Figure 2). Above this point, head-cutting has been a problem as erosion in the main channel propagated up into the Buena #3 subwatershed. Several years ago, a sewer line and other buried infrastructure was threatened by channel erosion and a grade control structure (GCS) was installed approximately 170 meters (m)(558 feet) downstream of the Fry Boulevard over crossing (Figure 2). The structure has protected the sewer line, but active erosion at the west wing wall has created an additional threat to the sewer line and other structures. The City has decided to repair the damaged wing wall and add further grade control structures to the channel.

Because of the continued commitment of the City to decrease impacts to the regional aquifer, it was decided to investigate the combination of grade control engineering with stormwater harvesting in a pilot project at Buena #3. In partial response to this request, WMG and Hassayampa proposed a reconnaissance investigation of the geomorphological stability of the channel to assess the applicability of stormwater harvesting and natural channel design concepts.

The reach of Buena #3 selected for this study was the 0.7 km (0.4 mile) section between Fry Boulevard and N. Lenzner Boulevard in the Bella Vista neighborhood. The channel runs northward for the first third and then bends around to the northeast for the remainder of the reach. All direct flow to the channel is shed from streets, yards and parking lots. A Center for Academic Success, pre-K to 12, public charter school is located on Carmelita Drive and uses culverts to shed stormwater into the channel.

The work described here is only a *reconnaissance*-level investigation, reflecting a professional level of care and accuracy; however, it was not based upon topographic survey data precise enough to actually design the structures described in the technical memorandum. Further, the recommendations represent a spectrum of design options that the City engineers, or their consultants, will need to specify. No hydrologic or hydraulic analysis was performed as part of this study and no assessment of flood hazards were made, which should be completed as part of any design process initiated by any party using this study. Hassayampa and WMG are not responsible for misuse of the recommendations of this study.

2.0 METHODS

The morphological geometric (morphometric) analysis of alluvial channels can be used to semi-quantitatively assess the stability of erosional processes and proper hydraulic and ecological function of the stream (Rosgen and Silvey, 1996; Zeedyk and Clothier, 2009). The data collection and analysis described in the technical memorandum follows the methods of Zeedyk and Clothier (2009) and Rosgen and Silvey (1996), as adapted to Arizona geology, hydrology and geomorphology. A series of nine transects were laid out roughly perpendicular to the channel longitudinal (downstream) axis (Figure 2). The transects were located using GIS covers of the channel and were distributed evenly, but then adjusted in the field to accommodate specific on-the-ground features of interest.

Field work occurred in November of 2014. Morphometric channel geometry data were acquired along each transect using a tape and hand-held GPS. Monuments were established at each end. Morphometric channel data were acquired for bankfull width and depth, and maximum flood depth and width of the flood-prone area; all were measured according to definitions and procedures of Rosgen and Silvey (1996). Channel slope and sinuosity were derived from a combination of Google Earth data, USGS topographic data, GPS locations and field observations of the channel. Cross-sectional area, width-to-depth ratios and entrenchment ratios were calculated from these data. No topographic survey was completed of the stream.

Channel substrate textures were hand sized in the field and each traverse was photographed. Detailed notes were made of the geological relations observed in cut banks and outcroppings, and drainage infrastructure was noted and located with the GPS unit, as appropriate.

3.0 RESULTS

The morphometric field data are overall consistent with a highly disturbed stream system establishing a new dynamic equilibrium (Table 1). Entrenchment ratios range from 1.00, which corresponds to a vertical trench, to 16.4, with an average of 4.42, the equivalent of a stream well integrated with its floodplain (Schumm et al., 1984; Rosgen and Silvey, 1996). It seems likely that the placement of the grade-control structure near Transect 2 has temporarily checked any advance of headcutting; however, the active erosion on the west wing wall suggests that it has only diverted the erosive advance into the surrounding drainage basin (Figures 3, 4).

Sinuosity of the channel is the ratio of the river length of the reach to the straight-line distance. A channel confined on either side by the built environment cannot establish channel meanders, thereby fitting its gradient to a stable downstream profile. While no specific sinuosity is ‘correct’, the study reach average is 1.1 suggesting a higher gradient than the actual slope of the channel bottom or floodplain. This suggests that the channel may exert a longitudinal erosive stress as it attempts to increase the sinuosity of the reach (Zeedyk and Clothier, 2009).

Further evidence of stream instability is found in some of the relatively steep (>3 m high) vertical cut banks approximately 50 m downstream of Transect 6 (Figure 5). At the time of the field work, active debris was accumulating at the toe of the escarpment and roots of green grasses were exposed. The escarpment shows up on aerial photos going back to at least 2011, suggesting it has moved about 5.4 m in four years.

Detailed examination of the escarpment walls reveals about 2 meters of reddish-brown, highly indurated (cemented), fine-grained sand to silt/clay, which is similar to shorter cut banks along the reach (Figure 6). It appears that the entire channel is inset (eroded) into this formation, possibly the M2 uppermost Pleistocene, <125 thousand years before present (kA) units of Dempsey and Pearthree (1994). These same authors describe extensive, but discontinuous, accumulations of calcium carbonate in the Sierra Vista area. This unit also appears to be the equivalent of hydrologic group 3 of GSA (2004).

Although the exposed channel substrate consists of medium- to coarse-grained sand and cobbles, this layer is active alluvium and represents only the top 0.3 m of bed material. Below the active alluvium is a layer of unknown thickness similar in composition and properties to the channel banks, reddish-brown, highly indurated, fine-grained clayey or silty sand.

