VALLEY VIEW WASH

FLOOD AND EROSION CONTROL STUDY FOR

THE CALLE DEL PANTERA AREA

(PHASE II: FLECHA CAIDA FLOOD

IMPROVEMENT STUDY)

Prepared For:

Pima County Department of Transportation
and Flood Control District
1313 S. Mission Road
Tucson, Arizona 85713

Prepared By:

Simons, Li & Associates, Inc.
110 South Church, Suite 2170
P.O. Box 2712
Tucson, Arizona 85702

February 15, 1989

Registered Professional Engineer (Civil)
CERTIFICATE NO. 12481
MICHAEL E. ZELLER
Date Signed 2-15-89
ARIZONA, U.S.A.

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Registered Professional Engineer (Civil)
CERTIFICATE NO. 12481
MICHAEL E. ZELLER
Michael E. Zeller
Date Signed 2-15-89
ARIZONA, U.S.A.

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Registered Professional Engineer
CERTIFICATE NO. 12481
MICHAEL E. ZELLER
Date Signed 2-15-89
ARIZONA, U.S.A.
Michael E. Zeller

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I. INTRODUCTION

This report presents the results and recommendations of a comprehensive drainage study of the Calle del Pantera area which is located adjacent to and west of Swan Road, approximately one half mile south of Sunrise Drive (see Figure 1). The study area, which includes Tucson Water's Valley View Reservoir site and Flecha Caida Ranch Estates #9, has experienced numerous drainage-related problems. These problems include inundation, erosion, and limited access during the rainy season. For the most part, these problems stem from the fact that a portion of the reservoir site and approximately one-half of the subdivision are both located in the 100-year flood plain of the Valley View Wash (see Figure 2).

This study was prepared for the Pima County Department of Transportation and Flood Control District. A brief discussion of the circumstances that led to this study, along with the study objectives, is provided in the following paragraphs.

1.1 Background Information

The Pima County Department of Transportation and Flood Control District has, on several occasions, investigated drainage problems within the Calle del Pantera subdivision (Flecha Caida Ranch Estates #9). Most of these drainage problems were addressed on a lot-by-lot basis. In response to several flooding complaints from the owner of Lot 496, a diversion dike was constructed along a portion of the northern boundary of said lot. A site-specific engineering study conducted in conjunction with Phase I of the Flecha Caida Flood Improvement Study (Reference 1), led to the construction of a soil-cement levee along the southwestern side of Calle del Pantera (see Figure 2). This levee provided some flood protection for Lots 495, 496, 498, and 499. In addition, channel improvements, combined with the construction of two earthen dikes, provided some flood protection for Lots 500 and 501.

These flood-control projects were designed to protect the affected lots from all flows up to and including the 5-year peak discharge. Although the site-specific study indicated that the homes on the remaining lots were not subject to flooding during the 5-year event, the finished-floor elevations obtained as part of the study did indicate that during less frequent events (i.e., the 10-year, 25-year, and 100-year storm events) more homes might be inundated.

The primary purpose of the Phase-I study was to map the 100-year flood plains for the Valley View Wash, the Finger Rock Canyon Wash, Sky Club Wash, and the Flecha Caida Wash between the limits of River Road, to the south, and the Coronado National Forest boundary, to the north. Once the Phase-I study was completed, in January of 1986, it was noted that 15 single-family residences along the Valley View Wash, including eight within Flecha Caida Ranch Estates #9, were within its 100-year flood plain. The seven remaining residences within its 100-year flood plain were located to the south, between the subdivision itself and River Road.

The finished-floor elevations of the eight affected homes within the site-specific study area were compared to the computed 100-year water-surface elevations of Valley View Wash (see Figure 1 and Appendix A). It was noted that

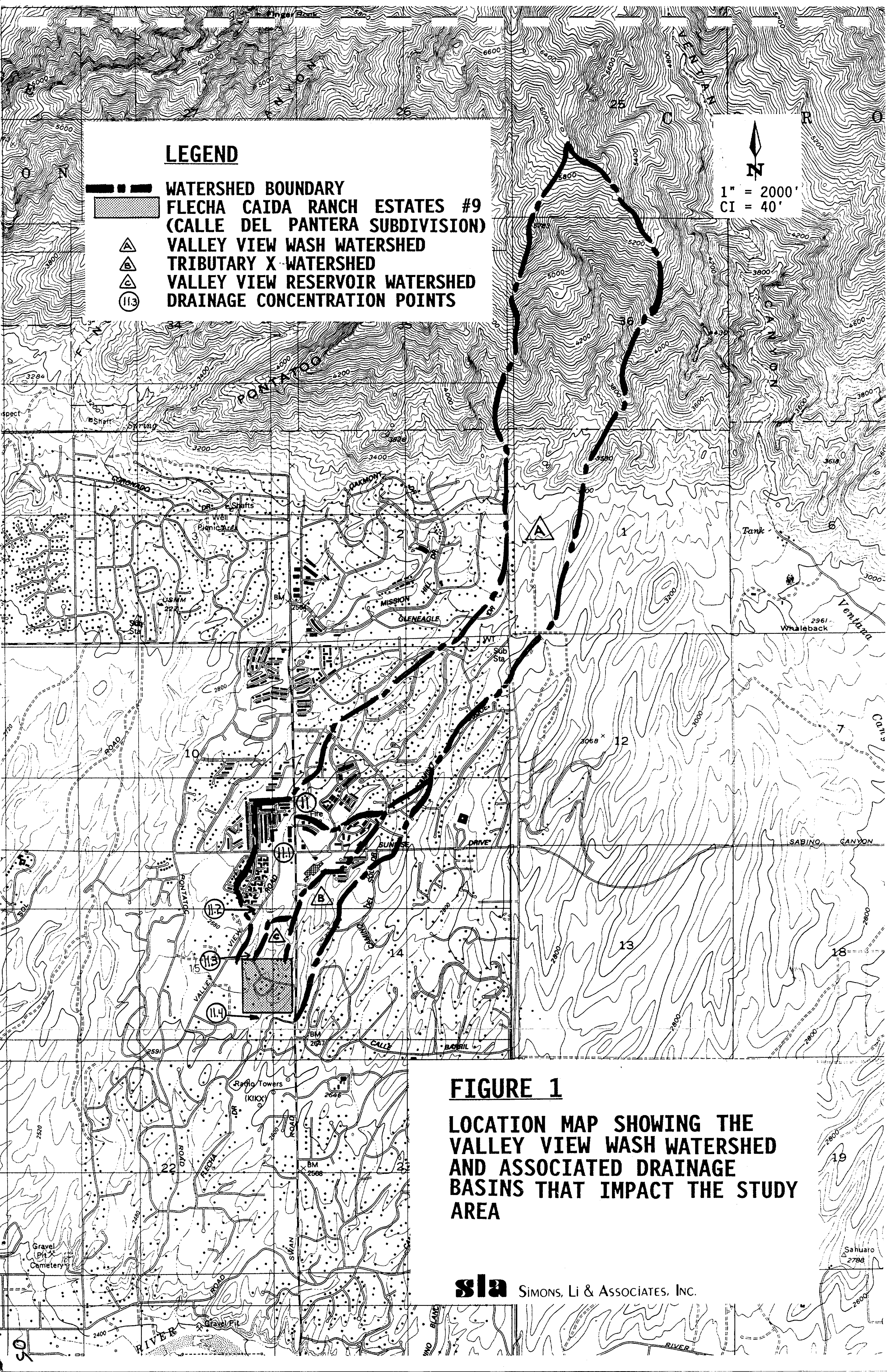

FIGURE 1

LOCATION MAP SHOWING THE VALLEY VIEW WASH WATERSHED AND ASSOCIATED DRAINAGE BASINS THAT IMPACT THE STUDY AREA

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FIGURE 2

FLOODPLAIN AND DRAINAGE FEATURES MAP
(FLECHA CAIDA RANCH ESTATES #9)

at least six of the eight homes were subject to flooding during this particular event. Pima County felt that if the homes located downstream of the Calle del Pantera subdivision were also subject to flooding, structural improvements on a lot-by-lot basis would be more expensive than providing a regional detention facility upstream of all affected lots. The cost effectiveness of a regional detention facility could be established through a multi-level flood study that included a cost/benefit analysis.

In cooperation with Tucson Water, Pima County initiated such a study to determine the feasibility of providing a regional detention facility to solve the area's drainage problems. Tucson Water agreed to allow a portion of their property to be used for the facility, in hopes that the facility's design would eliminate their erosion problems (to date, erosion along the wash has threatened the integrity of the reservoir access road, as well as the sewer, water, and electrical lines which are buried beneath it).

However, the preliminary results of the multi-level flood study, which included an evaluation of several detention basin designs, indicated that it was not feasible to provide a regional detention facility along the Valley View Wash. Pima County reviewed the results, and agreed that the concept of a regional detention basin was not cost effective. A more-detailed description of the analysis and results of that study are discussed in Section III of this report.

1.2 Study Objectives

Since a detention facility was not cost effective, the original scope-of-work was subsequently amended to eliminate the remaining tasks that were directly related to the detention concept. In their place, several tasks were formulated in an effort to ensure that the project's original objectives were achieved. These objectives were as follows:

1. Define the flood hazards associated with the Calle del Pantera area, and evaluate the cost effectiveness of both structural and non-structural mitigation measures;

2. Develop short-term and long-term mitigation measures to address local erosion problems within the reservoir site and the subdivision; and,

3. Determine if the existing culvert beneath Swan Road could be upgraded to convey the entire 100-year peak discharge without adversely impacting downstream properties.

With the exception of the local erosion problems within the Calle del Pantera subdivision, a regional detention facility would have addressed the other objectives. It was for this reason that the original scope-of-work concentrated on the design of a regional facility. However, since the cost of a detention facility significantly overshadowed the benefits derived, each objective had to be evaluated independently.

As stated above, the modified scope-of-work included a series of tasks that were formulated in an effort to satisfy the project's original objectives. These tasks were broken down into two separate studies. One study analyzed the upstream watershed in sufficient detail to address the relative impact of commercial developments and roadway improvements on the flood hazards associated with study area. The second study considered the feasibility of channelizing a portion of the Valley View Wash to eliminate flood hazards within Flecha Caida Ranch Estates #9. It also provided the design requirements for both short-term and long-term erosion-control structures.

The purpose of this comprehensive report is to present the results of all three studies (i.e., the regional detention feasibility study; the commercial development/roadway improvements impact study; and the channelization/erosion-control study). In addition, this report provides a brief description of the drainage features, patterns, and problems which are inherent to the study area.

A summary of the individual tasks which were performed for the overall project is provided in the following section.

1.3 Project Tasks

The most relevant tasks performed in conjunction with the regional detention feasibility study were:

1. Identify, from the Phase-I study, those homes that might be subject to flooding during the 100-year event;

2. Obtain, through field survey, the finished-floor elevation of each home identified in Task 1;

3. Obtain, from Pima County's Property Management Section, an estimate of the full-cash value of each home identified in Task 1;

4. Perform, as needed, a hydrologic analysis of the Valley View Wash at various concentration points to define the 2-year, 5-year, 10-year, 25-year, and 100-year peak discharges and their associated hydrographs;

5. Perform a multi-frequency hydraulic analysis of the study area using the peak discharge magnitudes defined in Task 4. Recent drainage improvements within the Calle del Pantera subdivision were to be included in the analyses; and,

6. Evaluate the effectiveness, from a peak-discharge-attenuation standpoint, of two detention-basin concepts (i.e., a single regional basin with a multi-stage outlet versus a segmented basin with a multi-stage outlet).

The most relevant tasks performed in conjunction with the commercial development/roadway improvements impact study were:

1. Perform a detailed hydraulic analysis of the Valley View Wash at Swan Road, north of Sunrise Drive, to determine the impact of upgrading the existing 72-inch CMP to a larger structure that would be capable of conveying the entire 100-year peak discharge;

2. Perform a detailed hydrologic analysis of the localized drainage areas that surround the Swan Road/Sunrise Drive intersection;

3. Determine the capacity of a recently installed 66-inch RCP which conveys storm runoff generated within the drainage areas identified in Task 2 directly to the Valley View Wash; and,

4. Analyze the feasibility of relocating the 66-inch RCP, as described in Task 3, such that it could discharge concentrated runoff into the Valley View Wash on the downstream side of the study area, instead of on the upstream side.

