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FLOODPLAIN STUDY
FOR
FLECHA CAIDA RANCH ESTATES # 9

INCLUDING PORTIONS OF
FLECHA CAIDA RANCH ESTATES #1 AND #2
AND
LAS LOMAS DE CATALINA

SECTIONS 15 & 22, TOWNSHIP 13 SOUTH, RANGE 14 EAST
PIMA COUNTY, ARIZONA

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January 8, 2008
Revised April 8, 2008

EXPIRES 06/30/2010

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I. INTRODUCTION

Background

The development of Flecha Caida Ranch Estates began in 1956. The subdivision plat for the 40-acre parcel currently known as Flecha Caida Ranch Estates #9 was prepared in December 1959 and recorded in February 1960 (Book 14 at Page 44). The subdivision plats for Flecha Caida Ranch Estates #1 (Book 11 at Page 74) and #2 (Book 12 at Page 69) were prepared and recorded in 1956 and 1957, respectively. The subdivision plat for Las Lomas de Catalina was prepared and recorded in 1978. Copies of the recorded plats are included in Appendix A. The subdivisions occupy portions of Sections 15 and 22 of Township 13 South, Range 14 East, Gila River Base and Meridian, Pima County, Arizona. Portions of the four subdivisions are impacted by the Valley View Wash, which is situated between Valley View Road and Pontatoc Road on the west and Swan Road on the east. This floodplain study of the Valley View Wash begins approximately one-half mile south of Sunrise Drive and extends downstream (south) to North Flecha Drive (see Figure 1).

Within the study reach, the Valley View Wash is a natural, privately-owned, unprotected watercourse that is subject to periodic flooding. In addition, since most of the soils within the foothills and valley region are unconsolidated alluvium, the Valley View Wash watercourse is subject to natural dynamic forces that cause localized erosion and sedimentation. Within the study reach, Flecha Caida Ranch Estates #9 is the most problematic area. Previous studies have determined that approximately one half of the 24 lots that comprise the subdivision are located in the 100-year flood plain for the Valley View Wash. Consequently, numerous drainage-related problems have been noted over the years. Common problems include overbank flooding, inundation of existing homes and secondary structures, bank erosion and degradation along the primary flow paths, and limited access to some lots during times of flooding. Although isolated portions of Flecha Caida Ranch Estates #1 and #2 and Las Lomas de Catalina have, to a lesser degree, experienced similar problems, the primary focus of this study was the sub-reach of the Valley View Wash that traverses the Ranch Estates #9 subdivision.

In response to the concerns of Ranch Estates #9 homeowners following a significant flow event on June 27, 1984, the Pima County Department of Transportation and Flood Control District (the District) commissioned an initial assessment of flooding problems in the subdivision, which was documented in a report prepared by Simons, Li and Associates (SLA) in 1984 (Reference 1). Homeowners at that time were concerned that new commercial developments within the area surrounding the Swan Road-Sunrise Drive intersection had contributed to the flooding observed on June 27th, which was estimated by SLA to be a 5 to 10-year storm event (530 cfs). Although SLA noted that the higher density developments had increased the discharge potential during the 100-year event by approximately five percent, the impact on the depth of flooding was minimal. SLA concluded that drainage problems are inherent to the area primarily because a portion of the subdivision was built in the natural flood plain for the Valley View Wash. This is best illustrated by overlaying a 2007 footprint of the subdivision on the 1941 aerial photograph (Figure 2).

Recognizing the impact a 5 to 10-year storm event had on the Ranch Estates #9 subdivision, the District commission a second study in 1985 to establish regulatory (100-year) discharges and 100-year flood plain maps for several Catalina foothills washes, including the Valley View Wash. This study was completed by SLA in 1986 (Reference 2). A detailed hydrology study was performed for the study using precipitation depths from Reference 3. A copy of the 1986 floodplain map that covers Ranch Estates #9 is included in Appendix A1 along with copies of the relevant hydrologic data sheets.

Between 1985 and 1988, Pima County attempted to address flooding problems on a lot by lot basis and on a regional basis. Two soil-cement berms were constructed along portions of Calle del Pantera and Cerco del Corazon to protect Lots 495, 496, 498, and 499 from all flows up to and including the 5-year event (i.e., the more-frequent flow events). Localized channel improvements, including channel widening, rock riprap bank protection, and two earthen berms were also constructed to provide flood and erosion protection for Lots 500 and 501. Most of these mitigation measures were constructed in 1985 and 1986. To address flooding on a regional basis, the District commissioned a third study in 1988 to evaluate the feasibility of either constructing a regional detention basin within the Tucson Water reservoir site to the north or channelizing a portion of the Valley View Wash to eliminate the flood hazards . This study was the direct result of the 1986 study that identified 15 flood prone structures within the Valley View Wash flood plain, including eight within Ranch Estates #9, five within Ranch Estates #1, and one in Ranch Estates #2. The study also included: (1) a more-detailed analysis of the upstream watershed to determine the relative impact of roadway/storm drain improvements (both existing and proposed) on the flood hazards associated with the subdivision; and (2) a more-detailed floodplain study within the boundary of the subdivision itself. This study was completed by SLA in 1989 (Reference 4). Copies of the revised drainage basin map and the revised floodplain map from the 1989 report for Ranch Estates #9 are included in Appendix A1. Appendices A2 through A4 provide archived copies of the 1984, 1986, and 1989 reports

The 1989 study concluded that regional solutions (i.e., detention and/or channelization) were not cost effective and that roadway/storm drain improvements would have little impact on the flooding. Consequently, the primary focus of the study's recommendations was erosion mitigation on a site by site basis and the purchase of residential flood insurance. Long-term degradation was identified as the most significant erosion problem and seven sites were specifically addressed. Two of the sites were upstream of the neighborhood on Tucson Water property, and five of the sites were within the neighborhood. The five sites and the problems noted at each site are summarized as follows:

Site No.	Description	Problem Noted
3	Downstream of Calle del Pantera, adjacent to northwest corner of Lot 498	1-foot (or less) drop in streambed profile on downstream side relative to roadway.
4	Downstream of access drive to Lot 501	1-foot (or less) drop in streambed profile on downstream side relative to roadway.
5	Downstream of Cerco de Corazon Circle, north boundary of Lot 497	2 to 3-foot drop in streambed profile on downstream side relative to roadway.
6	Downstream of access drive to Lot 503	2 to 3-foot drop in streambed profile on downstream side relative to roadway.
7	Downstream of Cerco de Corazon Circle, south boundary of Lot 497, north boundary Lot 504	2 to 3-foot drop in streambed profile on downstream side relative to roadway.

Given the severity of the problems noted and their potential to worsen as a result of their location relative to downstream control points, the order-of-concern was Site 6, 7, 5, 4, and 3. The long-term or ultimate degradation depth associated with each site was eight, seven, three, three, and eleven feet, respectively. The recommended short-term solution at Sites 5 through 7 was a gabion cut-off wall. The long-term solution included the addition of bank protection and aprons to the short-term structures. Gabions were selected as opposed to conventional cut-off walls or plunge basis to facilitate the addition of the long-term components. A wait and see approach was recommended for Sites 3 and 4.

It should be noted that between 1984 and 1989, the eastern flow path along Calle del Pantera was considered the primary flow path for the more frequent flow events and the western low-flow channel was considered the secondary flow path. In addition, the 1989 study did not specifically address the home on Lot 502, which was constructed in 1986, since it appears that most of the field data (i.e., topographic information and finish-floor elevations) were collected between 1984 and 1986, and initial development of the St Thomas Apostle church occurred in 1987, with additional improvements in 1997.

Subsequent to the 1989 study, some drainage/erosion-mitigation measures were constructed in the neighborhood, including (1) a concrete cut-off wall between Sites 3 and 4; (2) a concrete cut-off wall between the access drive to Lot 502 and 503 (Site 6); (3) a concrete cut-off wall on the downstream side of the access drive to Lot 502; (4) a drainage structure beneath the access drive to Lot 503; and, (5) grouted rock/gunite channel lining immediately downstream of the access drive to Lot 503. It appears that the cut-off wall between Sites 3 and 4 was constructed in conjunction with the 1997 improvements to the church. However, the other mitigation measures appear to have been constructed by the individual property owners.

Purpose

Recent flooding and erosion within the Ranch Estates #9 subdivision during the 2007 summer monsoon season raised the concerns of numerous homeowners who were not aware of the historical drainage problems inherent to the area. The Pima County Regional Flood Control District commissioned this study to (1) address the impact of recent improvements in the

drainage area upstream of the study area; (2) provide updated flood-hazard mapping for the area from just upstream of Calle del Pantera to North Flecha Drive, which is where the Federal Emergency Management Administration (FEMA) flood plain for the Valley View Wash begins; and (3) identify mitigation measures that could be implemented by the affected homeowners to address their flood/erosion hazards.

The key elements of the study as outlined in the scope-of-work are summarized as follows:

- Data collection and field investigation.
- Aerial topographic mapping at 1"=40', one-foot contour interval, including digital aerial photography and ground control. Obtain finish-floor elevations (FFEs) of flood prone structures within the Ranch Estates #9 subdivision. Use the newly acquired topographic information to estimate FFEs for the remaining floodprone structures.
- Revisit 1984 hydrology using NOAA 14 rainfall depths, including multiple return intervals. Compare results to 1984 study.
- Remap 100-year flood plain using new topography. Provide a comparison with the expanded mapping completed in 1989. Identify flood prone structures based on FFEs from survey.
- Identify and evaluate alternative mitigation measures, including their impact on the floodplain. Perform scour analyses as needed, including long-term degradation, to provide preliminary design parameters for the mitigation measures (e.g., floodwalls, bank protection, cut-off walls, etc.).

II. HYDROLOGY

Valley View Wash

In 1984, it was standard engineering practice to conduct hydrologic analyses under the assumption that the upstream drainage area would be fully developed in accordance with the zoning conditions that were in place at the time of the study (i.e., future-conditions hydrology). The 1984 hydrology study performed by SLA was based on future conditions. It included a drainage basin map that outlined the zoning boundaries that were used in the study. Although the majority of the upstream watershed was developed at the time of the study, the most recent (2005) aerial photographs of the watershed were reviewed to determine if these boundaries were still valid.

Based on the results of that review, it was noted that three small areas in the watershed were developed at a greater density than previously assumed. An exhibit showing the location of these areas relative to the key concentration points is provided in Appendix A5. To determine the impact of these higher-density developments, the original hydrologic data sheets were revised accordingly. A comparison between the original values and the updated values is shown in Table 2.1. The revised hydrologic data sheets are provided in Appendix B. The three concentration points (CP) included in the comparison were CP 18, 11.1, and 13. CP 18 is located immediately downstream of the area containing the higher density developments. CP 11.1 is located at Sunrise Drive and CP 13 is located at the northern boundary of the Ranch Estates #1 subdivision.

Table 2.1 Pre- and Post-Development Hydrology (100-year)

Conc. Pt.	Area (ac)	nb	I (%)	Cw	Tc (min.)	Q_{100} (cfs)	change (%)
11.1 (1986)	927	0.047	8	0.65	38	2263	
13 (1986)	1239	0.045	12	0.65	49	2512	
18 (1986)	610	0.055	3	0.68	22	2175	
11.1 (1986 updated)	927	0.047	9	0.66	38	2279	0.69%
13 (1986 updated)	1239	0.045	12.8	0.65	49	2527	0.61%
18 (1986 updated)	610	0.055	4.5	0.68	22	2192	0.81%

Description of hydrologic variables: nb (basin factor); I (Impervious Cover); Cw (weighted runoff coefficient); Tc (time of concentration); Q_{100} (100-year or regulatory discharge).

The largest increase (0.81%) occurs immediately downstream of the area containing the higher-density developments. Since the drainage area increases in the downstream direction, the relative impact decreases. The updated analysis clearly shows that the impact associated with the three higher-density developments is not significant. The impact in the immediate vicinity of Ranch Estates #9 is approximately 0.65%.

This comparative analysis was based on precipitation depths from Reference 3, which was used in Pima County in 1984. Today, the upper-bound of the 90% confidence interval values from Reference 5 are used in all hydrologic analyses. Therefore, a separate hydrologic analysis was conducted to define new discharge values at the key concentration points, which were used to remap the regulatory or 100-year floodplain.

The new regulatory discharge values are shown in Table 2.2. The associated hydrologic data sheets are provided in Appendix C.

Table 2.2 Updated Hydrology (100-year)

Conc. Pt.	Area (ac)	nb	I (%)	Cw	Tc (min.)	Q_{100} (cfs)
11	908	0.047	9	0.70	35	2802
11.1	927	0.047	9	0.70	35	2861
11.3	1034	0.046	12	0.70	40	2916
11.3a	1045	0.046	12	0.70	42	2855
11.3b	1055	0.046	12	0.70	43	2804
11.4	1073	0.046	12	0.69	43	2844
11.4a	1189	0.046	12	0.69	44	3119
13	1239	0.045	12.8	0.69	44	3219
13.1	1608	0.045	12.9	0.69	50	3797
18	610	0.055	4.5	0.73	19	2823

Description of hydrologic variables: nb (basin factor); I (Impervious Cover); Cw (weighted runoff coefficient); Tc (time of concentration); Q_{100} (100-year or regulatory discharge).

SLA's 1989 study included runoff concentration points at both the northern and southern boundaries of the Ranch Estates #9 subdivision. These concentration points were CP 11.3 and 11.4, respectively. For the purpose of this study, three additional concentration points were defined. Two (CP 11.3a and 11.3b) are within the subdivision itself, and the third (CP 11.4a) is located immediately downstream of the confluence of the tributary wash that traverses the southern boundary of Lot 505. An exhibit showing the locations of the key concentration points listed in Table 2.2 is provided in Appendix A5. Based on the results of the updated hydrologic analysis, the regulatory discharge associated with CP 11.3 was selected for the floodplain analysis of the sub-reach that traverses the Ranch Estates #9, since the discharge at CP 11.3 exceeds the values associated with CP 11.3a, 11.3b, and 11.4. The downstream reach that traverses Ranch Estates #1 and #2 and Las Lomas de Catalina was analyzed using the discharges associated with CP 11.4, 13, and 13.1.

In addition, peak discharges for the more-frequent runoff events were determined at CP 11.3. These are summarized in Table 2.3. The associated hydrologic data sheets are also provided in Appendix C.

Table 2.3 Updated Hydrology (multi-frequency)

Conc. Pt.	Return Interval (yr)	Tc (min.)	Cw	Q (cfs)
11.3	2	104	0.31	268.00
	5	71	0.45	680.00
	10	59	0.53	1092.00
	25	49	0.61	1740.00

Description of hydrologic variables: Tc (time of concentration); Cw (weighted runoff coefficient); Q (peak discharge).

St. Thomas Church

As previously noted, the initial development of the St Thomas Apostle church occurred in 1987, with additional improvements in 1997. Copies of the two development plans are included in Appendix D. Since there was some concern that the overall development of the church had increased the magnitude of runoff impacting the subdivision, the pre- and post-developed conditions associated with the church site were evaluated. Based on the results of that evaluation, it was determined that the church has had no significant impact on the hydrology for the Valley View Wash. In addition, some of the improvements associated with the church have actually benefited potions of the subdivision.

When the church was initially developed in 1987, four detentions basins were constructed. Two of these basins regulate runoff that ultimately enters the Ranch Estates #9 subdivision. In 1984, approximately 1.6 acres of the church site contributed direct runoff to the subdivision. The associated 100-year peak discharge under Natural/Rural conditions was determined to be approximately 8.3 cfs. Under developed conditions (i.e., post-1997), the drainage area increased to approximately 3.0 acres with a current 100-year runoff potential of 23 cfs. However, approximately 2.2 acres of this area drains to the two detentions basins constructed in 1987. In order to limit the magnitude of runoff impacting the subdivision to 8.3 cfs, a conservative estimate of the required storage volume for these two basins is approximately 0.33 acre-feet. This estimate was made using the procedures outlined in Reference 6. Although it appears that only approximately one-half of the this volume was actually provided, the two basins should effectively reduce the 100-year peak discharge entering the subdivision to approximately 13.6 cfs, which is only 5.3 cfs above the pre-developed condition. Considering the magnitude of runoff associated with Valley View Wash during the 100-year event (2916 cfs), this increase in not significant.

It should be noted that the results just discussed are based on the rainfall depths associated with Reference 5. When the church was developed, they would have been permitted to use the values associated with Reference 3 to determine the required storage volume for each detention basin. Therefore, the results are not intended to suggest that they did not meet the requirements in effect at the time of development.

A comparison of the 2007 topography versus the 1984 topography was also conducted to determined if either site grading or other site improvements has had an adverse impact on the subdivision. The results indicate that site grading has actually benefited the subdivision by capturing and diverting a small quantity of overbank flow from the Valley View Wash, which slightly reduces the flood hazard associated with Lot 501. It was also noted that a grade-control structure placed just upstream of the northwest corner of Lot 500 and one placed just north of the southwest corner of Lot 498 have effectively stabilized the bed profile along the western boundary of Lots 498 and 499, in addition to protecting the pavement at Calle del Pantera.

III. FLOODPLAIN ANALYSIS

Regulatory Flood Plain

Per Pima County's floodplain ordinance, the regulatory floodplain is defined as "that portion of the geologic floodplain associated with a watercourse….where the 100-year peak discharge is 100 cfs or greater, or those areas that are subject to sheet flooding." It is also important to note that the term 100-year flood or flood plain is a probability reference as opposed to a time period reference. The referenced year is divided into one (i.e., 1/100, or 1 divided by 100) to define the probability of occurrence in any given year. For example, the 100-year flood has a 1% chance of occurring in any given year. The 50-year flood has a 2% chance, the 5-year flood has a 20% chance, and so on.

The regulatory or 100-year flood plain within the study area was remapped using ground data from the 2007 topographic survey prepared by Cardinal Land Surveying, Inc. (CLS). A copy of the complete ground survey is provided in Appendix E. The floodplain analysis was performed using the HEC-RAS water-surface profile model (Reference 7). An exhibit showing the 2007 topography, including the 2007 aerial photograph for the study reach, is provided as Figure 3. The locations of the cross sections used in the HEC_RAS analysis are shown on Figure 4. The regulatory (100-year) floodplain boundary and floodprone area is also shown on Figure 4.

Sheet 1 of Figure 4 depicts three flood zones; whereas, Sheets 2 and 3 depict only one. This is due to the more-detailed floodplain analysis that was required to accurately map flooding conditions within Ranch Estates #9. The blue shaded area between the eastern and western-most 100-year or regulatory floodplain boundaries (on all sheets) is the approximate wetted portion or special flood hazard area as defined by the base flood elevations (e.g., WS= 2620.13) listed for each cross section. The base flood elevations relative to each cross section within the Ranch Estates #9 sub-reach are summarized in Tables 3.1a and 3.1b. The base flood elevations relative to each cross section within Ranch Estates #1 and #2 and Las Lomas de Catalina are summarized in Table 3.2. It should be noted that high ground or "islands" that may exist between the floodplain boundaries (i.e., locations where the ground is higher than the base flood elevations) were not identified or excluded from the floodprone area.

Table 3.1a Regulatory Discharges and Water-Surface Elevations for the Main Corridor of the Valley View Wash within Ranch Estates #9 (as shown on Sheet 1 of Figure 4)

Reach	River Station (or Cross Section)	Q Total (cfs)	W.S. Elevation (ft)
Main Corridor	1	3119	2608.88
Main Corridor	2	3119	2612.30
Main Corridor	3	1978	2613.85
Main Corridor	4	1775	2616.12
Main Corridor	5	1775	2620.59
Main Corridor	5.5	Lateral Weir	
Main Corridor	6	1816	2624.07
Main Corridor	6.5	Lateral Weir	
Main Corridor	7	2106	2627.85
Main Corridor	7.5	Lateral Weir	
Main Corridor	8	2366	2631.21
Main Corridor	8.5	Lateral Weir	
Main Corridor	9	2458	2634.48
Main Corridor	9.5	Lateral Weir	
Main Corridor	10	2551	2637.65
Main Corridor	10.5	Lateral Weir	
Main Corridor	11	2885	2641.20
Main Corridor	11.5	Lateral Weir	
Main Corridor	12	2916	2644.31
Main Corridor	13	2916	2646.94
Main Corridor	14	2916	2650.01
Main Corridor	15	2916	2652.79
Main Corridor	16	2916	2654.67
Main Corridor	17	2916	2656.90
Main Corridor	18	2916	2658.44
Main Corridor	19	2916	2660.90
Main Corridor	20	2916	2662.81

Table 3.1b Regulatory Discharges and Water-Surface Elevations for the West Channel of the Valley View Wash within Ranch Estates #9 (as shown on Sheet 1 of Figure 4)

Reach	River Station (or Cross Section)	Q Total (cfs)	W.S. Elevation (ft)
West Overflow	1	3119	2608.88
West Overflow	2	3119	2612.30
West Overflow	3	1140	2615.27
West Overflow	4	1140	2618.00
West Overflow	5	1140	2620.13
West Overflow	6	1099	2622.94
West Overflow	7	807	2625.59
West Overflow	8	551	2629.85
West Overflow	9	460	2633.54
West Overflow	10	367	2637.10
West Overflow	11	31	2640.67

Table 3.2 Requlatory Discharges and Water-Surface Elevations for the Valley View Wash Downstream of Ranch Estates #9 (as shown on Sheet 2 and 3 of Figure 4)

Reach	River Station (or Cross Section)	Q Total (cfs)	W.S. Elevation (ft)
Main Channel	0.01	3797	2530.72
Main Channel	0.02	3797	2533.42
Main Channel	0.03	3797	2536.97
Main Channel	0.04	3797	2541.83
Main Channel	0.05	3797	2543.86
Main Channel	0.06	3797	2545.92
Main Channel	0.07	3797	2549.46
Main Channel	0.08	3797	2551.01
Main Channel	0.09	3797	2553.56
Main Channel	0.10	3797	2557.19
Main Channel	0.11	3219	2561.73
Main Channel	0.12	3219	2565.60
Main Channel	0.13	3219	2569.43
Main Channel	0.14	3219	2574.09
Main Channel	0.15	3219	2578.48
Main Channel	0.16	3219	2582.23
Main Channel	0.17	3219	2585.84
Main Channel	0.18	3219	2589.53
Main Channel	0.19	3219	2592.94
Main Channel	0.20	3219	2596.39
Main Channel	0.21	3219	2599.43
Main Channel	0.22	3119	2601.46
Main Channel	0.23	3119	2605.49
Main Channel	0.24	3119	2608.88

Two water-surface profiles are represented by Tables 3.1a and 3.1b. Table 3.1a applies to the "main corridor" of the Valley View Wash within Ranch Estates #9, and Table 3.1b applies to the "west overflow" channel, which conveys flows that break out of the main corridor between Sections 5 and 12. The location of the drainage divide or "lateral weir" crest is shown on Sheet 1 of Figure 4 as a black-dashed line that extends south from the church's access drive to the house constructed on Lot 503. Breakout flows from the "main corridor" were used to map the flood plain associated with the "west overflow" channel. A section by section breakdown of the discharge associated with each overflow or "lateral weir" section is presented in Table 3.3. Since the average depth of flow over the weir between Section 6 and 10 was approximately one foot, a second flood zone characterized by shallow flooding with average depths equal to one foot was delineated between the weir crest and the eastern boundary of the "west overflow" flood plain.

The third delineated flood zone is characterized by shallow flooding with average depths less than one foot. This area is centered along Calle del Pantera in the vicinity of the Cerco del Corazon intersection.

Table 3.3 Lateral Weir Quantities and Hydraulic Properties

Reach	River Station	Q Leaving (cfs)	Lateral Weir			W.S. Elevation	
			Max Depth (ft)	Avg Depth (ft)	Min El (ft)	u/s (ft)	d/s (ft)
Main Corridor	5.5	42	0.87	0.45	2622.00	2624.07	2620.59
Main Corridor	6.5	292	1.38	1.14	2623.20	2627.85	2624.07
Main Corridor	7.5	256	1.24	1.11	2627.00	2631.21	2627.85
Main Corridor	8.5	91	1.16	0.67	2631.00	2634.48	2631.21
Main Corridor	9.5	93	1.65	1.10	2635.00	2637.65	2634.48
Main Corridor	10.5	337	1.42	1.17	2637.00	2641.20	2637.65
Main Corridor	11.5	31	0.69	0.42	2640.50	2643.63	2641.20

It should be noted that the water-surface elevations presented in Tables 3.1 and 3.2 are based on a critical flow regime, as opposed to a subcritical or supercritical flow regime. When the subcritical floodplain analysis defaulted to critical depth, a supercritical profile was evaluated. The results confirmed that critical flow dominates. This is significant in that the computed water-surface elevations are not subject to either upstream or downstream controls. For example, removing the culvert beneath the access drive to Lot 503 (Section 7) will not significantly change the water-surface elevation at either Section 6 or Section 8.

In addition to analyzing the 100-year or regulatory flood plain within Ranch Estates #9, a separate model was prepared to evaluate the capacity of the primary low-flow channel, which is currently the western-most watercourse, as opposed to the Calle del Pantera street section. The results indicate that the capacity falls between the 2-year and 5-year peak discharge, which is consistent with the results of the 1989 study. The bifurcation in the main channel located immediately upstream of Calle del Pantera was also evaluated to determine the approximate distribution of flow between the two flow paths. The results indicate that approximately half the flow will be conveyed along both watercourses, especially during high-flow events. During low-flow events, the current tendency is for most of the flow to be conveyed along the western watercourse. However, given the dynamic nature of this alluvial channel, this tendency can change, and there is no way to predict the actual flow path from one flow event to another (i.e., the relative distribution of flow can fluctuate from one event to another).

Flood Prone Structures

In addition to providing updated topographic information that could be used to remap the Valley View Wash flood plain, Cardinal Land Surveying obtained the finish-floor elevations (FFEs) of all homes within Ranch Estates #9 which were thought to be within the 100-year or regulatory flood plain per the 1989 study. Table 3.4 summarizes the results of their survey, in addition to providing a comparison between the regulatory water-surface elevation and the existing ground elevation (grade) on the upstream side of the structure.

The negative sign associated with the values in the "FFE versus $WS_{structure}$" column denotes that the regulatory water surface is above the finish-floor elevation. The values in the

"$WS_{structure}$ versus Grade" column denotes the approximate depth of flow adjacent the to upstream side of the structure during a regulatory event.

Per Pima County's floodplain ordinance, new homes constructed in or near a flood prone area must have their lowest finish floor elevated a minimum of one foot above the regulatory water-surface elevation on the upstream side of the structure. Only two structures in or near the remapped regulatory flood plain within Ranch Estates #9 meet this requirement – the main structure on Lot 493 and the garage on Lot 504. Although the finish-floor elevation for the structure on Lot 499 is above the computed water-surface elevation, the difference is only 0.45 feet.

Table 3.4 Comparison of FFE to W.S. Elevation and Existing Grade within Ranch Estates #9

Lot No.	Description	FFE (ft)	W.S. Elev at Structure (ft)	FFE versus $WS_{structure}$ (ft)	Existing Grade at Structure (ft)	$WS_{structure}$ versus Grade (ft)
498	main structure	2651.46	2652.79	-1.33	2651.30	1.49
493	main structure	2648.78	2644.85	3.93	2649.00	-4.15
499	main structure	2643.30	2642.85	0.45	2642.50	0.35
496	main structure	2635.54	2635.66	-0.12	2634.50	1.16
	sunken living	2635.16	2635.66	-0.50	n/a	n/a
495	main structure	2632.26	2632.31	-0.05	2631.80	0.51
497	main structure	2630.73	2631.21	-0.48	2630.30	0.91
	bedroom addition	2629.55	2629.83	-0.28	2629.00	0.83
504	garage	2622.98	2620.17	2.81	2623.00	-2.83
503	main structure	2621.78	2622.45	-0.67	2621.50	0.95
	carport	2617.90	2618.43	-0.53	2618.60	-0.17
502	main structure	2630.75	2631.78	-1.03	2630.30	1.48
501	main structure	2636.19	2636.59	-0.40	2635.00	1.59
500	main structure	2639.71	2640.95	-1.24	2639.40	1.55

Since all of the habitable structures within Ranch Estates #1 and #2 and Las Lomas de Catalina, which were thought to be flood prone in 1986, turned out to have FFEs from the 1989 survey that were above the previously defined base flood elevations, new FFEs were not obtained by Cardinal Land Surveying for these structures. Instead, FFEs for the previously-identified structures were estimated using the 1989 FFE plus a datum conversion constant. FFEs for any newly identified structures were estimated using the 2007 topographic survey as a guide. Table 3.5 summarizes the results of a comparison between the adjusted and/or estimated FFEs and the new regulatory (100-year) water-surface elevations.

Table 3.5 Comparison of FFE to W.S. Elevation and Existing Grade within Ranch Estates #1 and #2 and Las Lomas de Catalina

Lot No.[1]	Description	Finish Floor Elevation (FFE)[2]		W.S. Elev at Structure (ft)	FFE versus $WS_{structure}$ (ft)
		OLD (ft)	NEW (ft)		
55	secondary structure	2598.70	2600.95	2602.04	-1.09
12	main structure	--	2591.50	2591.52	-0.02
13	main structure	--	2589.90	2588.28	1.62
14	guest house	--	2585.90	2584.96	0.94
16	east half of lot (split)	--	2578.25	2576.96	1.29
43	main structure	2557.70	2559.95[3]	2558.90	1.05
44	main structure	2563.60	2565.85	2566.10	-0.25
45	main structure	2553.80	2556.05	2554.31	1.74
46	main structure	2549.60	2551.85	2547.78	4.07
47	main structure	--	2546.50	2544.00	2.50
48	main structure	2538.80	2541.05	2539.70	1.35
49	main structure	--	2536.00	2534.36	1.64

Note: [1]Lot 55 is in Flecha Caida Ranch Estates No. 2, Bk 12 Pg 69. Lots 12 and 13 are in Las Lomas de Catalina, Bk 29, Pg 79. The remaining lots are in Flecha Caida Ranch Estates, Bk 11, Pg 74.

[2]If applicable, the "Old" FFE (NGVD 29) is from the 1989 SLA report. The "New" FFE (NAVD 88) was computed by adding 2.25 feet to the "Old" FFE. For all lots lacking an "Old" FFE, the "New" FFE was estimated using the topographic survey as a guide.

[3]The adjusted FFE for Lot 43 is significantly higher than the topographic mapping suggests

As previously noted, structures located in or near the 100-year flood plain are generally considered protected from flooding when their lowest finish floor is elevated a minimum of one foot above the 100-year water-surface elevation on the upstream side of the structure. Only one structure in Ranch Estates #1 (FC1), one in Ranch Estates #2 (FC2), and two in Las Lomas de Catalina (LLdC) do not meet this requirement – the secondary structure on Lot 55 of FC1, the main structure on Lot 55 of FC2, and the main structure on Lot 12 and guest structure on Lot 14 of LLdC. However, the guest structure on Lot 14 of LLdC is close with a difference of 0.94 feet. To verify the potential floodprone status or risk of flooding for these structures, it recommended that the owners have their FFEs surveyed for comparison with the water-surface elevations provided in Table 3.4. In addition, since the FFE for Lot 43 is questionable (see footnote 3 in Table 3.5), it is recommended that the owner contact a land surveyor and arrange to have the FFE determined, such that it can be compared to the elevations provided in Table 3.4. Also, if the owners of any of the remaining lots listed in Table 3.5 are concerned about the potential for flooding, it is recommended that they have their FFEs surveyed to more accurately identify their flood risk.

Erosion and Sedimentation

In addition to flooding problems, the subdivisions, to a varying degree, are also prone to erosion and sedimentation problems. Long-term degradation, which is the gradual lowing of the channel bed in response to a reduction in the upstream sediment supply, was the primary focus of the 1989 study, since it stood out as the most significant erosion problem. That study identified five problems areas within the Ranch Estates #9 subdivision and discussed the extent of the problem associated with each. The location of these problem areas are shown on a excerpt of the 1989 exhibit, which is provided in Appendix A1 (page 18). The full exhibit is provided in Appendix A4 (page 63). Using the same site numbers referenced in the 1989 study, the five sites are described as follows: Site 3 is located along the west branch immediately downstream of Calle del Pantera; Site 4 is located immediately downstream of the access drive for Lot 501; Site 5 is located just downstream of Cerco de Corazon, near the northwest corner of Lot 497; Site 6 is located immediately downstream of the access drive for Lot 503; and, Site 7 is located within the boundary of Lot 504 immediately downstream of Cerco del Corazon.

During the field investigation associated with this 2007 study, it was noted that erosion and/or sedimentation problems still exist at all five locations, with the possible exception of Site 3. As previously noted, two grade-control structures constructed adjacent to Lots 498 and 499 have temporarily checked long-term degradation at Site 3. However, some local erosion near the access drive to Lot 498 was noted. In addition, the installation of a grade-control structure just downstream of the access drive to Lot 502, and the paved access drive itself, which was not considered during the 1989 study, has temporarily mitigated the long-term degradation problem at Site 4. To show the short-term effect both of these structures have had on the streambed profile, Figure 5 was prepared. It compares the 1998 streambed profile along the western watercourse to the 2007 profile. The approximate locations of the three referenced grade-control structures are Stations 5+15, 10+35, and 12+23.

It should be noted that the magnitude of degradation in the vicinity of the access drive for Lot 501 is misleading. The difference between the two profiles at this location was due to bank erosion as opposed to channel bed degradation. During the 2007 summer monsoon season, the area experienced a 2-year to 5-year flow event. During this event, the west bank in the vicinity of the access drive eroded between 20 and 30 feet in a southwesterly direction. Bank erosion extended approximately 50 to 60 feet in upstream direction and ceased just downstream of the access drive. As a result, most of the rock riprap bank protection along the upstream reach was lost, and the majority of the sediment removed from the bank was deposited in the area between the wash and the north elevation of the home on Lot 502. Bank erosion was also noted along the east bank in the vicinity of the home on Lot 500. Currently, this structure is located less than ten feet from the top of the eastern bank. However, it does not appear that bank protection in the form of rock riprap has ever been provided along this bank.

With the exception of the additional erosion and sedimentation problems noted in the vicinity of Site 4 and the stabilizing effect of the church's grade-control structure relative to Site 3, long-term degradation is still the most significant erosion problem at the remaining sites. The existing slope of the bed along the western channel ranges between 2.15% and 2.45%. The slope of the bed along the eastern channel segment located within the boundary of Lot 497 is

approximately 1.4%. The 1989 study estimated an equilibrium bed slope of approximately 1%. However, assuming a conservative sediment reduction factor of 40% relative to the upstream watershed, the equilibrium bed slope was estimated to be approximately 1.5% using the relationship provided in Chapter 6 of Reference 8.

The computed equilibrium slope of 1.5% was plotted on Figure 5 at several locations based on appropriate downstream pivot points. Typically, the pivot points around which a channel bed will adjust its grade are channel confluences, grade-control structures, and at-grade roadway crossings. For Sites 6 and 7, the downstream pivot point was the confluence with the eastern tributary channel that traverses the southern boundary of Lot 505. The estimated long-term degradation depth at Sites 6 and 7 were determined to be approximately 5.2 feet and 4.8 feet respectively. This is in addition to the drop height that currently exists, which is 1.0 feet and 2.0 feet, respectively. For the grade-control structure located just downstream of the access drive to Lot 502, the additional degradation depth was determined to be approximately 1.5 feet, with an existing drop of approximately 1.8 feet. Assuming this existing grade-control remains effective (i.e., does not fail), the approximate long-term degradation depth downstream of the access drive to Lot 502 is less than one foot, which is also applicable to the access drive for Lot 501, since the channel bed downstream of the access drive for Lot 501 pivots around the access drive for Lot 502. With respect to the existing grade-control structures at Station 10+35 and 12+23 and the downstream edge of pavement for Calle del Pantera, the long-term degradation depths were determined to be approximately 1.3 feet, 1.0 feet, and 2.5 feet, respectively. Again, these depths are in addition to the drop height that currently exists, which is approximately 2.0 feet, 1.0 feet, and 0.5 feet, respectively. Since the existing slope for the eastern channel is slightly less than the computed equilibrium slope, it is reasonable to assume that this channel is at or near its equilibrium slope; therefore, the existing drop height at Site 5, which is approximately four feet, is not expected to increase significantly.

An exhibit similar to Figure 5 was also prepared for the main branch of the Valley View Wash with Ranch Estates 31 and #2 and Las Lomas de Catalina. This exhibit is included in Appendix A5. Since the 2007 profile is very similar to the 1998 profile, long-term degradation does not appear to be a problem along this portion of the study reach. However, given the dynamic nature of alluvial channels, especially when man's activities are factored into the equation, the critical balance between the upstream sediment supply and the downstream sediment transport capacity could be upset at any time. The existing slope along this reach is approximately 2.3%, which is consistent with the existing slope of the west branch channel within Ranch Estates #9. Therefore, the estimated equilibrium bed slope for that reach (1.5%) is also applicable to this reach. Consequently, the long-term degradation depth for this reach is approximately one foot per 125 feet of channel relative to the downstream pivot.