The study reach through flow and direct precipitation is augmented by several drainage infrastructure elements. Storm flow running along the first 150 meters of Carl Hayden

Boulevard is directed into the channel by several down drains on the east bank and parking lot curb cuts on the west side of the channel. Immediately downstream of the GCS, sheet wash from a vacant lot and apartment complex parking lot appears to flow directly to the channel and some of this runoff may contribute to the failure of the GCS wing wall.

From Transect 3 to Transect 4, a large amount of runoff is conveyed to a 1.6-m-diameter corrugated-metal culvert (CMC) that outfalls at grade directly into the channel. Through a series of inlets, culverts and open drainage ditches, the CMC accumulates flows from a drainage ditch that flows eastwardly under N. 7th Street and drains the streets and lots from east of N. 5th Street between S. Denman Avenue and Fry Boulevard. In addition, the system adds N. 7th Street drainage and partial runoff from impervious surfaces at the Center for Academic Success charter school.

At Transect 5, a curb cut and asphalt-lined channel convey some fraction of street and lot runoff from E. Carmelita Drive to a down drain that outfalls to the channel (Figure 4). Massive erosion of the down drain has been treated with concrete construction debris rip-rap; however, erosion at the margins of the debris is active and extensive. Construction debris has also been placed along the western bank of the channel from the down drain to about 65 m upstream. Other than incidental yard runoff, no other major flow augmentation occurs from Transect 5 to the N. Lenzner Boulevard bridge.

The morphometric field data acquired can be plotted as a function of longitudinal position in the reach (Figure 7). There is a distinct break in most of the data above and below the GCS with a transition occurring primarily between Transects 3 and 5. It is within this interval that most of the additional drainage enters the channel from either the CMC or the Carmelita Avenue down drain. Downstream of Transect 5 there is slightly more stability, with the exception of the cut bank below Transect 6 (Figure 6).

4.0 RECOMMENDATIONS

4.1 Erosion Control

From the data and observations reported in Section 3, it appears that the channel degradation mitigated by the Buena #3 GCS is now propagating into the watershed along the channel tributaries. Specifically, the erosive potential energy is being dissipated by cutting back into the subwatershed between the GCS and Transect 6 at several points and attempting to increase its sinuosity at the cut bank below Transect 6. If this erosion continues, destruction of the GCS and adjacent structures is possible.

In order to slow or stop the destructive erosion in Buena #3, several alterations in the erosive power of the stream are necessary. In a sense, grade control has to be distributed from the subwatershed, and surface flows need to be intercepted. Flows entering the channel need to be slowed and spread over the adjacent terraces as much as is practical, given the restricted open space. In addition, the peak discharges should be reduced by on-site retention in the neighborhoods.

Detention ponds are an important mitigation device for decreasing the erosive power of runoff. By extending the flood hydrographs, the intensity of peak flows can be slowed and the erosive power of runoff can be mitigated. In addition, sediment storage in off-channel ponds can be used to create a balance of suspended material in runoff that reduces the ability of water to cut into channel banks. Finally, flows can be conveyed to channels in such a way that energy is dissipated and erosive power kept within design criteria (NRC, 2009; Smith et al., 2011).

Flows that originate at C and D (Figure 8) should be diverted over Zuni bowls (Zeedyk and Clothier, 2009) to Pond #1, located in the overbank area (E), the west bank of the channel between Transects 2 and 5. Zuni bowls (Figure 8) are small (meter-scale) head-cut armor slopes and plunge pools, composed of rock and sometimes installed in series, used to dissipate energy and protect erosive channels. The size and dimensions of Pond #1 will depend upon flood-flow (hydrologic and hydraulic) modeling and be sized upon the mean annualized flood (see for example, WEF/ASCE, 1998). The culvert connections to the CMC at D should be plugged or removed, and an outfall to Pond #1 should be constructed to keep flows at grade. Pond #1 should outfall to the channel using another set Zuni bowls and rock rundowns (J).

Pond #1 should take up about two-thirds of the overbank area (Figures 8, 9). A floodplain terrace, B, sloping downstream at grade, should extend between the pond and the existing channel. The floodplain should direct water to another Zuni bowl at G. From here downstream through H, the west bank transitions through a graded slope to the Pond #1 outfall.

Culverts in the parking lot of the Center for Academic Success charter school should be disconnected. Runoff should be directed through curb cuts to a linear, planted rainwater basin (I). Overflow of the basin should also be directed to Pond #1 at F.

Flows coming down the Carmelita Avenue channel should enter Pond #2 (L) past a rock rundown. The pond should be sized to provide sediment and erosive detention with a methodology similar to Pond #1. Pond # 2 should pass flows downstream through a Zuni bowl.

The Center for Academic Success charter school and the adjacent neighborhoods should be encouraged to implement rainwater harvesting to reduce runoff to the stream and conserve potable water currently used for irrigation. The construction and maintenance of rainwater basins would provide an excellent educational project and instruct students on conservation practices and stormwater quality behaviors. Neighborhood residents can form co-operative construction projects that use rainwater for landscaping and reduce runoff (Lancaster, 2008).