The most relevant tasks performed in conjunction with the channelization/erosion-control study were:

1. Prepare and evaluate the cost effectiveness of a concept channelization plan which addresses the 100-year flood hazards associated with the Calla del Pantera subdivision;

2. Evaluate the magnitude and extent of local erosion which currently threatens the reservoir access road and the utility lines that serve the reservoir;

3. Provide a short-term and long-term solution to the reservoir's erosion problems;

4. Identify those areas within the Calle del Pantera subdivision where local erosion problems exist; and,

5. Recommend measures that can be implemented to control and contain erosion within these problem areas.

II. DRAINAGE CHARACTERISTICS AND ASSOCIATED IMPROVEMENTS

The Valley View Wash watershed upstream of Swan Road encompasses approximately 900 acres (see Figure 1). The majority of this watershed is currently developed for single-family residences at relatively low densities (i.e., approximately one residence per acre).

Between Swan Road and the northern boundary of Flecha Caida Ranch Estates #9, approximately 125 acres of additional drainage area contributes runoff to the Valley View Wash. The majority of this area is currently developed for commercial and high-density residential uses, with the exception of the Valley View Reservoir site.

Along this latter reach, the Valley View Wash exists as a single, well-defined channel that, for the most part, is capable of conveying the entire 100-year peak discharge, which is approximately 2300 cfs. The average width of the flood plain along this reach is approximately 150 feet.

The well-defined channel abruptly ends at the subdivision's northern boundary. Since there is very little topographic relief in an east-west direction, runoff can easily spread across the area. As a result, the average width of the 100-year flood plain within the subdivision ranges between 500 and 600 feet. This places the western half of the subdivision in the 100-year flood plain. Although the depth of flow is relatively shallow, the finished floors of several homes within this area are only a few inches above the adjacent ground elevations.

Using historic aerial photographs it was noted that before the subdivision was established in 1959, two poorly-defined, low-flow swales existed within its northwestern quarter. Again, this is the area that has little or no topographic relief in the east-west direction. The eastern swale traversed a portion of the eastern half of Lots 498, 499, and 500. The western swale traversed the western boundary of these same lots; crossed the northern and eastern portion of Lot 501; and joined the eastern swale along that portion of Cerco de Corazon Circle that is situated between Lots 497 and 501. Downstream of this location, a single channel emerges. Although this channel could be considered the main channel, it lacks capacity to contain the entire 100-year peak discharge. This limited-capacity channel traverses Lot 504, before exiting the subdivision at the lot's southeast corner.

At this location, a tributary channel (Tributary "X"), which drains approximately 120 acres, joins the Valley View Wash. This channel conveys between 340 and 400 cfs during the 100-year event. Flow enters the subdivision along the eastern boundary of Lot 508; traverses a portion of Lots 509, 510, and 505; and exits the subdivision at the southwest corner of Lot 505, where it joins the Valley View Wash.

Immediately downstream of the confluence of the Valley View Wash and Tributary "X", the main channel is well incised. However, since the average width of the channel is limited to approximately 50 feet, this segment of the wash is not capable of conveying the 100-year peak discharge.

Although, the subdivision itself was not responsible for the expanded flood plain, the alignment of Calle del Pantera, in combination with Cerco de Corazon Circle, contributed to the drainage problems that currently exist. When Calle del Pantera was constructed, the street section essentially replaced the eastern swale. At its western intersection with Cerco de Corazon Circle, flows conveyed in the street section were required to make a 90-degree turn. Immediately downstream of the intersection is the house that occupies Lot 496. This was one of the first houses in the subdivision to experience problems. As previously mentioned, a diversion dike intended to protect this house was the first mitigation measure constructed.

Before a continuous soil-cement levee was constructed along Calle del Pantera as part of the 1985-1986 improvements, runoff in excess of the street capacity would overtop the roadway and follow the path of the eastern swale. However, the house that occupies Lot 499 lies adjacent to this poorly-defined swale, and was in the direct part of overflow from the street section. Therefore, the primary purpose of this levee was to protect the house on Lot 499 by increasing the capacity of the street section.

It appears that the eastern swale conveyed most of the annual runoff released at the northern boundary of the subdivision. However, on occasion, the quantity of runoff generated within the upstream watershed would be sufficient to initiate conveyance in the western swale. This actually occurred on June 27, 1984, when the area was subjected to a discharge magnitude (estimated at 570 cfs) that approximated the 5-year value (estimated at 590 cfs). As with the eastern swale, the poorly-defined nature of the western swale caused some problems with the homes that existed on Lots 500 and 501. This lead to the construction of two dikes. One dike served to protect the house on Lot 500, and one served to protect the house on Lot 501. In addition, the capacity of the western swale, from Lot 499 south to Cerco de Corazon Circle, was also upgraded to contain a greater quantity of flow.

III. REGIONAL DETENTION FEASIBILITY STUDY

As previously mentioned, the 100-year flood plain for the Valley View Wash was originally defined as part of the Phase-I flood-improvement study (Reference 1). In addition, the most recent flood-control improvements were designed considering the magnitude of runoff generated during the 5-year event. However, it was necessary to determine (1) if these improvements did, in fact, provide 5-year protection to the Calle del Pantera subdivision; and (2) if they reduced the impact of less frequent storm events (e.g., the 10-year, 25-year, and 100-year floods). Therefore, one of the initial goals of the feasibility study was to conduct a multi-frequency floodplain analysis of the Valley View Wash to address these questions. In addition, this analysis was used to make a determination of which homes would be flooded during each return-period event (i.e., the 2-year. 5-year, 10-year, 25-year, and 100-year floods).

The peak-discharge computation sheets for each return period at the respective concentration points shown on Figure 2 are contained in Appendix B. The input/output listing associated with the multi-frequency hydraulic analysis of the respective flood plains are contained in Appendix C.

The results of this multi-frequency floodplain analysis (see Appendix C) indicates that there should be no flooding of homes within the study area during the 2-year and 5-year events (see Figure 3). This includes all homes within the Calle del Pantera subdivision, and those homes located downstream in the area between the subdivision itself and River Road. It was further shown that the 10-year flood only inundates the floors of two homes, the 25-year event only inundates the floors of four homes, and the 100-year event only inundates the floors of five homes (compared to the six initially identified during the Phase-I study). All of the affected homes are within the Calle Pantera area.

To provide the maximum level of protection for the neighborhood, it would be desirable to construct a detention basin that could effectively reduce the peak discharge generated during the various flow events to the magnitude associated with the 5-year event. If it is not feasible to limit the outflow from the detention basin to the 5-year magnitude, the next preferable alternative would be to consider a reduction to the 10-year or 25-year magnitudes.

With this in mind, the volume of storage required to reduce (1) the 10-year to the 5-year; (2) the 25-year either to the 10-year or to the 5-year; and, (3) the 100-year either to the 25-year, to the 10-year, or to the 5-year was determined using the equations contained in the Pima County Stormwater Detention/Retention manual (Reference 4). The required volumes are provided as Figure 4. As one would expect, the results indicate that reducing the 10-year peak discharge to the 5-year peak discharge requires the least storage volume. This alternative would require 27 acre-feet of storage to accommodate storm runoff, and 11-acre feet of storage to account for siltation of the basin with time. Basin to be maintained. Silt removal will be done instead of providing for this extra volume.

FLECHA CAIDA FLOOD STUDY – RESIDENCES IN VALLEY VIEW WASH
(WSE BASED ON SUBCRITICAL RUN)

RESIDENCE* #	FFE	Q_{100} WSE	Q_{25} WSE	Q_{10} WSE	Q_5 WSE	Q_2 WSE
1	2648.7	0.6 2649.5	2648.9	2648.7	2648.5	2648.0
2	2640.8	2640.8	2640.5	2640.3	2640.2	2639.8
3	2636.3	0.4 2636.7	2636.4	2636.1	2635.9	2635.4
4	2633.6	2632.0	2631.7	2631.4	2631.2	2630.8
5	2632.7	2631.7	2631.4	2631.1	2630.9	2630.5
6	2630.2	0.6 2630.8	2630.5	2630.3	2630.0	2629.7
7	2628.5	0.3 2628.8	2628.5	2628.3	2628.1	2627.8
8	2619.5	2618.6	2617.9	2617.4	2617.0	2616.4
9	2598.7	0.6 2599.3	2598.8	2597.4	2596.6	2595.2
10	2557.7	2557.5	2557.0	2556.7	2556.4	2555.9
11	2563.6	2562.6	2562.1	2561.7	2561.4	2560.8
12	2553.8	2552.7	2552.3	2552.1	2551.8	2551.3
13	2549.6	2546.3	2545.9	2545.7	2545.0	2544.1
14	2538.8	2537.0	2536.4	2536.2	2535.3	2534.0
15	2441.5	2440.9	2439.6	2438.8	2438.3	2437.5

* SEE FIGURE 2 FOR RESIDENCE LOCATION

FIGURE 3

MULTI-FREQUENCY WATER-SURFACE ELEVATIONS VERSUS FINISHED-FLOOR ELEVATIONS WITHIN THE STUDY REACH

STORAGE VOLUME REQUIREMENTS FOR THE VARIOUS DESIGN SCENARIOS

FLECHA CAIDA BASIN MANAGEMENT STUDY

THERE IS NO DAMAGE EXPECTED FOR A 5-YEAR FLOOD. DAMAGE WOULD OCCUR AS THE FLOOD INCREASES BEYOND A 5-YEAR.

ESTIMATION OF VOLUME REQUIRED TO REDUCE THE PEAK DISCHARGE AT CALLE DEL PANTERA TO A 5-YEAR LEVEL FROM A 100-YEAR LEVEL:

USE $$V_s = \frac{C_w P A}{12}\left[1 - \frac{Q_o}{Q_i}\right]$$

$Q_5 = 584$ CFS

$Q_{100} = 2261$

$C_w = 0.650$

$P = 2.70$ INCHES

$A = 1023$ ACRES

$$V_s = \frac{0.65(2.7)(1023)}{12}\left[1 - \frac{584}{2261}\right]$$

$V_s = 111$ ACRE FEET

$V_{SD} = 500 A C_P = 500(1023)(.91) = 465{,}465\ FT^3$

$V_{SD} = 11$ ACRE FEET

TOTAL VOLUME = 122 ACRE FEET

FIGURE 4

SHEET 1 OF 4

TRY REDUCING TO A 10-YEAR DISCHARGE

$$V_S = 149.6\left(1 - \frac{907}{2261}\right)$$

V_S = 90 ACRE FEET

TOTAL VOLUME = 101 ACRE FEET

TRY REDUCING TO A 25-YEAR DISCHARGE

$$V_S = 149.6\left(1 - \frac{1350}{2261}\right)$$

V_S = 60 ACRE FEET

TOTAL VOLUME = 71 ACRE FEET

FIGURE 4

SHEET 2 OF 4

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PROJECT: VALLEY VIEW WASH
DETAIL: ATTACHMENT C

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VOLUME TO REDUCE 25 YEAR FLOOD TO 10 YEAR LEVEL

$$V_S = \frac{0.556\,(2.16)\,1025}{12}\left[1 - \frac{907}{1350}\right]$$

V_S = 34 ACRE FEET

TOTAL VOLUME = 45 ACRE FEET

FIGURE 4
SHEET 3 OF 4

VOLUME TO REDUCE 10-YEAR FLOOD TO 5-YEAR LEVEL

$$V_s = \frac{0.484(1.86)1023}{12}\left[1 - \frac{584}{907}\right]$$

V_s = 27 ACREFT + 11

No need to add 11 acre ft.

TOTAL VOLUME = 38 ACRE FEET

VOLUME TO REDUCE 25-YEAR FLOOD TO 5-YEAR LEVEL

$$V_s = \frac{0.556(2.16)1023}{12}\left[1 - \frac{584}{1350}\right]$$

V_s = 58 ACRE FEET

TOTAL VOLUME = 69 ACRE FEET

If basin is constructed a maintenance program to remove silt should be instituted.