In addition to estimating the long-term bed profile for the western channel within Ranch Estates #9, a single-event scour analysis was conducted in accordance with the procedure outlined in Chapter 6 of Reference 8. The associated computation sheets are provided in Appendix F. Based on the results of the scour analysis, the minimum design scour depth is three feet. A similar analysis was conducted for the main channel of the Valley View Wash within Ranch Estates #1 and #2 and Las Lomas de Catalina. Based on the results of that analysis the

minim design scour depth is 3.5 feet within Ranch Estates #1 and #2 and 3.3 feet within Las Lomas de Catalina. The associated computation sheets are also provided in Appendix F.

Typically, when bank protection is proposed, the minimum scour depth is combined with the long-term depth to determine the design toe-down depth. When protection is proposed in the vicinity of a channel drop and bed protection is not provided, then the design toe-down depth must be increased to account for the depth of local scour on the downstream side of the drop, which is a function of the ultimate drop height. Under existing conditions, the maximum drop height is expected to occur at Site 6 within Ranch Estates #1. Based on an ultimate drop height of 6.2 feet, the maximum local scour depth was determined to be approximately 9.9 feet. Assuming a minimum drop height of three feet (e.g., Site 3), the local scour depth would be approximately 7.3 feet. These depths emphasize the importance of bed protection when excessive drop heights are anticipated or the importance of adequately spaced grade-control structures to stabilize the bed profile.

An erosion-hazard setback analysis was also performed in conjunction with the single-event scour analysis using the procedure outlined in Chapter 7 of Reference 8. Based on the results of that analysis, the average building setback distance was determined to be approximately 73 feet. This value is consistent with the minimum distance (75 feet) specified in Pima County's current floodplain and erosion hazard management ordinance for unprotected banks along natural channels. Currently, the homes on Lots 498, 500, and 501 are within the erosion-hazard area for the western channel. The existing setback distances for these homes are approximately 60 feet, 10 feet, and 20 feet, respectively. Since the home on Lot 498 is located on the inside of the channel bend and approach flows are more dispersed, the existing setback distance (60 feet) may be adequate. However, the potential for bank erosion and/or lateral migration of the channel bank should be a major concern for the owners of Lots 500 and 501. The 75-foot setback specified in Pima County's current floodplain and erosion hazard management ordinance is also applicable to the structures within Ranch Estates #1 and #2 and Las Lomas de Catalina. Currently, the homes on Lots 16B and 43 through 51 (Ranch Estates #1), Lot 55 (Ranch Estates #2), and Lots 11 through 16 (Las Lomas de Catalina) are within the erosion-hazard area for either the main channel or one of the secondary channels. However, the actual potential for bank erosion and/or lateral migration was not evaluated as part of this study. Within the majority of these lots, mitigating factors such as soils, channel-specific discharges, armoring, etc., may result in a reduced potential for bank erosion and/or lateral migrations.

IV. FLOOD HAZARD MITIGATION

For the most part, the results of this 2007 study are consistent with the results of the 1989 study (see the floodplain comparison exhibit in Appendix A5). The floodplain boundaries are similar, as are the identified erosion hazards, with the possible exception of Site 4 within Ranch Estates #9. The 1989 study addressed the feasibility of regional solutions, including upstream detention and channelization of flows within the subdivision itself and determined that neither was cost-effective. To reduce the economic impact of flooding, flood insurance was recommended. To address long-term degradation, both short-term and long-term solutions in the form of gabion cut-off walls and grade-control structures were recommended. Although cut-off

walls and grade-control structures have been provided within Ranch Estates #9, they were not designed in accordance with the recommendations of the study (i.e., concrete cutoff walls, concrete grade-control structures, and grouted rock aprons were installed instead). At the time, bank erosion in the vicinity of Lots 500 and 501 was not discussed, since rock riprap protected berms had already been provided upstream of these lots. However, the loss of this protection must now be addressed and floodproofing options should also be considered, in addition to the purchase of flood insurance. Although long-term degradation continues to be a problem within Ranch Estates #9, particularly at Site 6, the recommendations of the 1989 study are still valid. Since the Valley View Wash along the study reach is a natural watercourse that is subject to periodic flooding and the natural dynamic forces of sedimentation and erosion, mitigation measures will be needed to protect flood/erosion prone structures. However, mitigation measures must be addressed by individual property owners, since the wash is privately owned.

Flood Prone Structures

As previously recommended, flood insurance should be purchased to reduce the economic impact of any flooding that may occur in the future. At a minimum, flood insurance should be purchased by the owners of all lots identified in Table 3.4 and 3.5, with the possible exception of Lots 493 and 504 (Ranch Estates #1), Lots 45-47 and Lot 49 (Ranch Estates #1), and Lot 13 (Las Lomas de Catalina). However, any lot owner concerned about the potential for flood damage should consider purchasing flood insurance. Since the subdivisions are not shown to be in a special flood hazard area on the effective FIRMs, Zone X insurance (the least expensive) can be purchased. Zone X premiums are significantly lower than Zone AE, Zone A01, or Shaded Zone X premiums. In addition, as along as the policy remains effective, owners who purchase insurance now will continue to qualify for this rate zone, even if the maps are revised to include the subdivision.

In addition, the owners should consider flood proofing their homes to minimize damage to the contents and to the structure itself. Floodproofing measures typically fall into to categories – wet and dry floodproofing. However, since wet floodproofing, which allows water to enter the structure, is typically limited to uninhabited parts of a residence, these measures would only apply to a garage or detached storage shed. Dry floodproofing involves sealing the home to prevent water from entering the structure. Typical dry floodproofing measures that are applicable to the flooding conditions that exist within the subdivision include:

- Adding a waterproof veneer (approx. two to three feet) along exterior walls
- Raising electrical system components approximately 1.5 feet
- Raising or waterproofing HVAC equipment approximately 1.5 feet
- Installing sewer backflow values

See Appendix G for more detailed information.

Although elevating, relocating, or demolishing (then rebuilding) the structure are other retrofitting measures, they would not be as cost-effective as simple floodproofing. Berms or low-profile levees and floodwalls are also effective retrofitting measures since they can

effectively divert flows around and/or away from structures; however, engineering studies that document the impact of these structures would have to be prepared by each property owner and submitted to the Regional Flood Control District for approval before floodplain-use permits could be issued. These types of structures typically increase the water-surface elevation and/or divert flows onto adjacent properties. Consequently, the Flood Control District needs to be assured that implementation of these types of mitigation measures will not adversely impact adjacent properties. Although some floodproofing measures would still require a floodplain-use permit, they are more likely to be approved without the requirement for a detailed floodplain study, since the impact of encroachment into the flood plain would be minimal.

Erosion and Sedimentation

The long-term degradation problem associated with Site 4 within Ranch Estates #9 appears to have been addressed, at least for now, by the combined effect of the grade-control structure installed downstream of the access drive to Lot 502 and the access drive itself. However, the loss of the rock riprap bank protection along the west bank should be addressed by the installation of new bank protection. If loss rock riprap protection is provided on a 3:1 (horizontal to vertical) side slope, the D50 diameter of the rock should be one foot, and the thickness of the rock layer should be a minimum of two feet. In addition, filter fabric should be installed under the rock and the toe of the protection should extend below channel bed a minimum of three feet plus the long-term degradation depth (Figure 5).

The loss rock riprap bank protection along the east bank adjacent to Lot 500 should also address by the installation of new protection. However, since the existing home is located within 10 feet of the bank, vertical gabion baskets (i.e., wire-tied baskets filled with rock), which can be placed vertically along the bank, would minimize disturbance of the soil in the vicinity of the homes foundation. Loose rock on a 3:1 slope would require 12-15 feet to install.

The sedimentation problem associated with Lot 502 appears to be directly related to the recent bank erosion associated with Lot 501. If a portion of the west bank adjacent to Lot 501 is reconstructed and adequately protected, the sediment transport capacity along the reach will be increased and low-flow will be redirected to its pre-Summer 2007 flow path. If desired, the bank between Sections 8 and 10 could be elevated to contain all or a portion of the 100-year discharge that currently breaks out of the western channel in the vicinity of Lots 501 and 502. A separate HEC-RAS model demonstrated that eliminating breakout in this area would not have an adverse impact on either the adjacent or downstream property owners. However, eliminating breakout upstream of Lot 501 could have an adverse impact of Lots 500 and 501. In addition, eliminating breakout downstream of Lot 502 could increase the flood hazard associated with Lot 503. Although elevating the bank in the vicinity of Lots 501 and 502 would eliminate breakout in this area and reduce low-flow hazards, it will not eliminate the high-flow hazards and vehicular access over the berm would have to be addressed.

Access during times of flooding is also a problem for the owners of Lots 501 and 502. However, installation of culvert crossing similar to the one installed for Lot 503 would increase the long-term degradation potential for Lot 500 and could undermine the existing grade-control structure installed on the church property. Consequently, crossings should not be provided,

unless detailed engineering plans are prepared for review and approval. The plans should include the installation of bank and bed protection and additional grade-control structures upstream of the crossings. It appears that the grade-control structure installed upstream of the access drive for Lot 503 was constructed to minimize the impact on the access drive for Lot 502, since some lowering the streambed would have been required to facilitate the crossing. However, it does not appear that plans were prepared for this crossing; therefore, the long-term stability of the structure is questionable.

The long-term degradation problem at Sites 6 and 7 can be addressed in the manner identified in the 1989 study (i.e., the installation of gabion grade-controls structures. Grade-control structures could also be constructed along the downstream reach to minimize the long-term degradation potential at these sites.

Since the slope of the eastern branch of the Valley View Wash within the boundary of Lot 497 appears to have reached equilibrium, long-term degradation should no longer be a problem for this reach, including Site 5. Therefore, the existing grouted rock bank protection along the east bank should continue to protect the home on Lot 497, which is within 30 feet of the bank. However, the addition of a loose rock riprap apron downstream of the existing grouted rock apron would provide some protection from local scour.

Since erosion and sedimentation problem areas were not previously identified in Ranch Estates #1 and #2 and Las Lomas de Catalina, no site specific evaluations or recommendations were prepared as part of this study.

V. REFERENCES

1. Pima County Department of Transportation and Flood Control District, *Preliminary Assessment of Flooding Problems along Valley View Was in the Vicinity of the Flecha Caida Subdivision*, July 26, 1984.

2. Pima County Department of Transportation and Flood Control District, *Flecha Caida Flood Improvement Study – Phase I, 100-year Peak Discharge Magnitudes and Floodplain Mapping*, January 28, 1986.

3. United States Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, *Precipitation – Frequency Atlas of the Western United States, Volume VIII, Arizona*, NOAA Atlas 2, 1973.

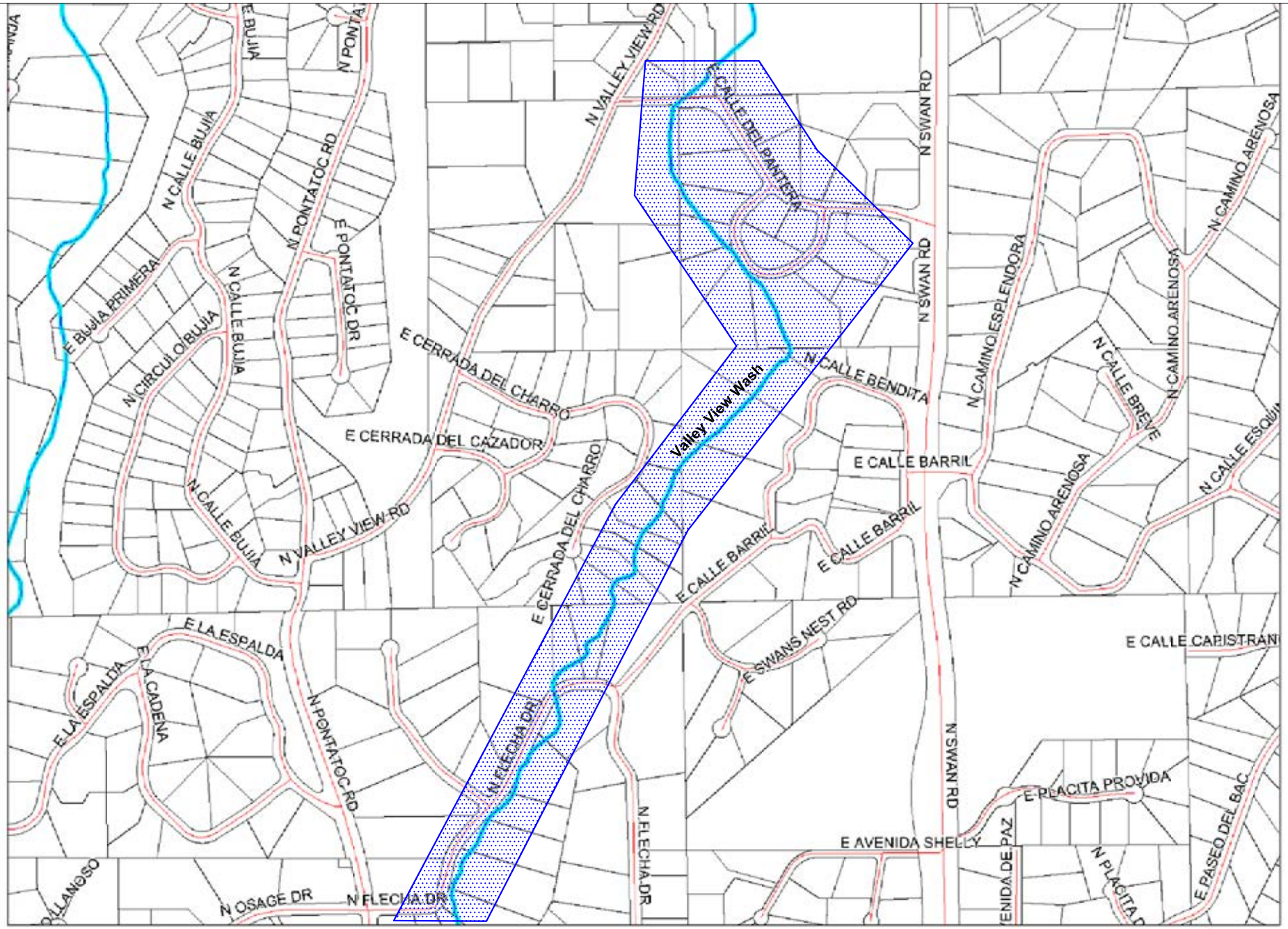
4. Pima County Department of Transportation and Flood Control District, *Valley View Wash, Flood and Erosion Control Study for the Calle Del Pantera Area (Phase II, Flecha Caida Flood Improvement Study)*, February 15, 1989.

5. United States Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service, *Precipitation – Frequency Atlas of the United States*, NOAA Atlas 14, Volume I, Version 4, 2004, revised 2006.

6. Pima County Department of Transportation and Flood Control District/City of Tucson, Department of Transportation, *Stormwater Detention/Retention Manual*, July, 1987.

7. U.S. Army Corps of Engineers, Hydraulic Engineering Center, *HEC-RAS River Analysis System, Version 4 Beta*, November 2006.

8. City of Tucson, Department of Transportation, Engineering Division, *Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona*, prepared by Simons, Li and Associates, Inc., December 1989, last revision, July 1998.



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500 0 500 1,000 1,500

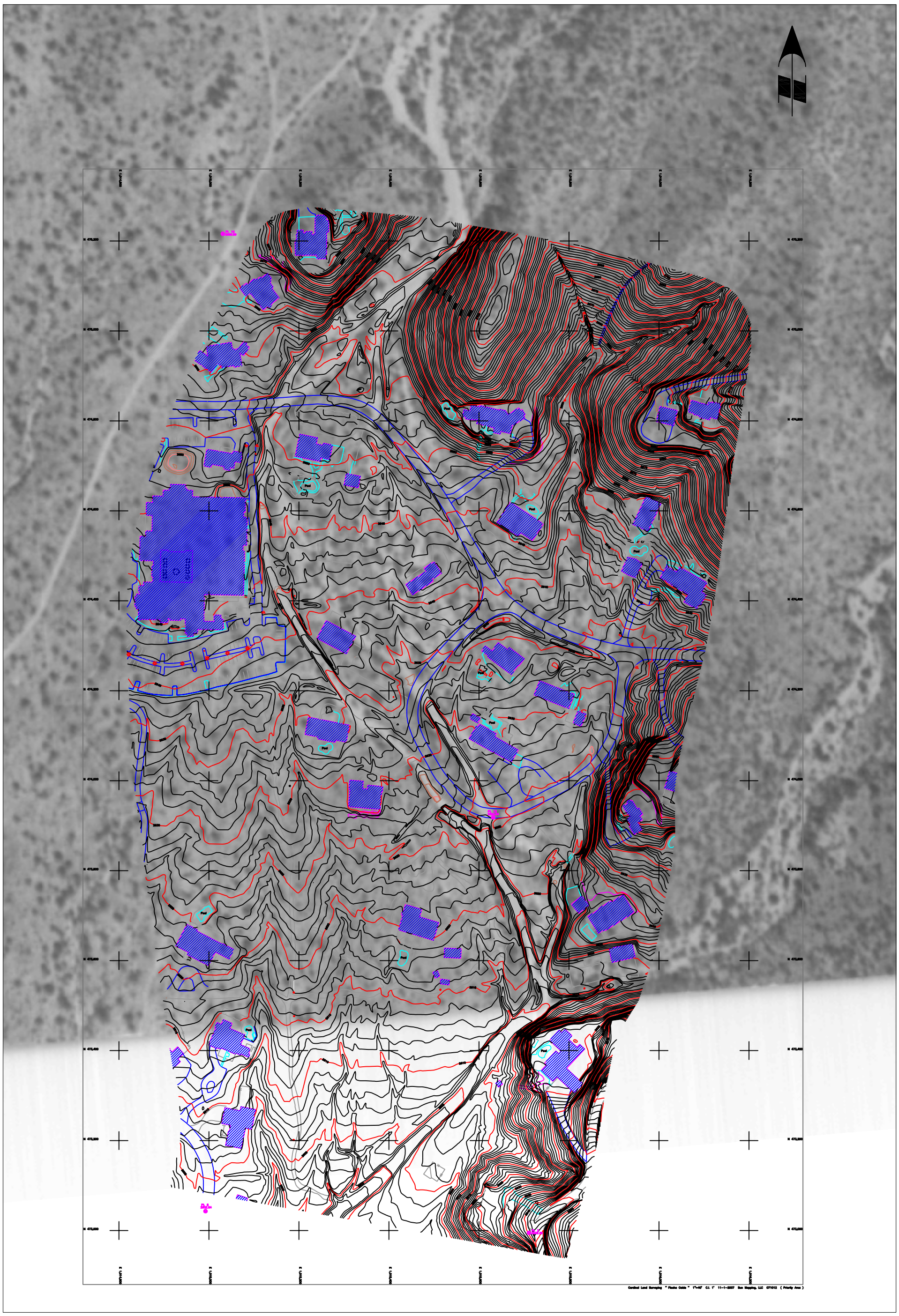
FEET

Study Reach

N

JE Fuller/Hydrology and Geomorphology, Inc.

Figure 1 - Location Map



200' 0' 200' 400'

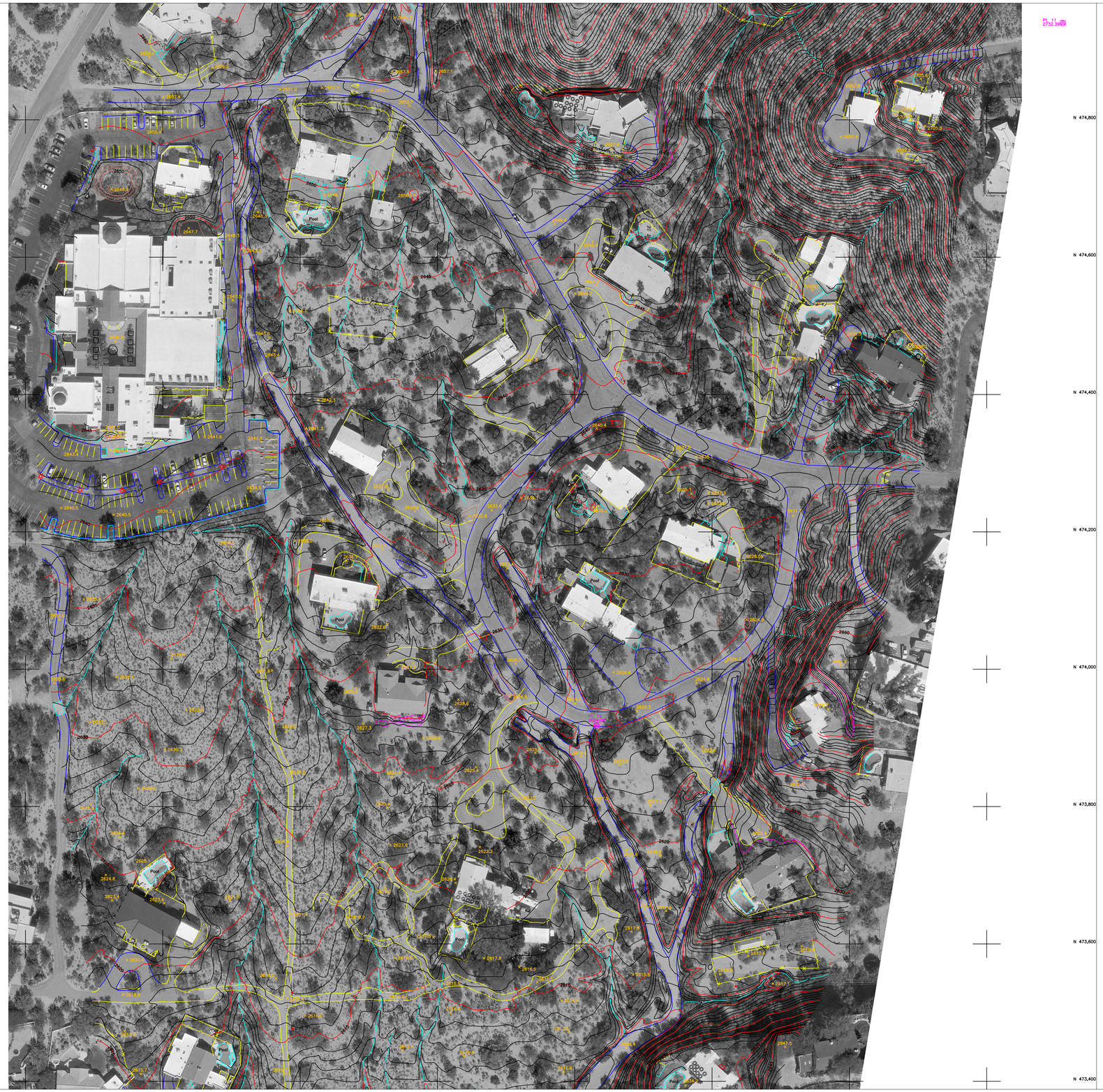
SCALE: 1 INCH = 200 FEET

Contour Interval = 1 FEET

Figure 2

2007 Topography (NAVD 88) and Existing Improvments superimposed on 1941 Aerial Photograph

JE Fuller / Hydrology & Geomorphology, Inc.



SCALE: 1 INCH = 80 FEET
Contour Interval = 1 FEET

Figure 3

2007 Topographic Mapping (NAVD 88)

2007 Aerial Photograph

Sheet 1 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

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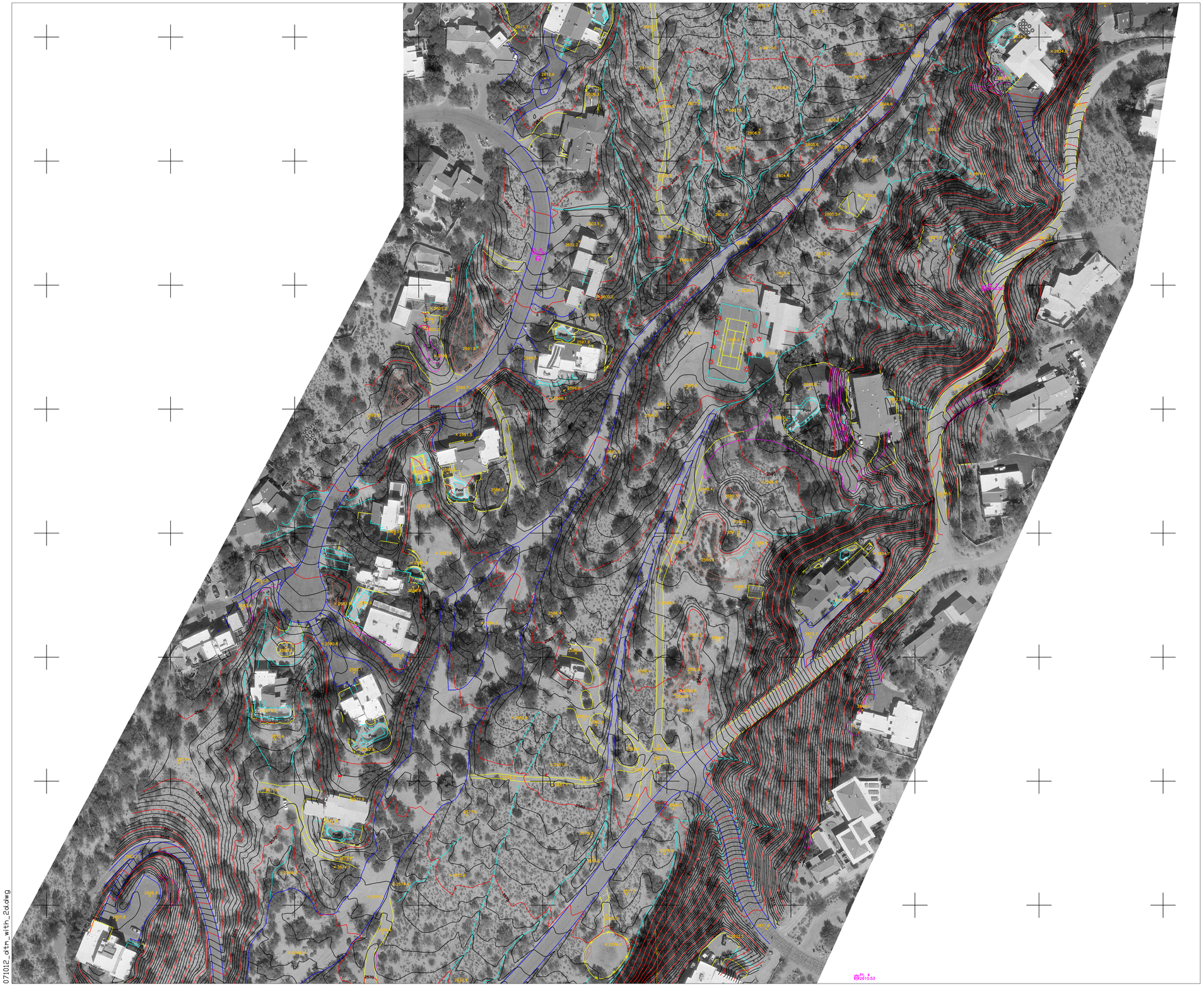


80' 0' 80' 160'

SCALE: 1 INCH = 80 FEET
Contour Interval = 1 FEET

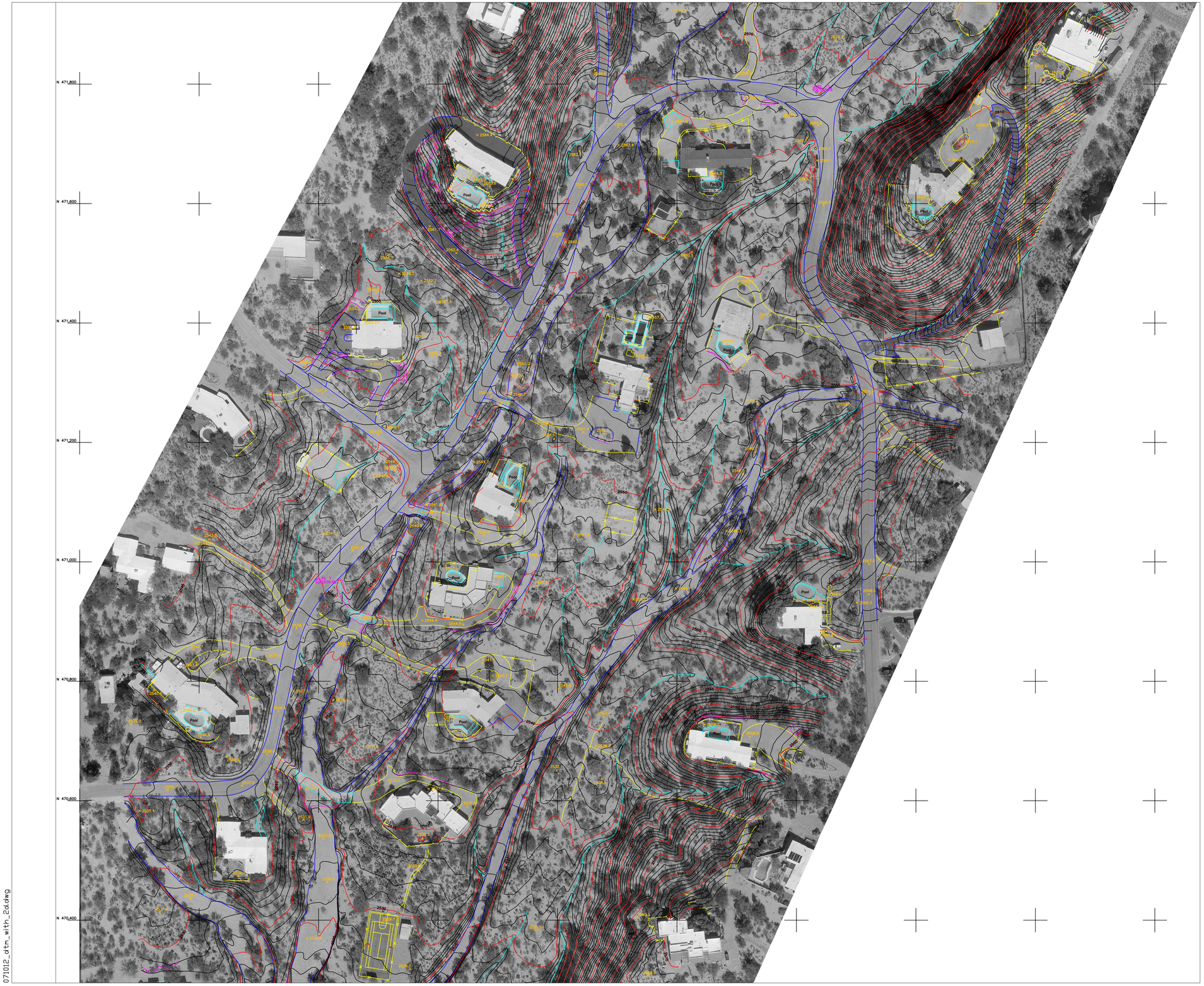
Figure 3

2007 Topographic Mapping (NAVD 88)
2007 Aerial Photograph

Sheet 2 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

071012_dtm_with_2d.dwg



SCALE: 1 INCH = 80 FEET
Contour Interval = 1 FEET

Figure 3

2007 Topographic Mapping (NAVD 88)

2007 Aerial Photograph

Sheet 3 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

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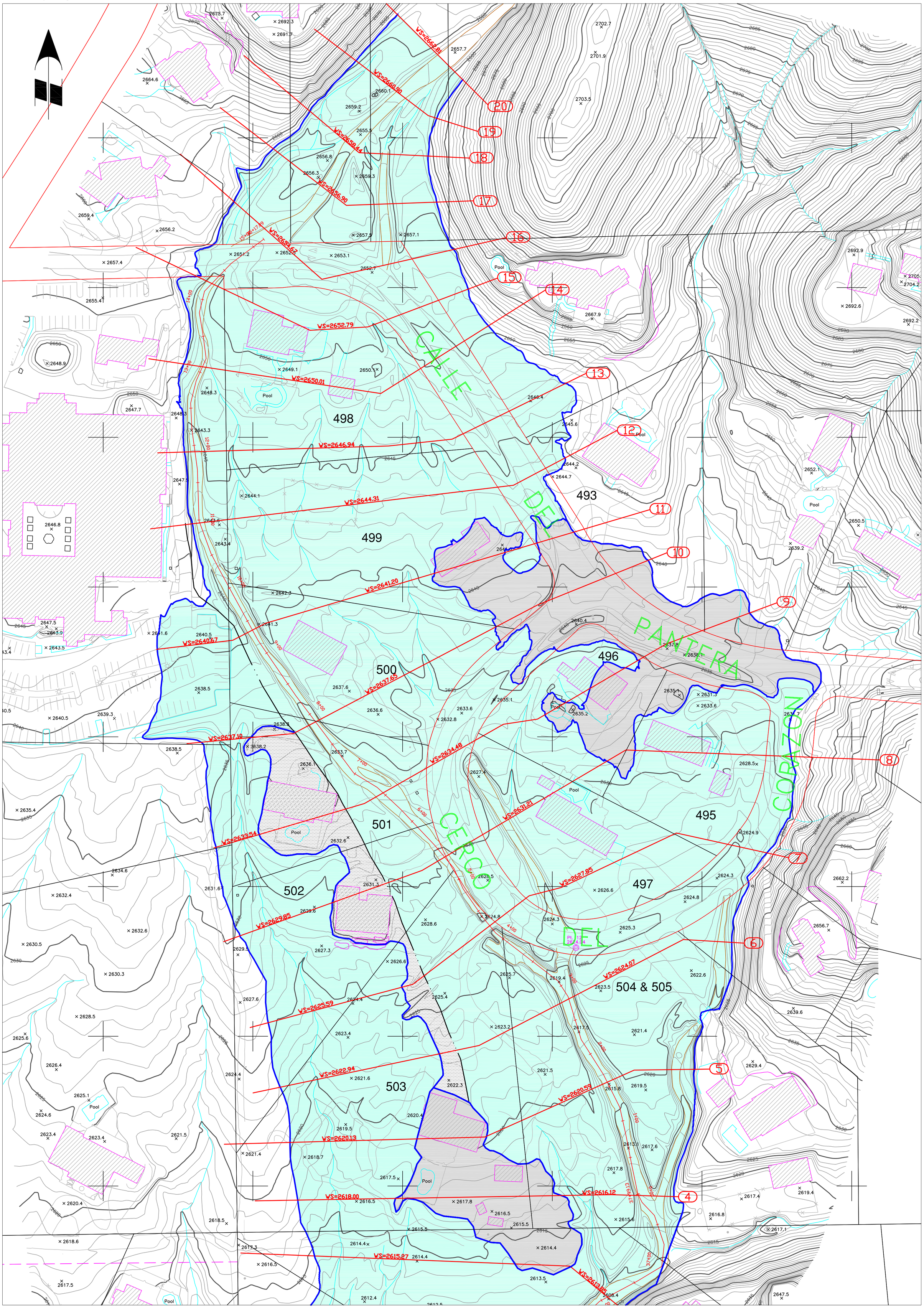
071012_3d-jef.dwg



80' 0' 60' 120'

SCALE: 1 INCH = 60 FEET
Contour Interval (NAVD 88) = 1 FEET

Legend

- HECRAS Cross Section
- 100-Year Floodplain Boundary
- Base Flood Elevations Determined
- Shallow Flooding (avg. depth < one foot)
- Shallow Flooding (avg. depth = one foot)
- Lateral Weir Crest
- Profile Control Line
- Base Flood Elevations (NAVD 88)

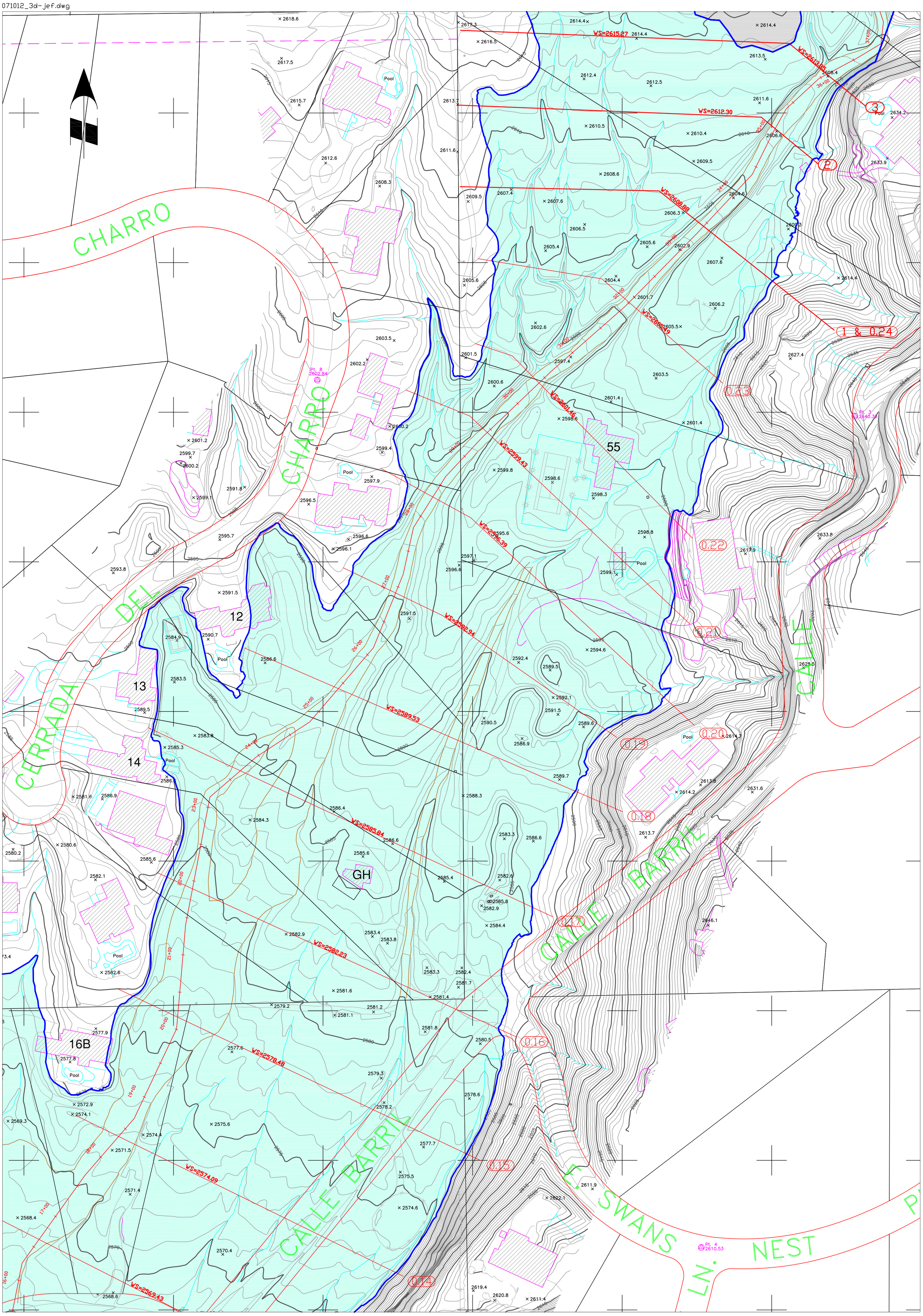
Figure 4

Flood Hazard Map

JE Fuller / Hydrology & Geomorphology, Inc.