Very little work is recommended for the channel itself, which appears to be approaching equilibrium. Despite the entrenchment, the average slope of the channel floor from Transect 5 downstream (0.009) is flatter than the slope above the GCS (0.016) (Table 1). One-rock dams (Zeedyk and Clothier, 2009) might be needed at K (Figure 8) where the channel appears to have scoured down to the fine-grained unit. Rock vanes should be constructed in areas of active cut banks. Rock vanes (Zeedyk and Clothier, 2009) extend into the channel from the toe of cut banks to slow velocity and accumulate sediment, which armors the toe of the slope.

4.2 Recharge Opportunities

The recommendations proposed above are intended to reduce peak flows, decrease erosive power of those flows and to detain, in the floodplain, water conveyed to and though the Buena #3 channel. This remedy could also promote recharge of the ground water, if possible. A number of studies have investigated the managed recharge potential of Sierra Vista for reducing impacts to the San Pedro River (GSA, 2001, 2004, 2006; Coes and Pool, 2005; Callegary et al., 2007; Lacher, 2012; Stewart, 2014). Several of these studies have generated data on the local subsurface and others have used this data to do predictive computer modeling (GSA, 2001, 2004; Coes and Pool, 2005 [field and lab saturated hydraulic conductivity, K_{sat}]; Callegary et al., 2007 [apparent electrical conductivity, $\tilde{A}_a$]; and Stewart, 2014 [transmission loss]).

In order to estimate recharge potential, GSA (2001, 2004) measured borehole and lab hydraulic conductivity at the Third Street Basins, a series of structures on Buena #3, located approximately 0.9 km upstream of the Fry Avenue bridge. Advanced tensiometers (ATs) were placed at intervals of 5, 10, 15, 20, 30 and 50 feet below ground surface (bgs) in on-channel boreholes and at 5, 10, 15, 20 and 30 feet bgs in off-channel locations. The ATs were used to measure infiltration during storm events and the data used to estimate effective recharge rates of 0.7 to 2 feet/day. Perching above the 30-foot bgs AT was assumed by GSA (2001).

In the 2004 investigation, two methods were used, cylinder infiltrometer data was obtained from surface soils and borehole permeameter data was obtained from depth. In addition, several samples were measured in the laboratory. Field-derived, in-situ K values gave an average of 1.7 feet per day (ft/day) or 52 centimeters per day (cm/day). Laboratory K_{sat} values ranged from 349.25 cm/day for coarse-grained units (0-9% fines) to 28.07 cm/day (0.91 ft/day) for fine-grained units (21-100% fines). Based upon these values, a deep percolation (assuming 42 inches of evapotranspiration per year) of 3.1 feet/year (10^{-4} cm/s) was estimated for the Third Street Basins (GSA, 2004). Overall, of these methods, GSA (2004) estimated an annual recharge flux to be from 2 to 3 feet per year (ft/yr). Similar work at the Fry Basin, located approximately 0.6 km west of Buena #3 resulted in an estimate of 1 to 2 ft/yr.

Coes and Pool (2005) measured hydraulic conductivity and deep percolation flux at a borehole (WC3) approximately 400 m downstream of the confluence of Buena #3 and Woodcutter Wash. The authors also measured tritium (H^3) and stable isotope subsurface migration. The authors described each at that location as 8 m wide, incised 1.8 to 3.0 m deep. Additionally, a few centimeters of sandy, active alluvium overlies a >22.6 m thickness of silty/clayey (30 to 60%) sand. Five samples were retrieved from the borehole for laboratory measurement of K_{sat} and ranged from 1.7×10^{-4} to 9.9×10^{-3} centimeters per second (cm/s), or 14.9 to 885 cm/day. Porosity ranged from 33 to 40% and soil moisture, from 11 to 19% (Coes and Pool, 2005).

At 14.9 cm/day, Coes and Pool's (2005) low boundary for K_{sat} is similar to GSA's (2004) fine-grained unit value of 28.07 cm/day. Coes and Pool's (2005) tritium data suggested a deep flux 10^{-9} cm/s, or 0.96 ft/yr, similar to the estimate of GSA (2004) for the Third Street Basins.

Callegary et al., (2007) generated electrical conductivity data that suggested higher permeability and recharge flux at both of these sites. This may be because the authors were mostly focusing on surface permeability, which is higher than the indurated, fine grain deep units. Stewart (2014) calculated transmission loss at the WC3 location and indicated 600 cubic meters was lost over the studied reach. Using her channel lengths and the average bankfull width in Table 1, that would result in a recharge flux of 0.16 ft/y, about an order of magnitude less than the Coes and Pool's (2005) estimate.

Regardless of the variations of these studies, the suggestion is that recharge in the vicinity of Buena #3 is in the low range relative to other parts of the Sierra Vista region. Any infiltrated stormwater in the proposed detention would probably perch on the indurated fine-grain unit and be consumed by evapotranspiration. If recharge was still required of the ponds, it might be necessary to construct an injection well, either within the pond or adjacent in order to locate a deeper and more permeable zone. Materials testing of the proposed pond site would be necessary before a final determination could be made.

5.0 CONCLUSIONS

The conclusions of this study are preliminary and could easily change with additional site characterization studies. Nevertheless, several items seem clear at this point.

1. The GCS located above Transect 2 appears to have done a good job of stabilizing the reach between it and the Fry Boulevard bridge. The accumulated sediment upstream of the bridge supports abundant grasses and small forbs and even several large cottonwoods; however, there is no evidence of any deep percolation of channel flows and perching of water at the root zone is suspected. Nevertheless, the City strategy of constructing GCS for erosion control and incidental channel recharge appears to be sound if subsurface conditions support downward movement of infiltrated water.