The next step was to determine the approximate storage volume available within the reservoir site. Although several acres of the reservoir site and adjacent property owned by Pima County were available for consideration as a detention facility, the topographic relief of the site creates a major physical constraint. The area immediately surrounding the wash, with the exception of Valley View Road, is very steep (see Figure 2). These adjacent hillsides rise approximately 35 feet from the bottom of the wash to the top of the ridge over a horizontal distance of approximately 150 feet. For this reason, the initial evaluation only included the area occupied by the wash. In addition, the height of the proposed embankment was limited to approximately six feet, in order to minimize the visual impact upon surrounding properties.

It was then assumed that the embankment would be located immediately upstream of Calle del Pantera subdivision. The first concept layout assumed that the detention facility would exist as two separate basins. The stormwater basin would extend from Calle del Pantera upstream along the wash to the access road serving the reservoir. A sedimentation basin would then be constructed along the wash from the access road upstream to Valley View Road. The access road would be modified to provide a structure that would trap sediments, while releasing stormwater into the downstream basin. Again,it was assumed that the height of this embankment would be limited to six feet. It was further assumed that the existing profile of the wash within each respective basin would be excavated to provide a minimum slope of 0.3 percent.

Under this scenario, the total available storage volume within the detention basin (lower reservoir) was determined to be approximately 8.73 acre-feet (see Figure 5). The available storage volume within the sediment basin was determined be approximately 7.63 feet. A rough estimate of the cost of excavation, at $4.00/cubic yard, is $225,000 (this includes the excavation of material not directly related to the required storage volume, but removal of which is required to form the respective basins).

As previously stated, the smallest detention facility required would be one that reduces the incoming 10-year peak discharge to the 5-year rate. This facility would require a 27-acre-foot detention basin, and an 11-acre-foot sedimentation basin. It is obvious that sufficient surface area is not available, within the limits described above, to accommodate the minimum storage volume required to achieve some reasonable form of peak-discharge attenuation for the subject area.

More volume could be obtained by (1) increasing the downstream height of the basins; (2) excavating below the existing channel invert on the upstream side of the downstream portions of the basins; (3) eliminating the sedimentation basins; and/or, (4) cutting away large portions of the adjacent hillsides. However, each of these additional measures, or combination of measures, would create additional design, aesthetic, and/or liability problems to be addressed. In addition, the cost of these additional alternatives may not be justified, considering the limited number of homes subjected to flooding during the various flow events. Therefore, a risk analysis was performed to determine the level benefits derived for any given flood-control project.

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RESERVOIR VOLUME CALCULATIONS

LOWER RESERVOIR WSE = 2656 FT MSE
MIN. ELEV = 2650 FT MSE

AREA AT ELEVATION 2656 = 2.11 ACRES
AREA OF BOTTOM = 1.38 ACRES
AVERAGE DEPTH = 5 FEET
TOTAL STORAGE VOLUME = 8.73 ACRE FEET
APPROXIMATE EXCAVATION VOLUME = 35,000 CU YD
EXCAVATION COST @ $4.00/CU YD = $140,000

UPPER RESERVOIR WSE = 2672 FT MSE
MIN ELEV = 2666 FT MSE

AREA AT ELEV 2672 = 2.07 ACRES
AREA OF BOTTOM = 0.98 ACRES
AVERAGE DEPTH = 5 FEET
TOTAL STORAGE VOLUME = 7.63 ACRE FEET
APPROXIMATE EXCAVATION VOLUME = 21,250 CU YD
EXCAVATION COST @ $4.00/CU YD = $85,000

FIGURE 5

RESERVOIR STORAGE CALCULATION FOR THE DUAL BASIN SCENARIO

This analysis was performed using some of the basic economic evaluation criteria outlined by the Arizona Department of Water Resources (ADWR). Figure 6 shows the relationship between the computed water-surface elevation for a given event and the finished-floor elevation of each residence located in the 100-year flood plain. Using this relationship, the percent damage to each residence was established using Figure 7 as a guide. The final percentages are shown on Figure 8. The average-annual risk was then determined using the full-cash value of each residence, as provided by Pima County's Property Management section (see Figure 9). The present net worth associated with the total average-annual risk was then determined assuming a 50-year design life at an annual percentage rate of seven percent.

The results of this economic evaluation indicate that the benefits derived from construction of a regional detention basin within the reservoir site would only support a $73,700 construction project. Therefore, the benefit derived would be less than the estimated cost ($225,000) of the concept of a dual detention basin. Since $73,700 is much less than the estimated cost of providing a regional detention basin capable of reducing the 10-year peak discharge to that associated with the 5-year event, there is no doubt that for this alternative, or alternatives requiring a greater magnitude of improvements, the benefit/cost ratio would always be less than one.

In view of these preliminary results, the monetary benefits derived for the study area does not justify the cost of providing a regional detention facility within the reservoir site.

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VALLEY VIEW WASH FLOOD STUDY
LOWEST FLOOR ELEVATION IN RELATION
TO WATER SURFACE ELEVATION
(WSE - FFE)
(FT)

RESIDENCE #	Q100	Q25	Q10	Q5	Q2
1	0.6	0.2	0	-0.2	-0.7
2	0	-0.3	-0.5	-0.6	-1.0
3	0.4	0.1	-0.2	-0.4	-0.9
4	-1.6	-1.9	-2.2	-2.4	-2.8
5	-1.0	-1.3	-1.6	-1.8	-2.2
6	0.6	0.3	0.1	-0.2	-0.5
7	0.3	0	-0.2	-0.4	-0.7
8	-0.9	-1.6	-2.1	-2.5	-3.1
9	0.6	0.1	-1.3	-2.1	-3.5
10	-0.2	-0.7	-1.0	-1.3	-1.8
11	-1.0	-1.5	-1.9	-2.2	-2.8
12	-1.1	-1.5	-1.7	-2.0	-2.5
13	-3.3	-3.7	-3.9	-4.6	-5.5
14	-1.8	-2.4	-2.6	-3.5	-4.8
15	-0.6	-1.9	-2.7	-3.2	-4.0

FIGURE 6

RELATIONSHIP BETWEEN THE LOWEST FINISHED-FLOOR ELEVATION
AND THE MULTI-FREQUENCY WATER-SURFACE ELEVATION

ARIZONA DEPARTMENT OF WATER RESOURCES
FLOOD DAMAGE ESTIMATES

FLOOR LEVEL IN RELATION TO WATER LEVEL (FT)	PERCENT DAMAGE TO IMPROVEMENTS
0.1	6
0	11
-0.5	15
-1.0	21
-1.5	27
-2.0	32
-2.5	37
-3.0	40

FIGURE 7

ARIZONA DEPARTMENT OF WATER RESOURCES' FLOOD DAMAGE ESTIMATES

PERCENT DAMAGE FOR EACH RESIDENCE USING THE A.D.W.R. PERCENTAGE METHODS

	% DAMAGE				
RESIDENCE	Q_{100}	Q_{25}	Q_{10}	Q_5	Q_2
1	16.2	12.6	11.0	0	0
2	11.0	0	0	0	0
3	14.2	11.8	0	0	0
4	0	0	0	0	0
5	0	0	0	0	0
6	16.2	13.4	11.8	0	0
7	13.4	11.0	0	0	0
8	0	0	0	0	0
9	0	0	0	0	0
10	0	0	0	0	0
11	0	0	0	0	0
12	0	0	0	0	0
13	0	0	0	0	0
14	0	0	0	0	0
15	0	0	0	0	0

FIGURE 8
SHEET 1 OF 2

FLECHA CAIDA RANCH ESTATES #9

LOT #	SLA #	PRICE
495	6	$109,994
496	7	108,357
497		112,891
498	1	101,732
499	2	102,430
500	3	91,466
501		132,866
502		110,292
503		141,965
504		VACANT $25,500

FLECHA CAIDA RANCH ESTATES #2

54	VACANT $30,000
55	$201,013
56	$196,182

FLECHA CAIDA RANCH ESTATES AMENDED

43	$119,896
44	146,342
45	129,078
46	124,011
47, 48, 49	148,739

BOOK	MAP	PARCEL	PRICE
110	02	001M	$99,492
110	02	001N	VACANT $24,000
110	02	001P	VACANT $24,000
110	02	001Q	VACANT $24,000

FIGURE 8

SHEET 2 OF 2

SIMONS, LI & ASSOCIATES, INC.

CLIENT PDOT
PROJECT VALLEY VIEW WASH
DETAIL
JOB NO. PAZPD0707
DATE CHECKED
CHECKED BY
PAGE 1 OF 1
DATE 2/12/88
COMPUTED BY PRC

COMPUTATION OF AVERAGE ANNUAL RISK

FLOOD RETURN PERIOD (YEARS)	FLOOD DAMAGE ($)	PROBABILITY	AVERAGE DAMAGE ($)	Δ PROBABILITY	AVERAGE ANNUAL RISK ($)
0	0	1			
			0	0.50	0
2	0	0.5			
			0	0.30	0
5	0	0.20			
			12,085	0.10	1,209
10	24,170	0.10			
			37,469	0.06	2,248
25	50,768	0.04			
			62,225	0.03	1,867
100	73,682	0.01			

TOTAL = $5,324

ASSUMING A 50 YEAR DESIGN LIFE, INTEREST = 7%, THE PRESENT NET WORTH OF AN ANNUAL PAYMENT OF $5,324 IS $73,737.00. ON A COST/BENEFIT BASIS, THIS IS THE MAXIMUM ALLOWABLE EXPENDITURE FOR FLOOD CONTROL.

FIGURE 9

COMPUTATION OF MAXIMUM ALLOWABLE EXPENDITURE FOR FLOOD-CONTROL IMPROVEMENTS

IV. COMMERCIAL-DEVELOPMENT/ROADWAY-IMPROVEMENT IMPACT STUDY

To date, all hydrologic and hydraulic analyses of the study area assumed that the existing 72-inch corrugated metal pipe (CMP) located beneath Swan Road and to the north of Sunrise Drive had no effect on reducing the downstream flow rates (i.e., it was assumed that this structure was capable of conveying all peak discharges up to and including the 100-year peak discharge). However, since the existing structure can not convey the entire 100-year discharge, previous studies may have over predicted those hazards associated with the Calle del Pantera subdivision. Therefore, if this structure were upgraded to convey the entire 100-year peak discharge, these hazards may indeed become a certainty.

Even if the structure is effective in reducing peak flows, the associated overtopping of Swan Road creates a major flood hazard for the Swan Road/Sunrise Drive intersection and surrounding properties. When the roadway is overtopped, these flows are captured by the Swan Road street section and conveyed to the Swan Road/Sunrise Drive intersection. Since the existing storm drain in the area lacks capacity to accept these flows, the surrounding commercial developments would be subject to inundation. In addition, a major traffic hazard would exist at and near the intersection.

Although the Swan Road CMP was to be upgraded as part of the Swan Road improvements, there was some concern that doing so would eliminate the possibility that the structure was, in fact, controlling the magnitude of runoff impacting the study area. Therefore, this study was initiated to define the hydraulic significance of this structure from a hydrologic standpoint.

To define the significance of this structure, two separate analyzes were performed. The first analysis considered the effect of sediment build up at the inlet. It was noted during a recent field investigation that the inlet area was clogged with enough sediment to effectively reduce the cross-sectional area by 50 percent. Previous field investigations conducted as part of the Phase-I study, along with a review of the project's topographic maps, confirmed that the inlet was consistently clogged in this manner. Therefore, it was assumed that the cross-sectional area of the clogged structure would approximate a 72-inch by 44-inch arched CMP.

The results of this analysis indicate that the structure would reduce the 2-year, 5-year, 10-year, 25-year, and 100-year peak discharges by approximately twenty percent, four percent, four percent, ~~three percent~~ and three percent, respectively.

The second analysis assumed that the inlet would either be maintained on a regular basis, or that a drop inlet would be constructed to control siltation. The results of this analysis indicates that the 72-inch CMP would reduce the 2-year, 5-year, 10-year, 25-year, and 100-year peak discharges by four percent, nine percent, five percent, three percent, and three percent, respectively.

Both analyses include an estimate of the amount of flow that might be expected to overtop Swan Road during each flow event. A composite summary sheet of the results of these two analyses is provided as Figure 10. Inflow and outflow hydrographs used in the overall analysis are provided in Appendix D.

