F-10/D006A SANTA FE VILLAGE

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

RECEIVED
AUG 5 1992
HYDROLOGY DIVISION

August 4, 1992

CERTIFICATE OF COMPLETION AND ACCEPTANCE

Sandy Thompson
SMC Properties, Inc.
P.O. Box 11519
Albuquerque, NM 87192

RE: PROJECT NO. 3116.90 , SANTA FE VILLAGE UNIT I PHASE V, (MAP NO. F-10)

Dear Mr. Thompson:

This is to certify that the City of Albuquerque accepts Project No. 3116.90 as being completed according to approved plans and construction specifications. If all required rights-of-way and/or easements have been dedicated, the City of Albuquerque will accept for continuous maintenance all public infrastructure improvements constructed as part of Project No. 3116.90. If the required rights-of-way and/or easements have not been dedicated, the City of Albuquerque cannot accept the project for continuous maintenance and said maintenance will be the responsibility of the developer. When a final plat has been filed it will be the developer's responsibility to provide the Construction Management Division with a copy, at which time the City will fully accept Project No. 3116.90.

The project is described as follows:

- The installation of water lines, sanitary sewer lines, storm sewer lines and inlets, curb and gutter, and paving in Santa Fe Village Unit I, Phase V.

- The contractor's correction period began July 8, 1992, and will be effective for a period of one (1) year.

Sincerely,

Andre Houle, P.E.
Division Manager
Design/Construction Division
Engineering Group
Public Works Department

AH:tp

PRUDENT LINE ANALYSIS
FOR THE
SOUTH BRANCH OF SAN ANTONIO ARROYO
ALBUQUERQUE, NEW MEXICO

prepared for

Roberson Construction, Company
6001 Atrisco Road, NW
Albuquerque, New Mexico 87120

by

Resource Technology, Incorporated
2620 San Mateo NE, Suite B
Albuquerque, New Mexico 87110

April 3, 1986

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INTRODUCTION

This report supplements a previous report titled "Drainage Report for Santa Fe Village Unit III, Phases I and II, Albuquerque, New Mexico." That report was prepared by Resource Technology, Inc. (RTI) and was submitted to the City of Albuquerque Design Hydrology Section on October 21, 1985. The Design Hydrology staff reviewed the report and had several relatively minor questions and one requirement which had to be resolved before approval of the report is granted. The minor questions have been dealt with and this report responds to the requirement that a Prudent Line Analysis be performed on the unlined reach of the South Branch of San Antonio Arroyo. This reach extends west from the Vulcan Parkway crossing to the toe of the volcanic escarpment, a distance of 1,015 feet, as shown in Figure 1.

Resource Technology, Incorporated, has conducted the Prudent Line Analysis in consultation with Dr. Peter Lagasse and Dr. James Schall, the principal authors of "Erosion Study to Determine Boundaries for Adjacent Development - Calabacillas Arroyo, Bernalillo County, New Mexico".

The concept of Prudent Line Analysis was applied by Simons, Li and Associates (SLA) to the Calabacillas Arroyo Channel and is described in the report just mentioned. Their methods were developed to determine boundaries (called offset tangents) along both sides of the channel within which development would not be prudent.

The procedure for Prudent Line Analysis used in this study is similar to that used in the Calabacillas Arroyo Study. A multiple level solution which "stresses that knowledge of governing physical processes plays the most important part in determining an appropriate level of mathematical analysis" was used. Only two levels of analysis were considered for this study. Level I analysis is qualitative and is based on application of geomorphic principles. Level II is quantitative and based on more complex geomorphic relationships and basic engineering relationships. A Level III analysis which is quantitative and based on detailed mathematical modeling was not considered necessary for the Calabacillas Arroyo, and consequently was not performed in the present study.

HYDROLOGY

A range of peak discharges for corresponding recurrence intervals is necessary to perform the Prudent Line Analysis.

The 100-yr. peak discharge for the 855-acre watershed of the South Branch of San Antonio Arroyo is 762 cfs. with a time to peak of 1.5 hours at Atrisco Road NW. This value was determined previously by Fred Denney and Associates, Inc. in "Hydrologic Report San Antonio - Mariposa and Boca Negra Arroyos" (Oct. 1980). The City of Albuquerque and AMAFCA have approved the report, and the hydrology presented in that report was used in this Prudent Line Analysis. ***

Resource Technology, Incorporated has previously performed hydrologic analyses on several basins on the West Mesa for the SPF, 100-yr., 50-yr., 5-yr. and 2-yr. frequency floods. One such basin is that for Boca Negra Arroyo which lies approximately 1 mile north of the South Branch of San Antonio Arroyo basin. The results for one of the sub-basins of Boca Negra Arroyo, which has very similar basin characteristics to those of the Middle Branch of San Antonio Arroyo, were used in this study. The area of this sub-basin, as shown in Figure 2 is 1,837 acres with a 100-yr. peak discharge of 1,237 cfs. and a time to peak of 1.52 hours. The shape of the sub-basin is also similar to that of the South Branch of San Antonio Arroyo.

