Floodplain Analysis for Hacienda Del Sol Wash

For

Pima County Regional Flood Control

March 4, 2008

Floodplain Analysis for Hacienda Del Sol Wash

Introduction

This study was performed to provide drainage information for Hacienda Del Sol Wash. The wash runs through Sections 13, 14, 15, 16, 20 and 21 of Township 13 South, Range 14 East, Pima County, Arizona. The entire watershed of the wash is in FEMA Zone X, as shown on the current Flood Insurance Rate Map (FIRM) number 04019C-1645K. While the watershed is in the Zone X, which is defined as non-flood hazard areas, a potential flood and erosion hazard is known in foothills areas where Hacienda del Sol Wash is located. Limited drainage information is currently available in foothills areas.

The purpose of this study is to provide flood and erosion hazard information for Hacienda del Sol Wash. Topographic, hydrologic and hydraulic analyses were performed to determine drainage conditions in Hacienda del Sol Wash. ArcGIS, Version 9.2, Pima County Hydrology Procedures (PC-Hydro), Version 5.3.1, HEC-RAS 4.0 Beta version (HEC-Ras), and HEC-GeoRas, Version 4.1.1 (HEC-GeoRas) were used for the analyses.

Description of the watershed

Most of the watershed of Hacienda del Sol Wash is located in Pima County, while the southern part of the watershed is in the City of Tucson (south of River Rd. Fig. 1). The watershed is 421.8 acres with a mean slope of 0.021. The watershed is located in a suburban foothills area covered with desert brush. Riparian B riparian habitat is mapped along the main channel, based on the 2005 Pima County Ordinance (Fig. 1). Zoning classification of the most of the watershed is CR-1 (single residence zone; Fig. 2).

Data processing procedures

Data processing method is summarized in Fig.3. Triangular Irregular Network (TIN) derived from Light Detection and Ranging (LiDAR) data was used to create a topographic map (Fig. 4). The contour interval is 5 feet. The boundary of the watershed and slope break points were determined using the derived contour lines. The locations of the stream centerline, cross-sections, river banks, culverts, and other physical attributes of the wash were determined by using a topographic map and 2005 PAG aerial photo (Fig. 4). Those physical attributes of the wash were digitized in ArcGIS with an extension of HEC-GeoRAS and then exported as an ASCII text file. The data was imported into HEC-RAS to create geospatially referenced geometric data. Other parameters for the steady analysis, such as Manning's n-vales, culvert data, expansion and contraction coefficients, normal depth boundary condition, ineffective flow areas, and peak discharge rates obtained from PC-Hydro are manually input into HEC-Ras. The hydraulic data obtained from HEC-RAS are imported into HEC-GeoRAS to delineate a floodplain in the study area.

Hydrologic analysis

The 100-year return interval peak discharge rates for the entire watershed and sub-watershed were computed by using PC-Hydro (Arroyo Engineering, 2007). NOAA Atlas 14 Upper 90% Confidence Interval rainfall data were used for the analysis. Hydrologic soils group map for the watershed is presented in Fig. 5. Hydrologic Soil Group B is the dominant soil type in the watershed, which occupies 82% of the watershed. The watercourse was divided into ten segments (Reaches) using slope break points. The basin factor for each segment was determined by using a 2005 PAG aerial photo (Figs. 6-7.10). Basin Factors were based on the tables in the PC Hydro User Guide. The Basin Factor ranges from 0.04 to 0.045 which corresponds to Suburban-Valley (< 1 house/acre; Arroyo Engineering, 2007). A vegetation cover density of 30% was used to select the SCS Curve Number for the hydrologic calculation of the study watershed. Impervious cover percentage was 15%, which was determined using a 2005 PAG aerial photo.

The results of hydrologic analysis are included in Appendix 1. The 100-year peak discharge rates for the entire watershed and the sub-watershed are 806 and 411 cubic feet per second (cfs), respectively. Time of concentration for the peak discharge is 53.5 minutes for the Hacienda del Sol watershed and 33.4 minutes for the sub-watershed. Rainfall intensity at the time of concentration is 3.02 inches per hour for the entire watershed and 4.24 inches per hour for the sub-watershed.

Hydraulic analysis

Steady flow analysis was performed to determine 100-year water surface elevations in the study area by using HEC-Ras. The study area is from the south of Camino Sumo to the confluence with Rillito River. As described above, geometric data for HEC-RAS including stream centerline, cross-sections, river banks, culverts, and block obstructions were obtained by using HEC-GeoRas. The locations of cross sections used for the analysis are show in Fig. 8. There are three culverts in the study area (Fig. 8: Pictures of the culverts are included in Appendix 2). The culverts were named Culvert #1, 2, and 3 from upstream to downstream. The dimensions of the culverts were summarized in Table 1. The data was obtained from Pima county Department of Transportation and Flood Control District improvement plans (CO 12-87-08, Co 12-87-08, and 4TRRCA, Appendix 3). The Culvert #1, 2, and 3 were designed to convey a 100-year discharge of 1611 cfs, 897 cfs, and 661 cfs, respectively.