2. Although the channel itself seems to have stabilized, the channel subwatershed continues to degrade in a lateral direction. This degradation includes head cuts at tributaries and the west wing wall of the GCS and cut banks downstream of the Carmelita Avenue outfall.

3. In order to reduce destructive erosion, we recommend slowing and spreading the introduction of stormwater to the channel. This would include several specific detention structures, bank armor and diversions of the current conveyance system (Figure 8).

4. To further reduce peak runoff erosional energy, we also recommend encouraging the neighborhood to retain stormwater on-site with the use of rainwater harvesting practices and structures, as per Lancaster (2008). An educational outreach program at The Center for Academic Success charter school that could be developed with projects and workshops in rainwater harvesting would also be helpful.

5. Managed groundwater recharge in the reach could be possible but seems less feasible than other sites in the City. Work by GSA (2002, 2004) and Coes and Pool (2005) suggested that the presence of the locally ubiquitous "fine-grained unit," possibly the M2 uppermost Pleistocene unit of Dempsey and Pearthree (1994), could be reducing the in-situ K_{sat} to a value that precludes efficient in-channel managed recharge or a recharge pond (Callegary et al., 2007). Alternately, a deep recharge well might be able to penetrate the shallow fine-grained unit and deliver water to a more permeable stratigraphic horizon. The acquisition of on-site subsurface data would be necessary to evaluate this option.

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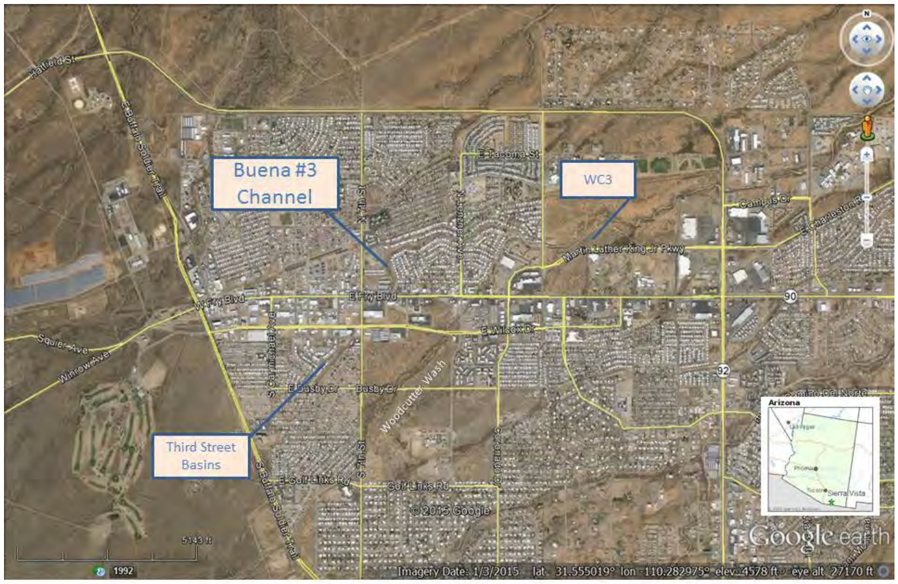

Figure 1 - - Vicinity map to Buena #3 Channel, between Fry Blvd. and Lenzner Ave., Sierra Vista, Arizona. Also located are the Third Street Basins (GSA, 2001, 2004) and soil boring WC3 (Coes and Pool, 2005).

Figure 2 - - Location map for analysis of Buena #3 Channel, between Fry Blvd. and Lenzner Ave.

Figure 3 - - Grade control structure directly downstream of Transect 2. Note destructive erosion of west wing wall.

Figure 4 - - Carmelita Avenue outfall looking north from channel grade. Severe erosion is obvious.

Figure 5 - - Approximately three-meter high cut bank on the southeast bank of Buena #3. Basal debris in center from a failure recent enough to expose roots of still living grasses.

Figure 6 - - Detail of bank exposure in Figure 5. Material is well-cemented, reddish brown, clayey very-fine-grained sand to silt, with gravel and cobble lenses at exposed base. See text for description.

Stream Name: Buena #3 Channel

Watershed Name: Woodcutter Wash, HUC150502020604 Drainage Area: 37.5 km^2

Location: Sierra Vista, Arizona, from E Fry Blvd to N Lenzner Ave

Observer: M.Murphy

Reach name/ number	1	2	3	4	5	6	7	8	9	Ave
Bankfull Width (m, Wbnk)	0.7	1.5	3.85	4.90	4.18	6.12	5.17	4.00	12.9	4.81
Mean Depth (m, dbnk)	0.25	0.60	0.60	0.87	0.65	1.30	1.18	0.83	0.80	0.79
X-Sectional Area (m2, Abnk)	0.18	0.90	2.31	4.26	2.72	7.96	6.10	3.32	10.30	4.23
W/D ratio (Wbnk/dbnk)	2.80	2.50	6.42	5.63	6.43	4.71	4.38	4.82	16.09	5.98
Max depth (m)	0.80	1.50	0.80	1.63	1.65	1.60	2.14	2.14	1.20	1.50
Width of annual flood area (m, Wfp)	11.5	16.22	8.00	11.5	7.45	7.84	5.17	10.8	17.9	10.70
Entrenchment ratio (Wfp/Wbnk)	16.4	10.8	2.08	2.34	1.78	1.28	1.00	2.70	1.39	4.42
Channel textures (hand sized)	xs sand & gravel	xs sand & gravel	xrs sand w cobbles	coarse sand w/gravel	coarse sand w/gravel & clay	coarse sand w\gravel	coarse sand w/gravel	coarse-sand	coarse-sand	coarse sand w/gravel & clay
Water surface (Ss) or bed (Sb) slope	0.031	0.016	0.004	0.012	0.011	0.009	0.003	0.009	0.011	0.012
Channel sinuosity (K)	1.094	1.113	1.096	1.233	1.121	1.079	1.070	1.086	1.037	1.103

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Table 1 - - Data sheet for Buena #3 traverses. Parameters are from Rosgen and Silvey (1996), as modified for Southwestern US ephemeral washes.