To develop a complete frequency range of hydrologic data for the South Branch of San Antonio Arroyo, the 2-yr., 5-yr. and 50-yr. peak discharges were determined as follows. The two basins (Boca Negra sub-basin and South Branch of San Antonio Arroyo) are similar in physical and hydrologic characteristics. To substantiate that the two basins are similar in hydrologic characteristics, the ratio of the 100-yr. storm runoff volumes was compared to the ratio of the drainage areas as follows:

1. 100-yr. storm runoff volume ratio -
   South Branch of San Antonio Arroyo/Boca Negra Sub-basin =
   87.64 ac.-ft./192 ac.-ft. = 0.46

2. Drainage area ratio -
   South Branch of San Antonio Arroyo/Boca Negra Sub-basin =
   855 ac./1,837 ac. = 0.47

Therefore the two basins produce almost exactly the same volume of


BOCA NEGRA SUB-BASIN
SOUTH BRANCH OF
SAN ANTONIO ARROYO BASIN
BOCA NEGRA SUB-BASIN
Water Tank
TOWN OF ATRISCO GRANT NORTH

FIGURE 2

water per unit area and thus both areas may respond almost the same to a hydrologic event.

Based on this discussion a factor of 0.46 was used to reduce the previously determined 100-yr., 50-yr., 5-yr. and 2-yr. peak discharge for the Boca Negra Sub-basin to determine the same frequency peak discharge for the South Branch of San Antonio Arroyo. Table 1 gives the results of this procedure.

These data were used to develop a flood frequency vs. discharge curve for the South Branch of San Antonio Arroyo, Figure 3. The 10-yr. and 25-yr. peak discharges were determined from Figure 3 as 265 cfs. and 379 cfs., respectively.

The peak discharges given in the top section of Table 1 are those for the total basin area of the South Branch of San Antonio Arroyo. However, the Prudent Line Analysis will only be performed for the drainage area contributing flow to Vulcan Parkway. Therefore the peak discharges at Vulcan Parkway were determined by a peak discharge to drainage area ratio. The 100-yr. peak at Vulcan Parkway equals (730 ac./855 ac.)(762 cfs.) = 651 cfs. The peak discharge for the other frequency storms were determined the same way and the bottom of Table 1 gives the adjusted peak discharges to be used in the Prudent Line Analysis.

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FLOOD PLAIN ANALYSIS

The HEC-2 Water Surface Profiles computer program developed by the

U.S. Army Corps of Engineers was used to determine the flow characteristics of the arroyo channel for the 100-yr., 25-yr., 10-yr., 5-yr., 2-yr. and 1/2(2-yr.) peak flows in the existing channel. Manning's roughness values used in this study are 0.03 for most cross section overbanks with grasses and small plants 0.05 was used for some cross sections with extremely dense bushes, 3 to 4 feet tall; and 0.035 was used for cross-sections with grasses, small plants and a few 3 to 4-foot tall bushes. Cross-section 11 is located on the escarpment and 0.08 was used in this section to account for the large basalt boulders.

This report defines the 100-yr. flood plain from the base of the escarpment to Vulcan Parkway more accurately than previous reports because the analysis is based on field surveyed cross-sections of the arroyo.

Figure 1 shows the location of the cross sections used in the HEC-2 program for the existing arroyo. The HEC-2 program was run assuming subcritical flow for the existing arroyo channel; in those reaches where the flow went to a supercritical condition, critical depth was used to define the flood plain limits because supercritical flow is very unstable. Table 2 contains the summary output for the 100-yr. and 10-yr. peak flows. The resulting 100-yr. flood plain limits are also shown in Figure 1. Figure 4 shows the 100-yr. water surface profile.

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QUALITATIVE GEOMORPHIC ANALYSIS

Qualitative geomorphic analysis involves the understanding of basic physical processes of watershed and arroyo response. To develop this understanding, field investigations were performed by RTI staff. Other qualitative analyses included comparision of aerial photographs taken over a period of several years, bed and bank material samples and composition, and arroyo profile analysis. All of the above qualitative analyses comprise the bulk of Level I analysis for Prudent Line definition.