Sheet 1 of 3

071012_3d-jef.dwg



80' 0' 60' 120'

SCALE: 1 INCH = 60 FEET

Contour Interval (NAVD 88) = 1 FEET

Legend

- HECRAS Cross Section
- 100-Year Floodplain Boundary
- Base Flood Elevations Determined
- Shallow Flooding (avg. depth < one foot)
- Shallow Flooding (avg. depth = one foot)
- Lateral Weir Crest
- Profile Control Line
- WS=2530.72 Base Flood Elevations (NAVD 88)

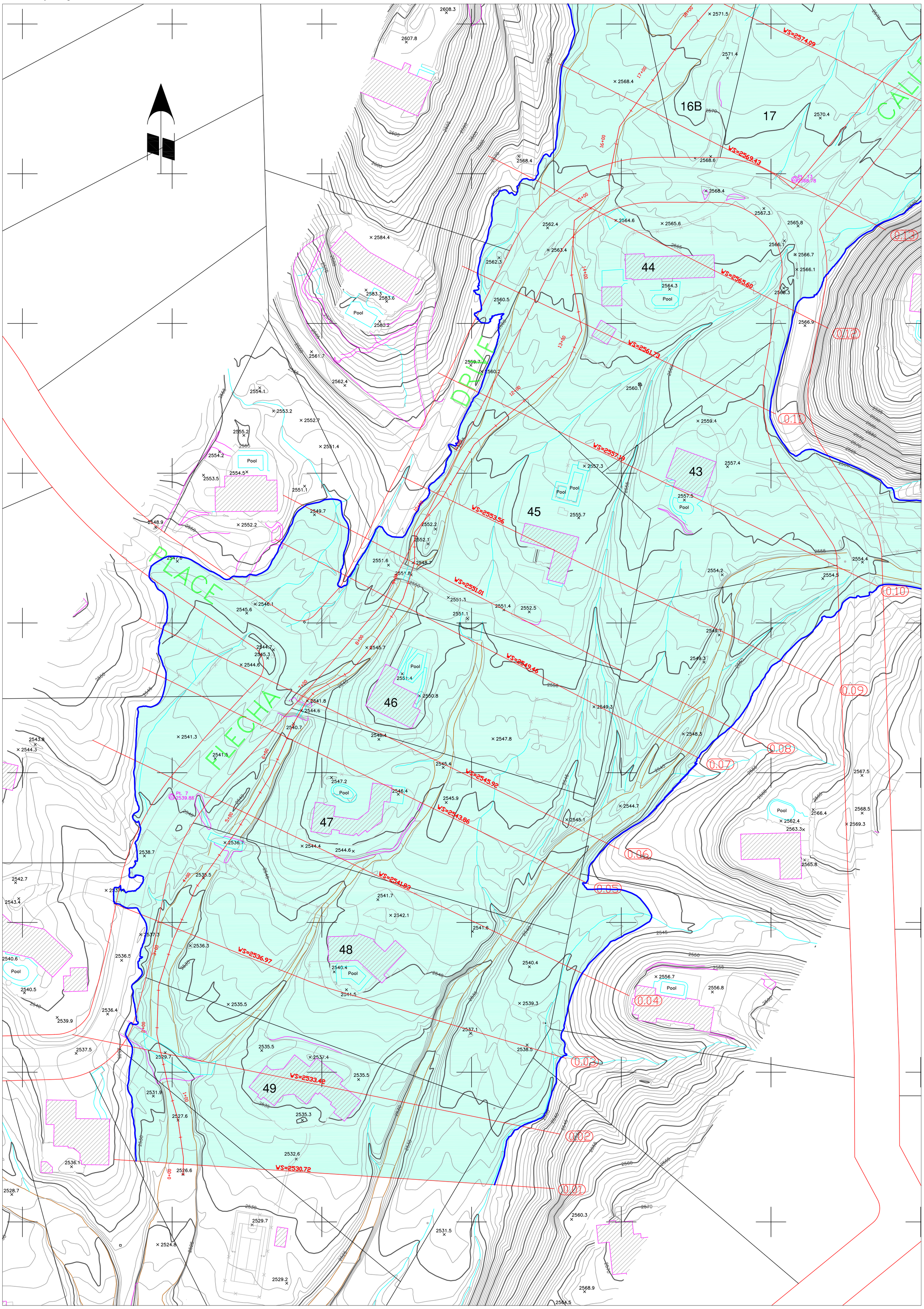
Figure 4

Flood Hazard Map

Sheet 2 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

071012_3d-jef.dwg



80' 0' 60' 120'

SCALE: 1 INCH = 60 FEET

Contour Interval (NAVD 88) = 1 FEET

Legend

- 6 — HECRAS Cross Section
- 100-Year Floodplain Boundary
- Base Flood Elevations Determined
- Shallow Flooding (avg. depth < one foot)
- Shallow Flooding (avg. depth = one foot)
- Lateral Weir Crest
- Profile Control Line
- WS=2530.72 — Base Flood Elevation (NAVD 88)

Figure 4

Flood Hazard Map

Sheet 3 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

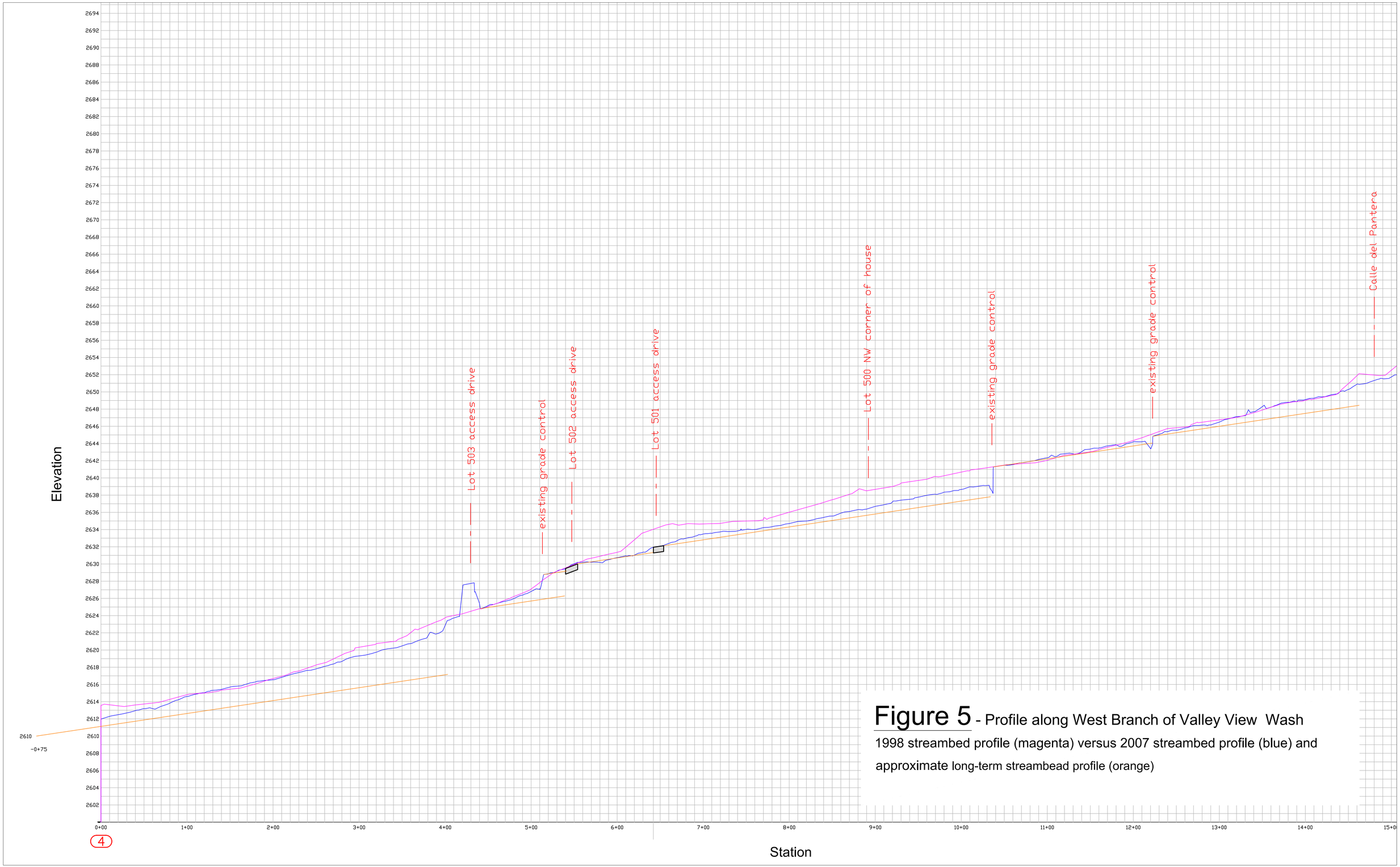


Figure 5 - Profile along West Branch of Valley View Wash

1998 streambed profile (magenta) versus 2007 streambed profile (blue) and approximate long-term streambead profile (orange)

Priority_topo_3d_jef.dwg

Appendix A

A1-Subdivision Plat/Historic Study Excerpts
A2-1984 SLA-Flooding Problems Report
A3-1986 SLA Phase 1-Hydrology and Mapping Report
A4- 1989 SLA Flood and Erosion Assessment/Mitigation Report
A5- Revised Drainage Basin/Floodplain Comparison Exhibits

[redacted]

Book 19 Page 44

4500 E. 4600 E. 4700 E.

UNSUBDIVIDED

5200-N.

5100-N.

5000-N.

4500-E. 4600 E. 4700-E.

5000 N.

NORTH

LOCATION PLAN
No Scale

CALLE DEL CORAZON

CALLE DEL PANTERA

SWAN ROAD

FLECHA CAIDA RANCH ESTATES NO 2
Book 17, Page 69 Nov 8, 1957

UNSUBDIVIDED

494 487 488 498 493 492 489 499 491 490 500 496 495 508 501 497 507 502 506 509 503 504 505 510

FLECHA CAIDA RANCH ESTATES NO 2
Book 17, Page 69 Nov 8, 1957

DEDICATION

[illegible]

PHOENIX TITLE AND TRUST COMPANY,
[illegible] Trust Agreement No 6121

By
[illegible] Vice President

Attest [illegible]
[illegible] Asst Secretary

STATE OF ARIZONA } ss
COUNTY OF PIMA

This instrument was acknowledged before me this [illegible] day of [illegible] 1960 by [illegible] as Vice President and [illegible] Fleming as Asst Secretary of the Phoenix Title and Trust Company [illegible]

Notary Public

My commission expires [illegible]

APPROVALS

Hereby certify that this plat was approved by the Board of Supervisors Pima County, Arizona on the [illegible] day of [illegible] 1960

Clerk Board of Supervisors — Date

County Engineer — Date 2-4-60

I, Mary Kinde Clerk of the City of Tucson hereby certify that this plat was approved by the Mayor and Council of the City of Tucson on the [illegible] day of [illegible] 1960

City Clerk — Date

City Engineer — Date

Date

Chairman City P & Z Comm — Date

Secy City P & Z Comm — Date

RECORDING DATA

STATE OF ARIZONA } ss
COUNTY OF PIMA

No. [illegible]
Fee [illegible]

Filed for record at the request of John W Bender on this [illegible] day of [illegible] 1960 at [illegible] M in Book [illegible] of Maps and Plats of Page [illegible] thereof

Anna Swanger
County Recorder
By [illegible]
Deputy

CERTIFICATION

Hereby certify that this plat represents a survey made under my direction and that all monuments as shown are as shown hereon

[illegible]
Registered Professional Engineer

CURVE DATA

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

NOTES

1. [illegible] water drainageway [illegible] not be changed or altered in any way except as approved by the Pima County Board of Supervisors.
2. [illegible] lot lines are hereby reserved along all subdivision boundary lines and 7' 6" on the [illegible] easements are dedicated along each side of all other lot lines. [illegible] 240 feet [illegible] of lots subject to the approval of the subdivider. [illegible] particular lot and of contiguous lots are [illegible] 7' 6" easements shall be retained on each side of the center line of such [illegible] easements necessary for municipal [illegible] water, sewer, gas, and along the [illegible] of the owner and [illegible] [illegible] easements are hereby reserved along [illegible] subdivision boundary lines [illegible] 30' on each side of all other lot lines.
3. Basis not established from centerline of the SE [illegible] of Sec. 5 as shown on plat recorded as Instrument No 37570 June 18, 1957 in Book 142 of Miscellaneous Records at Pages 417 to 422.
4. [illegible] of the P.C. P.R.C. & P.T. [illegible] with 1/2" pipe set at [illegible] other lot corners.
5. [illegible] Pima County Survey Monument [illegible]
6. [illegible]

MAP 48
ZONE CR 1
H.D.Z.-11

FLECHA CAIDA RANCH ESTATES #9

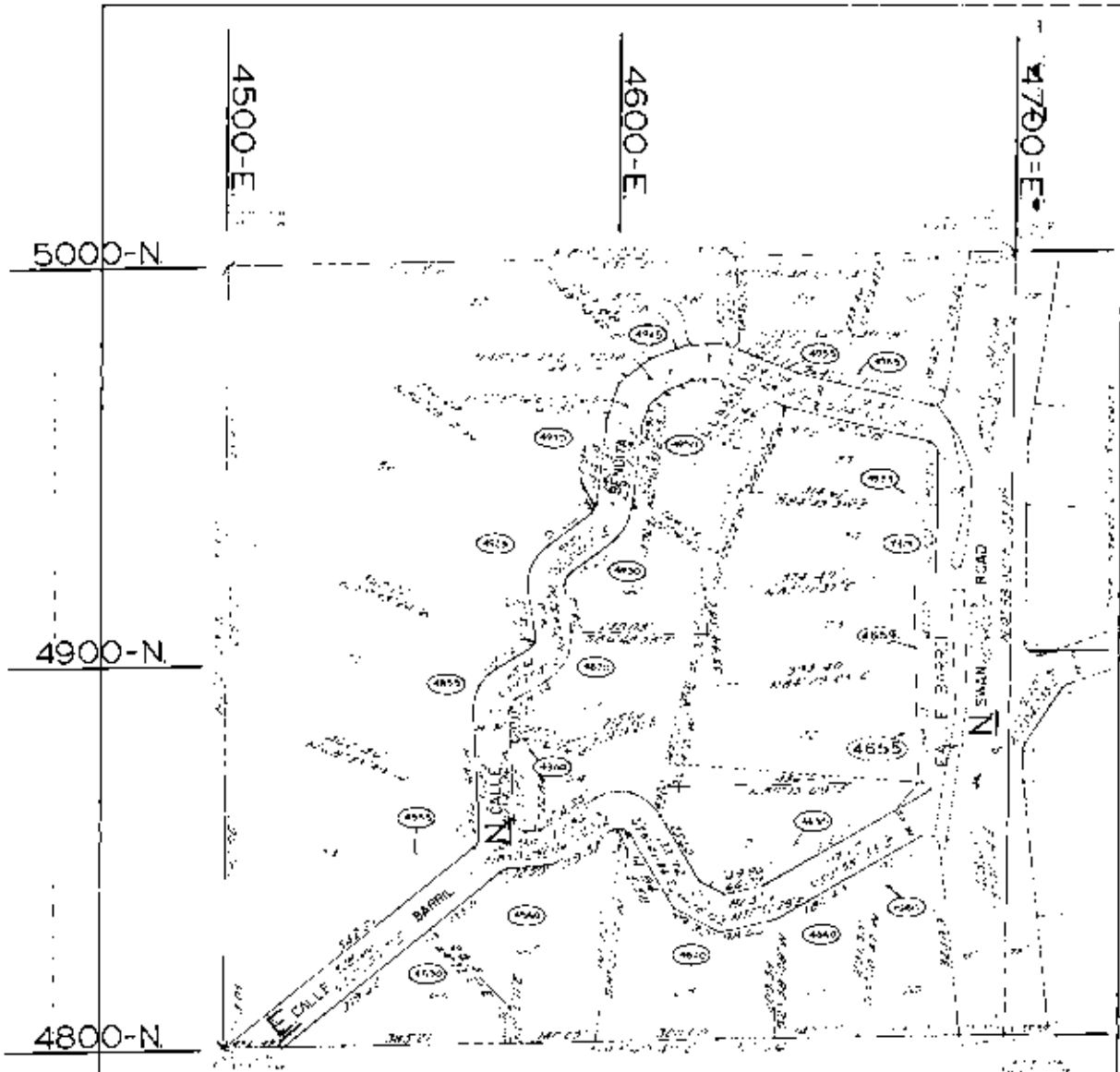
BEING A SUBDIVISION OF A PORTION OF THE NE1/4 OF THE SE1/4 OF SECTION 15, T 13 S, R 14 E, G & S R B & M, PIMA COUNTY, ARIZONA

MADDOCK & ASSOCIATES

SCALE 1" = 100' SHEET 1 OF 1 DECEMBER, 1959

CHECK FLOODPLAIN ON ALL LOTS

Book 19 Page 44



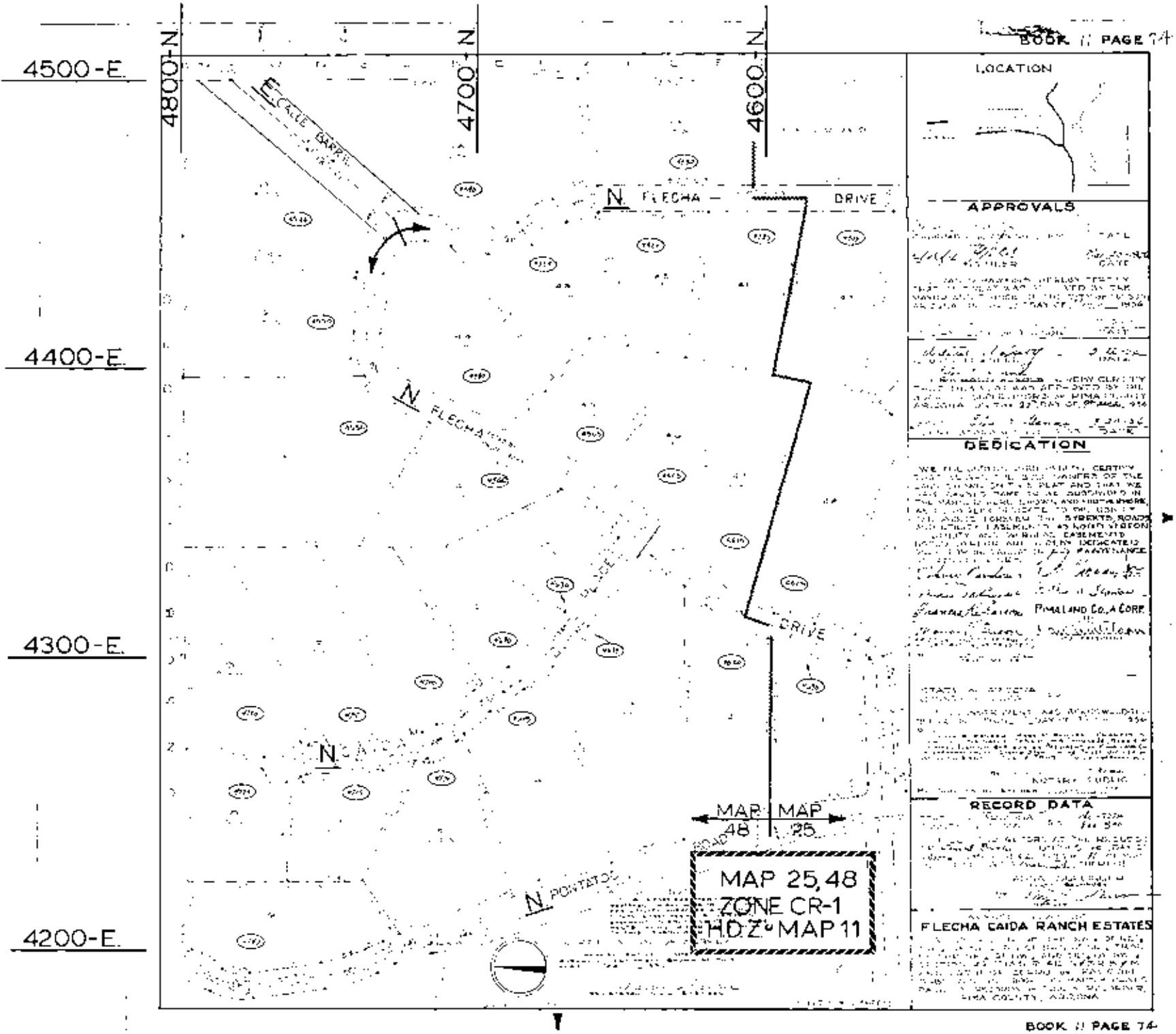
LOCATION

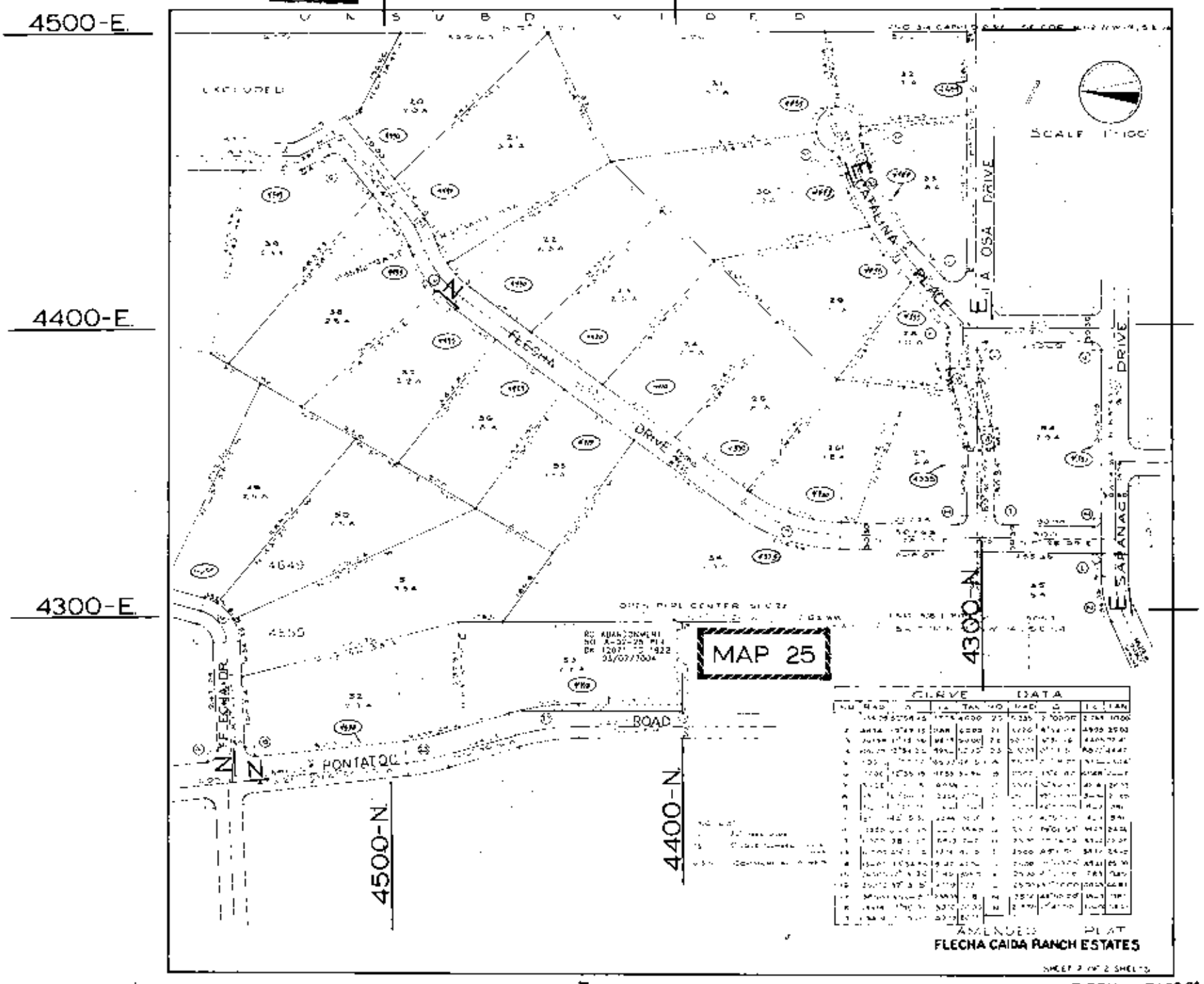
APPROVALS

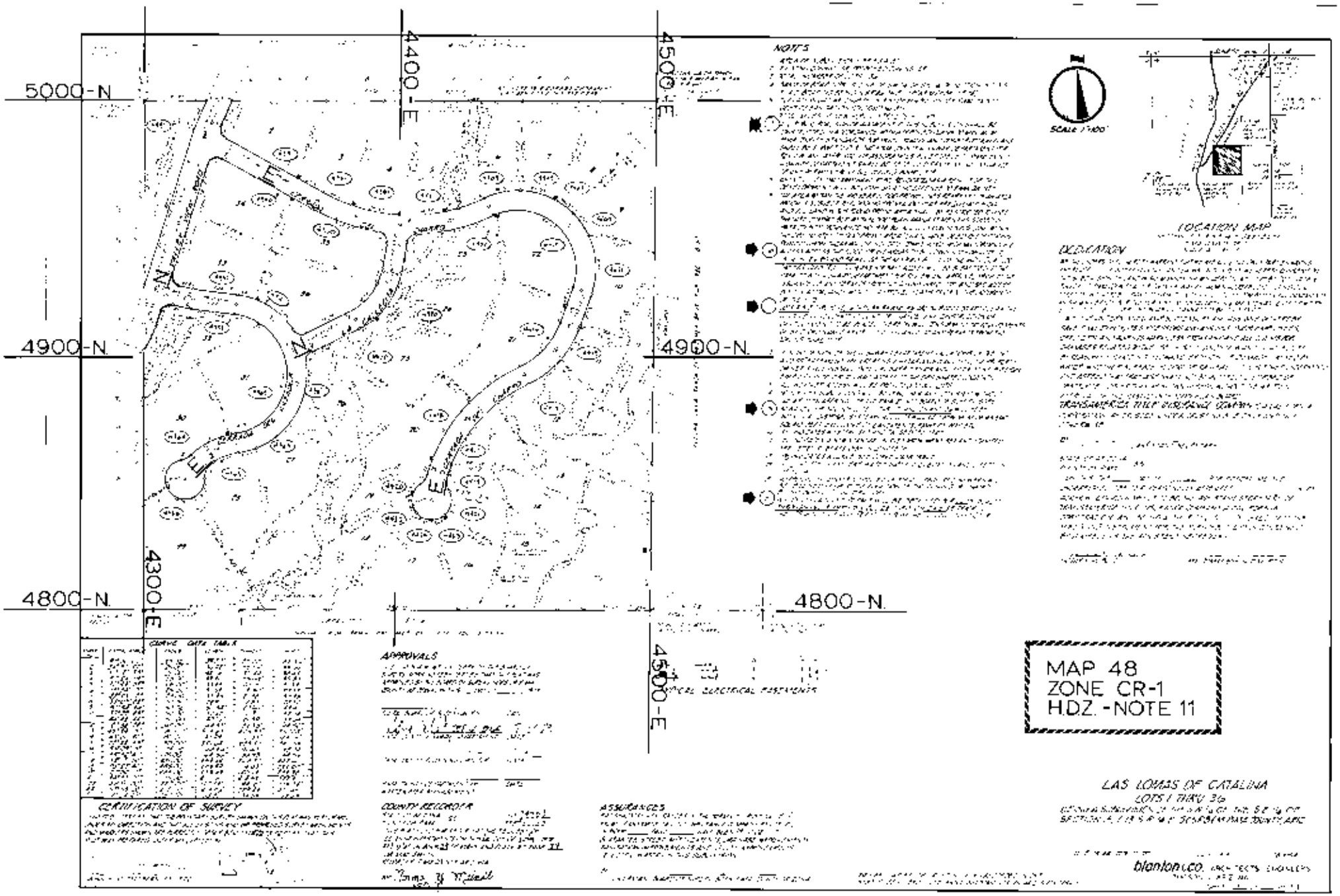
DEDICATION

RECORD DATA

FLECHA CAIDA RANCH ESTATES NO 2


4500-E.
4400-E.
4300-E.
4200-E.
4800-N.
4700-N.
4600-N.
E. CALLE BARRIL
N. FLECHA DRIVE
N. FLECHA
N. FLECHA DRIVE
N. POINTATOS
MAP 48
MAP 25
MAP 25,48
ZONE CR-1
H.D.Z. MAP 11
LOCATION
APPROVALS
DATE
DEDICATION
PIMALAND CO., A CORP.
RECORD DATA
FLECHA CAIDA RANCH ESTATES


4500-E.
4400-E.
4300-E.
UNSUBDIVIDED
EXCLUDED
SCALE 1"=100'
CATALINA PLACE
E. LA OSA DRIVE
E. SAPANAGO DRIVE
FLECHA DRIVE
PONTATOC ROAD
MAP 25
4300-N.
4400-N.
4500-N.
CURVE DATA
AMENDED PLAT
FLECHA CAIDA RANCH ESTATES
SHEET 2 OF 2 SHEETS


5000-N
4900-N
4800-N
4400-E
4500-E
4300-E
4900-N
4800-N
4500-E
NOTES
SCALE 1"=100'
LOCATION MAP
DEDICATION
APPROVALS
CERTIFICATION OF SURVEY
COUNTY RECORDER
ASSURANCES
MAP 48
ZONE CR-1
H.D.Z. - NOTE 11
LAS LOMAS DE CATALINA
LOTS 1 THRU 36
blanton.co.

PRELIMINARY ASSESSMENT

OF FLOODING PROBLEMS ALONG

VALLEY VIEW WASH IN THE VICINITY

OF THE FLECHA CAIDA SUBDIVISION

Property of:
Pima County
Flood Control District Library
740-6350

Prepared for:

Pima County Department of Transportation
& Flood Control District
1313 South Mission Road
Tucson, Arizona 85713

AUG 1984
FLOOD
PLAIN
MANAGEMENT

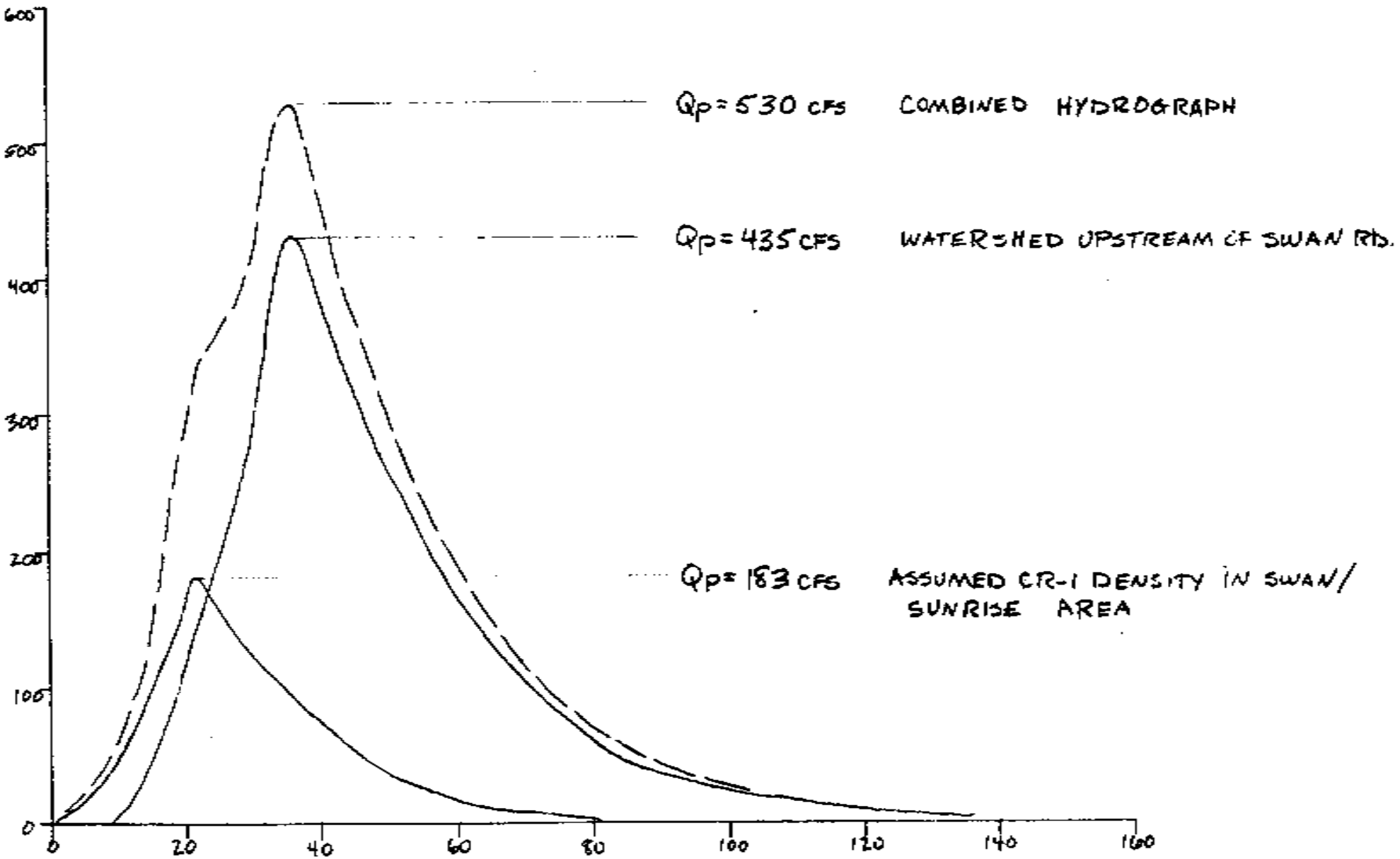
Prepared by:

Simons, Li & Associates, Inc.
120 W. Broadway, Suite 260
P.O. Box 2712
Tucson, Arizona 85702

July 26, 1984

VALLEY VIEW WASH

ESTIMATED HYDROGRAPHS FOR JUNE 27, 1984 FLOODS WITH ASSUMED CR-1 DENSITY IN SWAN/SUNRISE AREA



FLECHA CAIDA FLOOD IMPROVEMENT STUDY -

PHASE I

100-YEAR PEAK DISCHARGE

MAGNITUDES AND FLOODPLAIN MAPPING

Prepared for:

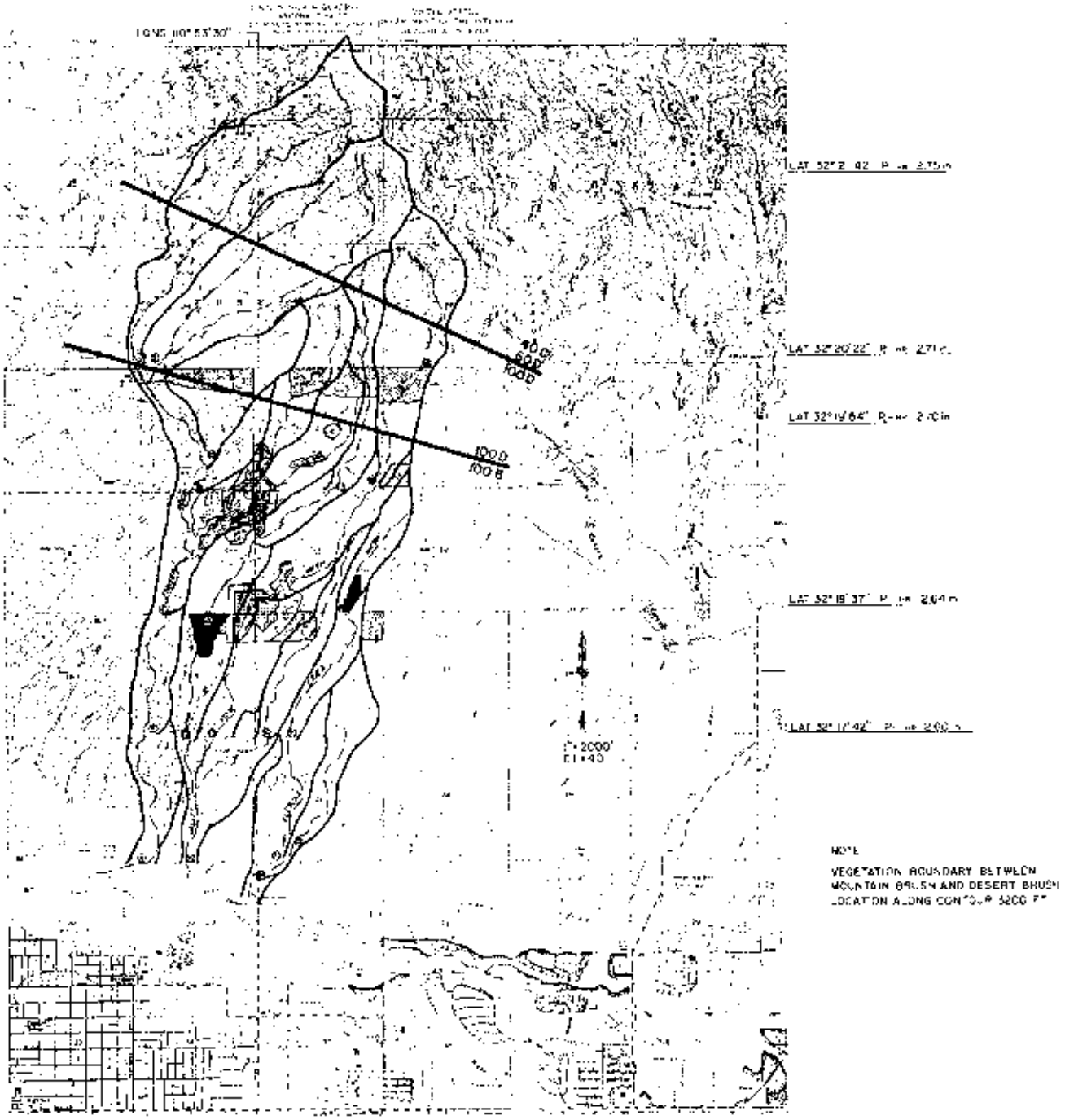
Pima County Department of Transportation
and Flood Control District
1313 South Mission Road
Tucson, Arizona 85713

Property of:
Pima County
Flood Control District Library
740-6350

Prepared by:

Simons, Li & Associates, Inc.
120 W. Broadway, Suite 170
P.O. Box 2712
Tucson, Arizona 85702

January 28, 1986



LEGEND

- WATERSHED BOUNDARY
- 1000 / 100B — HYDROLOGICAL SOIL GROUP BOUNDARY
- DRAINAGE CONCENTRATION POINT
- PRIMARY & SECONDARY WATERCOURSES
- RESIDENCES/ACRE
- SCHOOL (PERMANENT SITES OR PROPOSED)
- COMMERCIAL
- TRANSITIONAL

APPENDIX A

DRAINAGE BASIN MAP SHOWING HYDROLOGIC CHARACTERISTICS OF STUDY AREA

RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 32° 18' 37"

Return Period (Years)	Precipitation Values (inches)*			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.55	X_1 1.57	1.88	X_2 1.86
5	2.04	2.07	2.50	2.52
10	2.42	2.40	2.90	2.95
25	2.88	2.82	3.60	3.51
50	3.25	3.20	4.00	4.02
100	3.50	X_3 3.58	4.50	X_4 4.51

$$Y_2 = -0.011 + 0.942(X_1^2/X_2) = 1.24''$$

$$Y_{100} = 0.494 + 0.755(X_3^2/X_4) = 2.64''$$

RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 32° 19' 54"

Return Period (Years)	Precipitation Values (inches) *			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.60	X_1 1.63	1.95	X_2 1.94
5	2.10	2.13	2.58	2.60
10	2.50	2.46	3.00	3.03
25	2.95	2.90	3.65	3.60
50	3.32	3.29	4.10	4.11
100	3.60	X_3 3.67	4.60	X_4 4.61

$$Y_2 = -0.011 + 0.942\,(X_1^2/X_2) = 1.27''$$

$$Y_{100} = 0.494 + 0.755\,(X_3^2/X_4) = 2.70''$$

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E1/2 R14E N2/3 T13S)

Drainage Concentration Point: #11 (@ VALLEY VIEW WASH SWAN ROAD SEC LINE 10 & 11)

Watershed Area (A): 908 (927*) acres/square miles.