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TECHNICAL MEMORANDUM

To: Catlow Shipek
Watershed Management Group

From: Dr. Mark T. Murphy, Ph.D., R.G.
Hassayampa Associates

Date: May 6th, 2015

Subject: Opportunities for recharge using grade-control structures

Project Number: 13WMG001

As part of the scope of this project, Hassayampa Associates (Hassayampa) was asked to investigate the potential for the use of grade-control structures (GCSs) in Sierra Vista, Arizona (City) washes as channel recharge sites. Because of erosional problems in many of the washes crossing the city, GCSs are necessary throughout the city. Previous investigations have identified channel bottom seepage in tributary washes to the San Pedro River as important groundwater recharge process (GSA, 2004; 2006; Lacher, 2012; Stewart, 2014). Thus, it is reasonable to ask if GCSs can be located to enhance seepage behind the structures and presumptive recharge of ground water in and around Sierra Vista. Hassayampa performed this analysis by a literature review of spatially distributed hydrogeological data to determine if this data could assist in identifying where GCSs might be located.

1.0 BACKGROUND

Restoration of the base flow of the San Pedro River in Cochise County, Arizona has been the goal of extensive federal, state and local efforts (Stromberg and Tellman, 2009). Reduction of groundwater pumping and enhanced recharge are two of the keystone remedial strategies to achieve this goal, with enhanced channel[1] recharge an additional remdy (Lacher et al., 2014).

In a previous Technical Memorandum, Hassayampa (2015) investigated an erosion control problem that had been addressed with a large GCS in the Buena #3 channel bisecting the Bella Vista neighborhood, near Fry and N. Lenzner Boulevards. Buena #3 channel is a tributary of Woodcutter Wash, aka Charleston Wash, a drainage bisecting the City. The stream headwaters are located in the Huachuca Mountains, west of town and the wash runs (generally) northeasterly approximately 25 river kilometers (km)(15.5 miles) at an average slope of 1.39%. The wash enters the San Pedro River approximately 0.6 river km (0.4 miles) downstream from the East Charleston Road Bridge, draining an area of approximately 37.5 square kilometers (km^2)(Coes and Pool, 2005).

There are two other major washes that drain the City. Coyote Wash also traverses the City. It begins on the northeast-facing slopes of the Huachuca Mountains, between Huachuca and Garden Canyons, crossing South Buffalo Soldier Trail south of Golf Links Road. From here it heads northeast, leaving the developed parts of town between the City Public Works yard and Charleston Road. Along the way, Coyote Wash accumulates flow from several short tributaries and stormwater outfall structures. Further to the north, Graveyard Wash begins within the City and also heads northeasterly, leaving the urban area near the intersection of AZ 92 and Coronado Drive.

Over the years, the City has constructed more than fifty GCS on Coyote, Woodcutter and Graveyard Washes and their tributaries. Because of down-cutting throughout the City wash system, these structures need frequent repair or replacement and new structures have been contemplated that could fill the combined function of erosion control and enhanced channel recharge.

2.0 ANALYSIS

The potential for infiltration and channel recharge behind GCSs clearly depends upon the coincidental location of these structures in areas that both have good recharge potential and require mitigation of destructive erosion. The spatial distribution of recharge potential has been shown to be highly variable (GSA, 2004; 2006). A brief literature

[1] *Enhanced recharge* is generally created by human activities and *channel recharge* is natural seepage through the stream bottom. *Enhanced channel recharge* occurs when the natural recharge is artificially increased.

search was performed to examine if the geological and soils data might be useful to better constrain where these locations of high recharge potential might be present.

Field examination of the washes traversing Sierra Vista reveals that the active alluvium is composed of a medium- to coarse-grained sand, with occasional pockets of gravel- to cobble-sized material, generally uncemented (Hassayampa Associates, 2015). In Garden Canyon it is described (Huckleberry, 1996) as 'poorly-sorted boulders, cobbles, gravels, and sand,' and some calcium-carbonate cement was also noted. Further downstream, mapped alluvium is described (Dempsey and Pearthree, 1994) as 'silt and sand to extremely-gravelly sand, with abundant cobbles and boulders in canyons.'

Huckleberry (1996) and Dempsey and Pearthree (1994) subdivide the upper-most Holocene surficial deposits using geomorphic position and the development of calcic horizons. The use of age-dated, calcic soil horizons (chronosequnces) to determine the age of paleosoils[2] in southeastern Arizona is well established (Gile et al, 1966; Machette, 1985; Mc Fadden, 1978; Morrison, 1985; McAuliffe, 1994). By this method, Dempsey and Pearthree (1994) distinguish between a Y2 (younger) and Y1 (older) Holocene stream alluvium. Huckleberry (1996) did not map older and younger alluvium; however, he did note that calcic horizons had formed in some deeper, and geomorphically older, deposits.