Based on the results of the overall analysis, it was concluded that the inlet capacity of the existing 72-inch (CMP) beneath Swan Road, coupled with the upstream headwater pool, does not result in a significant attenuation of the 5-year, 10-year, 25-year or 100-year peak discharges. Although the clogged analysis using the 2-year peak discharge demonstrated a twenty-percent reduction in the peak discharge, the significance with respect to the study area is not that great, since the area is currently protected against that particular flow event even assuming no peak reduction.

Therefore, the 72-inch CMP can be removed and replaced with a structure that would be capable of conveying the entire 100-year peak discharge before the roadway is overtopped. This action will eliminate the potential flood hazards at or near the Swan Road/Sunrise Drive intersection without increasing the flood hazards associated with the Calle del Pantera subdivision. However, every effort should be made to design this structure to maximize the allowable headwater elevation at the inlet. This action will guarantee maintenance of the limited attenuation effects that the upstream area does provide.

Since the above analysis confirmed that flood hazards within the Calle del Pantera subdivision are not significantly affected by the Swan Road culvert, a detailed study was performed to identify the upstream drainage areas that include the Swan Road/Sunrise Drive intersection. The purpose of this study was to determine if something could be done within these upstream drainage areas to effectively reduce the flood hazards associated with the Calle del Pantera subdivision. By isolating the individual drainage areas, and identifying their associated concentration points, alternative flood-control measures might become apparent.

Once the individual drainage areas and concentration points were defined (see Figure 11), a hydrograph analysis was performed to establish how these areas would interact with each other, and to what degree such interaction would affect the study area. This analysis was performed using the procedures outlined in Pima County's hydrology manual (Reference 3). Figure 12 summarizes the peak-discharge magnitudes at the various concentration points for each return interval. The hydrologic computation sheets for the respective concentration points shown on Figure 11 are contained in Appendix E).

A synthetic hydrograph was computed at each concentration point for each return period (see Appendix F), with the exception of Concentration Point 11. At this location, the outflow hydrograph associated with the 72-inch CMP was used (see Appendix D). Average flow velocities from previous multi-frequency floodplain analyses were then used (see Appendix C) to lag the respective hydrographs to Calle del Pantera (Concentration Point 4, which is the same as concentration point 11.3). A composite hydrograph was then prepared for each return period to define the peak discharge associated with this analytical

ATTENUATION EFFECTS OF AN EXISTING 72-INCH CMP AT THE SWAN ROAD CROSSING OF THE VALLEY VIEW WASH

SUMMARY OF RESULTS

I. WITH CLOGGING (72"x44" CMPA)

RETURN PERIOD (YR)	PEAK DISCHARGE (cfs)	PEAK OUTFLOW (cfs)	PEAK REDUCTION (cfs)	
2	233	186	47	(20%)
5	592	569	23	(4%)
10	880	845	35	(4%)
25	1306	1269	37	(3%)
100	2196	2128	68	(3%)

II WITHOUT CLOGGING (72" CMP)

RETURN PERIOD	PEAK DISCHARGE	PEAK OUTFLOW	PEAK REDUCTION	
2	233	223	10	(4%)
5	592	536	56	(9%)
10	880	838	42	(5%)
25	1306	1268	38	(3%)
100	2196	2127	69	(3%)

FIGURE 10
SHEET 1 OF 3

III OUTFLOW VOLUME / ELEVATION (72"×44" CMPA)
(FLOWLINE @ INLET ≈ 41.4)

RETURN PERIOD (YR)	PEAK OUTFLOW (cfs)	STORAGE VOLUME (AC-FT)	ELEV.
2	186	2.47	49.31
5	569	5.55	52.20
10	845	6.47	52.89
25	1269	7.51	53.61
100	2128	9.42	54.81

IV OUTFLOW VOLUME / ELEVATION (72" CMP)
(FLOW LINE @ INLET ≈ 38.4)

RETURN PERIOD (YR	PEAK OUTFLOW (cfs)	STORAGE VOLUME (AC-FT)	ELEV.
2	223	0.42	45.50
5	536	4.71	51.52
10	838	5.94	52.50
25	1268	7.14	53.36
100	2127	9.07	54.60

FIGURE 10
SHEET 2 OF 3

IV OUTFLOW DISTRIBUTION (72"x44" CMPA)

RETURN PERIOD	PEAK OUTFLOW	CMPA DISCHARGE	WEIR DISCHARGE
2	186	186	—
5	569	230	339
10	845	240	605
25	1269	250	1019
100	2128	270	1858

V OUTFLOW DISTRIBUTION (72" CMP)

RETURN PERIOD	PEAK OUTFLOW	CMP DISCHARGE	WEIR DISCHARGE
2	223	223	—
5	536	370	166
10	838	390	448
25	1268	400	868
100	2127	430	1697

FIGURE 10

SHEET 3 OF 3

FIGURE 11

LOCALIZED DRAINAGE BASIN MAP SHOWING RELEVANT POINTS OF RUNOFF CONCENTRATION

FIGURE 12

PEAK DISCHARGE SUMMARY SHEET FOR LOCAL DRAINAGE AREAS SURROUNDING THE SWAN ROAD-SUNRISE DRIVE INTERSECTION

CONC. PT.	D.A. (ACRES)	PEAK DISCHARGE 2-YR	5-YR	10-YR	25-YR	100-YR
1	59	153	214	268	327	435
2	66	155	227	271	349	487
3	34	61	91	112	141	205
3A	53	105	156	196	252	354
4	94	150	228	280	366	503
11	908	223	592	880	1306	2196
11.3	1061	241	606	941	1400	2345

DRAINAGE BASIN DESCRIPTION

CP #1 – LOCAL DRAINAGE AREAS EAST OF SWAN Rd

CP #4 – LOCAL DRAINAGE AREAS WEST OF SWAN Rd. INCLUDES AREA BETWEEN CP# 1 & CP#2 PLUS AREAS ASSOCIATED WITH CP# 3 & CP # 3A.

CP # 2 – AREA INCLUDES AREA ASSOCIATED WITH CP # 1 & AREA BETWEEN SWAN Rd AND CP#2.

CP # 3 – UPPER PORTION OF AREA LOCATED TO THE WEST OF SWAN Rd EXCLUDING THAT PORTION ASSOCIATED WITH CP#2.

CP # 3A – LOWER PORTION OF AREA LOCATED TO THE WEST OF SWAN Rd

CP # 11 – VALLEY VIEW WASH UPSTREAM OF SWAN Rd.

CP # 11.3 – AREA INCLUDES CP #11, CP#1 PLUS CP#4 approach (see Figure 13). The resultant peaks were then compared to those peaks obtained using the entire Valley View Wash drainage area, relative to the same concentration point (Calle del Pantera).

The results of this analysis indicate that, for the 5-year, 10-year, 25-year, and 100-year events, the peak discharges computed using the entire watershed area (Method 1) exceeded the peak rate determined by combining runoff hydrographs (Method 2) from the individual drainage areas. However, it was observed that during the 2-year event, the combined discharge associated with Method 2 was greater than the one associated with Method 1.

During the 2-year event, the peak discharge associated with Method 2 was approximately 24 percent higher than the peak discharge associated with Method 1. However, it should be noted that Method 2 does not account for attenuation due to wedge storage; nor does it account for transmission losses within the channel. Therefore, the reflected increase is likely greater than would be expected under actual conditions. On the other hand, Method 1 does, to some extent, internally account for these attenuation effects when the time of concentration is computed. The summary sheets associated with the hydrograph analyses are provided as Figure 13.

It was also noted that, with respect to Swan Road, the localized drainage areas acting independently of the Valley View Wash watershed generate a significant quantity of runoff in a very short period of time. Therefore, the combined discharge from the local basins arrive first. They are then followed by the flows generated in the upstream portion of the Valley View Wash watershed. This effectively subjects the study area to a significant quantity of flow over an extended time frame, assuming that rainfall is uniformly distributed over the entire watershed. However, if the areal extent of the storm does not include the upper limits of the Valley View Wash watershed, the flow rates will decline almost as rapidly as they will rise. However, there is no doubt that the localized drainage areas subject the Calle del Pantera subdivision to a significant quantity of flow on a more frequent basis, since localized thunderstorms that do not encompass the entire watershed represent a more common occurrence.

Since all of the runoff generated east of Swan Road is captured by the recently constructed storm-drain system, the most downstream segment, a 66-inch RCP, was rated to verify that its design capacity was indeed the 100-year peak discharge associated with this drainage area. If this conduit conveys the entire 100-year peak discharge, it might be feasible to relocate this structure such that flows generated in this drainage area could be diverted around the Calle del Pantera Subdivision. This might significantly reduce the quantity of runoff impacting the subdivision.

The results of this analysis indicate that the 66-inch RCP has a uniform flow capacity equal to approximately 450 cfs. The 100-year peak discharge for the associated drainage basin (Concentration Point 1) was determined to be approximately 435 cfs. Therefore, assuming that the inlets to the individual catch basins and the upstream mains and laterals can collect and convey their

2-YEAR
HYDROGRAPH LAGGING DATA

PEAK DISCHARGE MAGNITUDES & TIME TO PEAK

CP#11 223 cfs @ 80 MIN.
CP#2 & #3 195 cfs @ 16 MIN.

AVE FLOW VELOCITIES FOR QP/2 FROM HEC-2

CP#11 → CP#4 : 4.31 fps (SUB)
SEC. 27 — 35 5.35 fps (SUPER)

CP#2 & CP#3 → CP#4 : 4.85 fps (SUB)
SEC. 27 — 30 5.30 fps (SUPER)

TRAVEL DISTANCES & TIMES (SUPER-VELOCITY)

CP#11 → CP#4 : 3900 ft IN 12.15 MIN

CP#2 & 3 → CP#4 : 2150 ft IN 6.76 MIN.

RESULTS (LAGGED SUB-BASINS VERSUS COMPOSITE)

Q_p @ CP#4 (LAGGED) = 299 cfs
Q_p @ CP#11.3 (COMPOSITE) = 241 cfs

FIGURE 13
HYDROGRAPH ANALYSIS SUMMARY SHEETS
(SHEET 1 OF 15)

2-YEAR
COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q TOTAL (CFS)
0	0			0
1	1.4			1.4
2	3.1			3.1
3	6.5			6.5
4	10.1			10.1
5	14.2			14.2
6	18.5			18.5
7	23.1	3.1		26.2
8	28.3	7.9		36.2
9	34.1	15.7		49.8
10	39.9	24.5		64.4
11	45.8	34.0		79.8
12	52.5	44.3	1.0	97.8
13	59.5	56.1	1.6	117.2
14	68.3	68.7	3.0	140.0
15	77.3	81.7	3.1	162.1
16	85.6	96.0	4.4	186.0
17	93.7	111.4	4.7	209.8
18	100.0	130.1	5.9	236.0
19	103.5	148.9	7.0	259.4
20	99.2	167.2	9.7	276.1
21	94.8	183.2	10.8	288.8
22	90.1	195.1	12.2 13.4	297.4 298.6 *
23	85.4	192.3	14.6	292.3
24	80.7	188.7	17.1	286.5
25	76.3	184.4	18.5	279.2

COMPOSITE HYDROGRAPH LISTING

METHOD B

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q_{TOTAL} (CFS)
26	71.9	179.5	20.9	272.3
27	67.7	173.3	22.7	263.7
28	63.6	163.4	25.3	252.3
29	60.2	153.8	27.2	241.2
30	56.9	144.6	29.7	231.2
31	53.4	136.4	31.7	221.5
32	49.9	127.8	34.2	211.9
33	46.7	119.3	36.1	202.1
34	43.6	111.3	38.6	193.5
35	40.6	103.7	41.2	185.5
36	37.8	96.5	43.7	178.0
37	35.1	89.5	46.2	170.8
38	32.6	83.2	48.7	164.5
39	30.3	77.5	51.3	159.1
40	28.0	71.7	53.8	153.5
41	25.7	66.1	56.3	148.1
42	23.9	61.3	59.3	144.5
43	22.1	56.6	62.6	141.3
44	20.4	52.1	65.6	138.1
45	18.7	48.4	68.7	135.8
* 87	1.2	2.4	232.8	236.4 *
91	1.0	2.4	223	226.4

FIGURE 13
3 OF 15

* PEAK VALUES ASSUMING SWAN RD CROSSING IS UPGRADED

5-YEAR HYDROGRAPH LAGGING DATA

PEAK DISCHARGE MAGNITUDES & TIME TO PEAK

CP #11 536.2 cfs @ 62 MIN
CP #2 & #3 289.9 cfs @ 15 MIN

AVE FLOW VELOCITIES FOR QP/2 FROM HEC-2

CP #11 → CP #4 : 5.34 fps (SUB)
6.89 fps (SUPER)

CP #2 & CP #3 → CP #4 : 5.26 fps (SUB)
5.85 fps (SUPER)

TRAVEL DISTANCES & TIMES (SUPER-VELOCITY)

CP #11 → CP #4 : 3900 ft IN 9.43 MIN

CP #2 & 3 → CP #4 : 2150 ft IN 6.12 MIN

RESULTS (LAGGED SUB-BASINS VERSUS COMPOSITE)

Q_P @ CP #4 (LAGGED) = 549 cfs.
Q_P @ CP #11.3 (COMPOSITE) = 606 cfs.