Length of Watercourse (L_c): 21700 ft. Length to Center of Gravity (L_{ca}): 10900 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
4000	5600 - 3800 = 1800
2200	3800 - 3400 = 400
1900	3400 - 3200 = 200
12500	3200 - 2740 = 460

Mean Slope (S_c): 0.0670 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (a_b): 0.047 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.69 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 38%B 13%C 49%D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 70% (DB) Impervious Cover: 8% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.11, 86.45, 92.41
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5501, 0.5323, 0.7086 (pervious areas) 0.9563 (impervious areas)

Runoff Supply Rate (q): 0.6519 i in./hr. (function of i)

Time of Concentration (T_c): 64 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 39 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 3.7103 in./hr.

Runoff Supply Rate (q) at T_c: 2.4187 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2214 (2260*) cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

* @ SUNRISE DRIVE (SEC LINE 10 & 15)

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CANYON FLOOD IMPROVEMENT STUDY (E½ RIVE N2/3 T13S)

Drainage Concentration Point: #13 (VALLEY VIEW WASH @ SEC LINE 15 & 22)

Watershed Area (A): 1239 acres/~~square miles~~.

Length of Watercourse (L_c): 27300 ft. Length to Center of Gravity (L_{ca}): 13700 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
2000	5600 - 3800 = 1800
2700	3800 - 3400 = 400
1800	3400 - 3200 = 200
12500	3200 - 2740 = 460
3600	2740 - 2580 = 160

Mean Slope (S_c): 0.0545 ft./ft. Watershed Type(s): _____ (future)

Basin Factor (a_b): 0.045 (future) Flood Frequency: _____ yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.68 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 53%B 10%C 37%D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30%(MB) 20%(DB) Impervious Cover: 12% (future)

CN(s): 83 82, 90 (pervious & impervious areas) CN*(s): 87.09, 86.43, 92.40
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5483, 0.5306, 0.7072 (pervious areas) 0.9562 (impervious areas)

Runoff Supply Rate (q): 0.6474 i in./hr. (function of i)

Time of Concentration (T_c): 77 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 49 hrs./mins.

Rainfall Intensity (i) at T_c: 3.1072 in./hr.

Runoff Supply Rate (q) at T_c: 2.0117 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2512 cfs.

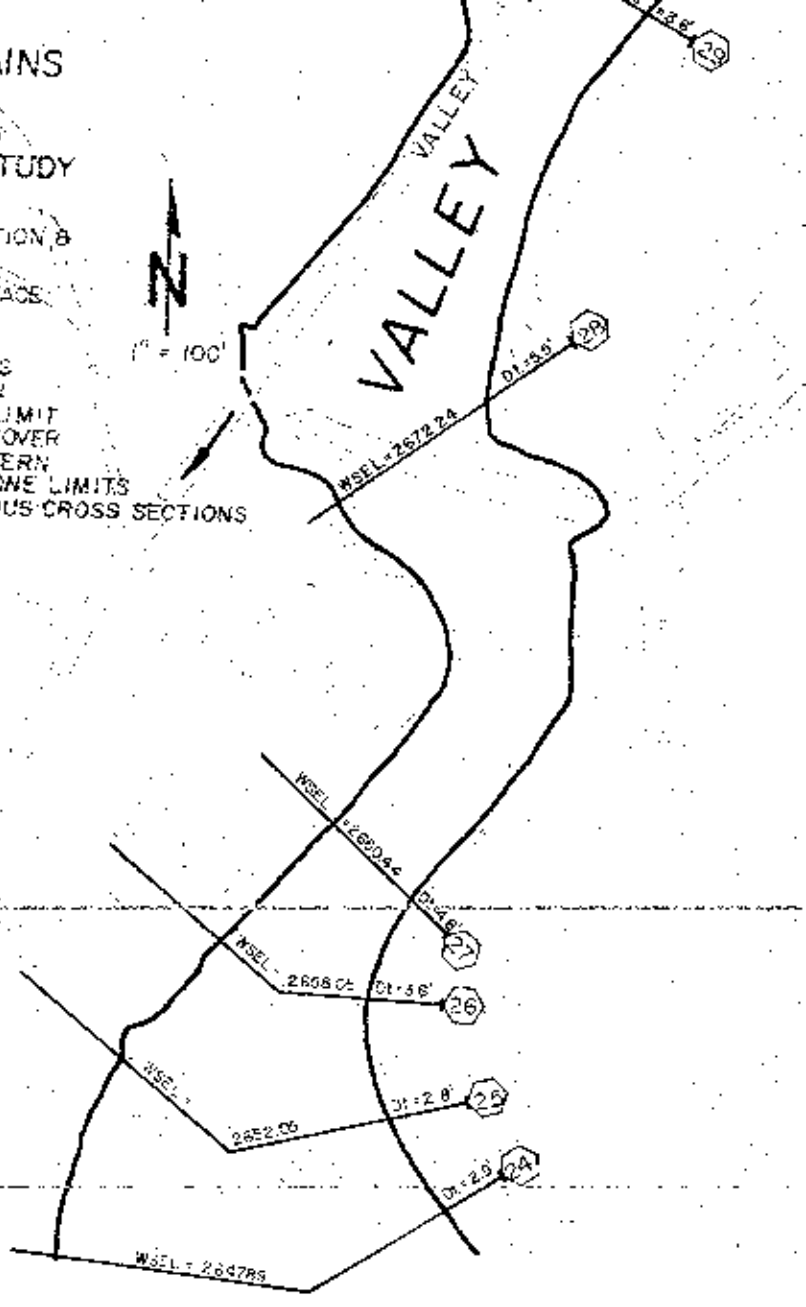
645.13qA (square miles): _____ cfs.

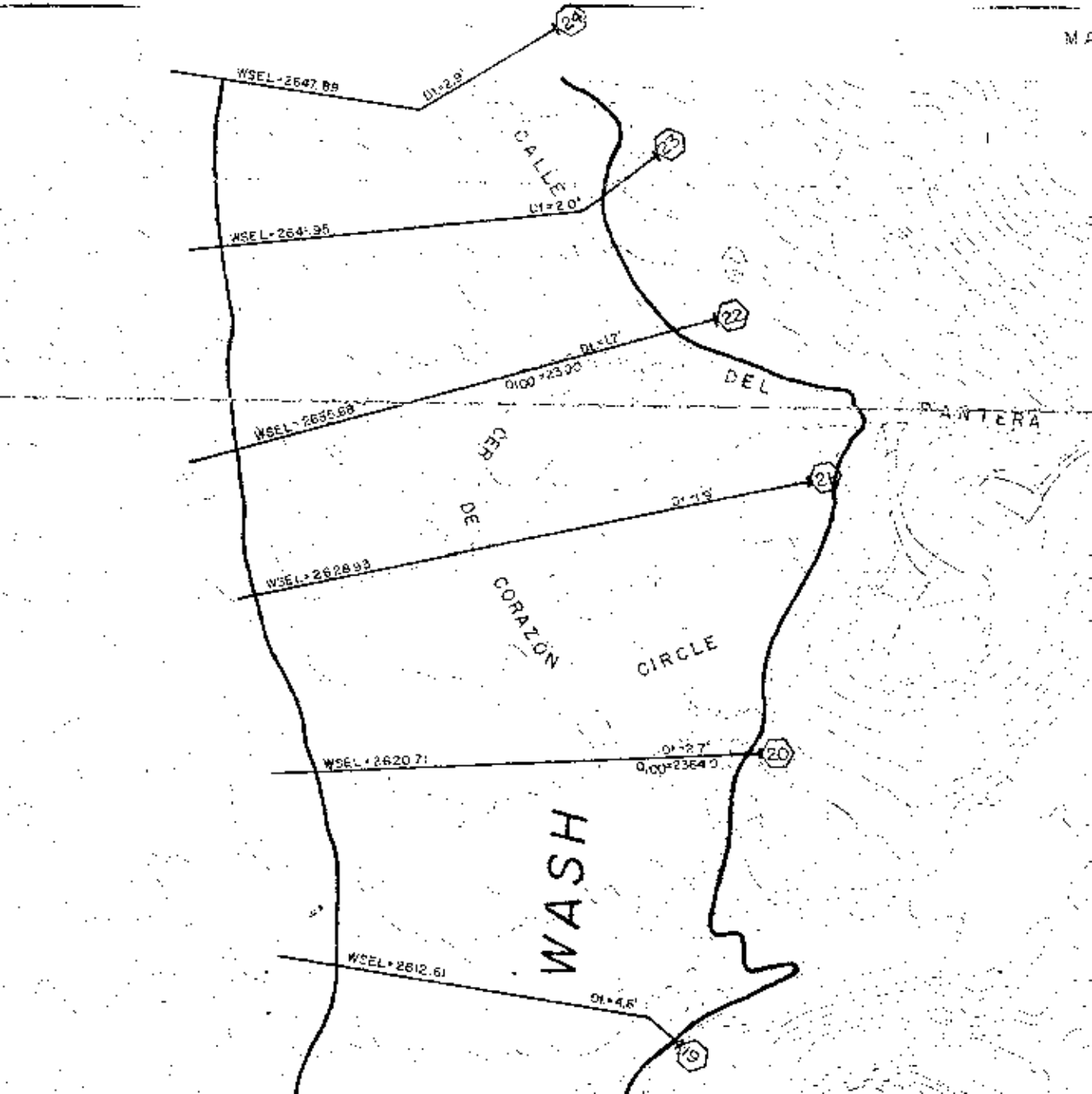
Note: For impervious areas, CN* = 99 (constant).

100-YEAR FLOOD PLAINS FOR THE FLECHA CAIDA FLOOD IMPROVEMENT STUDY

LEGEND

- (1) CROSS-SECTION LOCATION & STARTING STATION
- WSEL 100-YEAR WATER SURFACE ELEVATION
- D THALWEG DEPTH
- Q100 100-YEAR FLOW IN CFS
- 100-YEAR FLOOD PLAIN
- DISCONTINUOUS FLOOD LIMIT DUE TO POSSIBLE BREAKOVER
- BREAKOVER FLOW PATTERN
- PROJECTED FLOOD PRONE LIMITS BETWEEN DISCONTINUOUS CROSS SECTIONS




WSEL=2647.89
DL=2.9'
24
CALLE
23
DL=2.0'
WSEL=2641.95
22
DL=1.7'
Q100=2330
WSEL=2635.68
DEL
PANTERA
CER
DE
21
DL=1.9'
WSEL=2628.93
CORAZON
CIRCLE
WSEL=2620.71
DL=2.7'
Q100=2364.0
20
WASH
WSEL=2612.61
DL=4.6'
19

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

01

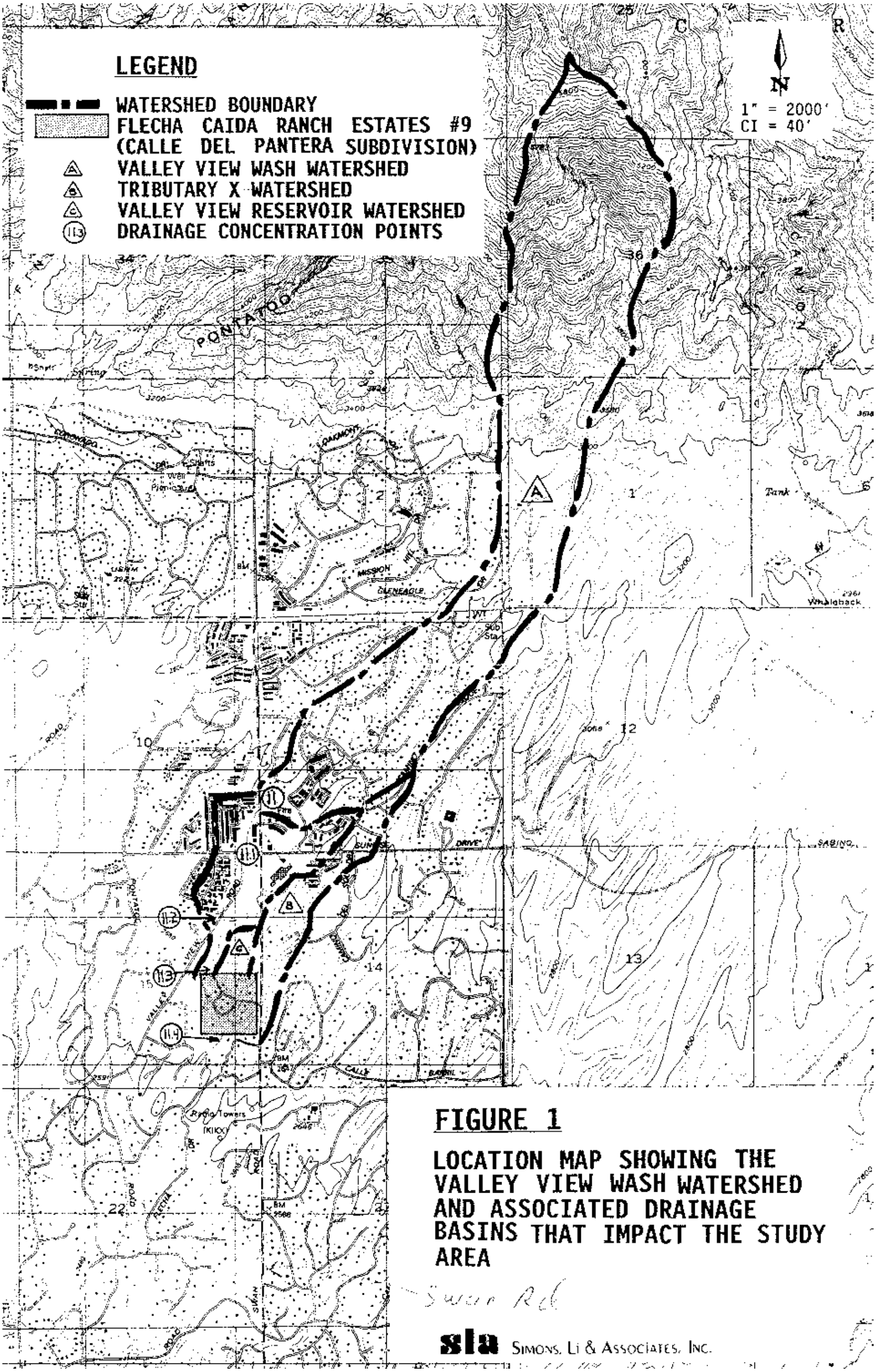


FIGURE 1

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

Susan Rd

sla SIMONS, LI & ASSOCIATES, INC.

494
487
488
498
493
492
489
499
491
490
CALLE DEL PANTERA
A
500
496
495
501
497
507
508
CERCO DE CORAZON CIRCLE
502
506
504
509
503
505
510
SWAN ROAD

PRELIMINARY ASSESSMENT

OF FLOODING PROBLEMS ALONG

VALLEY VIEW WASH IN THE VICINITY

OF THE FLECHA CAIDA SUBDIVISION

Property of:
Pima County
Flood Control District Library
740-6350

Prepared for:

Pima County Department of Transportation
& Flood Control District
1313 South Mission Road
Tucson, Arizona 85713

AUG 1984
FLOOD
PLAIN
MANAGEMENT

Prepared by:

Simons, Li & Associates, Inc.
120 W. Broadway, Suite 260
P.O. Box 2712
Tucson, Arizona 85702

July 26, 1984

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I. RESULTS AND CONCLUSIONS	1
II. SUMMARY OF DISCUSSION WITH NINO GAZZI OF M.M. SUNDT CONSTRUCTION COMPANY (MEMO PREPARED BY MR. JOHN WALLACE, SLA)	7
III. HYDROLOGIC AND HYDRAULIC ANALYSIS OF JUNE 27, 1984 EVENT	9

LIST OF APPENDICES

APPENDIX A - Hydrologic and Hydraulic Computations

LIST OF FIGURES

FIGURE 1.1 - 1941 Photograph, Valley View Wash through Flecha Caida

FIGURE 1.2 - 1978 Photograph, Valley View Wash through Flecha Caida

I. RESULTS AND CONCLUSIONS

This report presents a preliminary evaluation of the flooding problems which have impacted upon the Flecha Caida subdivision. This site is located southwest of the intersection of Swan Road and Sunrise Drive along Calle del Pantera.

The Pima County Department of Transportation & Flood Control District (PCDOT&FCD) requested Simons, Li & Associates, Inc. (SLA) to make an evaluation of the flooding problems which impact upon this subdivision, and to determine if any upstream construction activity resulted in an increase in flooding to the Flecha Caida subdivision. In particular, the PCDOT&FCD requested an assessment as whether or not the peak-flow rate was increased by any unusual diversion or concentration of flow caused by construction of a box culvert at the Valley View Wash crossing of ongoing Sunrise Drive during a rainfall runoff event that occurred on June 27, 1984. The first field investigation was made by Mr. Michael E. Zeller, P.E., and Mr. Clinton M. Glass, P.E. on June 29, 1984. A summary of the findings of both that field investigation and the ensuing study are as follows:

1. A determination of whether or not flood waters backed up behind an artificial dam in the vicinity of the Sunrise Road box culvert construction area could not be made during the field visit, since significant regrading and clean-up of the area had already been undertaken prior to our arrival. However, we were able to assess the elevation of flood waters based upon debris lines along the channel banks and along the side walls of the box culvert. To further understand the flooding conditions at the Sunrise Road box culvert site, we contacted Mr. Nino Gazzi of M.M. Sundt Construction Company, the Construction Manager for the Sunrise Road project, and Mr. Charles Music, with the Pima County Inspections Division. Both men were at the site on June 27th, and witnessed the flooding. In

addition, several photographs were taken in the area of the Valley View Wash crossing of Sunrise Road. A summary of the meeting with Mr. Gazzi, along with copies of the photographs which he took during this flood, are provided in the following section of this report.

2. A drive down Valley View Road revealed that a significant volume of runoff had collected against the curb of the roadway, and then drained south until it was recaptured by Valley View Wash at a point located approximately one-quarter mile south of Sunrise Road. This runoff apparently came from some of the high-density urbanized area in the vicinity of the Swan and Sunrise intersection.

3. Examination of old aerial photographs found that Calle del Pantera was constructed along the alignment of one of the old channel braids. During the June 27, 1984 event, flood waters broke out into this old channel braid just upstream of Calle del Pantera; and then flowed southeasterly in the roadway to Cerco del Corazon, where they then flowed southerly within that roadway (and across adjacent subdivision lots) until they were finally recaptured by the natural channel at the south end of the subdivision. A 1941 photograph and a 1978 photograph of the area is provided on Figures 1.1 and 1.2 of this report.

4. Runoff generated in the vicinity of the Swan/Sunrise intersection is either conveyed to Valley View Wash by storm-sewer systems, or if flows overland until intercepted by the curb along Valley View Road. In either instance, this runoff is collected back into the natural channel at a point approximately one-quarter mile upstream of the Calle del Pantera crossing. Other than these modifications to the upstream drainage,

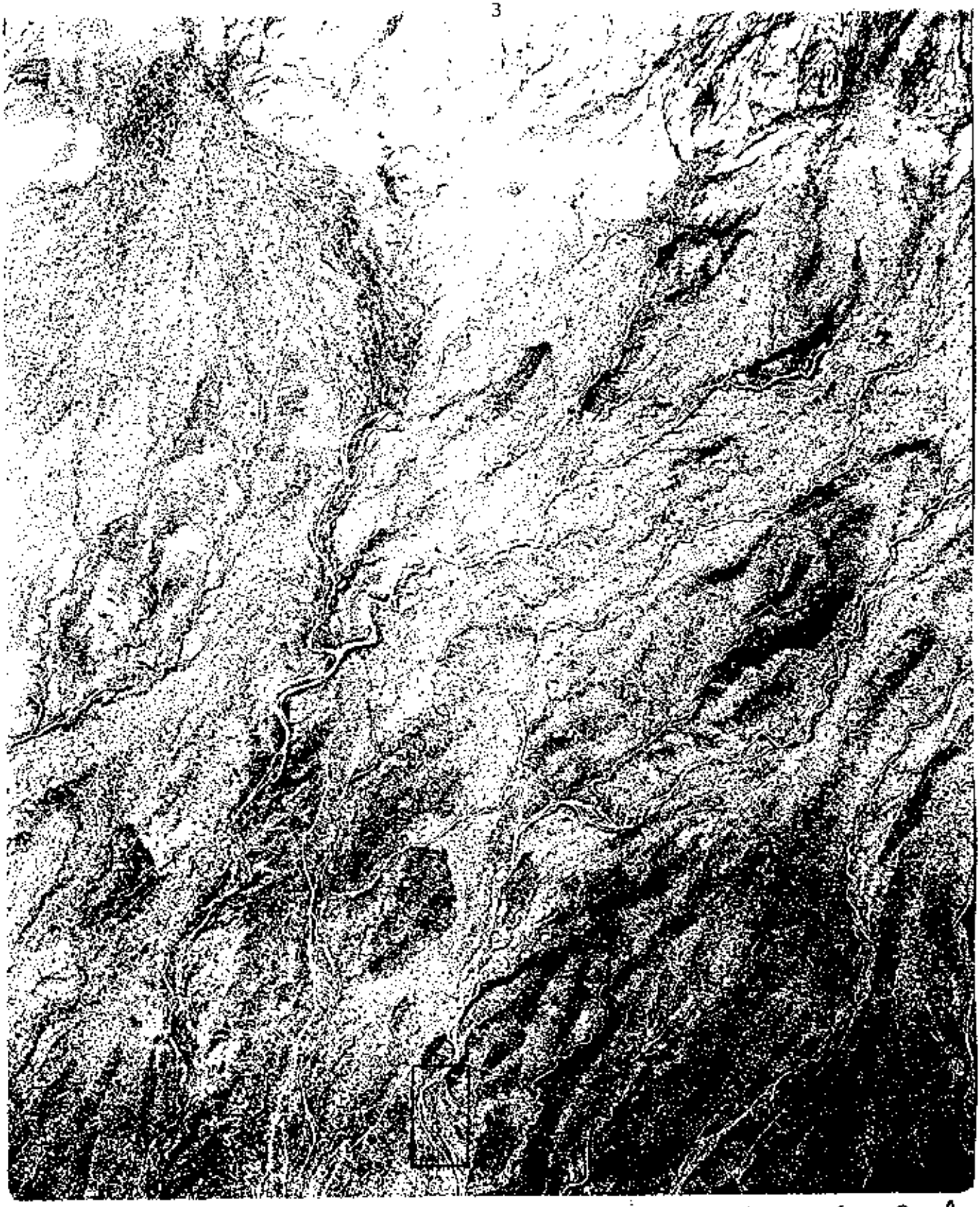


Figure 1.1 - 1941 Photograph, Valley View Wash through Fleche Caida

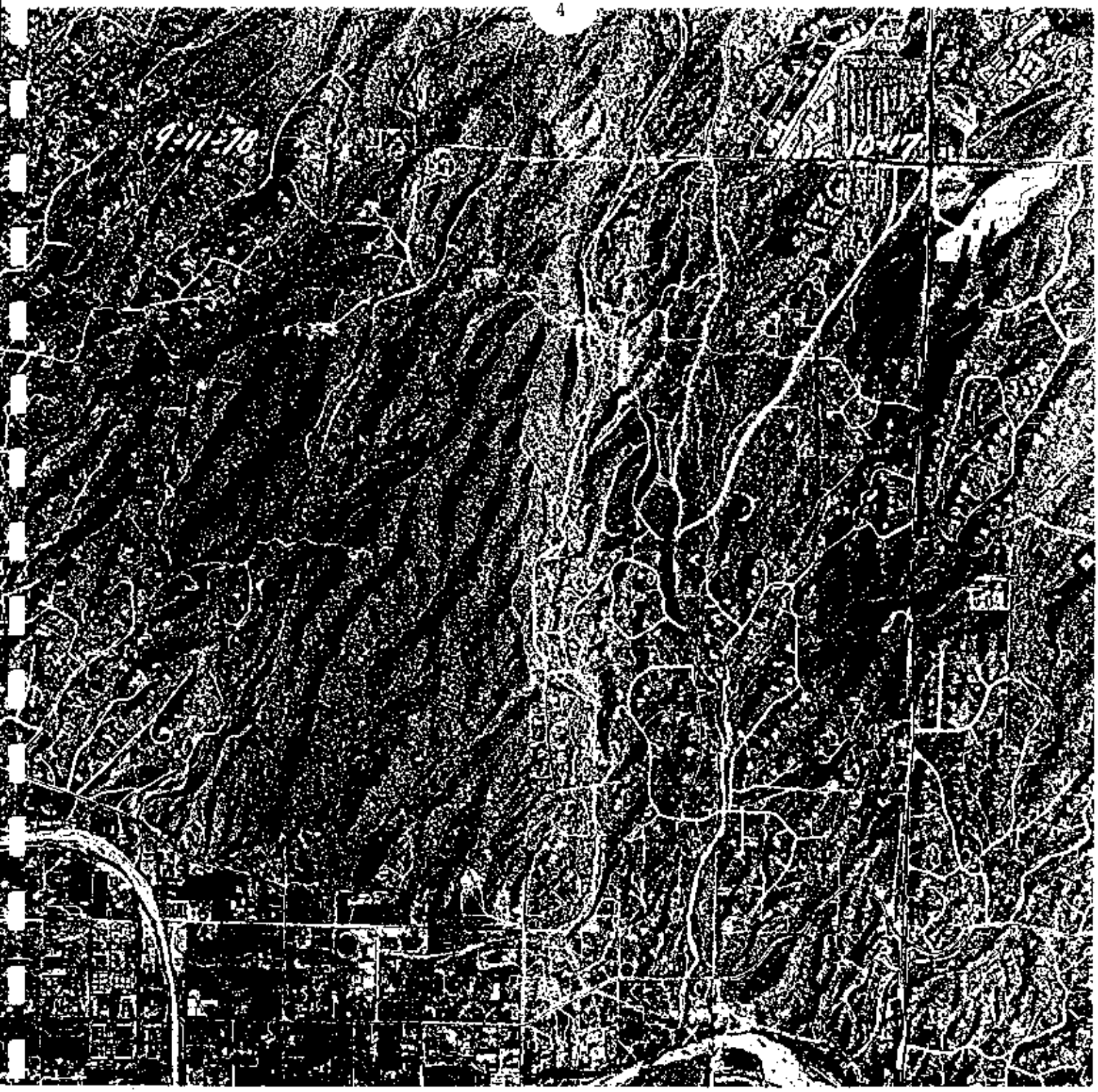


Figure 1.2 - 1978 Photograph, Valley View Wash through Flecha Caida

which resulted from recent urbanization, there were no apparent diversions of flow which either increased the contributing basin area or further concentrated the flow within the study area.

5. Examination of historical photographs showing the pre-development channel conditions through the Flecha Caida subdivision area revealed that the homes which have been impacted during recent flooding were constructed within the natural floodplain area of Valley View Wash. Prior to being developed, the channel through this reach consisted of a network of braided channels that spread across the entire low-lying area between adjacent terraces. Flooding along this type of alluvial sand-bed channel is typically characterized by periodic lateral shifts of the channel from one point to another across the floodplain area.

6. Considering the nature of the natural channel geomorphology and hydraulics, totally inadequate provisions for drainage were provided at the time of platting this subdivision. Many of the existing homes within the subdivision area are only located one to two feet above the adjacent channel thalweg, and within just a few feet of the channel banks. Based upon the historical photographs it appears that some of the structures within the subdivision were constructed directly on top of the old channel system.

7. Rainfall data provided by the PCDOT&FCD indicated that the event of June 27, 1984 measured 1.9 inches during a 2-1/2 hour period between 1:00 p.m. and 3:30 p.m. Based upon the magnitude of runoff that was generated and the normal characteristics of the summer thunderstorms in the Tucson area, it is likely that most of this rainfall occurred during a period of one hour, or

less. This event was compared with NOAA rainfall data to estimate its return period. The estimated frequency of this rainfall event, based upon the NOAA data, is between 5 and 10 years.

8. A hydrologic and hydraulic analysis was conducted to estimate the volume of runoff and peak-flow rate during the June 27, 1984 flood, and to assess the degree to which upstream urbanization, particularly in the vicinity of the Swan Road and Sunrise Drive intersection, has increased downstream flooding. An outline of the methodology used in this analysis is described in Section 3 of this report. The basic conclusions of this analysis are that the upstream urbanization has increased the frequency of runoff generating events and has increased the volume of runoff. However, the analysis also found that the actual peak rate of flow and depths of flooding were only marginally increased over and above what would have occurred had the upstream area been only developed at a CR-1 density, rather than the existing higher densities. The principal basis for this conclusion is the differences in time of concentration for runoff generated by the highly urbanized areas in the vicinity of Swan Road and Sunrise Drive versus that of the upstream watershed area, which stretches approximately 4 miles north of the subdivision. The analysis of the June 27th event found that the high-density urbanization increased the flow rate from 530 cfs to 570 cfs; or, corresponding, by a depth of between 0.1 to 0.2 feet.

clarify by %

define

7-8%

II. SUMMARY OF DISCUSSIONS WITH NINO GAZZI OF M.M. SUNDT CONSTRUCTION COMPANY (MEMO PREPARED BY MR. JOHN WALLACE, SLA)

On July 10, 1984 I met with Mr. Nino Gazzi to discuss the events that had occurred on June 27, 1984 at his job site (Sunrise Drive just west of Swan Road) which resulted from the rainfall occurring on that day.

At this site, 5-5'x8' concrete box culverts are presently being build to convey flows from a natural channel (wash) under Sunrise Drive at a point just west of Valley View Drive. On the morning of June 27th, the construction of these culverts was limited to that portion extending to the north of the centerline of Sunrise Drive. The westbound lane and supporting embankment of Sunrise Drive had been removed where it crosses the wash to allow construction to proceed. The eastbound lane and supporting embankment were still in place at this time.

Rainfall began in the area at approximately 1:30 p.m. and the photos that accompany this summary were taken at approximately the time the flows concentrating at the culvert construction site reached their peak.

As indicated in the photos, and confirmed by Mr. Gazzi, water from the wash backed up behind the undisturbed portion of the roadway embankment until it reached the top of the roadway, at which point it began to flow over the roadway into the downstream channel. According to Mr. Gazzi, the maximum depth of water flowing over the roadway at any time was approximately 1-foot. After the water had stopped flowing over the roadway, a worker at the site excavated a trench about 3 feet in width through the roadway embankment in order to drain the remaining water ponded behind the embankment. The photo showing the downstream channel was taken at approximately the time of peak flow over the top of the roadway.

According to Mr. Gazzi, there was also a large volume of runoff generated from the developed areas adjacent to the intersection of Sunrise Drive and Swan Road. This runoff flowed down

Valley View Drive and across the natural desert area between the wash and Valley View Drive until it was completely intercepted by the Wash where it crosses Valley View Drive. As a result of heavy flows on the roadway, considerable damage was done to a newly-installed curb alone Valley View Drive.

III. HYDROLOGIC AND HYDRAULIC ANALYSIS OF JUNE 27, 1984 EVENT

A hydrologic and hydraulic analysis was conducted to assess the degree to which upstream urbanization increased runoff during the June 27, 1984 event. This analysis was made as follows:

1. Rainfall measurements were obtained from the PCDOT&FCD at all available surrounding gage sites. The storm pattern or aerial extent of the rainfall over the contributing watershed was estimated from this information. The measured rainfall was then converted from its actual duration (about 2 hours) to a 1 hour storm based upon rainfall distribution data obtained from NOAA.

2. The magnitude of the peak-discharge rate was then computed for the basin area upstream of the Swan Road crossing, and for the urbanized area in the vicinity of the Swan Road and Sunrise Drive intersection. The lower portions of the basin in the vicinity of the Swan Road and Sunrise Drive intersection was computed as a separate tributary, because the degree of urbanization in this area has resulted in significantly different runoff characteristics than for the upstream watershed.

3. Flood hydrographs were developed for each of the contributing areas described in Item 2 above, then lagged to a concentration point at Calle del Pantera. These hydrographs were then summed to determine the peak-flow rate through Flecha Caida.