Site Observation

RTI staff visited the site to assess the physical condition and attributes of the South Branch of San Antonio Arroyo and the surrounding watershed. Potential problem areas were noted and geomorphic features within the arroyo and surrounding flood plain were observed.

The site observation extended from the watershed area above the escarpment to Atrisco Road. Detailed investigation began at the base of the escarpment and continued to Vulcan Parkway which has been staked out in the field. The arroyo above the escarpment is not well defined. Below the escarpment the channel is well defined from the base of the escarpment to the large meander (see Figure 1) and then becomes less defined downstream to the AMAFCA gabion grade control stucture at Atrisco Road.

The soil type within the study reach below the escarpment is the Bluepoint-Kokan association (BKD) as presented in the U.S. "Soil Survey for Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico". The soil appeared to be fine unconsolidated sand.

Beginning at the base of the escarpment the arroyo has a trapezoidal shape with a bottom width of about 40 feet. The north bank has a moderate slope (approximately 30 degrees) with moderate vegetation. The south bank has very steep to vertical faces on 10 - to 15 foot tall buttes which are located at the confluences of the small arroyos from the south bank and the main arroyo. The steep faces on the south have very little vegetation. The low flow channel is very small with a 2-foot width and 3-inch high banks. The bottom of the entire arroyo channel has moderate vegetation density with bushes 2 to 4 feet tall.

East of the escarpment at the confluence of the first small arroyo (Sub-basin J) and the main arroyo, the low flow channel is non-existant. The arroyo cross-section is trapezoidal with the north and south banks similar in slope (approximatley 30 degrees). The arroyo bottom width varies from about 25 to 40 feet. The arroyo bottom has more vegetation than the overbanks with 3 to 4-feet tall bushes becoming more numerous.

Near the confluence of the second small arroyo (Sub-basin I) and the main arroyo, the arroyo is trapezoidal in cross-section and a low flow channel is non-existant. The arroyo bottom is approximately 70 feet wide and densely vegetated with bushes 2 to 4 feet tall. The slopes of the banks are approximately the same (30 degrees), and the banks are moderately vegetated with grasses and tumbleweeds.

Approximately 200 feet west of Vulcan Parkway the arroyo is approximately trapezoidal in cross-section with the bank slopes becoming increasingly mild. The bottom width is approximately 70 feet with moderate vegetation. The 2 to 4- feet tall bushes become sparse. A low flow channel is visible as a 2 - to 4-foot wide shallow depression with no defined banks.

From Vulcan Parkway to the large meander a low flow channel becomes increasing more defined with moderate vegetation density grasses, tumbleweeds and brushes. The arroyo has a defined low flow channel for about 2,000 feet east of the large meander and then becomes poorly defined for the remainder of its distance to the AMAFCA gabion grade control structure.

The Prudent Line Analysis described in this report did not extend downstream (east) from Vulcan Parkway because this section of the arroyo will be concrete lined. Figure 5 is a typical arroyo cross-section west of Vulcan Parkway.

In summary, the soil within the study reach appeared to be fine sandy unconsolidated material, and the vegetation density is moderate to dense throughout the study reach. The arroyo generally has a trapezoidal cross-section with the side slopes becoming increasingly mild with distance from the escarpment. The bottom width also increases with distance from the escarpment. Throughout much of the study reach a defined low flow channel is not visible.

Aerial Photograph Analysis

Aerial photographs spanning a period of several years can provide information on channel geometry changes and historical trends. Four sets of photographs were obtained for the years 1935, 1959, 1967 and 1980.

In 1984, Denny-Gross and Associates developed a set of topographic maps based on photogrammatical methods. However, these photographs were not used because they were so near in time to the 1980 Albuquerque Ortho-photo thus offering very little difference from the 1980 ortho-photo in terms of a long time span.

The 1935 photograph was taken by the U.S. Soil Conservation Service at a scale of 1: 31,680. The 1959 photograph was taken by the U.S. Geological Survey at a scale of 1: 19,400. The 1967 photograph was also taken by the U.S. Geological Survey at a scale of 1: 25,952. The 1935, 1959 and 1967 photos were all enlarged to a scale of 1: 2,400 for comparison in this study. The 1980 map used is the Albuquerque ortho-photo topographic Map No. F-10 which was based on photography by Bohannon-Huston, Inc. at a scale of 1: 2,400.