Using the soil calcium-carbonate build-up and correlation to other local units, Dempsey and Pearthree (1994) estimated that the Y2 alluvium is between 100 and 11,000 years old. Older Quaternary alluvial fan and stream deposits (M2, M1) also include calcic soil horizons, from Stage I to Stage IV of Machette (1985). Stage IV is similar to what is informally called 'calcrete' or 'caliche.'

In their hydrogeological interpretation of the upper San Pedro River basin, Pool and Coes (1999) distinguish between the pre- and post-entrenchment alluvium of the upper San Pedro River valley (Layer 1 in their computer model), with the latter including the active sediment in the current alluvial channels. This designation has been used by subsequent hydrogeological analyses that are focused on channel recharge in ephemeral streams (Coes and Pool, 2005; Pool and Dickinson, 2007, Lacher, 2011, 2012).

[2] A *paleosoil* is a former soil preserved by burial underneath sediments or other subsequent deposits.

The down-cutting and entrenchment of the San Pedro River is a historic event recorded in the morphology of the river (see Hereford and Betancourt (2009) for a recent summary). It now appears that this event occurred between 1890 and 1908 (Hereford, 1993). This would suggest a correlation between the pre-entrenchment alluvium of Pool and Coes (1999) and the Y1, <100-year-old alluvium of Dempsey and Pearthree (1994).

Pool and Dickinson (2007) assigned a hydraulic conductivity of 7.5 m/d (24.6 ft/d) to the pre- and post-entrenchment stream alluvium (Layer 1). Callegary and co-workers (2007) measured electrical conductivity in the active alluvium of Woodcutter and Coyote Wash and related it to textural properties of the alluvium. These data were used to infer reaches of the stream of high or low recharge potential. Stewart (2012) looked at the same washes, plus Graveyard Wash and used streambed transmission losses to evaluate recharge potential of the alluvium; however, neither of these studies attempted to relate channel recharge to alluvial stratigraphy other than to note that changes downstream might relate to calcium carbonate cementation.

Actual measurements of hydraulic conductivity, both field and laboratory, were made by GSA (2001, 2004) at the 3rd Street Wash and at the Fry Basin sites. Surface soil hydraulic permeability was measured using the in-situ cylinder infiltrometer method of Bouwer et al. 1999). Sites in the Coyote Wash watershed ranged from 13.3 to 0.2 ft/d and Woodcutter Wash sites ranged from 1.7 to 0.2 ft/d (GSA, 2004), much lower than the regional estimate used by Pool and Dickenson (2007).

Another attempt at estimating channel recharge in area washes was completed by GSA (2004) using an application of the AGRA/KINEROS model (Semmens et al, 2007) to Coyote Wash. The study was primarily focused on developing a set of regression equations that would predict the amount of recharge available at downstream retention ponds based upon runoff contributing areas. The model evaluated hydraulic conductivities of 2.0 and 0.5 ft/d in the ponds. The model was not intended to consider variable channel recharge in the channels due to changes in alluvial or deeper geologic material.

Using the data of the NRCS Soil Survey, the City has projected the hydraulic conductivity of surface and near-surface units across the three watersheds to estimate initial abstraction of precipitation in rainfall-runoff models (Humphreys, unpublished data). The NRCS soil maps (NRCS, 2015) indicate that four soil map units are found within the city limits. The City used soil profiles for the map units to calculate a limiting composite hydraulic conductivity of 0.24 ft/d to 0.80 ft/d for models.

The City estimates maybe the best general picture of the three-dimensional spatial distribution of recharge potential. It might be expected that a combination of the surficial geology mapping (Dempsey and Pearthree 1994) and the NRCS soils maps might be a method for locating areas along the City washes that would be ideal for locating infiltration-enhancing GCSs. For example, areas where coarse-grained surface units, determined from surface geology, combined with an absence of deeper fine-grained units, from the soils map, might suggest areas favorable for construction of recharge-enhancing GCSs.

Figure 1 is a clipped portion of Dempsey and Pearthree (1994). Holocene channel, terrace, and alluvial-fan deposits (units Y, Y1, and Y2) can be traced as linearly elongated areas closely spaced to the active washes. Upper to uppermost Pleistocene, alluvial-fan and valley-fill deposits of unit M2 (including units M2a and M2b) and coarse middle Pleistocene (M1) alluvial fan remnants underlie most of the City, with some suggestion that the stratigraphically higher M2 deposits are located upslope of the M1. In some deeply incised places, the oldest alluvial units (O and Ogr) are exposed, coarse gravelly fan remnants of early Pleistocene age.

Although textures of these units include coarse-grained material that may have high hydraulic conductivity, i.e. the 13.3 ft/d measurements recorded by GSA (2004), the occurrence of these units at the surface does not assure that they persist at depth. The NRCS soils maps include data from deeper levels (Figure 2). Map Units 97, 40, 149 and 71 are mapped across the City. Unit 40 dominates the City area. Made up of 'gravelly loam,' the hydraulic conductivity assigned to the unit by the City analysis is 0.80 ft/d. Unit 97 follows the drainage pattern; however, the presence of a clay layer from 1 to 13 ft below the surface limits the assigned hydraulic conductivity to 0.24 ft/d (NRCS, 2015). The other two map units, 149 and 71, were assigned a value of 0.80 ft/d in the City analysis.