4. The magnitude of the peak-runoff rate was then computed for a condition that assumed a CR-1 urbanization density in the vicinity of the Swan Road/Sunrise Drive intersection area, rather than the high densities that currently exist. The CR-1 density was chosen as a basis of comparison because this is the general density

of development for much of the surrounding foothills area. A hydrograph for this latter condition was developed, as was done for the case of existing conditions, and lagged to a point of concentration at Calle del Pantera. This hydrograph was then summed with the hydrograph for the upstream area to determine a peak-flow rate for a more moderate urbanization density. The difference in the peak-flow rate from existing conditions (i.e., high-density urbanization) versus the hydrograph for the assumed CR-1 density is the estimate of the increase in peak-flow rate due to the higher-density urbanization.

5. These discharge estimates were checked by taking field cross sections of Valley View Wash between Sunrise Drive and determining the magnitude of the flow rate from high water marks using the standard "Slope-Area Method," as described by the United States Geological Survey. This cross-section data was also used to determine the difference in the depth of flow between existing conditions and the assumed CR-1 density condition.

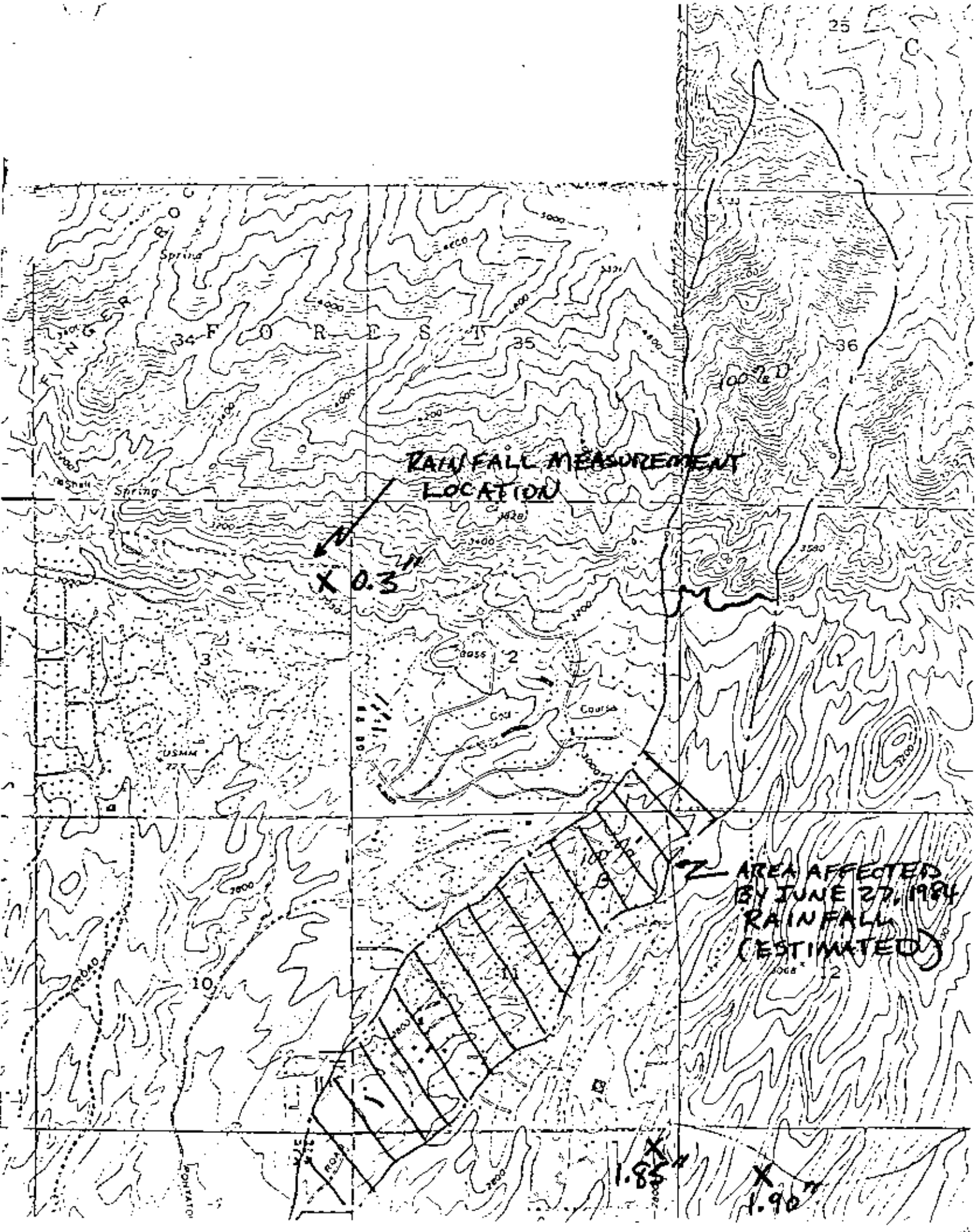
The results of the analysis described above found that the higher-density urbanization increased the peak-flow rate by about 5% over and above what would have occurred with a more moderate urbanization density. This increase in discharge increased the flow depth by about 0.1 to 0.2 feet at the channel cross-section locations surveyed as a part of this study.

All hydrologic and hydraulic computation sheets are provided in Appendix A of this report.

R21/R484-32

APPENDIX A

HYDROLOGIC AND HYDRAULIC COMPUTATIONS


RAINFALL MEASUREMENT LOCATION
X 0.3"
AREA AFFECTED BY JUNE 27, 1984 RAINFALL (ESTIMATED)
X 1.85"
X 1.90"
Spring
Spring
Golf
Course
USGM
34
35
36
25
10
3
2
1

HYDROLOGIC DATA SHEET

Project Name and Location: Valley View Wash, tributary areas in vicinity of Sun Ter.

Drainage Concentration Point: Pantano Dr. (existing urbanized conditions)

Watershed Area (A): 126 acres/square miles.

Length of Watercourse (L_c): 5300 ft. Length to Center of Gravity (L_{ca}): 2450 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
5300	2840 - 2670 = 170

Mean Slope (S_c): 0.032 ft./ft. Watershed Type(s): Urban (Heavy) (future)

Basin Factor (n_b): 0.030 (future) Flood Frequency: June 27, 1977 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 1.66 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 100% B Cover Type(s): Desert Brush, Urban

Cover Density (pervious areas): 30% Impervious Cover: 60% (future)

CN(s): 82 (pervious & impervious areas) CN*(s): 82.69
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.2784 (pervious areas) 0.955 (impervious areas)

Runoff Supply Rate (q): 0.6844 i in./hr. (function of i)

Time of Concentration (T_c): 0.3767 $i^{-.4}$ hrs. ~~mins~~. (function of i)

Iterative Solution of T_c: 13 ~~hrs~~/mins.

Rainfall Intensity (i) at T_c: 4.96 in./hr.

Runoff Supply Rate (q) at T_c: 2.77 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 352 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: Valley View Wash, tributaries in vicinity of Sun & Sunrise

Drainage Concentration Point: Pantera Dr. (assumed CR-1 density)

Watershed Area (A): 126 acres/square miles.

Length of Watercourse (L_c): 5300 ft. Length to Center of Gravity (L_{ca}): 2450 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
5300	2840-2670 = 170

Mean Slope (S_c): 0.032 ft./ft. Watershed Type(s): Suburban (future)

Basin Factor (n_b): 0.032 (future) Flood Frequency: June 27, 1984 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 1.66 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 100% B Cover Type(s): Desert Brush, Urban

Cover Density (pervious areas): 30% Impervious Cover: 20% (future)

CN(s): 82 (pervious & impervious areas) CN*(s): 82.69
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.2784 (pervious areas) 0.955 (impervious areas)

Runoff Supply Rate (q): 0.4137 i in./hr. (function of i)

Time of Concentration (T_c): 0.4914 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 18 hrs./mins.

Rainfall Intensity (i) at T_c: 3.48 in./hr.

Runoff Supply Rate (q) at T_c: 1.44 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 183 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

10

HYDROLOGIC DATA SHEET

Project Name and Location: Valley View Wash

Drainage Concentration Point: at Swan Road, area of watershed south of section 11

Watershed Area (A): 375 acres/square miles.

Length of Watercourse (L_c): 8300 ft. Length to Center of Gravity (L_{ca}): 4000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
8300	3000 - 2750 = 250

Mean Slope (S_c): 0.0301 ft./ft. Watershed Type(s): suburban (future)

Basin Factor (n_b): 0.032 (future) Flood Frequency: June 27, 1984 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 1.66 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 100% B Cover Type(s): Desert Brush

Cover Density (pervious areas): 30% Impervious Cover: 20% (future)

CN(s): 82 (curve number) (pervious & impervious areas) CN*(s): 82.69 (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.2784 (pervious areas) 0.955 (impervious areas)

Runoff Supply Rate (q): 0.4137 i in./hr. (function of i)

Time of Concentration (T_c): 0.6675 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 27 hrs./mins.

Rainfall Intensity (i) at T_c: 2.78 in./hr.

Runoff Supply Rate (q) at T_c: 1.15 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 435 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

$Q_p = 435$
$T_c = 27$
(CR-1)

VVW @ Sun/Sunrise

$T_c = 28$

	ΔT	ΔQ
(lag)	8	0
	11	11
	16	70
	22	151
	28	251
	33	386
	36	435
	39	402
	42	365
	47	295
	64	141
	81	59
	137	5

$Q_p = 352$
$T_c = 13$
(Urban)

VVW @ Calle Del Pantera

$T_c = 19$

ΔT	ΔQ
2	9
6	56
10	122
13	203
17	312
19	352
23	295
29	213
34	149
38	114
49	48
65	17
87	4

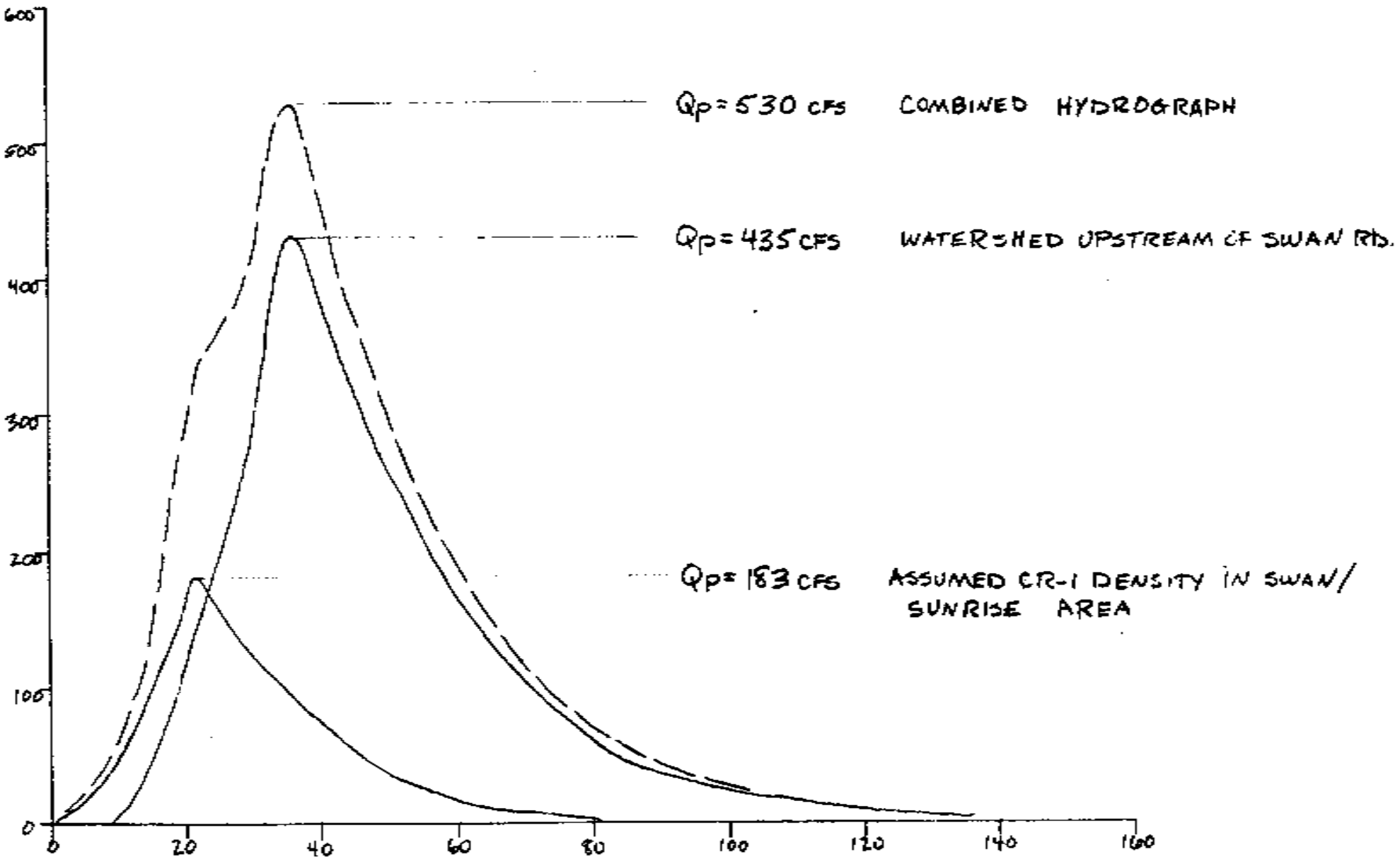
VVW @ Calle Del Pantera (CR-1)

2	5
7	29
11	63
15	105
20	162
22	183
24	169
26	154
33	111
44	59
57	25
101	2

$Q_p = 183$
$T_c = 18$
$T_c = 22$

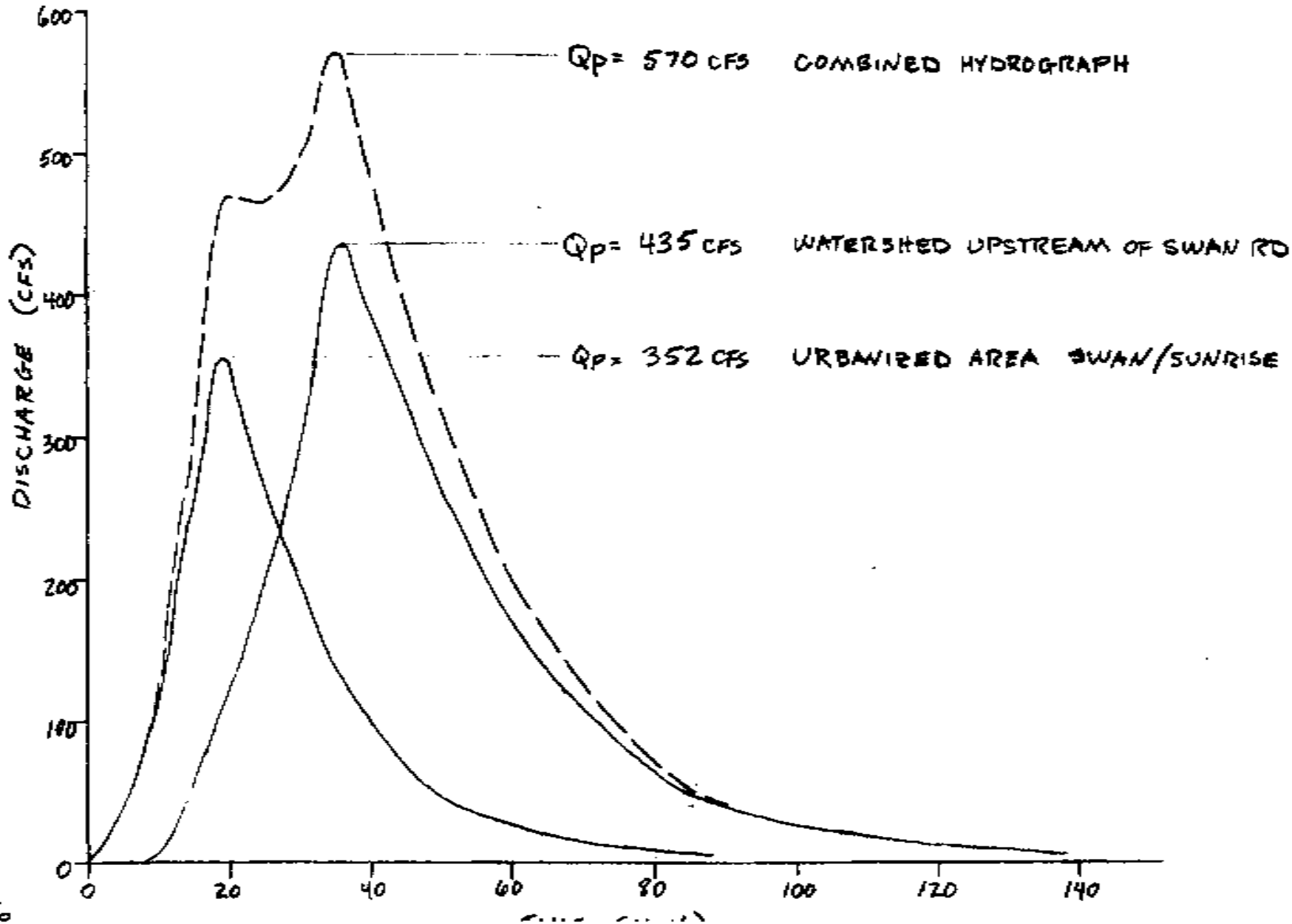
VALLEY VIEW WASH

ESTIMATED HYDROGRAPHS FOR JUNE 27, 1984 FLOODS WITH ASSUMED CR-1 DENSITY IN SWAN/SUNRISE AREA



VALLEY VIEW WASH

ESTIMATED HYDROGRAPHS FOR JUNE 27, 1984 FLOOD



DETERMINATION OF PEAK DISCHARGE BY SLOPE AREA METHOD (BY DALRYMPLE & BENSON)

$$Q = K_2 \sqrt{\frac{\Delta h}{\frac{K_2}{K_1} L + \frac{K_2^2}{2g A_2^2}\left[-\alpha\left(\frac{A_2}{A_1}\right)^2(1-k) + \alpha_2(1-k)\right]}}$$

WHERE
Q = PEAK DISCHARGE (CFS)
K_n = CONVEYANCE (CFS)
A_n = CROSS SECTIONAL AREA OF FLOW (FT^2)
α_n = VELOCITY HEAD COEFFICIENT
L = REACH LENGTH (FT)
Δh = " DROP (FT)
k = 0.5 IF Δh_v IS POSITIVE
= 0 IF " " NEGATIVE

SECTION 1-2:

K_1 = 3781
K_2 = 2247
A_1 = 54 FT^2
A_2 = 40 FT^2
$\alpha_1 = \alpha_2 = 1$
L = 260 FT
Δh = 5.28 FT
k = 0.5

$$Q = (2247)\sqrt{\frac{5.28}{\left(\frac{2247}{3781}\right)(260) + \frac{(2247)^2}{2g(40)^2}\left[-1\left(\frac{40}{54}\right)^2(1-.5)+(1-.5\right.}}$$

$$\boxed{Q = 401 \text{ CFS}}$$

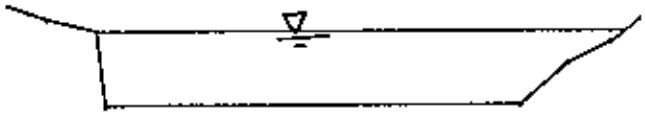
SECTION 3-4 :

$K_1 = 4580$
$K_2 = 3247$
$A_1 = 59\ ft^2$
$A_2 = 48\ ft^2$
$\alpha_1 = \alpha_2 = 1$
$L = 190$
$\Delta h = 4.06$
$K = 0.5$

$$Q = (3247)\sqrt{\frac{4.06}{\left(\frac{3247}{4580}\right)(190) + \frac{(3247)^2}{2g\,(48)^2}\left[-1\left(\frac{48}{59}\right)^2(1-0.5) + (1-0.5)\right]}}$$

$$\boxed{Q = 540\ CFS}$$

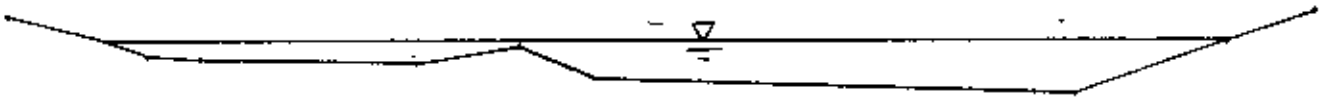
SECTION (4) : (190 FT D/S OF SECTION (3))



n = 0.03
S = 0.0214
D = 1.99 FT

Q = 475 CFS ; V = 9.9 FT/S
K = 3247 ; F = 1.32
A = 48 FT²

SECTION (5) : (109 FT D/S OF SECTION (4))



n = 0.03
S = 0.0137
D = 1.43 FT

Q = 245 ; V = 5.1 FT/S
K = 2093 ; F = 0.99
A = 48 FT²

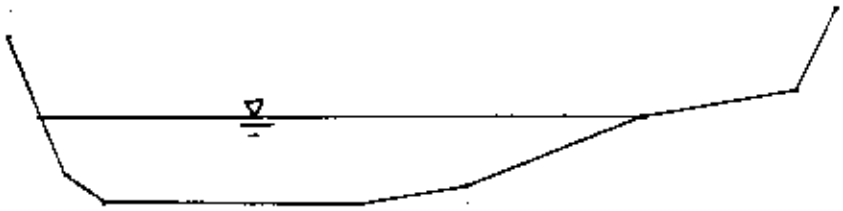
FOR Q = 475 , D = 1.87

Depth	A	P	V	Q
1.43	46.0	58	5.1	245
1.60	58.0	60	5.7	330
1.70	64.0	61	6.0	384
1.80	70.2	63	6.2	435
1.90	76.6	65	6.5	497
2.00	83.2	66	6.8	564

sla SIMONS, LI & ASSOCIATES, INC.

CLIENT: PIMA COUNTY | JOB NO. WR.84-32 | PAGE
PROJECT | DATE CHECKED | DATE
DETAIL | CHECKED BY | COMPUTED BY: J. W.

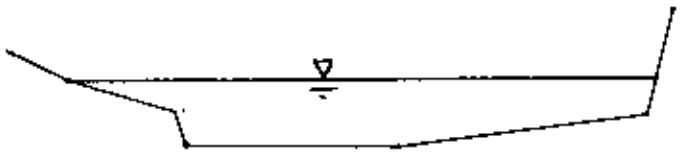
SECTION (1) (120 FT D/S FROM TEMPORARY DIP SECTION AT SUNRISE CROSSING)



n = 0.03
S = 0.0204
D = 2.28 FT

Q = 540 CFS, V = 10 FT/S
K = 3781, F = 1.34
A = 54 FT²

SECTION (2) (260 FT D/S FROM SECTION 1)

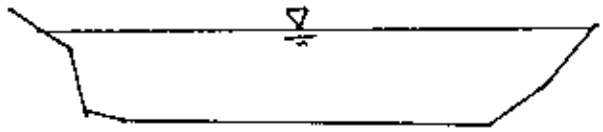


n = .03
S = 0.0204
D = 1.75

Q = 321 CFS; V = 8 FT/S
K = 2247, F = 1.26
A = 40 FT²

FOR Q = 475 CFS, D = 2.12 FT

SECTION (3) (253 FT D/S FROM CUT WATER STORAGE FACILITY ACCESS RD ℄)



n = 0.03
S = .0214
D = 2.51

Q = 670 CFS, V = 11.4 FT/S
K = 4510, F = 1.39
A = 59 FT²

FOR Q = 475 CFS; D = 2.06

Photo No. 1 - Valley View Wash, looking upstream across Sunrise Drive at Box Culvert under construction. Note south half of roadway still in place.

Photo No. 2 - Valley View Wash looking east along Sunrise Drive.

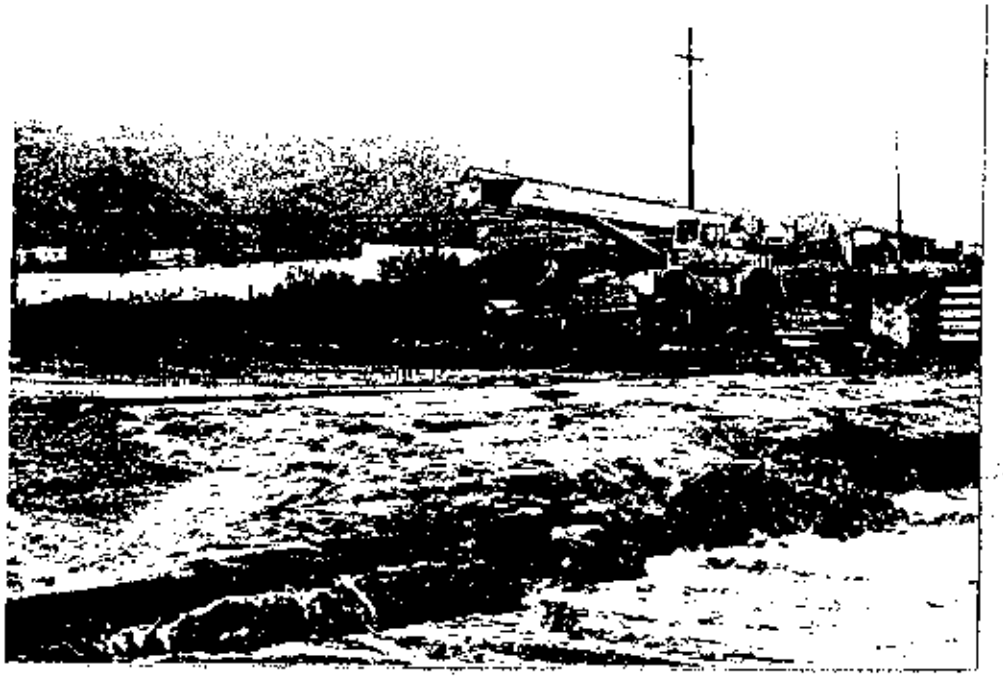


Photo No. 3 - Valley View Wash at Sunrise Drive looking northeast.

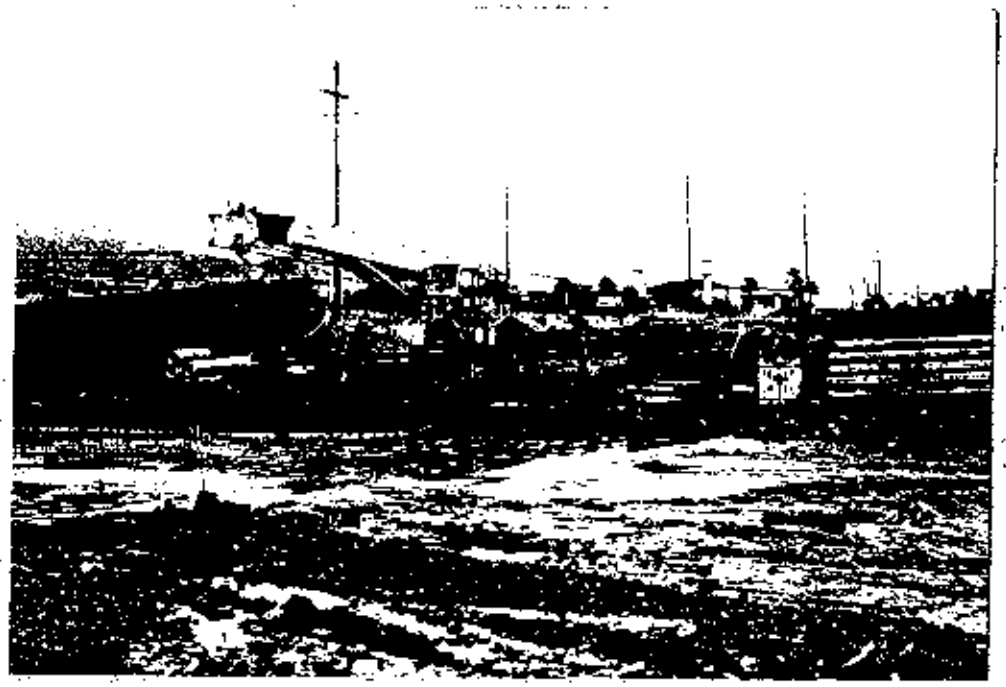


Photo No. 4 - Valley View Wash at Sunrise Drive looking northeast.

Photo No. 5 - Valley View Wash looking downstream from Sunrise Drive.

sla

Simons, Li & Associates, Inc.

120 WEST BROADWAY
SUITE 260
P.O. BOX 2712
TUCSON, ARIZONA 85702

TELEPHONE (602) 884-9594

Transmittal Letter

To Pima County Dept. of Trans. & Flood Control Dist.
1313 S. Mission Road

Attention Ms. Suzanne Shields, P.E.

Date July 26, 1984

Project Misc. Flood Invest. of Flood Problems for Pima Co.

Project Number AZ.WR.84.32

Enclosed please find:

Copies	Of
1	Preliminary Assessment of Flooding Problems Along Valley View Wash in the Vicinity of the Flecha Caida Subdivision

- ☐ Forwarded
- ☐ Returned
- ☐ Prints
- ☐ Originals
- ☐ Reports
- ☐
- ☐ For Approval
- ☐ As Requested
- ☐ For Your Use
- ☐ For Your Files
- ☐
- ☐ Mail
- ☐ Express Courier
- ☐

Remarks Attached is one copy of the above-referenced assessment for your review.

From Clinton M. Glass, P.E.

RECEIVED:

28

FLECHA CAIDA FLOOD IMPROVEMENT STUDY -

PHASE I

100-YEAR PEAK DISCHARGE

MAGNITUDES AND FLOODPLAIN MAPPING

Prepared for:

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

Property of:
Pima County
Flood Control District Library
740-6350

Prepared by:

Simons, Li & Associates, Inc.
120 W. Broadway, Suite 170
P.O. Box 2712
Tucson, Arizona 85702

January 28, 1986

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II.	PEAK DISCHARGE DETERMINATION	2
III.	FLOODPLAIN MAPPING	4

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Appendix B - Hydrologic Data Sheets

Appendix C - Culvert Performance Curves

Appendix D - Diskette Containing Input Data for Subcritical and Supercritical Runs

Appendix E - Index Map to the Floodplain Maps (Separate Document)

I. STUDY AREA

The Flecha Caida study area encompasses approximately eight square miles of the foothill region of the Santa Catalina Mountains. The area is bounded by the Coronado National Forest on the north, River Road on the south, Alvernon Way on the west, and Craycroft Road on the east.

The three major washes studied include Finger Rock Wash, Valley View Wash, and Flecha Caida Wash. One tributary (Sky Club Wash) to the Valley View Wash and three tributaries to the Finger Rock Wash were also included in this study.

II. PEAK DISCHARGE DETERMINATION

One-hundred year peak discharge magnitudes were determined at all concentration points shown on the drainage basin map which is contained in Appendix A. The criteria used to establish the location of each concentration point were 1) drainage area (0.5 square miles or larger), and 2) the location of the associated watercourse with respect to section lines or major roadways.

The soil group identifications are also shown on the drainage basin map. It should be noted that the upper limits of the drainage basins associated with the Finger Rock Wash, Valley View Wash, and Sky Club Wash are located within the Coronado National Forest. The soil group classifications for this area are 40 percent Soil Group C and 60 percent Soil Group D. The vegetation type is mountain brush at a cover density of 30 percent. The associated Curve Numbers are 82 and 90, respectively. The soil group classification for the lower limits of the drainage basins located within the study area are 100 percent Soil Group B. The vegetation type is desert brush at a cover density of 20 percent. The associated Curve Number is 83.

The impervious cover percentages used in the hydrologic calculations were based on projected densities as shown on the Catalina Foothillls Area Plan. The selected values are as follows:

Type of Development	Percent Impervious Cover
Natural	0
0.25 RAC	5
1 RAC	10
2 RAC	30
5 RAC	50
8 RAC	65
Commercial	65
Transitional	65

If the drainage basin contained a mixture of the above densities, a weighted value was computed. The weighted breakdown, where applicable, is shown on the back of the hydrologic data sheets which are contained in Appendix B.

In general, all basin factors were determined from aerial photographs and

field investigations of the areas which are currently developed. A basin factor for undeveloped areas was selected using the County's hydrology manual as a guide. If a particular watercourse traversed areas with different basin factors, a weighted value was computed. The weight breakdown where applicable is also shown on the back of each hydrologic data sheet.

Five one-hour precipitation depths (P_1-HR) were determined using the maps and procedure as presented in the County's hydrology manual. Those drainage basins extending into the Coronado National Forest are subject to a P_1-HR value approximately equal to 2.75 inches. Those drainage basins located in the southern-most region of the study area are subject to a P_1-HR value approximately equal to 2.60 inches. The P_1-HR value used in each hydrologic calculation was selected based on the location of the centroid of the respective drainage basin. The rainfall data sheet and depth versus return period curve used to determine the respective P_1-HR depths are also contained in Appendix B.

It should be noted that the 100-year peak discharge magnitudes for the Finger Rock Wash decrease in the downstream direction. This occurs primarily because the drainage area does not increase in proportion to the length of the watercourse. Therefore, the decrease in the effective rainfall intensity as computed using the County method has a more significant impact on the resultant peak discharge than does the increase in drainage area. Transmission losses is a natural process that can justify a slight reduction in the peak discharge, especially given the wide, sandy nature of the Finger Rock channel within the downstream reaches. However, the majority of the reduction is computational in nature.

III. FLOODPLAIN MAPPING

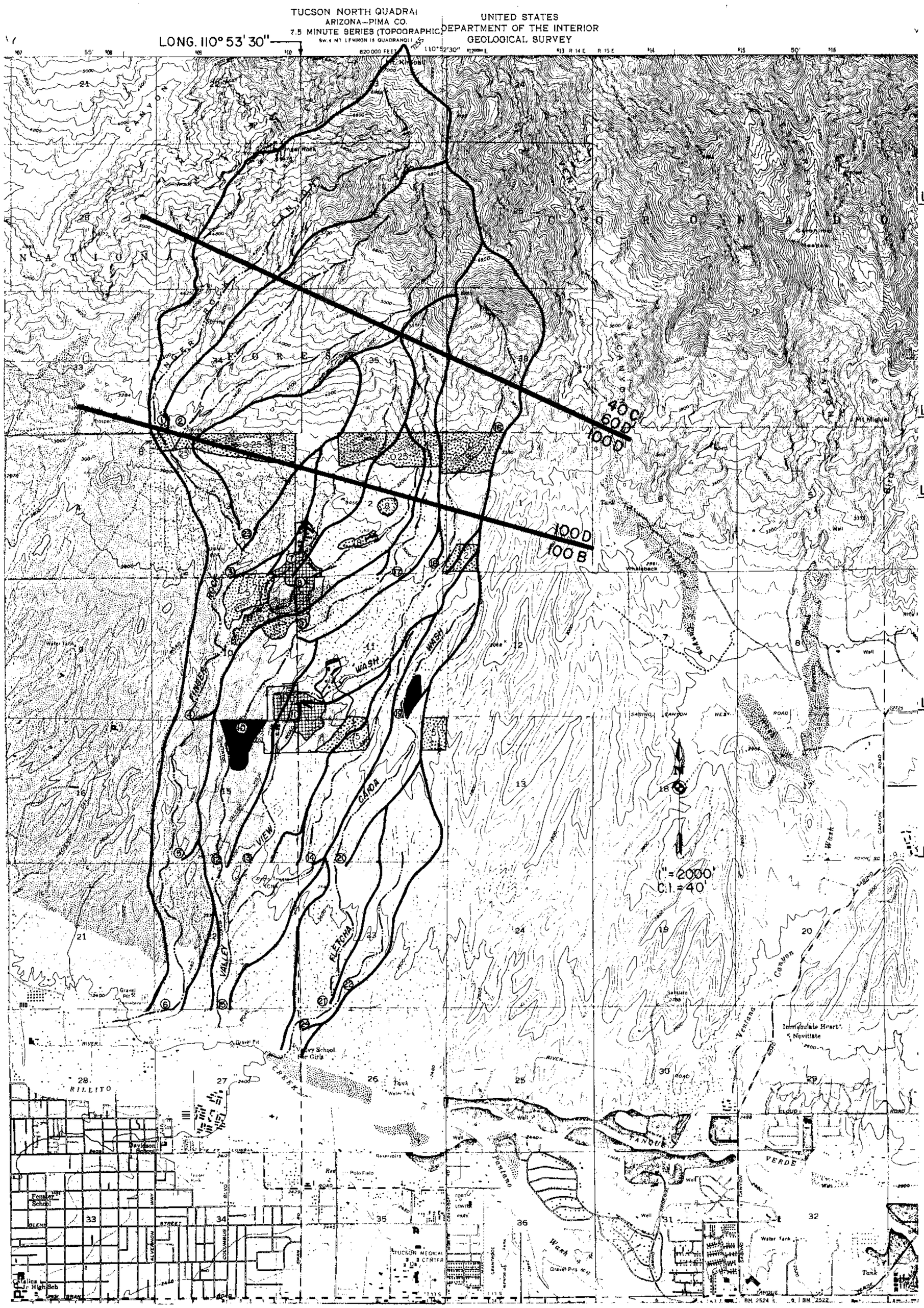
The 100-year floodplains associated with all reaches of the respective washes and their tributaries were modeled using the U.S. Army Corps of Engineers HEC-2 computer program. Approximately 77,500 feet or 14.7 miles of floodplain mapping was performed. Both subcritical and supercritical runs were generated. In general, the flow characteristic for all washes was supercritical. Therefore, the subcritical runs generated critical water-surface elevations. The 100-year floodplain maps (submitted under a separate document) prepared as a result of this study are based on the critical water-surface elevations. The flow velocities associated with the supercritical runs are provided to assist in determining the erosion potential along various reaches of the wash. They can also be used to establish design parameters for bank protection if the need arises.