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Therefore, a period of 45 years can be inspected to determine geomorphic changes of the South Branch of San Antonio Arroyo. Figure 6 is a comparison of the arroyo alignment shown by each photograph. Note that Figure 6 is approximate due to parralax and scaling errors on the enlarged photographs.

The following paragraphs discuss the change in channel configuration determined from the aerial photographs (with reference to Figure 6).

Between the escarpment and Vulcan Parkway the arroyo showed no change except for a very short reach immediately west of Vulcan Parkway. The arroyo alignment determined from the 1935 photo is approximately 10 feet south of the alignment shown on the 1959, 1967 and 1980 photos. Based on the aerial photographs the arroyo alignment in this reach has remained very stable.

East of Vulcan Parkway the arroyo showed varying alignments in 1935 and 1967 near the large meander and other locations. However, this reach of the arroyo is planned to be concrete lined.

Arroyo Classification

The slope between the base of the escarpment and Vulcan Parkway rip-rap transition structure was determined as 1.19 percent. The 2-yr. peak discharge used as the mean annual discharge for arroyo classification is 85 cfs.

Based on a slope - discharge relationship developed by Lane in "A Study of the Shape of Channels Formed by Natural Streams Flowing in Erodible Material" (1957), the South Branch of San Antonio Arroyo is classified as a braided stream.

The arroyo has grade controls at the top and bottom of the study reach. The top of the arroyo reach is controlled by the escarpment. The bottom will be controlled by the proposed rip-rap transition and concrete box culvert at Vulcan Parkway. These controls will therefore limit the net channel grade for the entire reach to 1.19 percent. The net channel grade cannot be lower than this slope, but shorter intermediate reaches could have steeper or lower slopes.

Arroyo Profile Analysis

The arroyo profile was analyzed using the cross-sections which were surveyed in the field in 1985 and the Albuquerque orthophoto topographic Map No. F-10 (1980). The cross-sections were located on the topographic map and Figure 4 shows the profiles from the proposed rip-rap transition at Vulcan Parkway (Section 1) to the base of the escarpment between sections 10 and 11.

Figure 4 shows that the arroyo is degrading very slightly from Section 1 to approximately Section 4. From about Section 4 to the base of the escarpment the arroyo appears to have aggraded since 1980.

Results and Conclusions

The South Branch of San Antonio Arroyo is a relatively straight arroyo with a mild slope (averaging 1.19 percent). The arroyo bottom width varies from 25 to 70 feet and bank heights vary from 15 feet at the base of the escarpment to mildly sloping sides which have no defined top or bottom.

Based on an analysis of aerial photographs spanning 45 years, the arroyo appears to be very stable with respect to lateral migration. The entire reach appeared to be aggrading except for a short reach approximately 100 feet west of Vulcan Parkway. In this reach an extremely small low flow channel was visable which may indicate degradation.

EROSION AND SEDIMENTATION ANALYSIS

Using the flood frequency distribution previously developed by comparison to a tributary of the Boca Negra Arroyo, additional flow frequency rates were selected by interpolation from the frequency curve (Figure 3) so that peak flow data for the 2-, 5-, 10-, 25-, 50-, and 100-yr. floods on the South Branch of San Antonio Arroyo at Vulcan Parkway were established.

Sediment Particle Size Analysis

During the field observation, RTI staff obtained six soil samples. The samples, classified by number and location in the watershed, are 3 bed samples, 1 floodplain sample and 2 watershed samples. A sieve analysis was performed on all samples and gradation curves were obtained. Figure 7 shows the representative bed material size distribution. The flood plain and watershed gradation curves are similar to the representative bed material.

Sediment Continuity

A. Representative Subreaches

The South Branch of San Antonio Arroyo was divided into 3 representative sub-reaches for analysis of sediment continuity. The sub-reaches are numbered beginning at the base of the escarpment and were delineated for physical characteristics of the channel, velocity, sediment gradation and relatively uniform reach lengths, where possible. The sub-reaches are shown on Figure 1 in plan view and on Figure 4 in profile.

B. Sediment Transport Relations

A sediment discharge per unit width equation was developed for the South Branch of San Antonio Arroyo based on the following data.

1. The representative bed material size distribution was broken into 6 intervals and the geometric mean of each interval was determined to be as follows:

Interval	Size Range mm. - mm.	Geometric Mean mm.	% Of Total Range
1	9.6 - 0.4	1.96	10%
2	0.4 - 0.28	0.33	10%
3	0.28 - 0.21	0.24	15%
4	0.21 - 0.15	0.18	20%
5	0.15 - 0.10	0.12	10%
6	0.10 - 0.074	0.086	10%
	< .074	--	25%

***

The D (50) size bed material was also determined as 0.17 mm.