FIGURE 13
SHEET 4 OF 15

5-YEAR
COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q_{TOTAL} (CFS)
0	0			0
1	2.2			2.2
2	5.3			5.3
3	10.8			10.8
4	16.9			16.9
5	23.4			23.4
6	30.6	5.0		35.6
7	38.0	12.9		50.9
8	47.2	25.2		72.4
9	56.4	39.1	5.2	100.7
10	65.8	54.3	5.5	125.6
11	76.1	70.4	10.4	156.9
12	87.3	89.9	10.9	188.1
13	100.9	109.7	15.6	226.2
14	115.4	130.5	19.8	265.7
15	128.7	154.3	28.6	311.6
16	141.1	180.9	33.1	355.1
17	151.2	210.7	41.7	403.6
18	152.7	239.2	46.5	438.4
19	145.9	265.5	54.8	466.2
20	138.5	289.9	61.8	490.2
21	130.9	285.6	70.0	486.5
22	123.5	280.4	76.9	480.8
23	116.2	273.6	83.0	472.8
24	109.2	266.3	89.4	464.9
25	102.4	253.6	97.5	453.5

FIGURE 13
SHEET 5 OF 15

5-YEAR

COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q_{TOTAL} (CFS)
26	95.8	238.3	106.3	440.4
27	90.3	223.2	115.2	428.7
28	85.0	209.7	122.0	416.2
29	79.4	196.4	127.2	403.0
30	73.8	182.9	134.2	390.9
31	68.7	170.1	142.9	381.7
32	63.8	157.7	152.6	374.1
33	59.1	146.4	161.9	367.4
34	54.7	135.3	169.2	359.2
35	50.3	125.1	177.6	353.0
36	46.7	116.1	186.8	349.6
37	43.0	107.0	196.7	346.7
38	39.4	97.9	206.0	343.3
39	36.2	90.5	211.9	338.6
40	33.4	83.2	218.6	335.2
41	30.6	76.0	226.1	332.7
42	27.9	70.4	234.4	332.7
43	26.0	64.8	242.2	333.0
44	24.0	59.5	248.3	331.8
45	22.1	54.9	255.2	332.2
↓ 64	↓ 5.7	↓ 12.7	↓ 592.1	↓ 608.1 *
70	3.9	8.6	536.2	548.7

* PEAK VALUE ASSUMING SWAN RD CROSSING IS UPGRADED

10-YEAR HYDROGRAPH LAGGING DATA

PEAK DISCHARGE MAGNITUDES & TIME TO PEAK

CP #11	838.2 @ 47 MIN
CP #2 & #3	353.7 @ 15 MIN

AVE FLOW VELOCITIES FOR $Q_P/2$ FROM HEC-2

CP #11 → CP #4:	5.67 fps (SUB)
	7.33 fps (SUPER)
CP #2 & CP #3 → CP #4:	5.45 fps (SUB)
	6.18 fps (SUBER)

TRAVEL DISTANCES & TIMES (SUPER-VELOCITY)

CP #11 → CP #4: 3900 ft IN 8.87 MIN

CP # 2 & 3 → CP #4: 2150 ft IN 5.80 MIN

RESULTS (LAGGED SUB-BASINS VERSUS COMPOSITE)

Q_P @ CP #4 (LAGGED) = 878 cfs

Q_P @ CP # 11.3 (COMPOSITE) = 941 cfs

10-YEAR

COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (cfs)	Q@CP2&3 (cfs)	Q@CP11 (cfs)	Q_{TOTAL} (cfs)
0	0			0
1	3			3
2	7.4			7.4
3	14.7			14.7
4	22.9			22.9
5	31.5			31.5
6	41.4	6.0		47.4
7	52.0	15.7		67.7
8	64.2	30.6		94.8
9	76.5	47.6	9.6	133.7
10	88.9	66.2	10.2	165.3
11	103.7	85.8	19.2	208.7
12	120.1	108.6	20.3	249.0
13	139.2	133.8	36.6	309.6
14	157.3	158.9	45.0	361.2
15	174.7	187.7	60.9	423.3
16	188.0	220.0	69.4	477.4
17	193.0	256.5	82.3	531.8
18	184.0	290.9	92.1	567.0
19	174.3	323.9	106.4	604.6
20	164.3	353.7	120.4	638.4
21	154.5	348.8	127.6	630.9
22	144.9	342.9	138.9	626.7
23	135.7	334.2	152.5	622.4
24	126.8	323.0	164.9	614.7
25	118.2	303.9	175.9	598.0

FIGURE 13

SHEET 8 OF 15

COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q_{TOTAL} (CFS)
26	111.2	285.0	188.9	
27	104.1	266.5	204.1	
28	96.6	249.5	213.6	
29	89.6	233.9	224.5	
30	82.8	217.7	236.9	537.4
31	76.7	202.2	246.5	
32	70.7	187.2	256.0	
33	64.9	173.4	266.7	
34	59.8	160.7	275.1	
35	55.0	148.2	282.9	486.1
36	50.1	137.1	291.8	
37	45.8	126.0	301.2	
38	42.1	115.1	311.1	
39	38.4	106.1	321.8	
40	34.9	97.4	329.8	462.1
41	32.4	88.8	337.1	
42	29.8	82.1	345.5	
43	27.3	75.5	375.5	
44	25.2	69.2	405.6	
45	23.1	63.6	443.6	530.3
53	12.6	33.3	880.1	926 *
55	10.9	28.5	838.2	877.6

* PEAK VALUE ASSUMING SWAN RD CROSSING IS UPGRADED

FIGURE 13

25 - YEAR HYDROGRAPH LAGGING DATA

PEAK DISCHARGE MAGNITUDES & TIME TO PEAK

CP #11 — 1267.6 cfs @ 41 MIN
CP #2 & #3 — 444.8 cfs @ 15 MIN.

AVE FLOW VELOCITIES FOR $Q_P/2$ FROM HEC-2

CP #11 → CP #4: 6.14 fps (SUB)
8.05 fps (SUPER)
CP #2 & CP #3 → CP #4: 5.75 fps (SUB)
6.54 fps (SUPER)

TRAVEL DISTANCES & TIMES (SUPER-VELOCITY)

CP #11 → CP #4: 3900 ft IN 8.07 MIN

CP # 2 & 3 → CP #4: 2150 ft IN 5.48 MIN.

RESULTS (LAGGED SUB-BASINS VERSUS COMPOSITE)

Q_P @ CP #4 (LAGGED) = 1338 cfs
Q_P @ CP # 11.3 (COMPOSITE) = 1400 cfs

FIGURE 13
SHEET 10 OF 15

25 - YEAR

COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (cfs)	Q@CP2&3 (cfs)	Q@CP11 (cfs)	Q TOTAL (cfs)
0	0			0
1	4			4
2	10.5			10.5
3	20.4			20.4
4	31.8			31.8
5	43.9	8.1		52.0
6	57.1	22.0		79.1
7	72.6	42.5		115.1
8	89.1	65.6	15.8	170.5
9	105.9	91.0	16.8	213.7
10	124.3	119.0	31.7	275.0
11	144.3	150.7	33.5	328.5
12	169.9	184.2	70.7	424.8
13	195.0	220.2	74.9	490.1
14	218.8	259.5	95.2	573.5
15	238.1	307.8	113.2	659.1
16	249.8	355.2	128.2	733.2
17	237.7	400.8	143.9	782.4
18	224.6	441.1	162.3	828.0
19	211.1	444.8	176.7	832.6
20	197.8	437.1	195.1	830.0
21	184.9	425.8	211.0	821.7
22	172.4	414.2	224.3	810.9
23	160.5	389.9	239.8	790.2
24	149.5	364.9	251.2	765.6
25	140.0	340.1	264.7	744.8

FIGURE 13
SHEET 11 OF 15

25 - YEAR

COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q_{TOTAL} (CFS)
26	130.1	317.8	276.2	
27	120.2	396.6	286.9	
28	110.9	274.8	298.8	
29	102.2	254.3	311.2	
30	93.9	234.7	324.9	653.5
31	86.1	216.2	333.1	
32	78.8	199.4	342.3	
33	72.2	183.8	370.5	
34	65.7	168.8	406.4	
35	59.5	154.0	458.7	672.2
36	54.6	141.2	533.7	
37	49.6	129.2	604.0	
38	44.8	117.6	691.3	853.7
47	20.4	52.9	1306.1	1379.4 *
48	19.0	51.0	1267.6	1337.6

* PEAK VALUE ASSUMING SWAN RD CROSSING IS UPGRADED

FIGURE 13
12 OF 15

100-YEAR HYDROGRAPH LAGGING DATA

PEAK DISCHARGE MAGNITUDES & TIME TO PEAK

CP #11	2127.4 @ 36 MIN
CP #2 & #3	639.1 @ 14 MIN

AVE FLOW VELOCITIES FOR $Q_P/2$ FROM HEC-2

CP #11 → CP #4:	7.09 fps (SUB)
	9.21 fps (SUPER)
CP #2 & CP #3 → CP #4:	5.47 fps (SUB)
	6.91 fps (SUPER)

TRAVEL DISTANCES & TIMES (SUPER-VELOCITY)

CP #11 → CP #4: 3900 ft IN 7.05 MIN

CP #2 & 3 → CP #4: 2150 ft IN 5.19 MIN

RESULTS (LAGGED SUB-BASINS VERSUS COMPOSITE)

Q_P @ CP #4 (LAGGED) = 2257 cfs.

Q_P @ CP #11.3 (COMPOSITE) = 2345 cfs.

FIGURE 13

SHEET 13 OF 15

100-YEAR
COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (cfs)	Q@CP2&3 (cfs)	Q@CP11 (cfs)	Q_{TOTAL} (cfs)
0	0			0
1	5.9			5.9
2	16.2			16.2
3	30.8			30.8
4	48.1			48.1
5	66.5	12.1		78.6
6	86.1	34.2		120.3
7	110.5	65.2	30.7	206.4
8	135.0	100.0	32.5	267.5
9	159.8	138.5	61.5	359.8
10	189.4	182.6	83.7	455.7
11	223.3	230.9	114.2	568.4
12	261.6	281.2	136.1	678.9
13	296.7	337.9	162.3	796.9
14	327.6	402.4	186.4	916.4
15	353.8	475.3	211.6	1040.7
16	335.8	545.9	231.8	1113.5
17	316.8	609.1	250.3	1176.2
18	296.7	639.1	269.0	1204.8
19	277.1	629.8	282.7	1189.6
20	258.1	613.1	298.3	1169.5
21	239.7	588.4	315.5	1143.6
22	222.2	549.9	330.8	1102.9
23	206.6	511.7	343.4	1061.7
24	192.6	474.8	388.0	1055.4
25	177.8	441.4	450.2	1069.4

FIGURE 13

100-YEAR
COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP2&3 (CFS)	Q@CP11 (CFS)	Q TOTAL (CFS)
26	163.5	409.9	555.5	1128.9
27	149.8	377.6	662.8	1190.2
28	137.6	347.4	809.1	1294.1
29	125.6	318.9	931.9	1376.4
30	114.1	293.2	1039.6	1446.9
31	104.4	268.2	1173.2	1545.8
32	94.7	245.3	1294.0	1537.0
33	85.1	223.2	1406.5	1714.8
34	77.8	202.0	1513.6	1793.4
35	70.5	184.4	1614.8	1869.7
41	40.0	103.4	2196.2	2339.6 *
42	36.0	93.5	2127.4	2256.9

* PEAK VALUE ASSUMING SWAN RD CROSSING IS UPGRADED

FIGURE 13
15 OF 15

respective flows, the 66-inch RCP was sized to convey the entire 100-year peak discharge.