At various locations along the respective washes and their tributaries, culverts have been constructed to provide a dry crossing. The design capacity of these structures vary. Some are capable of accommodating the entire 100-year peak discharge while others are designed only to accommodate runoff from more frequent storm events. The two main culvert types are concerte box culverts (CBC) and corrugated metal pipes (CMP, both round and arch shapes). Since washes within the study area are known to carry a significant sediment load accompanied by debris, a clogging factor was considered when the capacity of the existing structures were determined. Culvert performance curves (Appendix C) for the CMP's were prepared which incorporated a 50 percent clogging factor. The resulting headwater elevation at each crossing was then incorporated in the HEC-2 analysis as a known water-surface elevation. The HEC-2's special bridge routine was used to compute water-surface elevations associated with the CBC's. The cross-sectional area associated with each CBC was reduced to account for a 25 percent clogging factor.

The input data used in our HEC-2 analysis (both subcritical and supercritical) were copied onto a floppy disk (Appendix D) The floodplain maps (24 sheets) are submitted under separate document. However, a copy of the index map to the floodplain maps is contained in Appendix E. This map shows the reaches of each wash studied and identifies cross-section locations relative to each map sheet.

APPENDIX A

DRAINAGE BASIN MAP



NOTE:

VEGETATION BOUNDARY BETWEEN MOUNTAIN BRUSH AND DESERT BRUSH LOCATION ALONG CONTOUR 3200 FT.

LEGEND

- WATERSHED BOUNDARY
- 100D / 100B — HYDROLOGICAL SOIL GROUP BOUNDARY
- ① — DRAINAGE CONCENTRATION POINT
- PRIMARY & SECONDARY WATERCOURSES
- RESIDENCES/ACRE
- SCHOOL (ESTABLISHED OR PROPOSED)
- COMMERCIAL
- TRANSITIONAL

APPENDIX A

DRAINAGE BASIN MAP SHOWING HYDROLOGIC CHARACTERISTICS OF STUDY AREA

APPENDIX 8

HYDROLOGIC DATA SHEETS

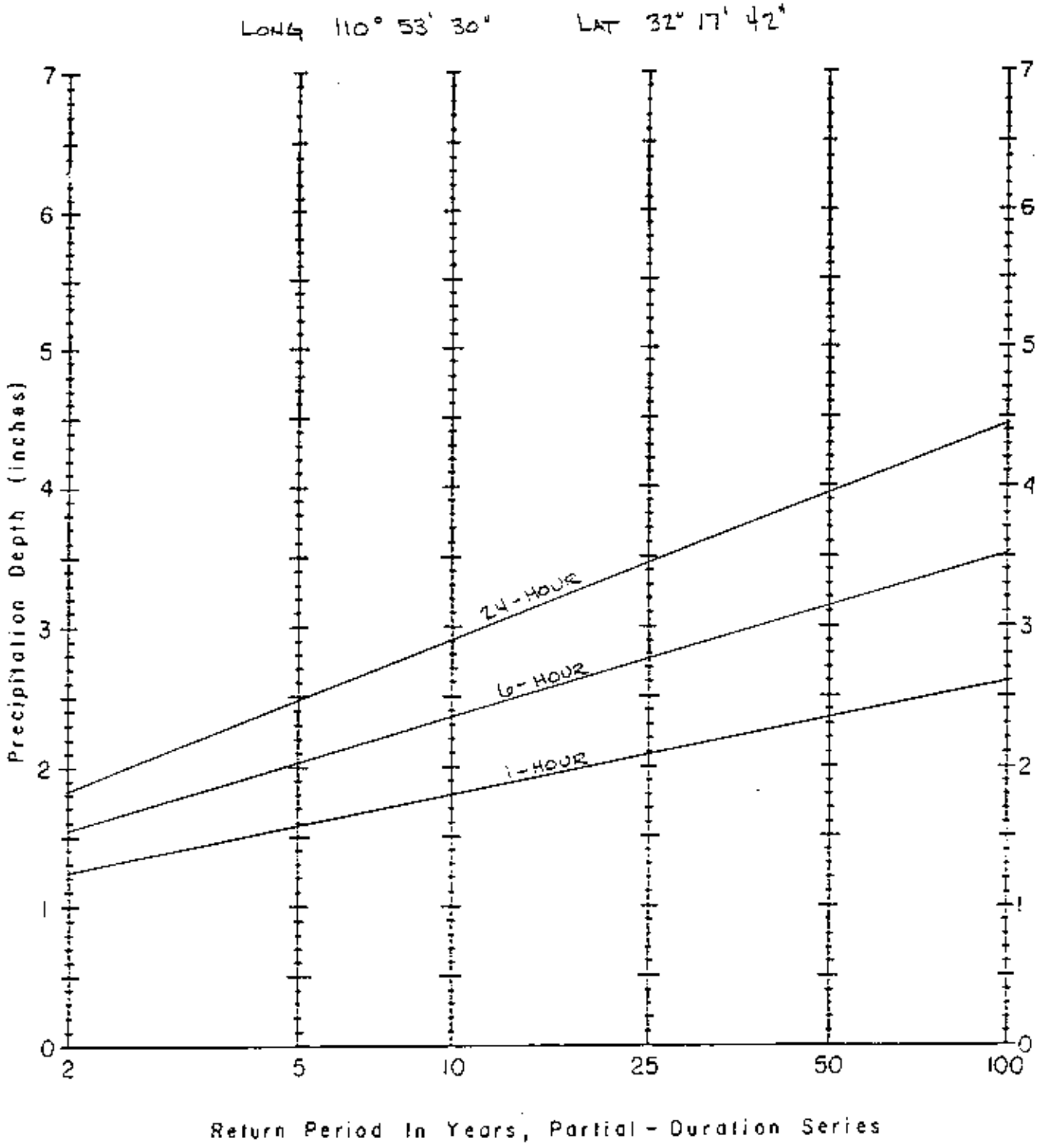
RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 32° 17' 42"

Return Period (Years)	Precipitation Values (inches)*			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.53	X_1 1.55	1.82	X_2 1.82
5	2.00	2.03	2.45	2.47
10	2.38	2.36	2.85	2.89
25	2.82	2.77	3.55	3.45
50	3.20	3.15	3.95	3.95
100	3.45	X_3 3.52	4.40	X_4 4.44

$$Y_2 = -0.011 + 0.942\,(X_1^2/X_2) = 1.23''$$

$$Y_{100} = 0.494 + 0.755\,(X_3^2/X_4) = 2.60''$$



Precipitation Depth Versus Return Period for Partial-Duration Series

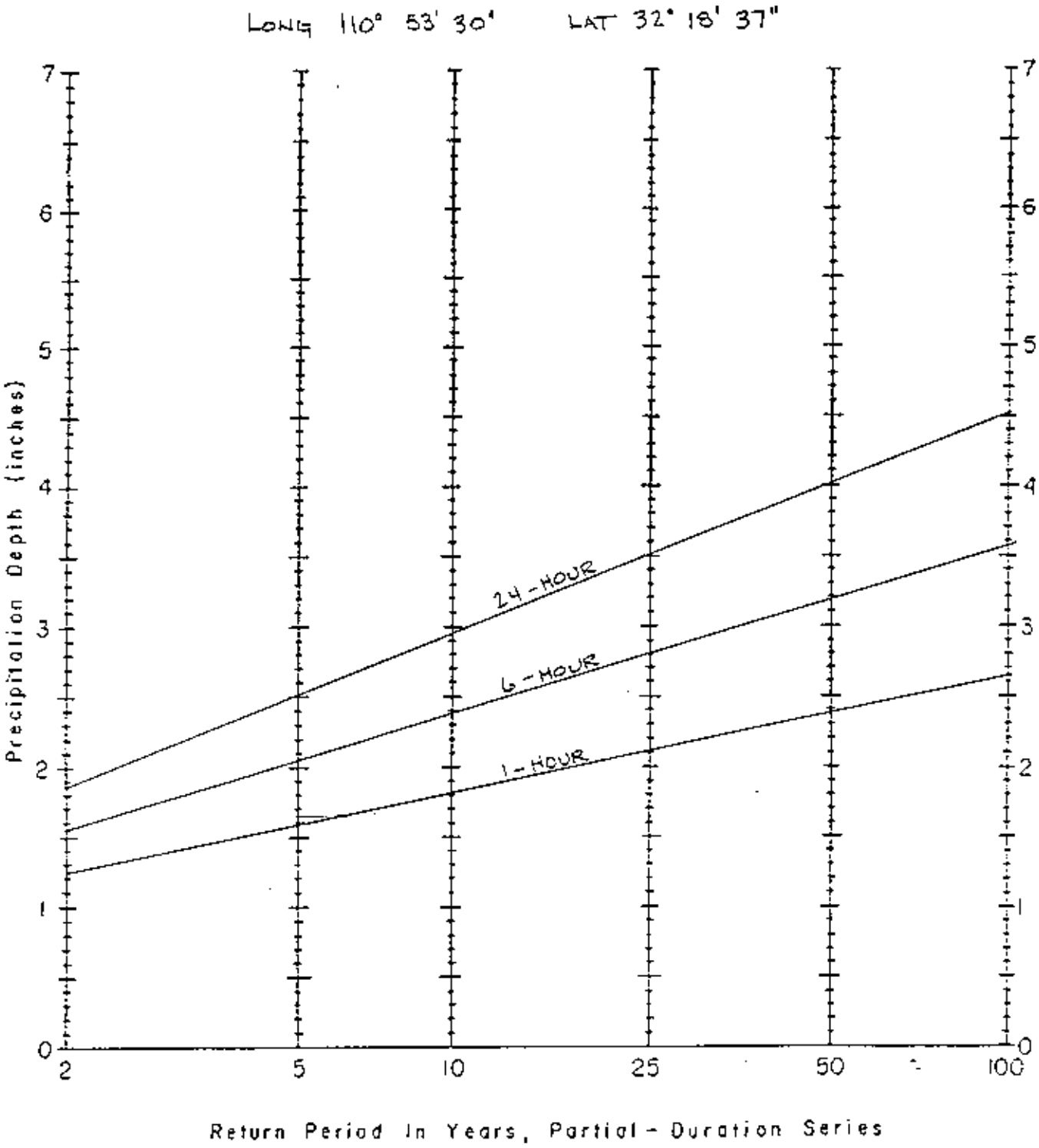
RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 32° 18' 37"

Return Period (Years)	Precipitation Values (inches)*			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.55	X_1 1.57	1.88	X_2 1.86
5	2.04	2.07	2.50	2.52
10	2.42	2.40	2.90	2.95
25	2.88	2.82	3.60	3.51
50	3.25	3.20	4.00	4.02
100	3.50	X_3 3.58	4.50	X_4 4.51

$$Y_2 = -0.011 + 0.942(X_1^2/X_2) = 1.24''$$

$$Y_{100} = 0.494 + 0.755(X_3^2/X_4) = 2.64''$$



Precipitation Depth Versus Return Period for Partial-Duration Series

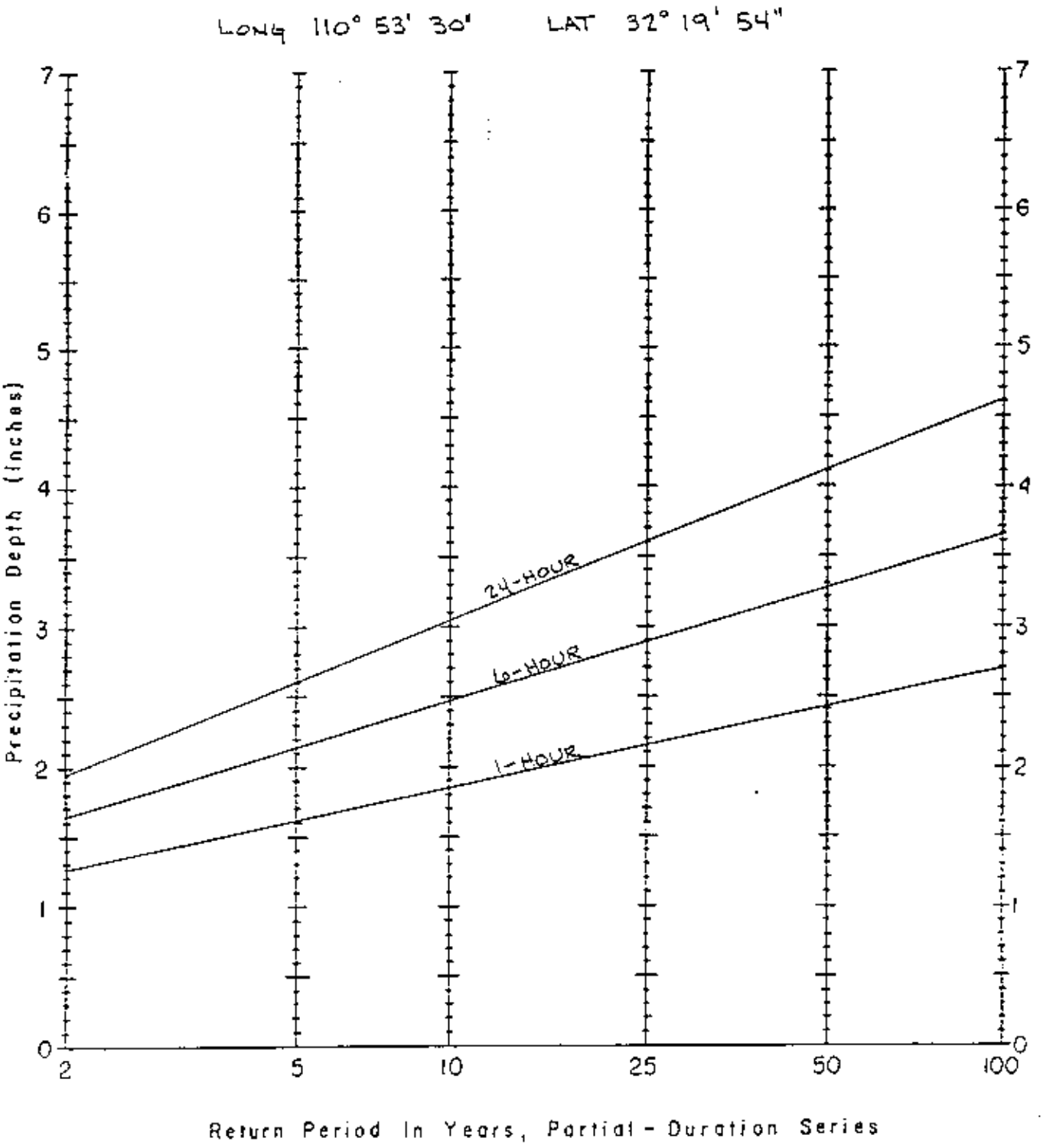
RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 32° 19' 54"

Return Period (Years)	Precipitation Values (inches) *			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.60	X_1 1.63	1.95	X_2 1.94
5	2.10	2.13	2.58	2.60
10	2.50	2.46	3.00	3.03
25	2.95	2.90	3.65	3.60
50	3.32	3.29	4.10	4.11
100	3.60	X_3 3.67	4.60	X_4 4.61

$$Y_2 = -0.011 + 0.942\,(X_1^2/X_2) = 1.27''$$

$$Y_{100} = 0.494 + 0.755\,(X_3^2/X_4) = 2.70''$$



Precipitation Depth Versus Return Period for Partial-Duration Series

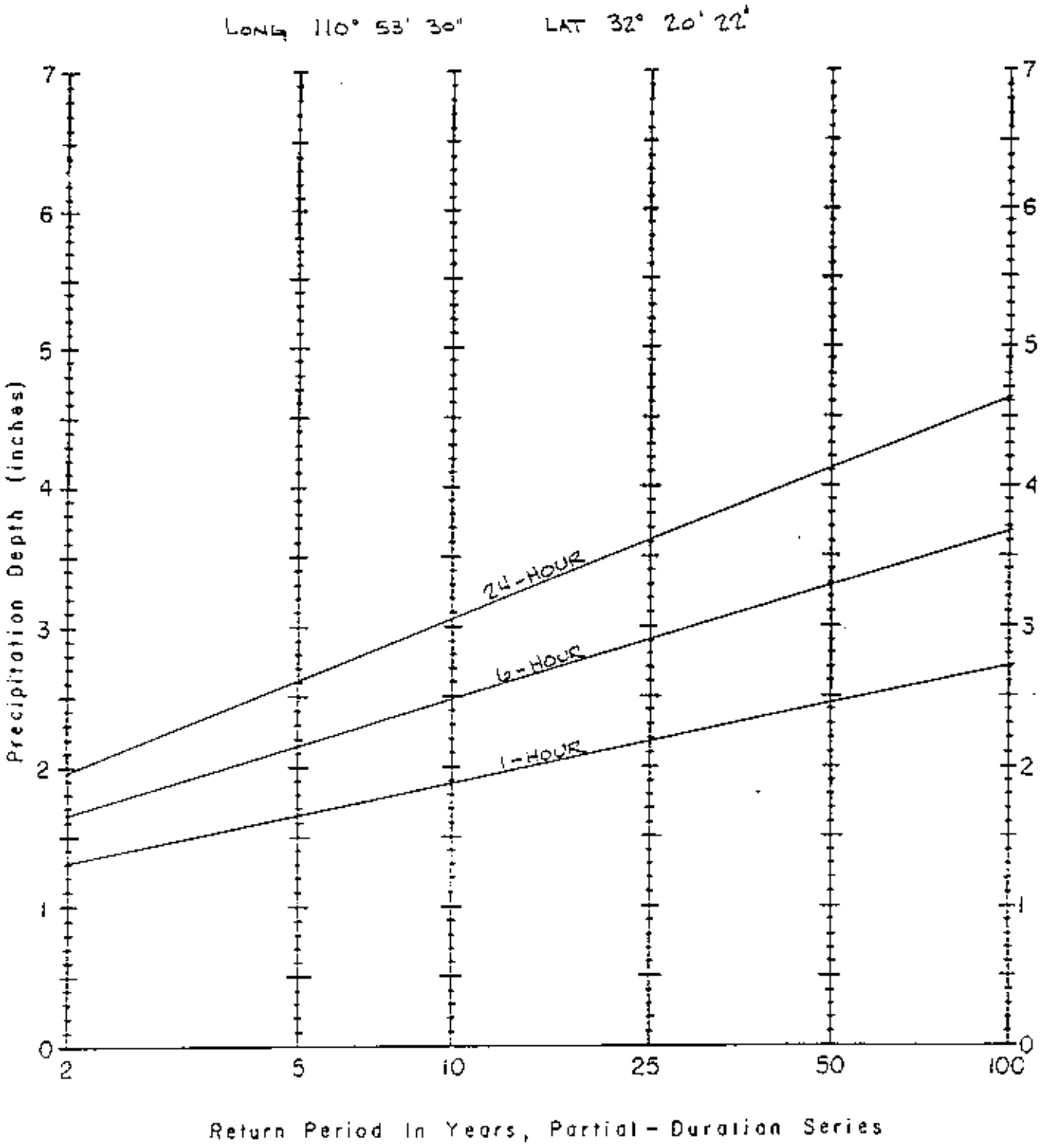
RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 37° 20' 22"

Return Period (Years)	Precipitation Values (inches)*			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.62	X_1 1.65	1.98	X_2 1.96
5	2.12	2.15	2.60	2.61
10	2.52	2.48	3.00	3.05
25	2.98	2.92	3.65	3.61
50	3.35	3.30	4.12	4.12
100	3.60	X_3 3.68	4.62	X_4 4.62

$$Y_2 = -0.011 + 0.942\,(X_1^2/X_2) = 1.30''$$

$$Y_{100} = 0.494 + 0.755\,(X_3^2/X_4) = 2.71''$$



Precipitation Depth Versus Return Period for Partial-Duration Series

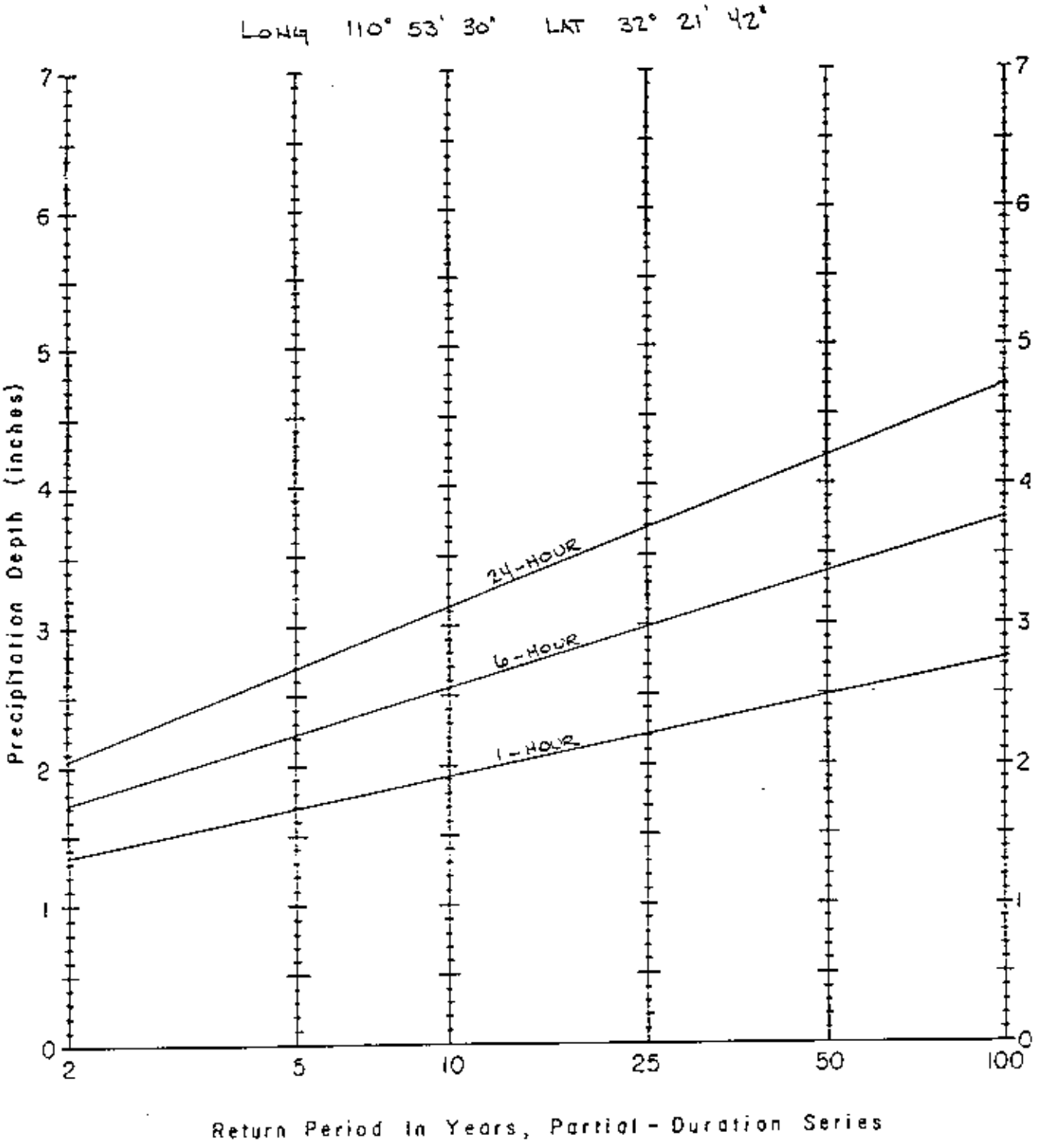
RAINFALL DATA SHEET

LONG 110° 53' 30" LAT 32° 21' 42"

Return Period (Years)	Precipitation Values (inches) *			
	6 Hour Duration		24 Hour Duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.70	X_1 1.71	2.08	X_2 2.04
5	2.18	2.22	2.65	2.70
10	2.58	2.55	3.08	3.13
25	3.02	2.98	3.72	3.70
50	3.40	3.37	4.30	4.21
100	3.70	X_3 3.75	4.65	X_4 4.71

$$Y_2 = -0.011 + 0.942\,(X_1^2 / X_2) = 1.34''$$

$$Y_{100} = 0.494 + 0.755\,(X_3^2 / X_4) = 2.75''$$



Precipitation Depth Versus Return Period for Partial-Duration Series

CHART FOR ADJUSTED CURVE NUMBERS (CN*'s)

CN		K_1		K_2
95		98.67		80.08
94		98.33		78.76
93		98.00		77.44
92		97.50		76.12
91		97.00		75.24
90	SOIL GROUP D @ 30%	96.50	MOUNTAIN BRUSH	73.92
89		96.00		72.60
88		95.50		71.72
87		95.00		70.40
86		94.50		69.52
85		94.00		68.20
84		93.50		66.88
83	SOIL GROUP B @ 20%	93.00	DESERT BRUSH	66.00
82	SOIL GROUP C @ 30%	92.50	MOUNTAIN BRUSH	65.12
81		92.00		63.80
80		91.50		62.92
79		91.00		62.04
78		90.00		60.72
77		89.50		59.84
76		89.00		58.96
75		88.50		58.08
74		88.00		56.76
73		87.00		55.88
72		86.50		55.00
71		86.00		54.12
70		85.00		53.24
69		84.50		52.36
68		84.00		51.04
67		83.00		50.16
66		82.50		49.28
65		82.00		48.40
64		81.00		47.52
63		80.00		46.64
62		79.00		45.76
61		78.50		44.88
60		78.00		44.00

$$CN^* = \frac{K_1(P_1 - .88) + K_2}{P_1}$$

Where,

P_1 = 1 hour rainfall depth.

(Use areal depth, if applicable).

Notes: (1) P_1 must always be greater than .88 inches.

(2) For impervious areas, CN* = 99 (constant).

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HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E½ R14E N2/3T13S)

Drainage Concentration Point: #1 (FINGER ROCK WASH @ FOREST SERVICE BOUNDARY)

Watershed Area (A): 1033 acres/~~square miles~~.

Length of Watercourse (L_c): 19400 ft. Length to Center of Gravity (L_{ca}): 13000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480

Mean Slope (S_c): 0.1592 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) (EXIST ~~Future~~)

Basin Factor (n_b): 0.060 (EXIST ~~Future~~) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.75 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 30% C 70% D Cover Type(s): ~~DESERT BRUSH~~ MOUNTAIN BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 0 % (EXIST ~~Future~~)

CN(s): 82 90 (pervious & impervious areas) CN*(s): 86.58, 92.50
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5426, 0.7165 (pervious areas) 0.9565 (impervious areas

Runoff Supply Rate (q): 0.6643 i in./hr. (function of i)

Time of Concentration (T_c): 55 $i^{-.4}$ ~~hrs~~ /mins. (function of i)

Iterative Solution of T_c: 30 ~~hrs~~ /mins.

Rainfall Intensity (i) at T_c: 4.4000 in./hr.

Runoff Supply Rate (q) at T_c: 2.9231 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 3044 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY

Drainage Concentration Point: #3A (@ CONFLUENCE WITH #27)

Watershed Area (A): 2896 acres/square miles.

Length of Watercourse (L_c): 28900 ft. Length to Center of Gravity (L_{ca}): 14900 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7235 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
6800	3080 - 2760 = 320
2700	2760 - 2690 = 70

Mean Slope (S_c): 0.0977 ft./ft. Watershed Type(s): MOUNTAIN UNDEVELOPED FOOTHILLS DEVELOPED (future)

Basin Factor (n_b): 0.053 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.72 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 17%B 18%C 65%D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30%(MB) 20%(DB) Impervious Cover: 3 % (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.15, 86.51, 92.46
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5557, 0.5375, 0.7126 (pervious areas) 0.9568 (impervious areas)

Runoff Supply Rate (q): 0.6634 i in./hr. (function of i)

Time of Concentration (T_c): 74 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 46 hrs./mins.

Rainfall Intensity (i) at T_c: 3.2970 in./hr.

Runoff Supply Rate (q) at T_c: 2.1872 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 6385 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E½ R14E N½3 T13S)

Drainage Concentration Point: #3 (@ SKYLINE DRIVE, SEC LINE 3 & 10)

Watershed Area (A): 2806 acres/~~square miles~~.

Length of Watercourse (L_c): 26200 ft. Length to Center of Gravity (L_{ca}): 14000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
6800	3080 - 2760 = 320

Mean Slope (S_c): 0.1190 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.055 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour):	in.	Areal Value:	in.
P_6 (6 hour):	in.	Areal Value:	in.
P_1 (1 hour):	2.73 in.	Areal Value:	in.
P_2 (2 hour):	in.	Areal Value:	in.
P_3 (3 hour):	in.	Areal Value:	in.

Soil Group(s): 14% B 19% C 67% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 2% (future)

CN(s): 83 82 90 (pervious & impervious areas) CN*(s): 87.20, 86.54, 92.47
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5569, 0.5392, 0.7139 (pervious areas) 0.9552 (impervious areas)

Runoff Supply Rate (q): 0.6647 i in./hr. (function of i)

Time of Concentration (T_c): 68 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 41 ~~hrs~~/mins.

Rainfall Intensity (i) at T_c: 3.5803 in./hr.

Runoff Supply Rate (q) at T_c: 2.3798 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 6731 cfs.

645.33qA (square miles): _____ cfs.

Note: For Impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E1/2 R14E N2/3 T13S)

Drainage Concentration Point: #4 (FINGER ROCK WASH @ SUNRISE DRIVE SEC LINE 10 & 15)

Watershed Area (A): 3716 acres/~~square miles~~

Length of Watercourse (L_c): 32200 ft. Length to Center of Gravity (L_{ca}): 19000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
6800	3080 - 2760 = 320
6000	2760 - 2630 = 130

Mean Slope (S_c): 0.0761 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (a_b): 0.050 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour):	in.	Areal Value:	in.
P_6 (6 hour):	in.	Areal Value:	in.
P_1 (1 hour): 2.72	in.	Areal Value:	in.
P_2 (2 hour):	in.	Areal Value:	in.
P_3 (3 hour):	in.	Areal Value:	in.

Soil Group(s): 26% B 15% C 59% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 6% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.18, 86.51, 92.46
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5552, 0.5375, 0.7126 (pervious areas) 0.9568 (impervious areas)

Runoff Supply Rate (q): 0.6641 i in./hr. (function of i)

Time of Concentration (T_c): 86 $i^{-.4}$ ~~hrs.~~/mins. (function of i)

Iterative Solution of T_c: 57 ~~hrs.~~/mins.

Rainfall Intensity (i) at T_c: 2.8260 in./hr.

Runoff Supply Rate (q) at T_c: 1.8767 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 7030 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLEGAL CANDA FLOOD IMPROVEMENT STUDY

Drainage Concentration Point: #4 (FINGER ROCK WASH @ SUNRISE DRIVE ROUTED

Watershed Area (A): 3713 acres/square miles.

Length of Watercourse (L_c): 32000 ft. Length to Center of Gravity (L_{cg}): 19000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
6800	3080 - 2760 = 320
2700	2760 - 2690 = 70
3100	2690 - 2630 = 60

Mean Slope (S_c): 0.0774 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.049 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.72 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 26% B 15% C 59% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 7% (future)

CN(s): 83, 92, 90 (pervious & impervious areas) CN*(s): 87.18, 86.51, 92.46
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5557, 0.5315, 0.7126 (pervious areas) 0.9568 (impervious areas)

Runoff Supply Rate (q): 0.6672 i in./hr. (function of i)

Time of Concentration (T_c): 83 $i^{-.4}$ hrs./mins. (function of i)

Iteractive Solution of T_c: 54 hrs./mins.

Rainfall Intensity (i) at T_c: 2.9405 in./hr.

Runoff Supply Rate (q) at T_c: 1.9619 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{cg})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 7343 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: ELEHA CAIDA FLOOD IMPROVEMENT STUDY (E½ R14E N½T13S)

Drainage Concentration Point: #5 (FINGER ROCK WASH @ SEC LINE 15 & 22)

Watershed Area (A): 3913 acres/~~square miles~~.

Length of Watercourse (L_c): 37800 ft. Length to Center of Gravity (L_{cg}): 19000 ft

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
6500	3080 - 2760 = 320
6000	2760 - 2630 = 130
5600	2630 - 2530 = 100

Mean Slope (S_c): 0.0562 ft./ft. Watershed Type(s): MOUNTAINS (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.047 (future) Flood Frequency: 100 yrs

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.71 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 30% B 14% C 56% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 7% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.16, 86.49, 92.44
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5535, 0.5336, 0.7113 (pervious areas) 0.9566 (impervious area)

Runoff Supply Rate (q): 0.66616 i in./hr. (function of i)

Time of Concentration (T_c): 95 $i^{-.4}$ ~~hrs~~/mins. (function of i)

Iterative Solution of T_c: 66 ~~hrs~~/mins.

Rainfall Intensity (i) at T_c: 2.5209 in./hr.

Runoff Supply Rate (q) at T_c: 1.66678 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{cg})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 6578 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLOOD IMPROVEMENT STUDY (E 1/2 R14E N 2/3 T13S)

Drainage Concentration Point: #6 (FINGER ROCK WASH @ ALVERNON WAY SEC CORNER 21, 22, 27, 28)

Watershed Area (A): 4124 acres/~~square miles~~.

Length of Watercourse (L_c): 44200 ft. Length to Center of Gravity (L_{ca}): 23000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
4500	3080 - 2760 = 320
6000	2760 - 2630 = 130
12000	2630 - 2420 = 210

Mean Slope (S_c): 0.0454 ft./ft. Watershed Type(s): ~~FOOTHILLS (DEVELOPED~~ MOUNTAINS (UNDEVELOPED) (future)

Basin Factor (n_b): 0.045 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.71 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 34% B 13% C 53% D Cover Type(s): DESERT BRUSH

Cover Density (pervious areas): 30% (MB) 20% (OB Impervious Cover: 6% (future)

CN(s): 83, 87, 90 (pervious & impervious areas) CN*(s): 87.16, 86.49, 92.44
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5535, 0.5358, 0.7113 (pervious areas) 0.9566 (impervious areas)

Runoff Supply Rate (q): 0.6541 i in./hr. (function of i)

Time of Concentration (T_c): 111 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iteracive Solution of T_c: 82 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 2.1255 in./hr.

Runoff Supply Rate (q) at T_c: 1.3903 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 5779 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E½ R14E N½ T13S)

Drainage Concentration Point: # 7 (@ SWAN Rd SEC LINE 2 & 3)

Watershed Area (A): 312 (340 ④) acres/~~square miles~~.

Length of Watercourse (L_c): 10200 ft. Length to Center of Gravity (L_{ca}): 6000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	5280 - 4000 = 1280
2200	4000 - 3400 = 600
1800	3400 - 3020 = 320
4200	3020 - 2850 = 230

Mean Slope (S_c): 0.1243 ft./ft. Watershed Type(s): ~~FOOTHILLS (DEVELOPED)~~ MOUNTAIN (UNDEVELOPED) (future)

Basin Factor (n_b): 0.054 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.71 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 25% B 75% D Cover Type(s): ~~DESERT~~ MOUNTAIN BRUSH BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 8% (future)

CN(s): 83 90 (pervious & impervious areas) CN*(s): 87.16 92.44
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5535 0.7113 (pervious areas) 0.9566 (impervious areas)

Runoff Supply Rate (q): 0.6946 i in./hr. (function of i)

Time of Concentration (T_c): 37 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 19 hrs./mins.

Rainfall Intensity (i) at T_c: 5.5590 in./hr.

Runoff Supply Rate (q) at T_c: 3.8613 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1214 (1323 ④) cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

④ @ SKYLINE DRIVE (SEC LINE 3 & 10)

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CANDA FLOOD IMPROVEMENT STUDY

Drainage Concentration Point: #7A (UPSTREAM OF CONFLUENCE WITH 8A)

Watershed Area (A): 348 acres/square miles.

Length of Watercourse (L_c): 12600 ft. Length to Center of Gravity (L_{ca}): 7200 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	5280 - 4000 = 1280
2200	4000 - 3400 = 600
800	3400 - 3080 = 320
1200	3080 - 2850 = 230
2400	2850 - 2765 = 85

Mean Slope (S_c): 0.0914 ft./ft. Watershed Type(s): MOUNTAIN UNDEVELOPED ~~FOOTHILLS DEVELOPED~~ (future)

Basin Factor (a_b): 0.050 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.71 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 33% B 67% D Cover Type(s): MOUNTAIN BRUSH ~~DESERT BRUSH MIX~~

Cover Density (pervious areas): 30% (MB) & 20% Impervious Cover: 11% (future)

CN(s): 83 90 (pervious & impervious areas) CN*(s): 87.16 92.44
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5535 0.7113 (pervious areas) 0.9566 (impervious areas)

Runoff Supply Rate (q): 0.6919 i in./hr. (function of i)

Time of Concentration (T_c): 44 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 23 hrs./mins.