2. The maximum and minimum slopes within the total reach were determined to be 1.95% and 0.69%, respectively.

3. The minimum and maximum unit discharges were determined from the 2-yr. peak Q (85 cfs.) in a section with the largest flow top width (T = 77.13 ft.), and the 100-yr. peak Q (651 cfs.); and in a section with the smallest flow top width (T = 51.91 ft.). Therefore, the minimum unit discharge is 1.1 cfs./ft., and the maximum unit discharge is 12.5 cfs./ft.. These hydraulic data were obtained from the HEC-2 output.

4. The maximum velocity and maximum Froude No. were determined to be 8.97 fps. and 1.09, respectively.

5. Manning's roughness coefficient was assumed to be 0.035.

These data were input into a computer program which solves for the Meyer - Peter Meuller bed load equation combined with the Einstein suspended load equation. The resulting sediment transport per unit width equation is

qs = 0.000062 (Y**0.803)(V**3.992)

where Y = hydraulic depth and V = velocity.

Using this unit sediment discharge equation, the total sediment transport capacity of each sub-reach was determined as follows:

Step 1 -

The HEC-2 computer program was run for the total reach (including the proposed rip-rap transition into the box culvert at Vulcan Parkway) for the following discharges.

(1/2)	2-yr. Qp =	43	cfs.
	2-yr. Qp =	85	cfs.
	5-yr. Qp =	169	cfs.
	10-yr. Qp =	226	cfs.
	25-yr. Qp =	324	cfs.
	100-yr. Qp =	651	cfs.

For each sub-reach and each Qp above, the qs was determined (using the derived equation above) for each cross-section.

Step 2 -

The average qs for the cross-sections within a sub-reach was then multiplied by the average top width of the sections in the sub-reach to obtain the total sediment discharge (Qs) for that sub-reach.

Therefore, the range of peak discharges (Qp) and the corresponding sediment discharges (Qs) were determined for each sub-reach. A regression equation of the form

$$Qs = a\ Qp^{b}$$

***

was then developed for each sub-reach. Coefficients a and b were derived for each sub-reach.

Step 3 -

Triangular shaped hydrographs were developed from the runoff hydrographs of the 2-yr., 10-yr., 25-yr., and 100-yr. storms as shown on Figure 8. The procedure for determining the triangular hydrographs is as follows.

1. The volume under the triangular hydrograph and the runoff hydrograph must be equal.

2. The rising and falling limbs of the triangular hydrograph should be equal (the triangular hydrograph is symmetrical about the peak).

This procedure along with the fact that a runoff hydrograph has more volume to the right of the peak than to the left (not symmetrical about the peak) accounts for the lagged triangular hydrograph peak as shown in Figure 8.

The total duration of each triangular hydrograph (d) was then determined from Figure 8.

Step 4 -

The total sediment transport capacity per storm for each sub-reach was then determined using an equation of the form

$$Qs = (a\ Qp^{**}b)\ d/(b+1)$$

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where a and b were previously derived, d = the total duration of the triangular hydrograph in seconds, Qp = the peak discharge in cfs., and

Qs = the sediment transport capacity in cu./ft.

C. Storm Related Aggradation/Degradation

The aggradation/degradation volumes for each reach were determined using the sediment transport capacity of each reach by application of sediment continuity principles. In short, if the transport capacity of an upstream sub-reach is greater than that of a downstream sub-reach, then deposition will occur in the downstream sub-reach. If the transport capacity of the upstream sub-reach is less than the downstream sub-reach, then erosion will occur in the downstream sub-reach until the volume of sediment being transported equals the transport capacity (Qs).

For aggrading reaches, a bulking factor of 0.6 was applied to account for compaction of the sediment. Therefore, the volume of deposited sediment was assumed to be 60 percent greater than the volume of sediment removed from the water flow. However, in degrading reaches the volume of sediment eroded and the volume in the water flow were assumed to be equal.

Table 3 lists the aggradation or degradation volume for each sub-reach for the 2-, 10-, 25-, and 100-yr. floods.

Equilibrium Slope Analysis

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Equilibrium slope is determined by assuming that, if over the long term there are no changes in channel geometry or sediment supply, an equilibrium slope would be achieved. Equilibrium slope implies that Qs(in) = Qs(out), therefore no aggradation or degradation would occur.