The City used these estimates as regional parameters for a surface hydrology model. It is difficult to assess how relevant these values are on a site-specific basis. Since Unit 97 has a lower hydraulic conductivity, exploring washes underlain by Unit 40 might be more successful at locating recharge-favorable GCSs; however, the variable occurrence of fine-grained material in the units may be problematic.

Compounding the problem of textural variability in these units is the extensive calcium carbonate cementation in the soils and alluvium of the Sierra Vista area. The formation of calcic soils is widespread in the southwestern US (Machette 1985), southeastern Arizona (Mc Fadden, 1978; Morrison, 1985; McAuliffe, 1994) and particularly in the Sierra Vista area (Dempsey and Pearthree, 1994; Huckleberry, 1997). As previously noted, Dempsey and Pearthree (1994) use the development of calcic horizons to age-date buried soils of the Sierra Vista region. Since all of these buried soils were once at the surface, this suggests that calcic horizons could occur at any depth in the region and may increase with the age of the unit through dissolution and re-deposition of calcium carbonate. Such a phenomenon is common (Machette 1985).

3.0 RECOMMENDATIONS

Based upon this very limited examination of the published geology of the Sierra Vista area, it appears that areas favorable for recharge may be uncommon within the City; however, there does not appear to be enough data to make a clear interpretation. Although material of sufficient permeability to allow for surface infiltration can be found on the surface, the presence of older calcic paleosoils at depth, could limit the deeper percolation of surface water.

If GCSs are to be used to facilitate enhanced channel recharge, a hydrogeological site investigation would be required that included borehole permeameter testing, similar to that employed by GSA (2004) at the Third Street Basin or at the Palominas/Schoolhouse Wash area (Milczarek et al, 2012).

Although site-specific subsurface data will be needed, a surface mapping of the existing washes might help plan for the selection of candidate sites. Using the geological criteria presented in Dempsey and Pearthree (1994) and Huckleberry (1997), along with a good understanding of calcic soils, a qualified geologist should be able to map the Quaternary units in major washes on a more detailed scale. This would help determine the relative age of inactive, pre-Holocene alluvium and the likelihood of encountering calcic horizons in the near surface. This could help screen out reaches that are unlikely to accommodate much channel recharge.

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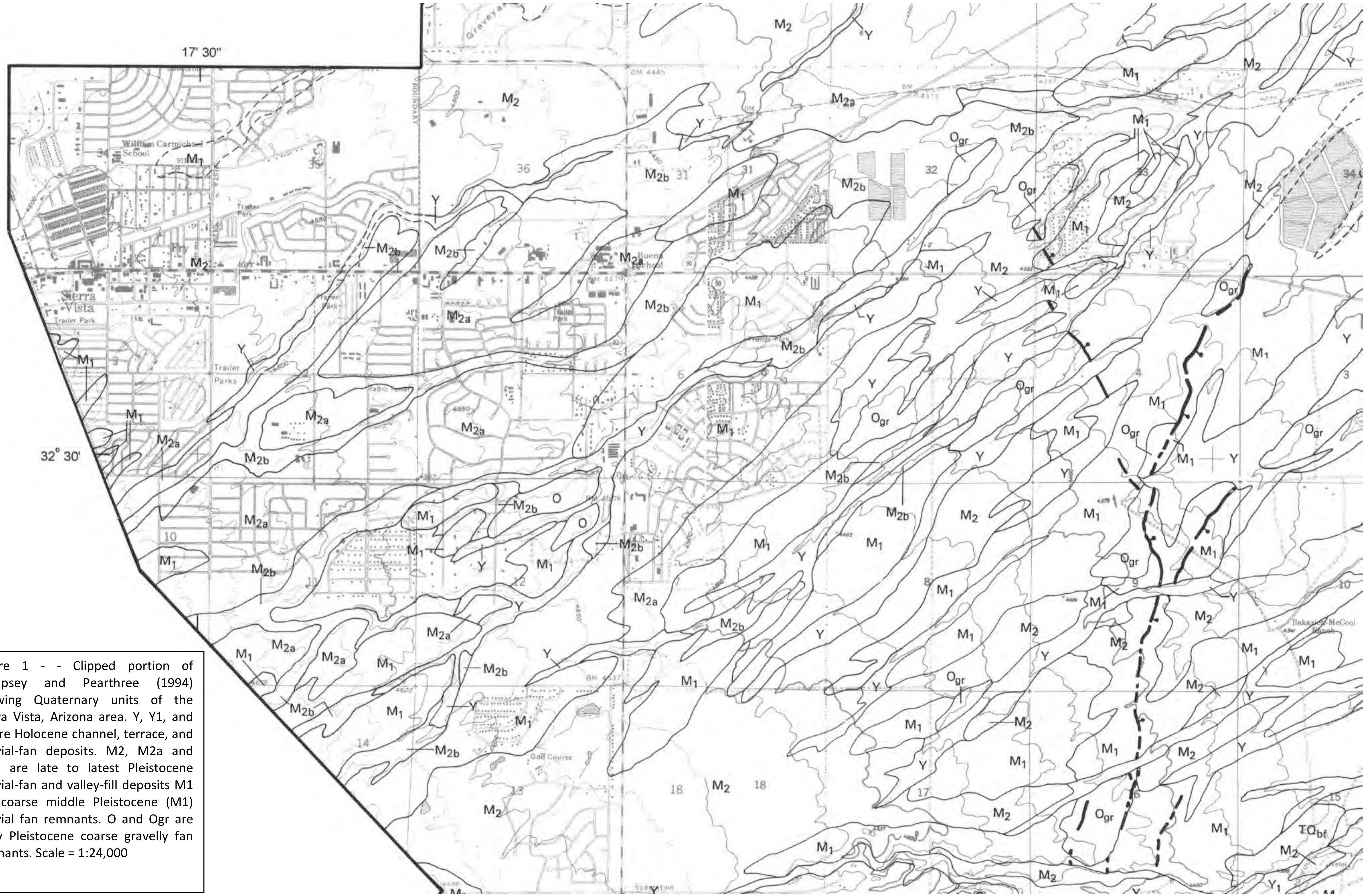