If runoff conveyed within the 66-inch RCP is to be diverted around the Calle del Pantera subdivision, a similar conduit must be constructed within the Swan Road right-of-way. This conduit would extend along the western side of Swan Road from just south of Sunrise Drive to Calle del Pantera (see Figure 2). All flow would then be released into Tributary "X", which joins the Valley View Wash on the downstream side of the Calle del Pantera area.

The results of the relocation analysis indicates that the lagged 2-year peak discharge (Method 2) impacting the Calle del Pantera area would be reduced by approximately 25 percent under this scenario. However, the reduction associated with Method 1 during the 2-year event was limited to approximately six percent. The associated reductions for the remaining discharge events were even less significant. Since homes in the area will not be inundated during the 2-year or 5-year events, there appears to be no justification for relocating the existing conduit.

In addition, it was noted that (1) each of the newly constructed driveway entrances would need to be removed and reconstructed to accommodate the relocated conduit; (2) the western right-of-way, which was recently landscaped, would be significantly disturbed--thus requiring a major relandscaping effort; and (3) excessive burial depths would be encountered.

Ignoring the costs associated with these items, the approximate cost of the conduit alone would be on the order of $345,000. This dollar amount was determined by assuming that standard materials and installation costs would amount to approximately two dollars per inch (diameter) per linear foot. Therefore, it was concluded that it is not cost effective to relocate the 66-inch RCP such that flows would be re-routed around the study area. The results of this relocation analysis are summarized on Figure 14.

ANALYSIS OF THE FEASIBILITY OF RELOCATING THE EXISTING 66-INCH RCP SUCH THAT THE OUTLET WOULD DISCHARGE INTO THE VALLEY VIEW WASH DOWNSTREAM OF THE CALLE DE PANTERA AREA

I. COMPARISON OF COMPUTED PEAK DISCHARGE MAGNITUDES*

	CONC. PT	2-YR	5-YR	10-YR	25-YR	100-YR
COMPOSITE	11.3 w/	241	606	941	1400	2345
	11.3 w/o	219	550	855	1272	2131
LAGGED	4 w/	299	549	878	1338	2257
	4 w/o	224	550	874	1327	2242

II PRELIMINARY DESIGN PARAMETERS

ASSUME 66" WOULD EXTEND FROM STATION 364+75, 63.5 LT TO 338+67, 100' LT

℄ @ 364+75, 63.5' LT	=	2709.21	
℄ @ 338+67, 100' LT	=	2651.2	
DISTANCE	2608 ft		58 ft

CONTINUOUS SLOPE 2.22%

MAXIMUM BURIAL DEPTH @ RIDGE 75 ft
WITH ℄ @ 2684.75 & GROUND ELEV. @ 2760.0

III STANDARD INSTALLATION COST

ASSUME \$2.00 /IN./LF OR \$132.00 /LF

TOTAL ESTIMATED COST \$344,256.00

IV CONCLUSION

A. INSIGNIFICANT REDUCTION IN PEAK DISCHARGE (5-YR TO 100-YR)

B. NOT COST EFFECTIVE

FIGURE 14

V. CHANNELIZATION/EROSION-CONTROL STUDY

Considering the results of the previous analysis, the only re-routing of flows that would be effective in mitigating the current drainage problems and flood hazards within the Calle del Pantera area would involve construction of a diversion channel through the area itself. To provide the maximum protection for the area, this channel should capture the entire 100-year peak discharge and convey it around the subdivision. This diversion channel could rejoin the natural channel a short distance downstream of the subdivision. The diversion channel could be designed and constructed in such a manner so as to minimize its aesthetic impacts. However, since it was not certain that the benefits derived from a diversion channel could justify the associated design, construction, and landscaping costs, a feasibility study was required.

Therefore, the purpose of this portion of the overall study was to evaluate the cost effectiveness of providing a diversion channel adjacent to the subdivision. In addition, the erosion hazards associated with the Calle del Pantera subdivision and the Valley View Reservoir site were evaluated as part of this study. Both short-term and long-term mitigation measures were considered. The general design requirements and anticipated construction costs were provided.

With regard to the concept channelization scheme, the first step was to select the most appropriate alignment. The primary goal was to select an alignment that would (1) minimize the length of the channelization reach, and (2) minimize the physical and aesthetic impacts on the neighborhood. After due consideration, there was only one alignment that met these criteria. This alignment is shown on Figure 2.

The diversion channel would begin at the upstream limit of the western low-flow swale, as described in Section II. The alignment would, for the most part, correspond to the alignment of the western swale as it crosses Calle del Pantera and traverses the western boundary of Lots 498 and 499. However, the channel would not be located on these lots. Instead, the channel would be located within a 50-foot easement that would parallel the western boundary of Flecha Caida Ranch Estates #9. The diversion channel would then join the main channel, which is located approximately 450 feet downstream of the subdivision's southern boundary.

If existing structures (i.e., homes, churches and associated improvements) are to remain, the top width of this channel could not exceed 50 feet. This assumes that a separate access and maintenance road is not required. If such a road is required to meet Pima County's channel design standards (Reference 5), the top width could not exceed 34 feet.

Therefore, the initial analysis assumed a 50-foot channel would be provided with a design slope equal to approximately 2.8 percent. This slope corresponds to the average existing slope along the proposed alignment. Since flow velocities within this channel would exceed 18 feet per second, the banks would need to be protected to prevent extensive erosion. In addition, approximately 20 grade-control structures would be required, under the assumption that the

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projected equilibrium slope of the channel would become approximately equal to one percent.

This concept design assumes that the drop associated with each grade-control structure would be limited to two feet. However, due to the concentrated flow conditions associated with this concept channel, it is likely that the equilibrium slope will be significantly flatter than one percent. Therefore, it may be necessary to provide more than 20 grade-control structures along the channel.

Considering the magnitude of these improvements alone, it does not appear that a 100-year diversion channel would be cost effective, considering the limited number of homes that are flooded during the 100-year event. In addition, a channel of this type would have a very negative impact upon the area, from both an aesthetic standpoint and an erosion standpoint.

Although the design discharge could be reduced to accommodate more frequent flow events, the design requirements and the negative impacts would not be significantly reduced. Therefore, initial findings indicate that a diversion channel is not a "preferred" flood-control alternative for the area. Since Pima County concurred with these initial findings and conclusions, the diversion-channel feasibility study was terminated to concentrate on the erosion-mitigation study.

With regard to local erosion within the study area, which includes Tucson Water's Valley View Reservoir site, the results of recent field investigations indicate that long-term degradation stands out as the most significant erosion problem along the primary and secondary channels that traverse the study area. A major cause of this problem is urbanization within the associated watershed. The term urbanization includes both residential and commercial developments, and the construction of roadways to serve these developments.

Urbanization has not only increased the frequency and volumes of stormwater runoff, but it has, at the same time, reduced the quantity of sediment supplied to the study area. As a result, channel grades within the area are decreasing in an effort to balance sediment-transport capacity, associated with the primary and secondary watercourses, with the upstream sediment supply.

In general, the pivot points around which the beds are adjusting their grades are located at channel confluences, and at the various at-grade roadway crossings within the study area. The problem areas that warrant the most attention are those areas which are located immediately downstream of these at-grade road crossings.

Although there are several locations within the Calle del Pantera area where long-term degradation is evident, there are only seven locations along the Valley View Wash watercourse that will require special attention, either now or in the near future. These seven locations are shown on Figure 2. A brief description of each of these sites is provided in the following paragraph.

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Erosion Sites #1 and #2 are located immediately downstream of the reservoir access road. Site #3 is located immediately downstream of Calle del Pantera, along the western branch of the Valley View Wash. Site #4 is located immediately downstream of the access drive for Lot 501 (Flecha Caida Ranch Estates #9). Site #5 is located on the downstream side of Cerco de Corazon, near the northwest corner of Lot 497. Site #6 is located immediately downstream of the access drive for Lot 503. Site #7 is located immediately downstream of Cerco de Corazon Circle, within the boundary of Lot 504.

Of the seven sites just described, all will require some form of structural improvement to prevent the roadway and associated structures from being undermined. Currently, the channel-bed elevation, at each of the locations described above, is one to three feet below the corresponding roadway elevation. Left unchecked, the ultimate drop height at selected locations could reach as much as ten feet.

Due to the importance of maintaining access to the reservoir, and due to the severity of erosion that currently exists, a "quick-fix" approach (short-term solution) is needed at Site #1 and, to some extent, at Site #2, while a long-term mitigation plan is designed and funding established. Since the remaining erosion sites are less sensitive, short-term mitigation measures are not needed at this time. However, to control erosion on a permanent basis, the long-term solution relative to these sites could be constructed in phases, as funds become available. It should also be noted that these sites are located on private property, which complicates funding and the right to access.

140 yd³
23 y

With regard to the short-term, erosion-control measure associated with Site #1, it is recommend that approximately 140 cubic yards of rock riprap be placed immediately downstream of the access road within the primary channel. The rock riprap should be installed in such a manner so as to form a blanket that is approximately 60 feet long, approximately 18 feet wide, and approximately 3.5 feet thick. The incline of the blanket from the top of the access road to the existing flow line of the primary channel should be approximately six feet horizontal to one foot vertical. The D_{50} diameter of the rock should be 12 inches, or more.

Before the rock-riprap blanket is installed, the existing gunite apron and underlying rubble should be removed. A filter blanket should then be provided before the riprap blanket is installed. In addition, care must be taken to ensure that the electrical conduit that parallels the roadway is not damaged during installation of the riprap blanket. It may be necessary to hand place the rock in the immediate vicinity of the conduit. Once the rock surrounding the conduit is secure, all remaining rock could be dumped in place before it is shaped to form the blanket.

A similar design could be provided at Site #2. However, the need for short-term improvements to this site are not as critical as it is at Site #1. In addition, the quantity of rock required is significantly reduced. Overall, the blanket at this site should be approximately 15 feet long, approximately 12 feet wide and approximately 3.5 feet thick. Therefore, only approximately 23 cubic yards of rock is required at this location. As with Site #1, the rock riprap blanket should be placed over a filter blanket at a 6:1 slope. The D_{50} diameter of this blanket should be a minimum of 10 inches.

The design calculations for the rock riprap blanket associated with Site #1 are provided in Appendix G. These design calculations also apply to Site #2.

The long-term, erosion-control measures associated with Site #1 and Site #2 will require the installation of three grade-control structures along the Valley View Wash between Calle Del Pantera and the reservoir access road. The location of these structures are shown on Figure 15. The long-term measures associated with Sites #3 through #7, as shown on Figure 2, will require the installation of cut-off walls and splash pads (or aprons) at selected locations. It is recommended that gabions be used to create each of these structures. A typical plan view of the proposed structures is provided as Figure 16. Typical cross-sections are provided as Figure 17.

With respect to Site #1 and Site #2, the reach length between the proposed grade-control structures was first approximated using Equation 9-V, as provided in Pima County's drainage and channel design manual (Reference 5). The ultimate height of the drop was assumed equal to three feet. This value was considered to be the maximum allowable, for safety reasons.

The stable or "equilibrium" slope was assumed to be approximately one-third of the natural slope associated with the reach under consideration. Therefore, the "equilibrium" slope for the reach located in the immediate vicinity of the reservoir access road and for the reach that traverses Flecha Caida Ranch Estates #9 was determined to be approximately one percent. However, the "equilibrium" slope for the reach located immediately upstream of Calle Del Pantera was determined to be approximately 0.7 percent.