The unit sediment transport equation derived previously was based on factors for the entire reach and therefore is applicable for equilibrium slope determination. The following equation (developed by Simons Li and Associates) was used to determine the equilibrium slope for each reach.

$$\text{Seq.} = \text{Sex.}\ (\text{Qs supply}/\text{Qs capacity})^{**}(2/(b-x))$$

where $x = 0.6(0.666\ b + c)$
Seq. = equilibrium slope
Sex. = existing slope
Qs supply = supply of sediment to the reach
Qs capacity = sediment transport capacity of the reach

b and c are defined in the unit sediment transport equation as the coefficients of velocity and depth respectively.

The unit sediment transport equation is

$$qs = 0.000062(Y^{**}0.803)(V^{**}3.992)$$

Therefore b = 3.992, = 0.803
and consequently x = 2.079 and 2/(b − x) = 1.045

The resulting equilibrium slope equation is

$$\text{Seq.} = \text{Sex.}\ (\text{Qs supply}/\text{Qs capacity})^{**}1.045$$

Table 4 lists the results of the equilibrium slope analysis.

Definition of the Representative Storm

As discussed previously, erosion potential was considered for the short term (100-yr.) and the long term (25-yr.) which is the cummulative effect of many storms over a 25-yr. period.

The long term (25-yr.) erosion potential is based on the representative annual flood multiplied by 25 years. Simons Li & Associates developed the following equation "which considers the annual probable occurrence of storms with various return periods."

$$(Vols\ r) = 0.01\ (Vols\ 100) + 0.01\ (Vols\ 50) + 0.02\ (Vols\ 25) + 0.06\ (Vols\ 10) + 0.1\ (Vols\ 5) + 0.3\ (Vols\ 2)$$

where Vols = sediment yield
r = representative annual storm
100, 50, 25, 10, 5, 2, = recurrence interval

Sediment volumes were determined previously for the 100-, 25-, 10-, and 2-yr. storms and the values for the 50- and 5-yr. storms were obtained by interpolation.

Table 3 lists the representative annual aggradation/degradation volumes and the resulting long term (25-yr.) aggradation/degradation volumes.

LATERAL MIGRATION ANALYSIS

The lateral migration potential for each sub-reach was determined based on the qualitative geomorphic analysis, sediment continuity, and equilibrium slope analysis. Sub-reach 1 not if supply reach and 2 will aggrade over the short and long term and consequently the erosion potential beyond the 100-yr. flood limits is unlikely.

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Vertical bank failure is possible in Sub-reach 1 along the south bank at the vertical walls between Sub-basin J and the base of the escarpment (see Figure 1). However, no development is planned west of Sub-basin J. Vertical bank failure is possible along the steep faces of Sub-basin I at the location of the apartment building (Figure 1). The problem will be eliminated by the construction of a gabion retaining wall which will provide protection from bank failure.

Vertical bank failure will not occur in any other locations within the study reach due too very mild side slopes (see Figure 5). Therefore the 100-yr. flood plain boundary as shown on Figure 1 is the prudent line for sub-reaches 1 and 2.

Sub-reach 3 will degrade under both the short and long term conditions. Sub-reach 3 will have the most degradation in the long term and therefore the lateral migration distances were determined for the long term.

The total degradation volume within Sub-reach 3 was assumed to be eroded from the banks and none from the channel to give a conservative estimate of the distance for lateral migration. Furthermore, because the direction of lateral migration is not known for certain, the total volume of sediment was assumed to come from either bank. The sediment was asssumed to erode uniformly throughout each sub-reach. The distance from the thalweg to the erosion face for each cross-section as given in Table 5 was determined as follows.

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The volume of sediment eroded between cross-sections was divided by the channel distance between cross-sections, to determine the average cross-sectional area that could be lost to provide the volume of sediment. Using the cross-section plots and assuming an angle of repose of 26 degrees (2H:1V), the required cross-sectional area and consequently the distances from the thalweg were determined.

The channel relocation discovered in the 1935 aerial photograph as shown on Figure 6 lies within the lateral migration limit.

OFFSET TANGENTS

As discussed previously, sub-reaches 1 and 2 are in an aggrading mode and historical evidence (aerial photograph analysis) indicates that the arroyo will remain within the 100-yr. flood plain boundaries as shown on Figure 1. Therefore, offset tangents were not determined in sub-reaches 1 and 2 because the 100-yr. flood plain is the prudent line. Consequently, a proposed drainage right-of-way including the entire 100-yr. flood plain was plotted for future platting purposes.