Figure 1 - - Clipped portion of Dempsey and Pearthree (1994) showing Quaternary units of the Sierra Vista, Arizona area. Y, Y1, and Y2 are Holocene channel, terrace, and alluvial-fan deposits. M2, M2a and M2b are late to latest Pleistocene alluvial-fan and valley-fill deposits M1 are coarse middle Pleistocene (M1) alluvial fan remnants. O and Ogr are early Pleistocene coarse gravelly fan remnants. Scale = 1:24,000

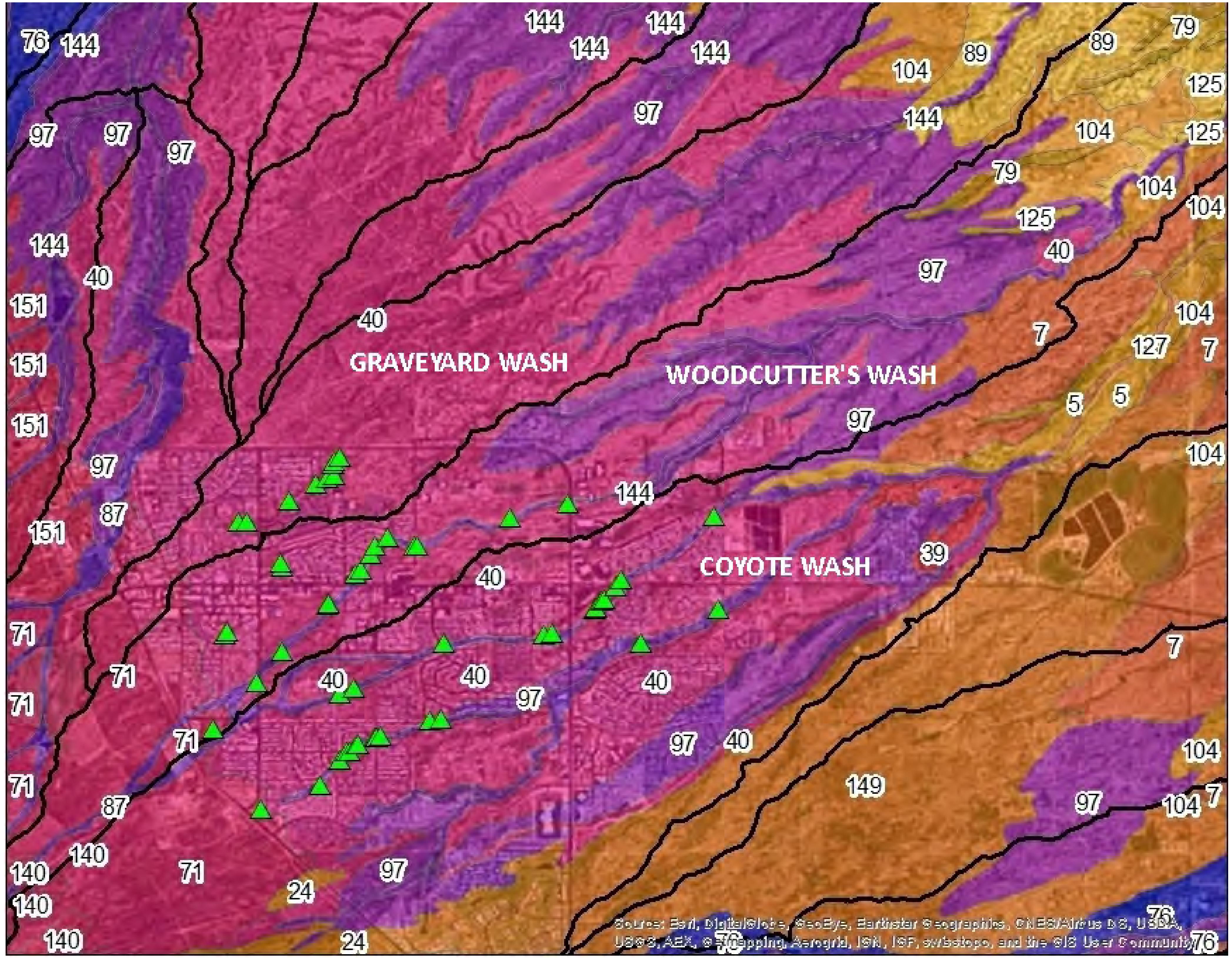

Figure 2 - - Portion of the USDA, NRCS Soil Survey Map for Cochise County, Douglas-Tombstone. Soil map units are discussed in text. Existent City of Sierra Vista grade-control structure locations indicated by green triangles. Scale (get from Cydney)