Rainfall Intensity (i) at T_c: 5.0419 in./hr.

Runoff Supply Rate (q) at T_c: 3.4885 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1224 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLEYA CAIDA FLOOD IMPROVEMENT STUDY (E½ RHE N 2/3 T13S)

Drainage Concentration Point: #8 (@ SWAN ROAD SEC LINE 10 & 11).

Watershed Area (A): 285 acres/~~square miles~~

Length of Watercourse (L_c): 8600 ft. Length to Center of Gravity (L_{ca}): 4400 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	4000 - 3400 = 600
800	3400 - 3200 = 200
2500	3200 - 3000 = 200
4200	3000 - 2820 = 180

Mean Slope (S_c): 0.0713 ft./ft. Watershed Type(s): FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.043 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.69 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 75% B 25% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 22% (future)

CN(s): 83 90 (pervious & impervious areas) CN*(s): 87.11, 92.41
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5501, 0.7086 (pervious areas) 0.9563 (impervious areas)

Runoff Supply Rate (q): 0.6703 i in./hr. (function of i)

Time of Concentration (T_c): 33 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 16 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 5.9778 in./hr.

Runoff Supply Rate (q) at T_c: 4.0077 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1151 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CANOA FLOOD IMPROVEMENT STUDY

Drainage Concentration Point: # 8A (UPSTREAM OF CONFLUENCE WITH 7A)

Watershed Area (A): 315 acres/square miles.

Length of Watercourse (L_c): 10200 ft. Length to Center of Gravity (L_{ca}): 5200 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	4000 - 3400 = 600
800	3400 - 3200 = 200
2500	3200 - 3000 = 200
4200	3000 - 2820 = 180
1600	2820 - 2765 = 55

Mean Slope (S_c): 0.0624 ft./ft. Watershed Type(s): MOUNTAIN UNDEVELOPED FOOTHILLS DEVELOPED (future)

Basin Factor (n_b): 0.042 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.69 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 77% B 23% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 26% (future)

CN(s): 83 90 (pervious & impervious areas) CN*(s): 87.11 92.41
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5501, 0.7086 (pervious areas) 0.9563 (impervious areas)

Runoff Supply Rate (q): 0.6827 i in./hr. (function of i)

Time of Concentration (T_c): 37 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 19 hrs./mins.

Rainfall Intensity (i) at T_c: 5.5179 in./hr.

Runoff Supply Rate (q) at T_c: 3.7669 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 1196 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLE/43 GAIDE FLOOD IMPROVEMENT STUDY (E½ R14E N²/3T13S)

Drainage Concentration Point: #9 (@ SWAN ROAD SEC LINE 10 & 11)

Watershed Area (A): 367 acres/~~square miles~~.

Length of Watercourse (L_c): 15200 ft. Length to Center of Gravity (L_{cg}): 7600 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
5900	5780 - 3400 = 2380
1300	3400 - 3200 = 200
4000	3200 - 2950 = 250
4000	2950 - 2820 = 130

Mean Slope (S_c): 0.0896 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (a_b): 0.048 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour):	in.	Areal Value:	in.
P_6 (6 hour):	in.	Areal Value:	in.
P_1 (1 hour):	2.70 in.	Areal Value:	in.
P_2 (2 hour):	in.	Areal Value:	in.
P_3 (3 hour):	in.	Areal Value:	in.

Soil Group(s): 48% B 6% C 46% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 7% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.13, 86.47, 92.46
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5518, 0.5340, 0.7099 (pervious areas) 0.9565 (impervious areas)

Runoff Supply Rate (q): 0.6468 i in./hr. (function of i)

Time of Concentration (T_c): 47 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 25 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 4.800 in./hr.

Runoff Supply Rate (q) at T_c: 3.1045 in./hr.

Equation for T_c:

$$T_c = \frac{nh}{50} \frac{(L_c L_{cg})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1148 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLEWA CAIDA FLOOD IMPROVEMENT STUDY (E½ RIVE N2/3 T35

Drainage Concentration Point: #10 (SUNRISE DRIVE SEC LINE 10 & 15)

Watershed Area (A): 537 acres/~~square miles~~.

Length of Watercourse (L_c): 19000 ft. Length to Center of Gravity (L_{ca}): 9500 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
5900	5780 - 3400 = 2380
1300	3400 - 3200 = 200
4000	3200 - 2950 = 250
7800	2950 - 2720 = 230

Mean Slope (S_c): 0.0659 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (a_b): 0.044 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.68 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 65% B 4% C 31% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 10% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.09, 86.43, 97.40
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5483, 0.5306, 0.7072 (pervious areas) 0.9562 (impervious areas)

Runoff Supply Rate (q): 0.6328 i in./hr. (function of i)

Time of Concentration (T_c): 56 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 32 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 4.1231 in./hr.

Runoff Supply Rate (q) at T_c: 2.6091 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 1412 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E1/2 R14E N2/3 T13S)

Drainage Concentration Point: #11 (@ VALLEY VIEW WASH SWAN ROAD SEC LINE 10 & 11)

Watershed Area (A): 908 (927*) acres/square miles.

Length of Watercourse (L_c): 21700 ft. Length to Center of Gravity (L_{ca}): 10900 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
4000	5600 - 3800 = 1800
2200	3800 - 3400 = 400
1900	3400 - 3200 = 200
12500	3200 - 2740 = 460

Mean Slope (S_c): 0.0670 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (a_b): 0.047 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.69 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 38%B 13%C 49%D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 70% (DB) Impervious Cover: 8% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87.11, 86.45, 92.41
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5501, 0.5323, 0.7086 (pervious areas) 0.9563 (impervious areas)

Runoff Supply Rate (q): 0.6519 i in./hr. (function of i)

Time of Concentration (T_c): 64 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 39 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 3.7103 in./hr.

Runoff Supply Rate (q) at T_c: 2.4187 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2214 (2260*) cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

* @ SUNRISE DRIVE (SEC LINE 10 & 15)

22 Impervious cover breakdown

5% @ 65%
2% @ 30%
8% @ 5%
47% @ 0%
38% @ 10%

8%

basin factor breakdown

8400 @ 0.035
3400 @ 0.040
9900 @ 0.060

21700

0.047

CP11

HYDROLOGIC DATA SHEET

Project Name and Location: EL CHA CANDA FLOOD IMPROVEMENT STUDY (E½ RIVE N½ T13S)

Drainage Concentration Point: #12 (@ SEC LINE 15-22)

Watershed Area (A): 794 acres/~~square miles~~. 32.012A

Length of Watercourse (L_c): 24500 ft. Length to Center of Gravity (L_{cg}): 12250 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
5900	5780 - 3400 = 2380
1300	3400 - 3200 = 200
4000	3200 - 2950 = 250
13200	2950 - 2580 = 370

Mean Slope (S_c): 0.0511 ft./ft. Watershed Type(s): ~~FOOTHILLS (DEVELOPED)~~ MOUNTAIN (UNDEVELOPED) (future)

Basin Factor (a_b): 0.047 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.67 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 76%B 3%C 21%D Cover Type(s): ~~DESERT~~ MOUNTAIN BRUSH BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 11% (future)

CN(s): 83, 87, 90 (pervious & impervious areas) CN*(s): 87.07, 86.40, 92.38
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5466, 0.5288, 0.7058 (pervious areas) 0.9560 (impervious areas)

Runoff Supply Rate (q): 0.6209 i in./hr. (function of i)

Time of Concentration (T_c): 70 $i^{-.4}$ ~~hrs.~~/mins. (function of i)

Iterative Solution of T_c: 43 hrs./mins.

Rainfall Intensity (i) at T_c: 3.3905 in./hr.

Runoff Supply Rate (q) at T_c: 2.1052 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{cg})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1685 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CANYON FLOOD IMPROVEMENT STUDY (E½ RIVE N2/3 T13S)

Drainage Concentration Point: #13 (VALLEY VIEW WASH @ SEC LINE 15 & 22)

Watershed Area (A): 1239 acres/~~square miles~~.

Length of Watercourse (L_c): 27300 ft. Length to Center of Gravity (L_{ca}): 13700 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
2000	5600 - 3800 = 1800
2700	3800 - 3400 = 400
1800	3400 - 3200 = 200
12500	3200 - 2740 = 460
3600	2740 - 2580 = 160

Mean Slope (S_c): 0.0545 ft./ft. Watershed Type(s): _____ (future)

Basin Factor (a_b): 0.045 (future) Flood Frequency: _____ yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.68 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 53%B 10%C 37%D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30%(MB) 20%(DB) Impervious Cover: 12% (future)

CN(s): 83 82, 90 (pervious & impervious areas) CN*(s): 87.09, 86.43, 92.40
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5483, 0.5306, 0.7072 (pervious areas) 0.9562 (impervious areas)

Runoff Supply Rate (q): 0.6474 i in./hr. (function of i)

Time of Concentration (T_c): 77 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 49 hrs./mins.

Rainfall Intensity (i) at T_c: 3.1072 in./hr.

Runoff Supply Rate (q) at T_c: 2.0117 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2512 cfs.

645.13qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

1239 unserviceable cover breakdown

9%	@	65%
3%	@	30%
6%	@	5%
35%	@	0%
46%	@	10%
1%	@	50%
12%		

warm factor breakdown

14000	@	0.035
3400	@	0.040
9900	@	0.060
27300		

CP 13

HYDROLOGIC DATA SHEET

roject Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E 1/2 R14E N 2/3 T13S

Drainage Concentration Point: #14 (@ SEC LINE 14 & 23)

atershed Area (A): 239 acres/~~square miles~~.

Length of Watercourse (L_c): 10800 ft. Length to Center of Gravity (L_{cg}): 5000 ft.

ncremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4200	3000 - 2800 = 200
6600	2800 - 2610 = 190

ean Slope (S_c): 0.0345 ft./ft. Watershed Type(s): FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.035 (future) Flood Frequency: 100 yrs.

24 (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.63 in. Areal Value: _____ in.

2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

oil Group(s): 100% B Cover Type(s): DESERT BRUSH

Cover Density (pervious areas): 20% Impervious Cover: 15% (future)

N(s): 83 (curve number) (pervious & impervious areas) CN*(s): 86.98 (adjusted curve number)

unoff to Rainfall Ratio(s),(C): 0.5395 (pervious areas) 0.9553 (impervious areas)

Runoff Supply Rate (q): 0.6019 i in./hr. (function of i)

ime of Concentration (T_c): 41 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 22 hrs./mins.

Rainfall Intensity (i) at T_c: 5.0095 in./hr.

unoff Supply Rate (q) at T_c: 3.0152 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{cg})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 726 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E½ RIVE N½ T13S)

Drainage Concentration Point: #15 (VALLEY VIEW WASH @ SEC LINE 22 & 27 (APPROX. RIVER RD)

Watershed Area (A): 2561 acres/square miles.

Length of Watercourse (L_c): 32900 ft. Length to Center of Gravity (L_{ca}): 16500 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
4000	5600 - 3800 = 1800
2200	3800 - 3400 = 400
1900	3400 - 3200 = 200
12500	3200 - 2740 = 160
11200	2740 - 2440 = 300

Mean Slope (S_c): 0.0277 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.047 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.166 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 70% B 6% C 24% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 12% (future)

CN(s): 83 82, 90 (pervious & impervious areas) CN*(s): 87.05, 86.38, 91.36
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5448, 0.5270, 0.7044 (pervious areas) 0.9558 (impervious areas)

Runoff Supply Rate (q): 0.62169 i in./hr. (function of i)

Time of Concentration (T_c): 106 $i^{-.4}$ ~~hrs.~~/mins. (function of i)

Iterative Solution of T_c: 78 hrs./mins.

Rainfall Intensity (i) at T_c: 2.1714 in./hr.

Runoff Supply Rate (q) at T_c: 1.3613 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For Impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 3514 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

roject Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E 1/2 R14E N 2/3 T13S)

Drainage Concentration Point: #16 (@ FOREST SERVICE BOUNDARY)

atershed Area (A): 390 acres/~~square miles~~.

Length of Watercourse (L_c): 7300 ft. Length to Center of Gravity (L_{ca}): 3700 ft.

ncremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
4000	5600 - 3800 = 1800
2200	3800 - 3400 = 400

ean Slope (S_c): 0.3259 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) (EXIST ~~future~~)

Basin Factor (a_b): 0.060 (EXIST ~~future~~) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.72 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 29% C 71% D Cover Type(s): ~~DESERT~~ MOUNTAIN BRUSH BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 0% (EXIST ~~future~~)

CN(s): 62, 90 (pervious & impervious areas) CN*(s): 86.51, 92.46
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5375, 0.7126 (pervious areas) 0.9568 (impervious areas)

Runoff Supply Rate (q): 0.6618 i in./hr. (function of i)

Time of Concentration (T_c): 23 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 10 hrs./mins.

Rainfall Intensity (i) at T_c: 7.3440 in./hr.

Runoff Supply Rate (q) at T_c: 4.8603 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1911 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: ELEC-P. CAIDA FLOOD IMPROVEMENT STUDY (E1/2 R14E N2/3 T13S)

Drainage Concentration Point: #17 (SEC. LINE 2 & 11 ALONG SKYLINE DRIVE ALIGNMENT)

Watershed Area (A): 257 acres/~~square miles~~.

Length of Watercourse (L_c): 11200 ft. Length to Center of Gravity (L_{ca}): 5600 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
5900	5780 - 3400 = 2380
1300	3400 - 3200 = 200
4000	3200 - 2950 = 250

Mean Slope (S_c): 0.1533 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) ~~FOOTHILLS (DEVELOPED)~~ (future)

Basin Factor (n_b): 0.053 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.71 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 26% B 9% C 65% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 4% (future)

CN(s): 83, 87, 90 (pervious & impervious areas) CN*(s): 87.16, 86.49, 92.44
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5535, 0.5358, 0.7113 (pervious areas) 0.9566 (impervious areas)

Runoff Supply Rate (q): 0.66665 i in./hr. (function of i)

Time of Concentration (T_c): 35 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 17 hrs./mins.

Rainfall Intensity (i) at T_c: 5.8595 in./hr.

Runoff Supply Rate (q) at T_c: 3.9055 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 1012 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E 1/2 RIVE N 2/3 T13S.

Drainage Concentration Point: #18 (SEC LINE 7 & 11 SKYLINE DRIVE ALIGNMENT)

Watershed Area (A): 610 acres/~~square miles~~.

Length of Watercourse (L_c): 13300 ft. Length to Center of Gravity (L_{ca}): 7000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
1100	6080 - 5600 = 480
4000	5600 - 3800 = 1800
2200	3800 - 3400 = 400
1900	3400 - 3200 = 200
4100	3200 - 2980 = 220

Mean Slope (S_c): 0.1339 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) HILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.055 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.72 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 8%B 19%C 73%D Cover Type(s): MOUNTAIN BRUSH DESERT SCRUB MIX

Cover Density (pervious areas): 30%(MB) 20%(DS) Impervious Cover: 30% (future)

CN(s): 83, 82, 90 (pervious & impervious areas) CN*(s): 87, 86, 86, 51, 92, -- (adjusted curve number)
(curve number)

Runoff to Rainfall Ratio(s), (C): 0.5552, 0.5375, 0.7126 (pervious areas) 0.9568 (impervious areas)

Runoff Supply Rate (q): 0.6754 i in./hr. (function of i)

Time of Concentration (T_c): 42 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 22 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 5.1810 in./hr.

Runoff Supply Rate (q) at T_c: 3.4994 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2152 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E 1/2 RIDGE N 2/3 T13S)

Drainage Concentration Point: #19 (FLECHA CAIDA WASH @ SUNRISE DRIVE SEC LINE 11 & 14)

Watershed Area (A): 161 acres/square miles.

Length of Watercourse (L_c): 7000 ft. Length to Center of Gravity (L_{ca}): 3500 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	3080 - 2960 = 120
5000	2960 - 2830 = 130

Mean Slope (S_c): 0.0319 ft./ft. Watershed Type(s): FOOTHILLS (DEVELOPED) (future)

Basin Factor (a_b): 0.039 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.666 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 100% B Cover Type(s): DESERT BRUSH

Cover Density (pervious areas): 20% Impervious Cover: 10% (future)

CN(s): 83 (curve number) (pervious & impervious areas) CN*(s): 87.05 (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5448 (pervious areas) 0.9558 (impervious areas)

Runoff Supply Rate (q): 0.5859 i in./hr. (function of i)

Time of Concentration (T_c): 38 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 19 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 5.4564 in./hr.

Runoff Supply Rate (q) at T_c: 3.1971 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 519 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E1/2 R14E T2S - T3S)

Drainage Concentration Point: #20 (FLECHA CAIDA WASH @ SEC LINE 14 & 23)

Watershed Area (A): 321 acres/~~square miles~~.

Length of Watercourse (L_c): 13600 ft. Length to Center of Gravity (L_{ca}): 6800 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	3080 - 2960 = 120
5000	2960 - 2830 = 130
6600	2830 - 2660 = 170

Mean Slope (S_c): 0.0287 ft./ft. Watershed Type(s): ~~SOUTHWEST DEVELOPED~~ (future)

Basin Factor (n_b): 0.037 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.64 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 100% B Cover Type(s): DESERT BRUSH

Cover Density (pervious areas): 20% Impervious Cover: 10% (future)

CN(s): 83 (curve number) (pervious & impervious areas) CN*(s): 87.00 (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5413 (pervious areas) 0.9555 (impervious areas)

Runoff Supply Rate (q): 0.5827 i in./hr. (function of i)

Time of Concentration (T_c): 56 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 32 hrs./mins.

Rainfall Intensity (i) at T_c: 4.0615 in./hr.

Runoff Supply Rate (q) at T_c: 2.3668 in./hr.

Peak Discharge:

1.008 qA (acres): 7666 cfs.

645.33qA (square miles): _____ cfs.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E½ R14E N⅔T13S)

Drainage Concentration Point: #21 (FLECHA CAIDA WASH @ SEC. LINE 23 & 26 RIVER ROAD)

Watershed Area (A): 521 acres/~~square miles~~.

Length of Watercourse (L_c): 19400 ft. Length to Center of Gravity (L_{ca}): 9500 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	3080 - 2960 = 120
5000	2960 - 2830 = 130
6600	2830 - 2660 = 170
5800	2660 - 2480 = 180

Mean Slope (S_c): 0.0294 ft./ft. Watershed Type(s): ~~FORMULA DEFINED~~ (future)

Basin Factor (a_b): 0.036 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour):	in.	Areal Value:	in.
P_6 (6 hour):	in.	Areal Value:	in.
P_1 (1 hour):	2.62 in.	Areal Value:	in.
P_2 (2 hour):	in.	Areal Value:	in.
P_3 (3 hour):	in.	Areal Value:	in.

Soil Group(s): 100% B Cover Type(s): ~~DESERT SCRUB~~

Cover Density (pervious areas): 20% Impervious Cover: 10% (future)

CN(s): 83 (curve number) (pervious & impervious areas) CN*(s): 86.95 (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5377 (pervious areas) 0.9552 (impervious areas)

Runoff Supply Rate (q): 0.5795 i in./hr. (function of i)

Time of Concentration (T_c): 67 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 41 hrs./mins.

Rainfall Intensity (i) at T_c: 3.4361 in./hr.

Runoff Supply Rate (q) at T_c: 1.9911 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 1046 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLEWA CAISE FLOOD IMPROVEMENT STUDY (E½ RNE N½ T35)

Drainage Concentration Point: #22 (SEC LINE 23 & 26)

Watershed Area (A): 253 acres/square miles.

Length of Watercourse (L_c): 10200 ft. Length to Center of Gravity (L_{ca}): 5100 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
3800	2850 - 2640 = 210
6400	2640 - 2480 = 160

Mean Slope (S_c): 0.0324 ft./ft. Watershed Type(s): Bottom (Developed) (future)

Basin Factor (a_b): 0.035 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.59 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 90% B Cover Type(s): Desert Brush

Cover Density (pervious areas): 20% Impervious Cover: 10% (future)

CN(s): 53 (pervious & impervious areas) CN*(s): 86.88
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5323 (pervious areas) 0.9547 (impervious areas)

Runoff Supply Rate (q): 0.5745 i in./hr. (function of i)

Time of Concentration (T_c): 43 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 23 hrs./mins.

Rainfall Intensity (i) at T_c: 4.8186 in./hr.

Runoff Supply Rate (q) at T_c: 2.7683 in./hr.

Peak Discharge:

1.008 qA (acres): 706 cfs.

645.33qA (square miles): _____ cfs.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E 1/2 R14E N 2/3 T13S)

Drainage Concentration Point: #23 (WITHIN SE 1/4 SEC 3) @ FINGER ROCK WASH

Watershed Area (A): 390 acres/~~square miles~~.

Length of Watercourse (L_c): 8900 ft. Length to Center of Gravity (L_{ca}): 4400 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	5040 - 3680 = 1360
2100	3680 - 3200 = 480
1400	3200 - 3000 = 200
3400	3000 - 2800 = 200

Mean Slope (S_c): 0.1315 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) FOOTHILLS (DEVELOPED) (future)

Basin Factor (n_b): 0.050 (future) Flood Frequency: _____ yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.71 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 33% B 67% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 8% (future)

CN(s): 83 90 (pervious & impervious areas) CN*(s): 87.16, 92.44
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5535, 0.7113 (pervious areas) 0.9566 (impervious areas)

Runoff Supply Rate (q): 0.6830 i in./hr. (function of i)

Time of Concentration (T_c): 30 $i^{-.4}$ ~~hrs.~~ /mins. (function of i)

Iterative Solution of T_c: 14 hrs./mins.

Rainfall Intensity (i) at T_c: 6.3765 in./hr.

Runoff Supply Rate (q) at T_c: 4.3550 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 1712 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E 1/2 R14E N 1/2 T12S)

Drainage Concentration Point: #24 (FLECHA CAIDA WASH @ SEC LINE 26 & 27 SWAN ROAD)

Watershed Area (A): 850 acres/~~square miles~~.

Length of Watercourse (L_c): 20600 ft. Length to Center of Gravity (L_{ca}): 10300 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
3000	3080 - 2960 = 120
5000	2960 - 2830 = 130
6600	2830 - 2660 = 170
7000	2660 - 2450 = 210

Mean Slope (S_c): 0.0291 ft./ft. Watershed Type(s): ~~MOUNTAINS (DESERT)~~ (future)

Basin Factor (a_b): 0.036 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.60 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 100% B Cover Type(s): Desert Brush

Cover Density (pervious areas): 20% Impervious Cover: 10% (future)

CN(s): 83 (curve number) (pervious & impervious areas) CN*(s): 86.91 (adjusted curve number)

Runoff to Rainfall Ratio(s),(C): 0.5341 (pervious areas) 0.9548 (impervious areas)

Runoff Supply Rate (q): 0.5762 i in./hr. (function of i)

Time of Concentration (T_c): 70 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 44 hrs./mins.

Rainfall Intensity (i) at T_c: 3.2500 in./hr.

Runoff Supply Rate (q) at T_c: 1.8726 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Note: For impervious areas, CN* = 99 (constant).

Peak Discharge:

1.008 qA (acres): 1604 cfs.

645.33qA (square miles): _____ cfs.

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY (E½ R14E N⅔T13S)

Drainage Concentration Point: #25 (24) FINGER ROCK WASH JUST SOUTH OF FIRST SERVICE BOUNDARY

Watershed Area (A): 2167 (2254) acres/~~square miles~~

Length of Watercourse (L_c): 20400 (19700) ft. Length to Center of Gravity (L_{ca}): 9500 (9800) ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
4300	7255 - 6240 = 1015
10100	6240 - 3560 = 2680
5000	3560 - 3080 = 480
1000	3080 - 3000 = 80

Mean Slope (S_c): 0.1796 (0.1790) ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) EXIST ~~(Future)~~

Basin Factor (a_b): 0.060 (0.049) EXIST ~~(Future)~~ Flood Frequency: 100 yrs.

P_{24} (24 hour):	in.	Areal Value:	in.
P_6 (6 hour):	in.	Areal Value:	in.
P_1 (1 hour): 2.74	in.	Areal Value:	in.
P_2 (2 hour):	in.	Areal Value:	in.
P_3 (3 hour):	in.	Areal Value:	in.

Soil Group(s): 10% B 25% C 74% D Cover Type(s): MOUNTAIN BRUSH / DESERT BRUSH

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 0% EXIST ~~(Future)~~

CN(s): 83 82, 90 (curve number) (pervious & impervious areas) CN*(s): 87.22, 86.56, 92.49 (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5525, 0.5409, 0.7152 (pervious areas) 0.9571 (impervious areas)

Runoff Supply Rate (q): 0.6701 i in./hr. (function of i)

Time of Concentration (T_c): 51 $i^{-.4}$ ~~hrs~~./mins. (function of i)

Iterative Solution of T_c: 28 ~~hrs~~./mins.

Rainfall Intensity (i) at T_c: 4.36667 in./hr.

Runoff Supply Rate (q) at T_c: 3.0600 in./hr.

Equation for T_c:

$$T_c = \frac{a_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 6684 (6600) cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY

Drainage Concentration Point: #26 (CONFLUENCE #7 & #8) (FINGER ROCK TRIBUTARY)

Watershed Area (A): 663 acres/square miles.

Length of Watercourse (L_c): 12600 ft. Length to Center of Gravity (L_{ca}): 6000 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	5280 - 4000 = 1280
2200	4000 - 3400 = 600
1800	3400 - 3080 = 320
4200	3080 - 2850 = 230
2400	2850 - 2765 = 85

Mean Slope (S_c): 0.0914 ft./ft. Watershed Type(s): MOUNTAIN (UNDEVELOPED) (future)

Basin Factor (n_b): 0.046 (future) Flood Frequency: 100 yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.70 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 54% B 46% D Cover Type(s): MOUNTAIN BRUSH DESERT BRUSH MIX

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 20 % (future)

CN(s): 83, 90 (pervious & impervious areas) CN*(s): 87.13, 86.47, 92.45
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5516 0.5340 0.7000 (pervious areas) 0.9565 (impervious areas)

Runoff Supply Rate (q): 0.6875 i in./hr. (function of i)

Time of Concentration (T_c): 38 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 20 hrs./mins.

Rainfall Intensity (i) at T_c: 5.4000 in./hr.

Runoff Supply Rate (q) at T_c: 3.7125 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2481 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

HYDROLOGIC DATA SHEET

Project Name and Location: FLECHA CAIDA FLOOD IMPROVEMENT STUDY

Drainage Concentration Point: #27 (CONFLUENCE WITH FINGER ROCK WASH)

Watershed Area (A): 1698 acres/square miles.

Length of Watercourse (L_c): 14900 ft. Length to Center of Gravity (L_{ca}): 7200 ft.

Incremental Change in Length (L_i) - ft.	Incremental Change in Elevation (H_i) - ft.
2000	5280 - 4000 = 1280
2200	4000 - 3400 = 600
1800	3400 - 3080 = 320
4200	3080 - 2850 = 230
2400	2850 - 2765 = 85
2300	2765 - 2690 = 75

Mean Slope (S_c): 0.0750 ft./ft. Watershed Type(s): MOUNTAIN UNDEVELOPED FOOTHILLS DEVELOPED (future)

Basin Factor (a_b): 0.044 (future) Flood Frequency: _____ yrs.

P_{24} (24 hour): _____ in. Areal Value: _____ in.

P_6 (6 hour): _____ in. Areal Value: _____ in.

P_1 (1 hour): 2.69 in. Areal Value: _____ in.

P_2 (2 hour): _____ in. Areal Value: _____ in.

P_3 (3 hour): _____ in. Areal Value: _____ in.

Soil Group(s): 56% B 44% D Cover Type(s): MOUNTAIN BRUSH & DESERT BRUSH

Cover Density (pervious areas): 30% (MB) 20% (DB) Impervious Cover: 20% (future)

CN(s): 83, 96 (pervious & impervious areas) CN*(s): 87.11, 92.41
(curve number) (adjusted curve number)

Runoff to Rainfall Ratio(s), (C): 0.5501, 0.7086 (pervious areas) 0.9563 (impervious areas)

Runoff Supply Rate (q): 0.6871 i in./hr. (function of i)

Time of Concentration (T_c): 44 $i^{-.4}$ hrs./mins. (function of i)

Iterative Solution of T_c: 24 hrs./mins.

Rainfall Intensity (i) at T_c: 4.8909 in./hr.

Runoff Supply Rate (q) at T_c: 3.3605 in./hr.

Equation for T_c:

$$T_c = \frac{n_b}{50} \frac{(L_c L_{ca})^{.3}}{(S_c)^{.4}} q^{-.4} \text{ hours.}$$

Peak Discharge:

1.008 qA (acres): 2364 cfs.

645.33qA (square miles): _____ cfs.

Note: For impervious areas, CN* = 99 (constant).

APPENDIX C

CULVERT PERFORMANCE CURVES

sla
SIMONS, LI & ASSOCIATES, INC.

CLIENT _____ JOB NO. _____ PAGE _____
PROJECT _____ DATE CHECKED _____ DATE _____
DETAIL _____ CHECKED BY _____ COMPUTED BY _____

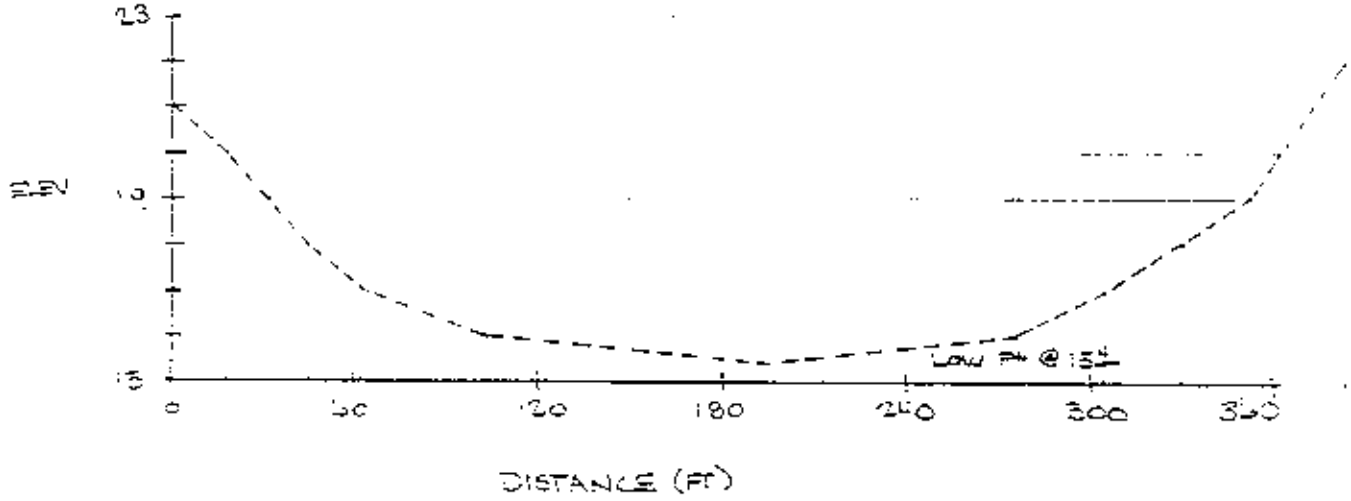
FINGER ROCK WASH

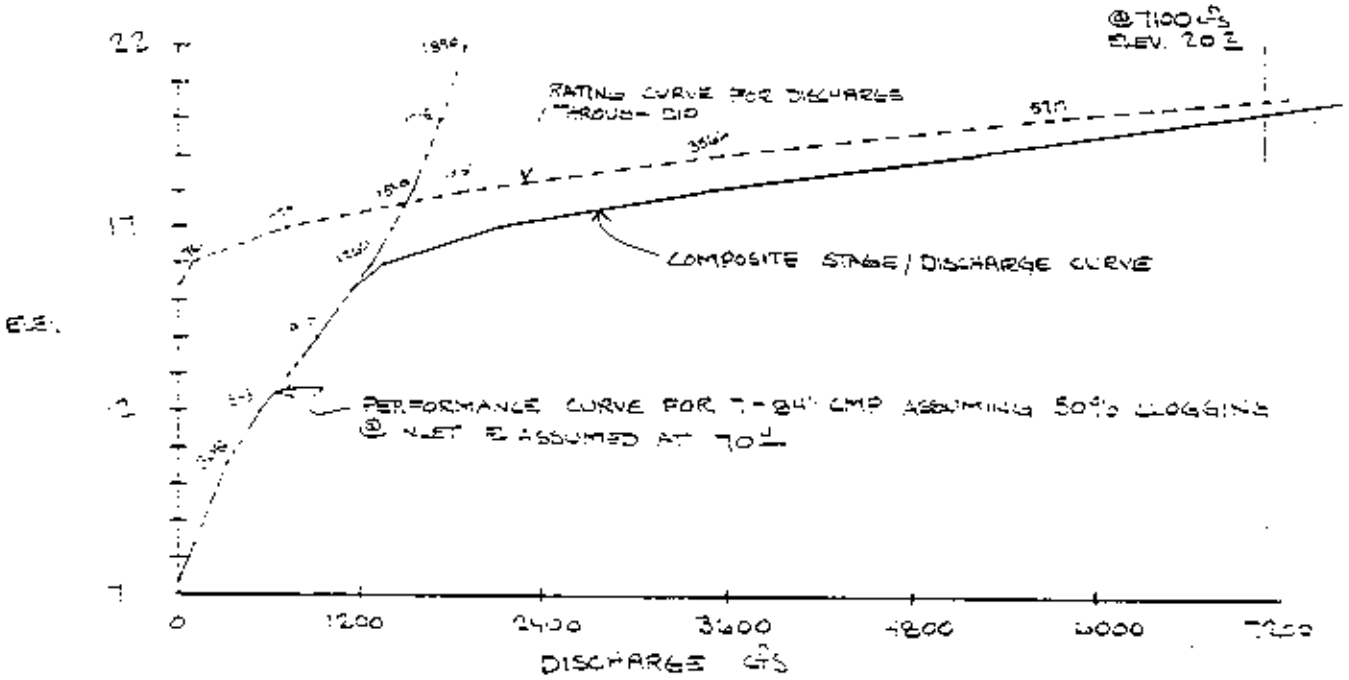
DISCHARGE RATING CURVE FOR 7-84" CMP & DIP SECTION AT PONTATOC CANYON DRIVE $Q_{100} \approx 7100$ cfs
$C_{WEIR} = 2.7$

DIP SECTION

9/21 18/20 45/18 63/17 103/16 195/15.4 275/16 305/17

330/18 353/19 362/20 383/22





APPENDIX C
SHEET 1 OF 8

049

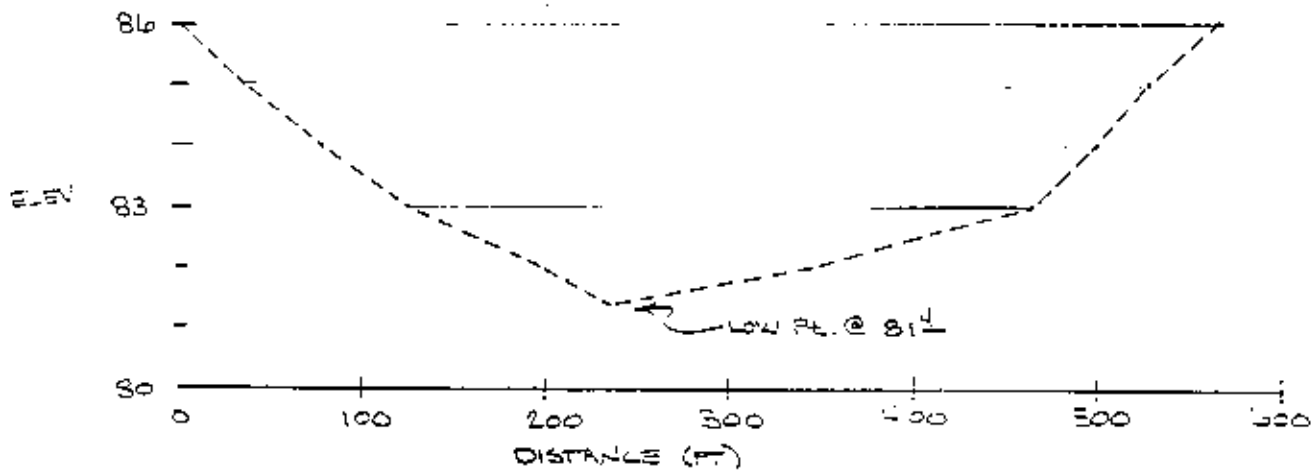
FINGER ROCK WASH

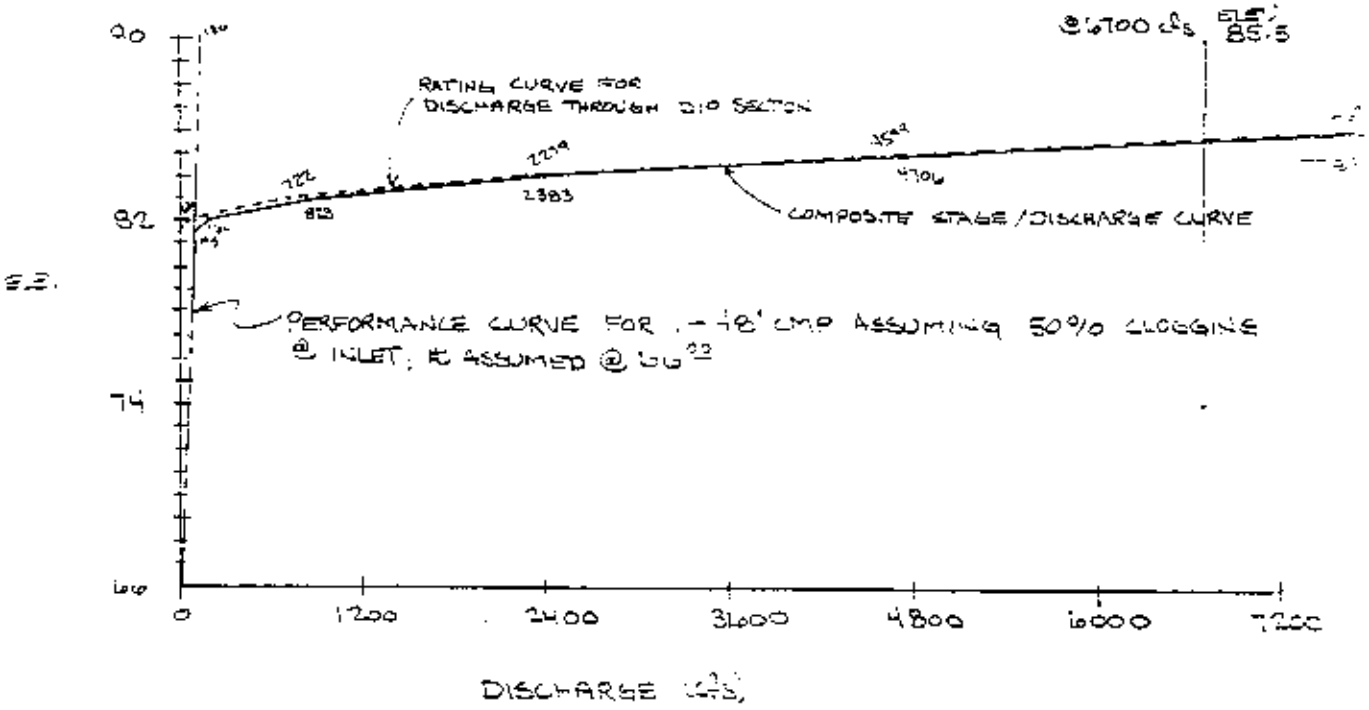
DISCHARGE RATING CURVE FOR 1-48" CMP
& DIP SECTION AT SKYLINE DRIVE $Q_{100} \approx 6700$ cfs
$C_{WEIR} = 2.7$

DIP SECTION

0/86 33/85 78/84 124/83 193/82 233/81.4 343/82

415/82.6 463/83 500/84 529/85 565/86





APPENDIX C
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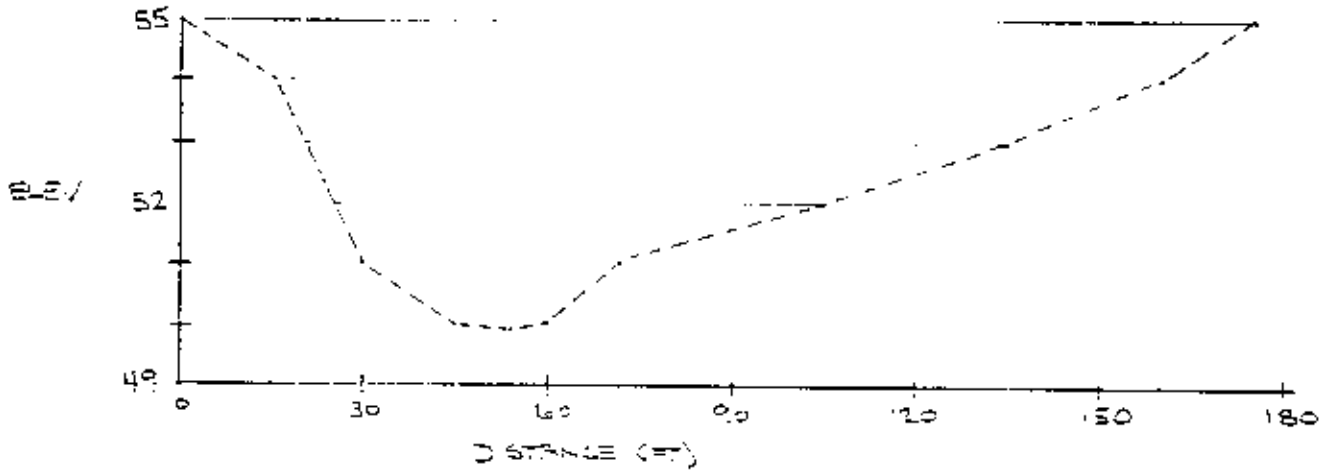
sla
SIMONS, LI & ASSOCIATES, INC.