Sub-reach 3 will degrade according to the sediment continuity analysis and the equilibrium slope analysis and therefore offset tangents were determined. The offset tangent lines in this sub-reach as determined by the lateral migration analysis are shown on Figure 1. The offset tangent lines generally exceed the 100-yr. flood plain throughout Sub-reach 1.

Figure 1 shows that the proposed drainage right-of-way is beyond the 100-yr. flood plain limits in sub-reaches 1 and 2. The proposed right-of-way limit within Sub-reach 3 is the offset tangent line or a line which is outside the floodplain and offset tangent line. This allows for use of straight line boundaries to simplify platting.

OPERATION AND MAINTENANCE

The maintenance costs associated with the 2-yr., 10-yr., 25-yr., and representative annual storm, and the long term effects of annual storms are based on two factors. The aggradation/degradation volumes as given in Table 3 and revegetation that may be necessary due to aggradation or degradation.

Table 6 lists the costs for removal and hauling of sediment from the aggrading reaches and the cost of fill for degrading reaches. The unit cost for removal and hauling was $4.00/cu. yd. and the unit cost for borrow and hauling of suitable fill material is $5.00/cu. yd. as given in "Contract Documents for City Wide Utilities and Cash Paving No. 31".

Table 6 also lists the revegetation cost associated with each storm. The revegetation cost is $3,000/acre for hydro mulching with irrigation twice a day for the first two weeks and once a day for the second two weeks. This cost and irrigation schedule was obtained from the City Parks and Recreation Open Space Division and was based on their experience with revegetation projects on the west mesa.

Revegetation costs were only determined for Sub-reach 3 which may degrade and thus vegetation may be lost. The surface area requiring *** revegetation in Sub-reach 3 was determined as follows. Using the same method discussed in the lateral migration section and using a typical arroyo cross-section (Figure 5) the lateral migration distance and corresponding eroded perimeter was determined. The average eroded perimeter multiplied by the reach length equals the average eroded area. Table 7 gives the results of this procedure and these results were used to determine the revegetation cost given in Table 6.

Sub-reach 2 is expected to aggrade and is not expected to require revegetation for the following two reasons. The estimated deposition depth for Sub-reach 2 was determined by using a typical arroyo cross-section (Figure 5) with the wetted perimeter corresponding the 2-yr., 10-yr., 25-yr. and 100-yr. peak flows. The wetted perimeter multiplied by the reach length determines the total deposition area (assuming uniform deposition across the wetted perimeter and occuring from the hydrograph peak to the hydrograph terminus). The aggradation volumes in Table 3 divided by the wetted area for each storm determines the estimated sediment depth throughout Sub-reach 2. The results of this procedure are given in Table 8.

The Representative Annual aggradation depth will be between the 2-yr. and 10-yr. depths based on aggradation volumes (Table 3). The long term aggraded sediment depth will occur very gradually and therefore, the vegetation should be able to adapt to this gradual change. The 100-yr. storm may produce an average sediment depth of 0.63 feet and the other storms would produce insignificant sediment depths. Therefore, as described in the following paragraph, revegetation will not be necessary.

As discussed in the qualitative geomorphic analysis, the arroyo in Sub-reach 2 is moderatly to densely vegetated and very dense 2 - 4 ft. tall bushes are present. The estimated 100-yr. sediment depth is only 0.63 feet and therefore the bushes would not be buried danger. The smaller grasses and plants may be buried; however, the existing vegetation and mild slope in Sub-reach 2 (see Figure 4) indicates that this reach may infiltrate a considerable volume of storm runoff. Thus the area will naturally revegetate with grasses and other vegetation.

Based on the above discussion, a revegetation cost was only computed for Sub-reach 3 and is given in Table 6 which also gives the total revegetation, soil removal and fill costs for sub-reaches 1 and 2. The costs given in Table 6 for removal of sediment in Sub-reach 2 is very unlikely to be a reality based on the average sediment depths given in Table 8. Therefore, the only maintenance cost expected for the study reach is within Sub-reach 3. The vertical bank failure that may occur on the south bank between Sub-basin J and the base of the escrpment (see Figure 1), will need to be addressed during design of the open space trail/bike path that is proposed at this location.

A gabion grade control structure is a possible solution to accomodate the possible degradation in Sub-reach 3. The structure would be located at the junction of sub-reaches 2 and 3. The elevation drop necessary to reach the equilibrium slope for Sub-reach 3 is 3.77 feet and was** determined as the difference between the existing slope multiplied by the estimated distance of 315 feet the equilibrium slope multiplied by

315 feet. The unit cost for gabions is $130/cu. yd. and filter cloth is $1/sq. ft. as given in "Contract Documents for City Wide Utilities and Cash Paving No. 31".