The first gabion grade-control structure should be located approximately 110 feet downstream of the reservoir access road (see Figure 15). However, in an effort to eliminate the three-foot drop that currently exists on the downstream side of the access road, the crest of this structure should be placed approximately four feet above the existing flow-line elevation at this location. All low areas between the access road and this first grade-control structure should then be backfilled in such a manner so as to create a one percent slope from the crest of this structure upstream to the cut-off walls that currently exist at Site #1 and Site #2.

The two subsequent grade-control structures should be placed such that their crest elevations correspond to the existing flow-line elevations at the respective locations. Based on the locations shown on Figure 15, each structure should be placed at 250-foot intervals. Therefore, the ultimate drop height associated with the first structure would be approximately seven feet. This includes the initial four-foot drop and the ultimate three-foot degradation depth. However, a stepped gabion structure is recommended in order to limit the drop associated with each step to a maximum of three feet. This could be

FIGURE 15

VALLEY VIEW RESERVOIR SITE MAP SHOWING ASSOCIATED EROSION SITE, LOCATION OF EROSION-CONTROL STRUCTURES, AND CROSS-SECTION LOCATIONS USED IN THE HYDRAULIC DESIGN ANALYSIS

sla

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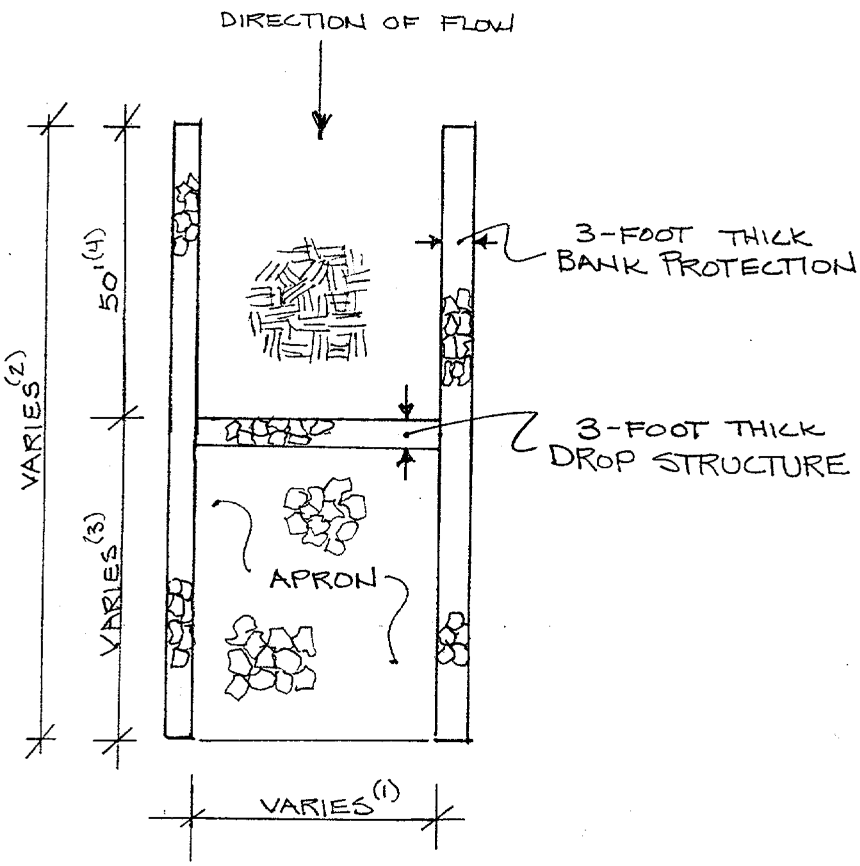
CLIENT PIMA COUNTY JOB NO. PAZ-PDOT07 PAGE 1 OF 1

PROJECT ______ DATE CHECKED ______ DATE 11-10-88

DETAIL ______ CHECKED BY ______ COMPUTED BY RLS

TYPICAL PLAN VIEW
OF
GABION GRADE-CONTROL/DROP STRUCTURE



(1) WIDTH OF APRON MATCHES WIDTH OF PRIMARY OR LOW-FLOW CHANNEL

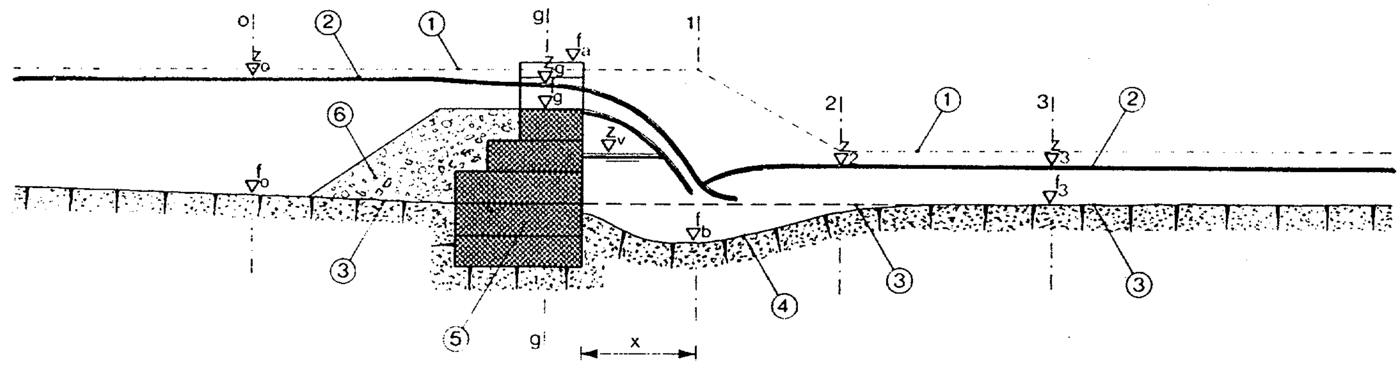
(2) OVERALL LENGTH FOR SINGLE DROP WOULD BE APPROX. 103 FT.; FOR STEPPED STRUCTURE LENGTH WOULD BE APPROX. 139 FT

(3) DOWNSTREAM LENGTH FOR SINGLE DROP WOULD BE APPROX. 53 FT; FOR STEPPED STRUCTURE LENGTH WOULD BE APPROX. 83 FT; FOR EROSION SITES 5, 6 & 7 LENGTH WOULD BE APPROX. 43 FT.

(4) DOES NOT APPLY TO EROSION SITES 5, 6 & 7.

FIGURE 16

PLAN VIEW OF PROPOSED GABION STRUCTURES

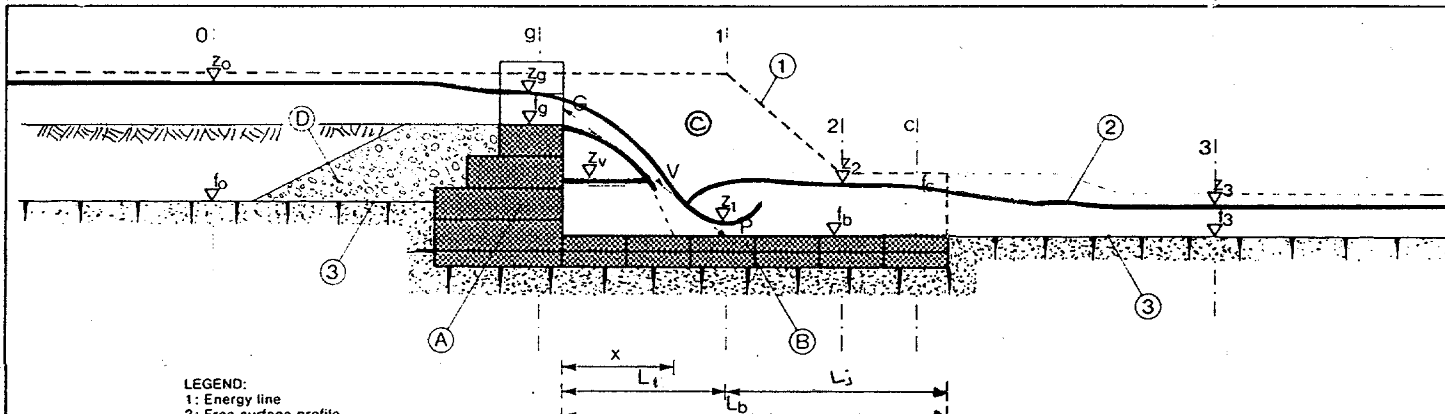


LEGEND:
1: Energy line
2: Free surface profile
3: Original river bed
4: Profile of max. bed scour
5: Gabion weir
6: Backfill
X: Distance of the free fall from the downstream face of the weir

ELEVATIONS:
z: Water levels
f: River bed and structure elevations
a: Weir wings elevation

SECTIONS:
0: Section upstream of the weir
g: Section at crest
1: Section at initial depth
2: Section at sequent depth
3: Section downstream of the weir

TYPICAL SECTION OF DROP STRUCTURE WITHOUT A DOWNSTREAM APRON



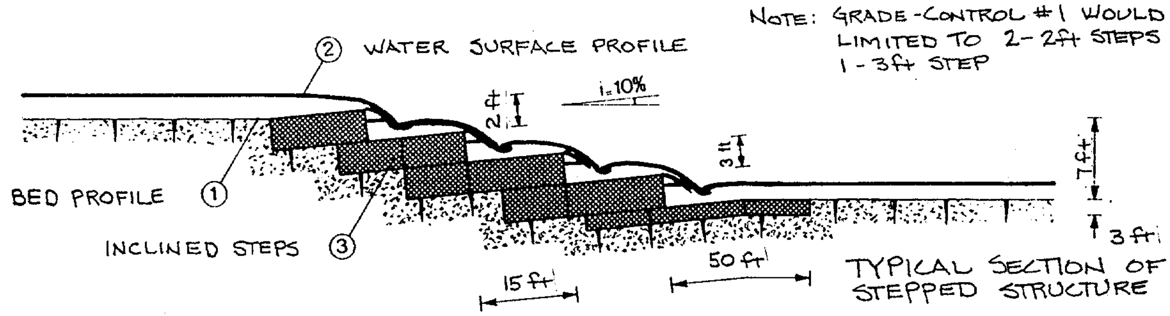
LEGEND:
1: Energy line
2: Free surface profile
3: Original river bed
4: Profile of river bed after training
A: Weir
B. APRON
C. BANK PROTECTION
D. ORIGINAL BED OR BACKFILL
X: Distance of the free fall from the downstream face of the weir

ELEVATIONS:
z: Water levels
f: River bed and structure elevations
a: Elevation of wings of weir

SECTIONS:
0: Section upstream of the weir
g: Section at crest
1: Section at max. bed scour
2: Section at sequent depth
c: Section at counterweir
3: Section downstream of the weir

TYPICAL SECTION OF GRADE-CONTROL STRUCTURE WITH DOWNSTREAM APRON

NOTE: GRADE-CONTROL #1 WOULD BE LIMITED TO 2-2ft STEPS AND 1-3ft STEP



TYPICAL SECTION OF STEPPED STRUCTURE

FIGURE 17

CROSS SECTION OF PROPOSED GABION STRUCTURES

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accomplished by providing two 24-inch steps and one 36-inch step. The ultimate drop height for the two subsequent structures would be limited to approximately two feet.

The concept design of the splash pad or apron which must be constructed in conjunction with the proposed drop structures is based upon the procedures outlined in References 6 and 7. However, before it was decided that an apron would be required, the depth of scour on the downstream side of the drop was computed to determine if the depth was excessive enough to justify the use of an apron to dissipate energy. Since the average depth of scour for the conditions studied ranged from six to seven feet, it appears that a moderately-sized grade-control structure with an apron would be more cost-effective than would be a massive grade-control structure.

The equations used to estimate the depth of scour varied depending on whether or not the structure in question was submerged during the design flow event (100-year event). The degree of submergence was determined from individual backwater analyses of the ultimate channel section under equilibrium conditions. The input/output listings associated with these hydraulic analyses are contained in Appendix H.

Since the results of the hydraulic analyses indicate that the first grade-control structure will not be submerged by the associated downstream tailwater, the Veronese equation, which is provided in Reference 5 as Equation 10-V, was used. However, the scour depth associated with the remaining structures was determined using a modified version of an equation which was developed by SLA from physical-model studies. These physical-model studies were conducted for Pima County by SLA, and the results are presented in Reference 8.