Manning's roughness coefficients for the main channel and the over-bank areas were determined by using a 2005 PAG aerial photo. An area-weighted average of the Manning's roughness coefficients for the main channel and the over-bank areas was used to determine a water surface elevation at each cross section. The weighted average of the roughness coefficients was determined by estimating area for a main channel and over-bank areas and assigning weighing factors that are proportional to the total area (Jacb

Dividian, USGS, Technical of Water-Resources Investigations, Book 3, Chapter A15, see Appendix 4). The method to assign composite n value to an entire cross section is used in Maricopa County (Phillips and Tadayon, 2006)

Entrance loss coefficient and Manning's roughness coefficient of the culverts are obtained from HEC-RAS Hydraulic Reference Manual version 3.1. Culvert #3 has a horizontal bend between the entrance and exit. Bend can result in additional losses for the culvert operating under outlet control (Floodplain Online Book, provided by Haestad Methods, Inc.). The detail of the method is provided in Appendix 5. The bend loss was estimated following the method described in Floodplain Online Book. Angle of bend is approximately 30 degree (development plan 4TRRCA). The culvert opening is assumed to be equivalent to 3.56 feet diameter, based on the opening of Culvert #4. The estimated entrance coefficient for Culvert #3 is 0.65.

Contraction and expansion coefficients are 0.3 and 0.5 for just upstream and downstream of culverts, and 0.1 and 0.3 for other cross sections, which were obtained from HEC-RAS Hydraulic Reference Manual. Normal depth with a slope of 0.02 was used as a boundary condition for the steady flow analysis.

The results of hydraulic analysis are shown in Figs. 9, 10 and 11. The results of hydraulic analysis using HEC-RAS were summarized in Appendix 6. Fig. 9 shows the profile of a 100-year water elevation. Fig. 10 shows a spatial variation of flood depths in the study area. Fig.11 shows a floodplain limit in the study area. The flood prone area becomes relatively wide between Culvert #2 and Culvert #3 (Figs. 10 and 11). There is a divided flow at approximately 500 feet upstream of River Rd, based on the steady flow analysis using HEC-RAS. The flood elevation exceeds 4 feet around the downstream end of a tributary, at the upstream of Culvert #1 and 2, and at the downstream of Culvert #3 (Fig. 10).

References

Floodplain Online Book, provided by Haestad Methods, Inc http://www.haestad.com/library/books/FMRAS/FloodplainOnlineBook/javascript/wwhelp/wwhimpl/common/html/wwhelp.htm?context=Floodplain_with_HEC_RAS&file=Floodplain%20with%20HEC-RAS-16-10.html

HEC-RAS Hydraulic Reference Manual version 3.1, US Army Corps of Engineering Center, November 2002.

Jacb Dividian, USGS, Technical of Water-Resources Investigations, Book 3, Chapter A15

PC-Hydro User Guide, PC-Hydro V5 Pima County Hydrology Procedures, A couputer program for predicting peak discharge of surface runoff from small semi-arid watersheds in Pima County, Arizona, Arroyo Engineering, LLC, 2007

Phillips J., and Tadayon, S., Selection of Manning's Roughness Coefficient for Natural and Constructed Vegetated and Non-Vegetated channels, and Vegetation Maintenance Plan Guidelines for Vegetated Channels in Central Arizona, Flood Control District of Maricopa County, Scientific Investigations Report 2006-5108.

The method to estimate a bend loss is described below. The article (Floodplain Online Book, provided by Haestad Methods, Inc) can be found at http://www.haestad.com/library/books/FMRAS/FloodplainOnlineBook/javascript/wwhelp/wwhimpl/common/html/wwhelp.htm?context=Floodplain_with_HEC_RAS&file=Floodplain%20with%20HEC-RAS-16-10.html

Horizontal Bends in Culverts.

Although most culverts are uniform in shape, size, and slope from the upstream to downstream end, there are exceptions.

A culvert can have one or more horizontal bends between its entrance and exit to bypass utility lines or other in-place items. Bends can result in additional losses for culverts operating under outlet control. If the bends are less than 15 degrees and are at least 50 ft (15 m) apart, the additional losses are considered insignificant and can be neglected (FHWA, 1985). When bends do not meet these criteria, there are additional culvert losses. Bend losses (vertical or horizontal) can be estimated using

$$h_b = K_b \frac{V^2}{2g}$$

where	h_b	=	the loss in each bend (ft, m)
	K_b	=	the bend loss coefficient (dimensionless)
	V	=	culvert velocity under full conduit flow (ft/s, m/s)

(7.8)

The bend loss coefficient is a function of the angle of the bend and the size of the culvert. Table 7.4 lists estimates of K_b for culverts flowing full. The cross-sectional area of noncircular shapes may be used to compute an "equivalent diameter" for that shape.

Table 7.4 Bend loss coefficients for culverts flowing full (Linsley et al., 1992).

Equivalent Diameter, ft	**Angle of Bend, deg.**		
	90	**45**	**22.5**
1	0.50	0.32	0.25
2	0.30	0.22	0.15
4	0.25	0.19	0.12
6	0.15	0.11	0.08
8	0.15	0.11	0.08

Example 7.3 Bend losses in a culvert.

A 5-ft diameter culvert is 250 ft long and has a 30-degree bend at a point along its length. For a discharge of 150 ft^3/s, compute the bend loss, assuming that the culvert is flowing full.

Table 7.4 is used to linearly interpolate a bend loss coefficient of 0.117, rounded to 0.12, for the culvert diameter and bend angle. The velocity in the pipe, assuming full pipe flow, is $V = Q/A$ = 150/19.63 = 7.64 ft/s. The bend loss is then determined with Equation 7.8 to be

$$K_b = 0.12\frac{7.64^2}{2g} = 0.11 \text{ ft}$$

.

In HEC-RAS, bend losses are not part of the information entered into the Culvert Data Editor. Therefore, to include this bend loss in the culvert data, 0.12 could be added to the entrance loss coefficient.