A conceptual gabion grade control structure may consist of a 110-foot wide structure with 1-foot thick and 20-foot long gabion mats at either end, and a 5-foot drop. This structure would have a total volume of 261 cu. yd. and 7,700 sq. ft. of filter cloth would be required. Therefore, the estimated cost of the gabion is $33,930 and the filter cloth would cost $7,700 for a total cost of $41,630.

The channel between Vulcan Parkway and the gabion grade control structure would be graded at the equilibrium slope; and revegetation would be necessary. A rough calculation of the earthwork volume to be removed was determined as the area of a right angle triangle with a 315-foot base and a vertical side of 3.77 feet multiplied by a 10-foot width equals 220 cu. yd.. The cost of removal equals (220 cu. yd)(4.00/cu. yd.) = $880. The revegetation cost was estimated as 10-foot bottom width plus two (20-foot graded widths to reach existing undisturbed ground on each bank, multiplied by a 315-foot length. This area equals 0.36 acres. The cost of revegetation = (0.36 acres)($3,000/acre = $1,080).

Another added cost that is not included in this cost estimate for a gabion grade control structure is surveying of the equilibrium slope in Sub-reach 3. Therefore, a rough cost estimate for the gabion grade control structure and excavation to the equilibrium slope with *** revegetation is $43,590.

RECOMMENDATIONS

The recommendation for a plan of action is based on economics. As discussed previously, a rough cost estimate for a gabion grade control structure and associated works is $43,590. The total costs for leaving the arroyo in its natural state are given in Table 6 for Sub-reach 3. The annual cost may be $730, a 100-yr. storm may cost $11,460 and the representative annual cost (considers the annual probable occurence of storms with various return periods) may be $1,612.

Based on the above discussion, leaving the arroyo in its natural state appears to be a feasible plan of action.

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F10/D006

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February 16, 2009

Brad Bingham
City of Albuquerque
Development and Building Services
Plaza Del Sol Building, 2nd Floor
600 2nd St NW
Albuquerque, NM 87102

Re: Petroglyph National Monument – Rinconada Day Use Area Improvements

Mr Bingham,
We are resubmitting our 60% submittal package along with storm runoff calculations for your review.

Per our discussion on 2/10, I have also included a quick storm runoff calculation using the rational method and assumptions that should be conservative for a project of this size (less than an acre). This spreadsheet highlights the worst case scenario as far as increases to flow volumes are concerned, showing projected increases to the flows for the 2, 10 and 100 year storms. All flow within the site will be along the surface and will be conveyed via curb and gutter, valley pans, curb cuts and swales. The offsite drainage will be routed via a swale on the north and east sides of the proposed parking lot to the existing roadside ditch running to the northeast from the intersection of Unser Blvd and St Josephs.

We were able to route significant portions of the runoff through the center island within rock-lined swales. This will account for some amount of infiltration that is not reflected in the calculations, providing you with a more conservative estimate of the flows.

Please let me know if you have any more questions regarding this project or if there are any more steps we need to take with the City of Albuquerque. I appreciate your continued help with this project and we looking forward to a successful completion later this summer.

Sincerely,

Drew Chandler, P.E.
Project Engineer

Sub-Basin Label	Sub-Basin Label (Acres)	Duration Tc (min)	2 Year Intesity (in/hr)	10 Year Intesity (in/hr)	100 Year Intesity (in/hr)	2 Year Area Coefficient	10 Year Area Coefficient	100 Year Area Coefficient	2 Year Flow Rate (cfs)	10 Year Flow Rate (cfs)	100 Year Flow Rate (cfs)		Area 1 (Acres)	Area 2 (Acres)	Area 3 (Acres)	2 YR Area 1 C	2 YR Area 2 C	2 YR Area 3 C	10 YR Area 1 C	10 YR Area 2 C	10 YR Area 3 C	100 YR Area 1 C	100 YR Area 2 C	100 YR Area 3 C
Existing	0.94	10.00	2.75	5.50	10.00	0.15	0.33	0.54	0.40	1.71	5.06		0.49	0.45		0.04	0.28		0.25	0.42		0.50	0.58	
													grass	gravel										
Developed	0.94	10.00	2.75	5.50	10.00	0.78	0.83	0.90	2.02	4.31	8.47		0.12	0.82		0.04	0.89		0.25	0.92		0.50	0.96	
							Net increase in flow (cfs)		1.62	2.60	3.41		grass	asphalt										
														concrete										