The overall results of the design analysis indicate that a 50-foot-long apron should be constructed in conjunction with the three proposed grade-control structures associated with Site #1 and Site #2. This apron would be approximately 25 feet wide. This width corresponds to the average width of the existing low-flow channel between Calle Del Pantera and the reservoir access. In addition, some bank protection will be required to protect the individual grade-control structures. The purpose of the bank protection is to ensure that the alignment and geometry of the low-flow channel remains stable in the immediate vicinity of each grade-control structure. It is further recommended that the bank protection be constructed using gabion baskets, which would extend approximately 50 feet upstream and downstream of each grade-control structure.

Collectively, the estimated cost associated with these grade-control structures was determined to be approximately $107,500.00. This estimate includes the grade-control structures, the downstream aprons, and the associated bank protection in both the upstream and downstream directions. The estimated cost assumes that approximately 1800 cubic yards of gabions, with filter fabric, would be required at a cost of approximately $60.00 per cubic yard. The cost of the backfill material was estimated using $4.00 per cubic yard. Approximately 1300 cubic yards of backfill material would be required. Therefore, the additional cost to backfill the area between the first grade-control structure and the reservoir access road is approximately $5200.00.

An alternative to the structural measures described above would be to provide a continuous cut-off wall along the majority of the reservoir access road. The existing cut-off wall is approximately three feet deep, with respect to the top of the roadway. Since the existing flow-line elevation on the downstream side of the access road is approximately three feet below the roadway elevation, the existing cut-off wall could fail during a major flow event. Therefore, the existing wall must either be modified, by increasing the toe-down depth, or be replaced. Replacement would be the most logical and cost-effective approach.

However, installation of a continuous cut-off along approximately 300 feet of the reservoir access road will not eliminate the need for grade-control structures within the downstream reach. Nevertheless, it will significantly reduce the size of the first grade-control structure, and eliminate the backfill requirement. The cost of the first grade-control structure would be reduced by approximately $13,800.00. Considering the additional savings for backfill material, the total savings under this option comes to approximately $19,000.00. Therefore, the total cost of the grade-control structures for this alternative would be approximately $93,700.00. However, a ten-foot-deep cut-off wall would be required to replace the existing three-foot wall. Assuming the wall is twelve inches thick, with structural reinforcement, the cost of the replacement cut-wall would be approximately $30,500.00. Therefore, this cut-off wall alternative would cost approximately $11,500.00 more than would the backfill alternative.

It should be noted that the cost associated with the backfill alternative does not include the cost of landscaping the backfill area. Therefore, the costs associated with these two alternatives are similar. However, the backfill alternative would eliminate a three-foot to four-foot drop on the downstream side of the access road. From a safety standpoint, this alternative would be more desirable. Therefore, the backfill alternative appears to be the most cost-effective, long-term solution to the existing erosion problem at Site #1 and Site #2.

The design computations for the grade-control structures are provided in Appendix G.

With regard to Sites #3 through #7, the severity of erosion that currently exists at each site and the distribution of flow relative to each of these sites warrants the establishment of a priority list for the installation of erosion-control structures.

Based on field observations, Site #6 should be given first priority. Site #7 should be second on the list, followed by Site #5, then Site #4, and finally Site #3. The amount of protection provided will vary from site to site. As previously stated, it is recommend that all erosion-control structures be designed using gabions, as opposed to standard cut-off walls or riprap plunge basins. The use of gabions will provide structures that can be constructed in phases as the need arises, and as funds become available. In addition, gabions provide a more aesthetically pleasing solution for the neighborhood.

Under existing conditions, at Site #5, Site #6, and Site #7 the flow-line elevations on the downstream side of the respective roadway crossings are approximately two to three feet below the roadway elevation. However, at Site #3 and Site #4 the difference between these two elevations (drop height) is limited to approximately one-foot, or less. Again, the severity of erosion at each site is a function of the location of the site with respect to the downstream pivot points; the quantity of flow conveyed in the low-flow channel; and the frequency of flow within the low-flow channel.

Under existing conditions, Site #3, Site #4, and Site #6 are located along the western low-flow swale between Calle del Pantera and Cerco de Corazon Circle. Generally, this channel is not subjected to runoff during the more frequent flow events (i.e., 2-year and 5-year flows). As a result, the severity of erosion is relatively minor, with the exception of Site #6. Since Site #6 is located on the downstream leg of this channel, and is located approximately 485 feet upstream of its natural pivot point, erosion at this location exceeds that at Site #3 and Site #4. Since the stability of the access drive at Site #6 is questionable at this time, this site was ranked number one on the priority list.

Since Site #5 and Site #7 are located along the primary low-flow channel (eastern swale), the severity of erosion at these locations exceeds that associated with Site #3, Site #4 and Site #5. As a result, these sites have received attention in the past in the form of grouted rock protection on the downstream face of the drop and along the adjacent banks. However, these protection measures should fail as the wash continues to degrade.

Both short-term and long-term mitigation measures were considered as part of this study. The short-term measures will provide immediate protection to the access drives and to Cerco de Corazon Circle. The long-term measures, which represent an extension of the short-term measures, will protect the bed and banks of the channel immediately downstream of the roadway and access drives. For the most part, the long-term solution requires the construction of a gabion drop structure which includes a cut-off wall along the downstream edge of the pavement; an energy dissipation apron on the channel bed; and bank protection adjacent to the apron. The short-term measure requires that only the cut-off wall be provided. As additional funding becomes available, the remaining elements can be added. The design requirements associated with the long-term solution, with respect to each site, are discussed in the following paragraphs.

As previously stated, it is recommend that a gabion drop structure be installed on the downstream side of the access road associated with Site #6. This structure will require the installation of a cut-off wall that is approximately 20 feet long; approximately three feet wide; and approximately six feet deep (i.e., from the top of pavement to the toe of the wall). In addition, a 40-foot-long apron and 43 lineal feet of bank protection should be provided adjacent to and downstream of the proposed cut-off wall (see Figure 16). This

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design is intended as a long-term mitigation measure. As such, it will address the existing problem for several years.

In addition, this structure can be upgraded should the wash continue to degrade--thus exposing a portion of the three-foot-thick apron, which will initially be buried. The ultimate degradation depth associated with this site was determined to be approximately eight feet. If and when the wash continues to degrade, additional drop structures, with aprons and bank protection, can be provided as needed. However, to ensure that the access drive is protected, the short-term feature (i.e., a cut-off wall) may need to be constructed in the very near future. This would be followed by the long-term features. A wait-and-see approach can then be followed with regard to continued degradation of the channel bed.

Site #7 might also require implementation of a short-term measure, since it is located along the primary channel (i.e., eastern low-flow swale) and the level of protection that currently exists is not sufficient to protect the roadway with any degree of certainty. Again, long-term features (downstream apron and bank protection) similar to the ones just described in conjunction with Site #6 would address the existing problem for several years, and allow for future additions and/or modifications. The ultimate degradation depth associated with this site was determined to be approximately seven feet.

The existing grouted rock apron at Site #5 should continue to protect the roadway for some time. Therefore, it is not necessary to install a short-term measure at this time. However, as the wash continues to degrade, more of the downstream toe of the existing structure will be exposed, and it will begin to fail. The long-term solution would be to remove the grouted rock apron and install a gabion drop structure, which collectively includes the cut-off wall, the downstream apron, and the bank protection. Again, this structure would be similar to the one described previously; and can be upgraded over time, if need be. The ultimate degradation depth associated with this site was determined to be approximately three feet.

A wait-and-see approach could be applied to both Site #3 and Site #4. However, if Pima County would like to provide some protection at this time, a three-foot-wide by three-foot-high gabion cut-off wall could be installed on the downstream side of the roadway at each location. Again, if the western swale begins to degrade, an apron and associated bank protection could then be added at that time. The ultimate degradation depths associated with Site #3 and Site #4 were determined to be approximately eleven feet and three feet, respectively. However, for the reasons mentioned above, the ultimate depth associated with Site #3 may be overstated.

The initial cost for the long-term gabion structures associated with Site #6 and Site #7 would be approximately $9800.00 per site. The cost associated with the short-term feature would be approximately $1600.00 per site. Assuming that the severity of erosion at Site #5 ultimately requires the installation of a similar long-term structure, the total cost associated with long-term improvements at all three sites would be approximately $29,400. In addition,

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approximately $800.00 per site would be required to provide a 40-foot-long gabion cut-off wall at both Site #3 and Site #4.

VI. CONCLUSIONS AND RECOMMENDATIONS

The Calle del Pantera subdivision (Flecha Caida Ranch Estates #9) has received considerable drainage attention in the last three to four years. A major floodplain delineation project was performed under Phase I of the Flecha Caida Flood Improvement Study, which included the subdivision. Flood-control improvements were provided to protect homes from flooding during the more frequent flow events (i.e., 2-year and 5-year events), and a comprehensive flood-control study was performed in an attempt to develop mitigation measures that would protect the subdivision from inundation during the 100-year event. However, this latter study has demonstrated that it is not feasible to provide 10-year, 25-year, or 100-year protection to the subdivision due to the limited number of homes that are impacted during these events. Therefore, it appears that non-structural measures should be considered for the area.

The two non-structural measures that are most appropriate for the area are the purchase of flood insurance or removal of the affected structures from the flood plain. Clearly, from the property owner's standpoint, flood insurance is the obvious approach. Removal of the structures would require the residents to either relocate temporarily, while an adequate structure is built, or relocate permanently. Either option could place the owner in a situation where they might lose some of their investment in the property. As long as the homes remain in the floodplain, their value will always be questionable. Therefore, the investment potential is significantly limited without the existence of structural flood-control improvements. In addition, it may be difficult to attract prospective buyers when these buyers are informed of the flooding conditions inherent to these selected homes. However, the purchase of flood insurance by the existing owners or prospective owners will, to some degree, offset this negative impact.

Although Pima County is not responsible for the flooding conditions that currently exist, they have initiated this study in an effort to assist the neighborhood in resolving their drainage problems (both flooding and erosion). If Pima County were to accept responsibility for resolving the current situation, the most cost-effective approach would be to purchase the affected homes and physically remove these structures from the floodplain. The lots could then be sold as undeveloped lots or remain as open space. The buyers of these lots, if the lots were sold, would then be required under Pima County's floodplain ordinance (Reference 9) to elevate the finished floors such that they would not be inundated during the regulatory flow event.

With regard to those existing structures that are subject to flooding during the 100-year event, it is recommended that flood insurance be purchased to offset any future losses due to flooding and/or erosion. In addition, lot owners must accept responsibility for protecting their access drives from erosion. Although the erosion problems that exist are directly related to upstream development, they are not uncommon problems. Any disruption to a natural watershed, from the construction of a single home with an access drive to the construction of intense urban centers, can cause these types of local problems. This study specifically addressed the problems and solutions associated with Site #4 and Site #6, which apply to the access drives associated with Lots 501 and 503, respectively. Future problem areas can be addressed in a similar fashion, as the need arises.

Pima County is responsible for maintaining roadways within their rights-of-way. This includes Calle del Pantera and Cerco de Corazon Circle. In response to problems that could be mitigated within public rights-of-way, Pima County provided a soil-cement berm along Calle del Pantera and part of Cerco de Corazon Circle. In addition, they provided the grouted rock protection for the roadway at Site #5, and continue to mitigate the problem that exists at Site #7; which, for the most part, is located on private property (see Figure 2). Their responsibility was expanded, by mutual agreement with Tucson Water, to provide short-term improvements to the reservoir access road--thus providing a long-term solution to the existing erosion problem.

In an effort to assist home owners with their flooding problems, Pima County has also provided improvements within private property. This includes the channel/levee improvement project provided within Lot 500 and 501. However, their ability to resolve all local erosion problems on behalf of the private property owners is limited by available funding. Therefore, we recommend that Pima County and the affected property owners work together to secure a mutual agreement with regard to the phasing and funding of the erosion-control improvements outlined in this study.

VII. REFERENCES

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5. Pima County Department of Transportation and Flood Control District, "Drainage and Channel Design Standards for Local Drainage for Flood Plain Management Within Pima County, Arizona," Adopted on May 15, 1984, Effective on June 1, 1984.

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