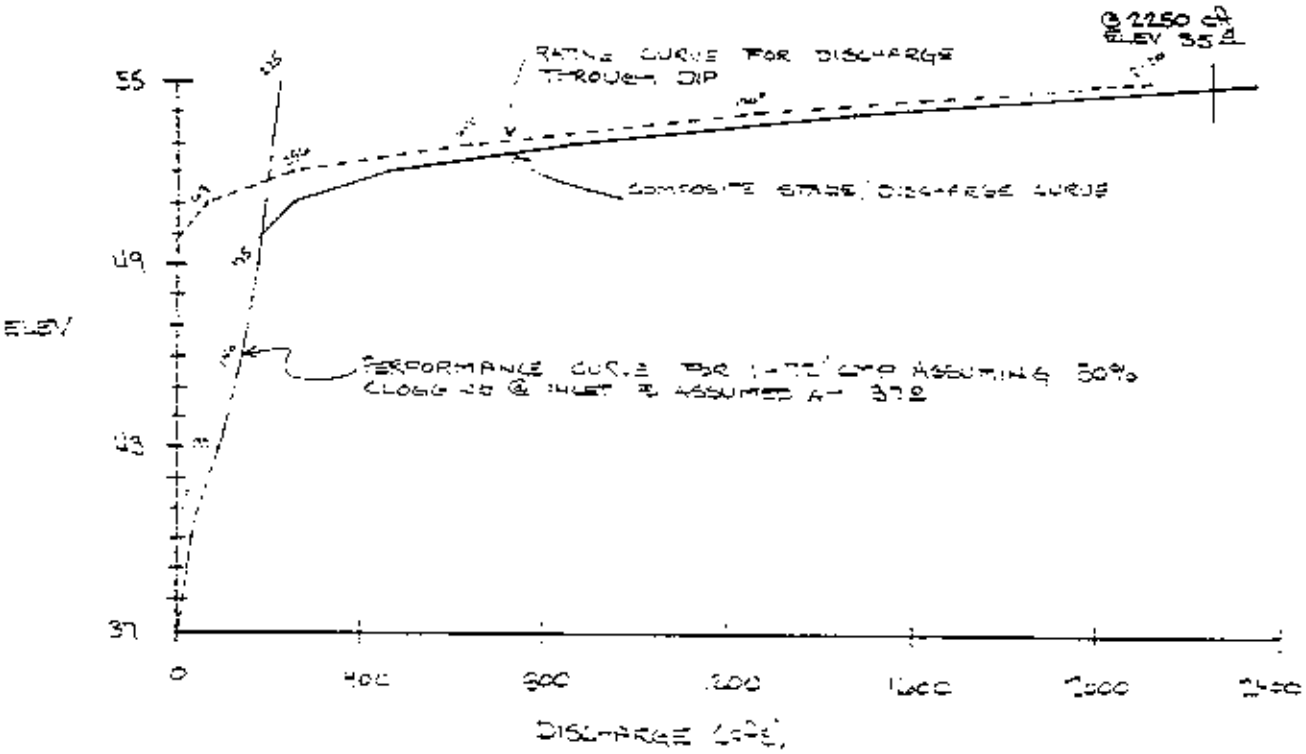
CLIENT _____ JOB NO. _____ PAGE _____
PROJECT _____ DATE CHECKED _____ DATE _____
DETAIL _____ CHECKED BY _____ COMPUTED BY _____

VALLEY VIEW WASH
HEC-2 SECTION NO 36
DISCHARGE RATING CURVE FOR 1-72" CMP & DIP
SECTION AT SWAN RD Q_{100} = 2250 Q_{WRC} = 1270

DIP SECTION

0/55 15/54 30/51 45/50 55/49.2 60/50 72/51 105/52
135/53 160/54 175/55





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sla
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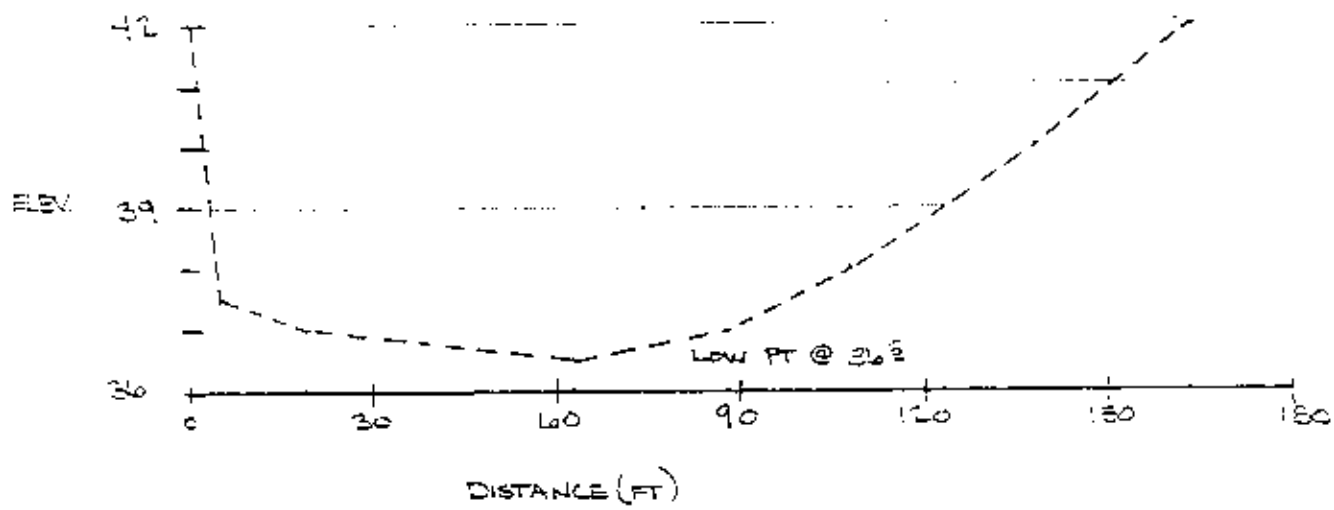
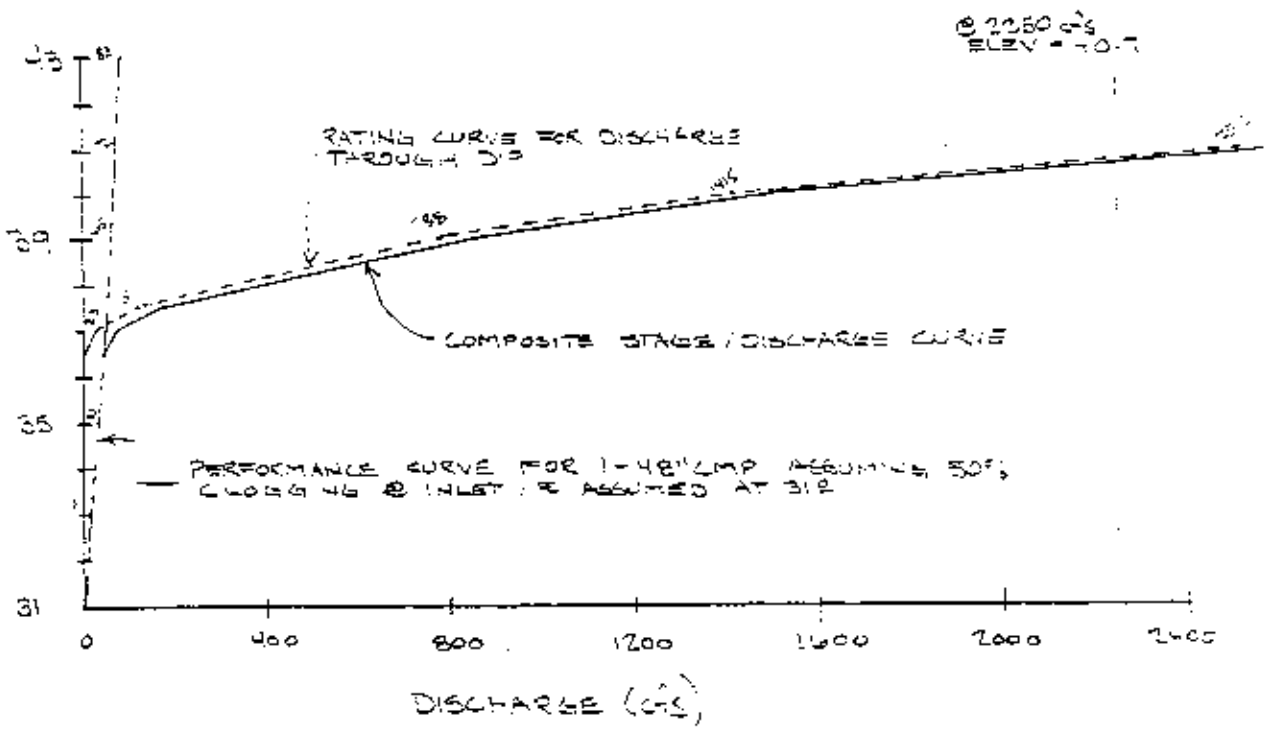
CLIENT _____ JOB NO. _____ PAGE _____
PROJECT _____ DATE CHECKED _____ DATE _____
DETAIL _____ CHECKED BY _____ COMPUTED BY _____

VALLEY VIEW WASH
HEC-2 SECTION No. 51
DISCHARGE RATING CURVE FOR 1-48" CMP & DIP SECTION AT CMO. ARCO $Q_{100} = 2250$ $C_{WEIR} = 2.7$

DIP SECTION

0/42 5/37.5 19/37 63/36.5 88/37 108/38 123/39 138/40 163/42





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SHEET No. 8

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sla SIMONS, LI & ASSOCIATES, INC.

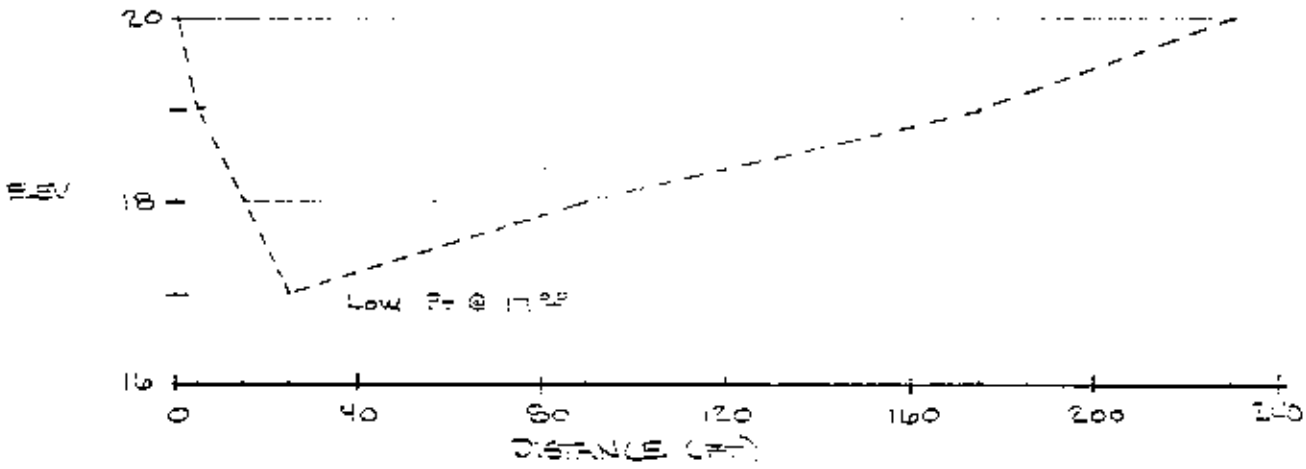
CLIENT _____ JOB NO. _____ PAGE _____
PROJECT _____ DATE CHECKED _____ DATE _____
DETAIL _____ CHECKED BY _____ COMPUTED BY _____

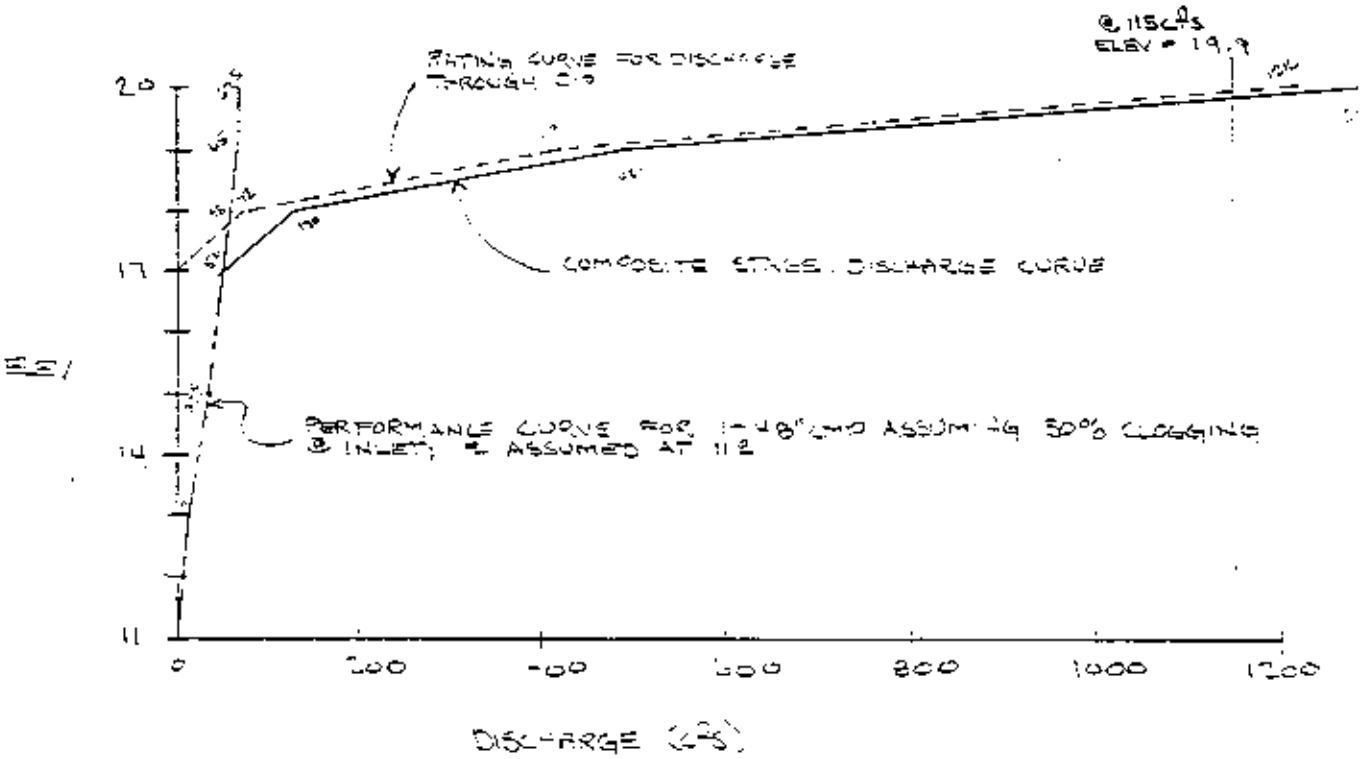
VALLEY VIEW WASH TRIBUTARY
(SKY CLUB WASH)

DISCHARGE RATING CURVE FOR 1-48" CMP & DIP
SECTION AT SWAN ROAD $Q_{100} = 1150$ $C_{WSR} = 2.7$

DIP SECTION

0/20 5/19 15/18 25/17 90/18 175/19 230/20





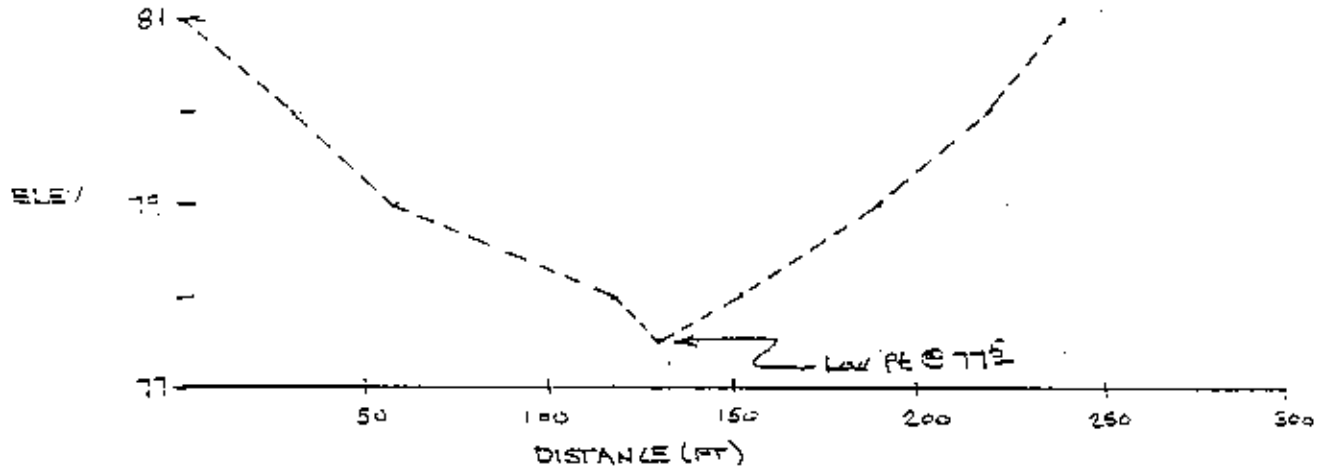
APPENDIX C
SHEET 5 OF 8

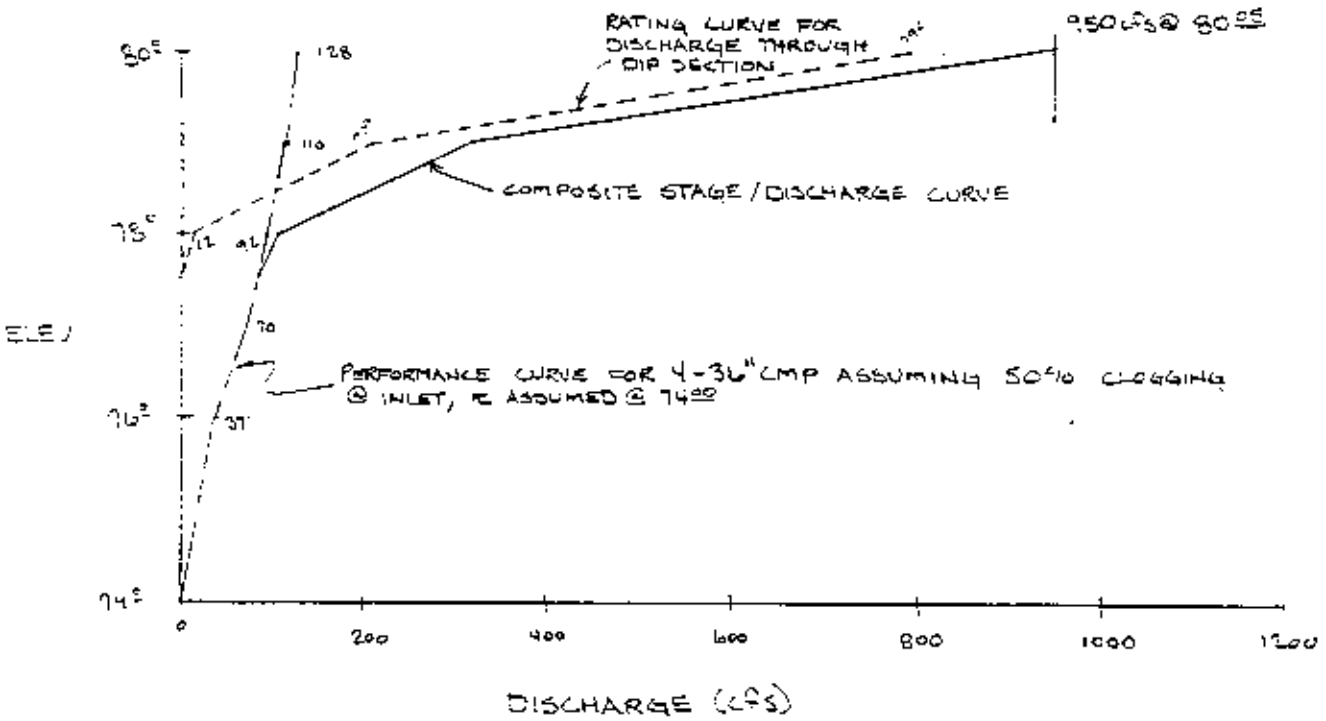
FLECHA CAIDA WASH

DISCHARGE RATING CURVE FOR 4-36" CMP & DIP SECTION AT CALLE DE LAS CHARAS $Q_{100} = 950$ cfs.
$C_{WEIR} = 2.70$

DIP SECTION

0/81 30/80 58/79 118/78 130/77.5 153/78 190/79 220/80 240/81





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SIMONS, Li & ASSOCIATES, INC.

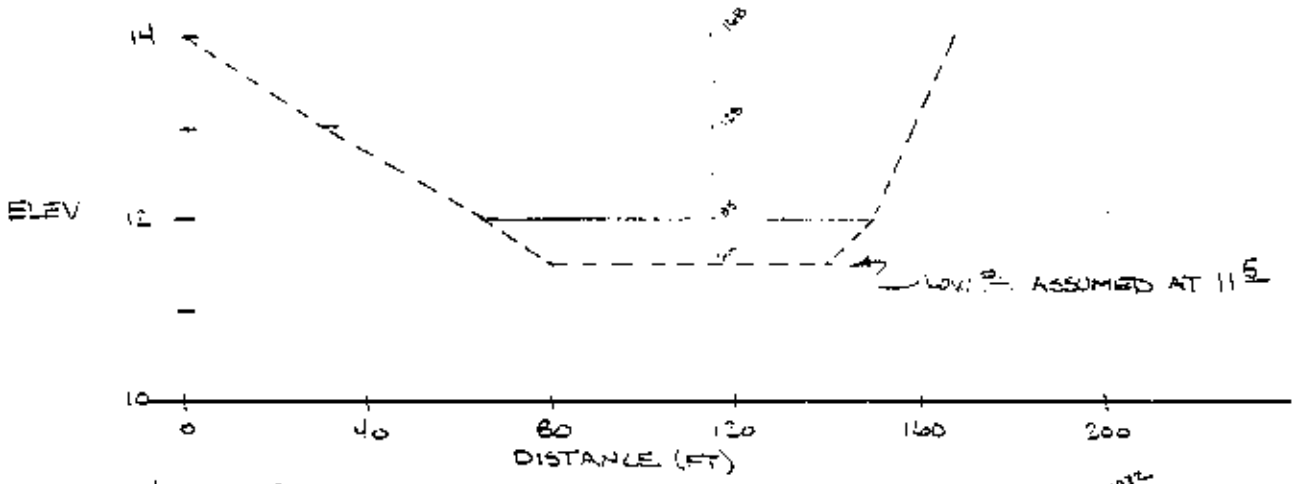
CLIENT ____ JOB NO. ____ PAGE ____
PROJECT ____ DATE CHECKED ____ DATE ____
DETAIL ____ CHECKED BY ____ COMPUTED BY ____

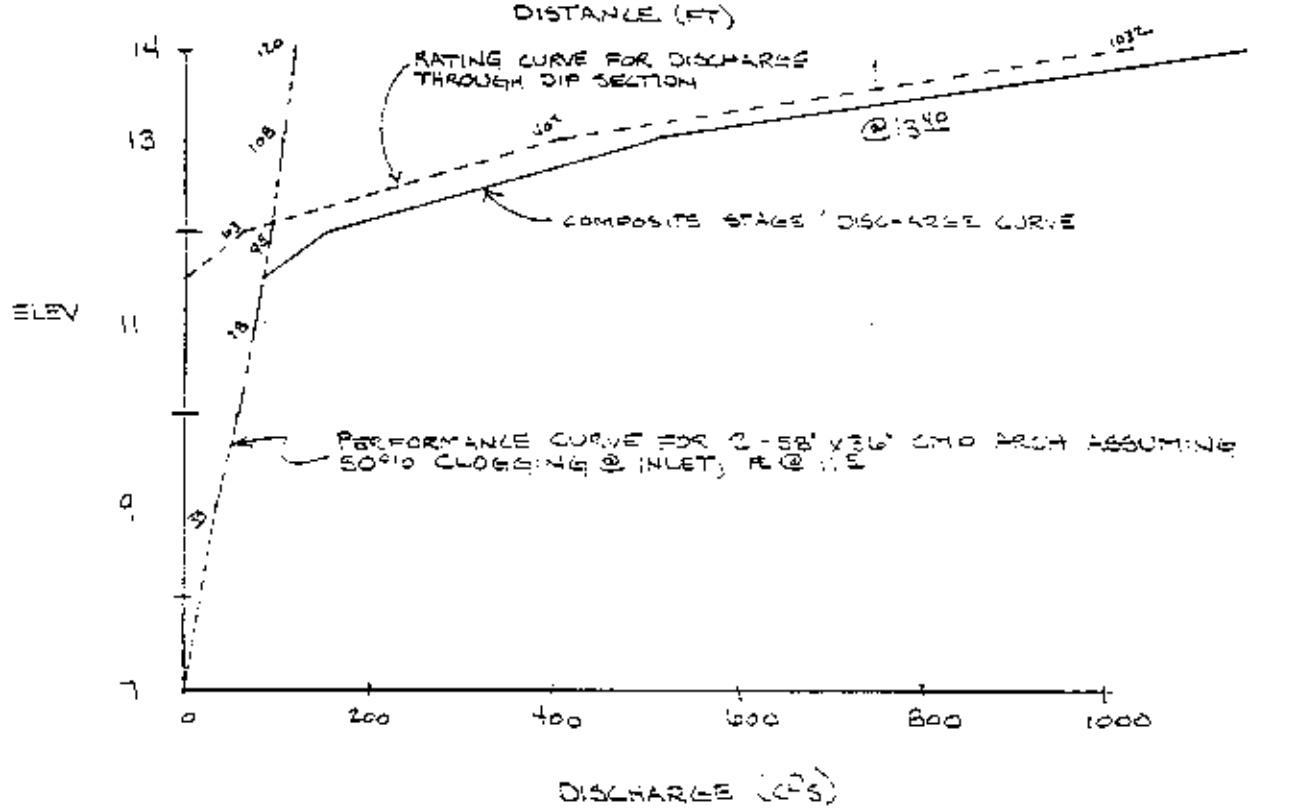
FLECHA CAIDA WASH

DISCHARGE RATING CURVE FOR 2 - 58" x 36" CMP ARCH & DIP CROSSING AT PASEO DEL BAC Q_{100} = 750 cfs C_{weir} = 2.70

DIP SECTION

0/14 30/13 65/12 80/11.5 140/11.5 150/12 166/14





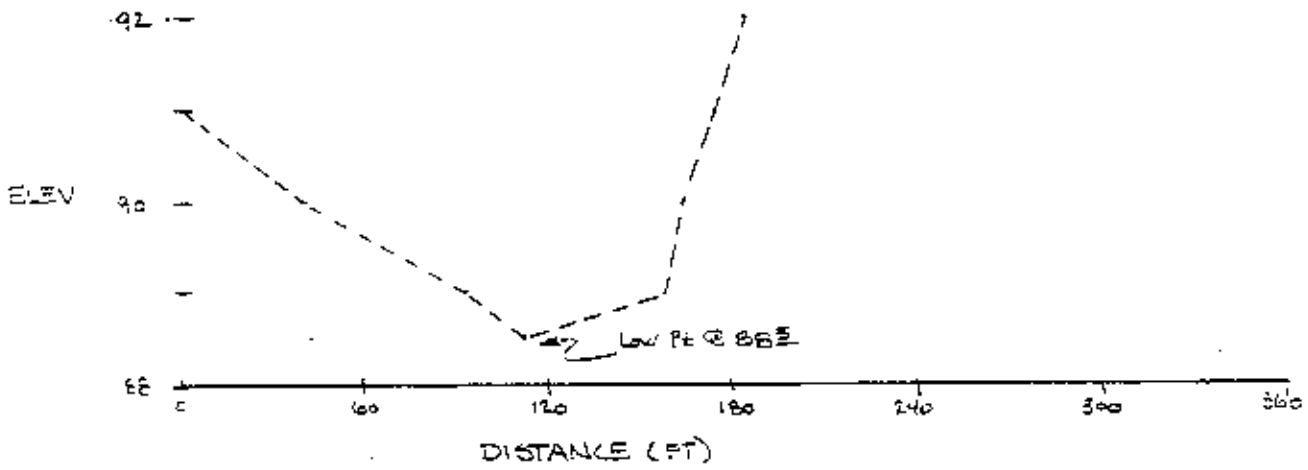
APPENDIX C
SHEET 7 OF 8

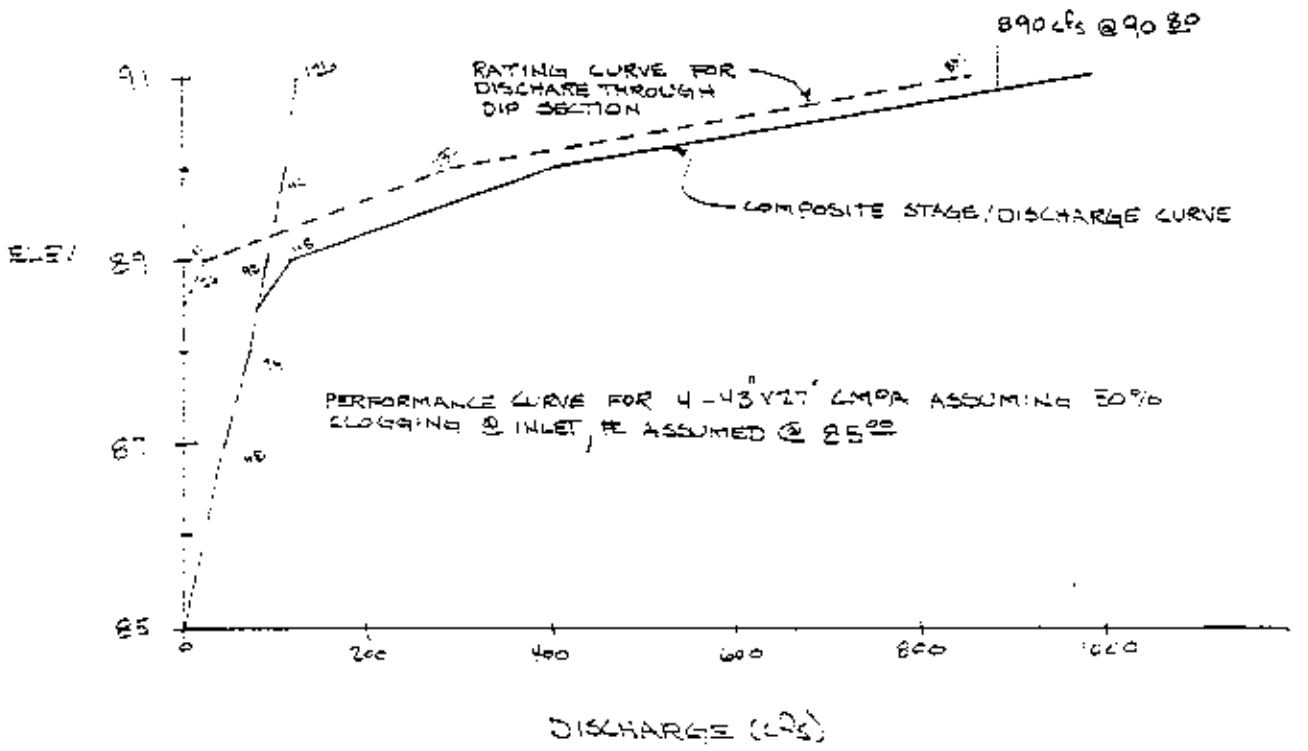
FLECHA CAIDA WASH

DISCHARGE RATING CURVE FOR 4 - 43" X 27" CMPA & DIP CROSSING OF COMO CARDENAL Q_{100} = 890 cfs C_{WEIR} = 2.70

DIP SECTION

0/91 40/90 92/89 112/88.5 158/89 165/90 295/90.5 175/90





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DISKETTE CONTAINING INPUT DATA FOR SUBCRITICAL AND SUPERCRITICAL RUNS

APPENDIX E

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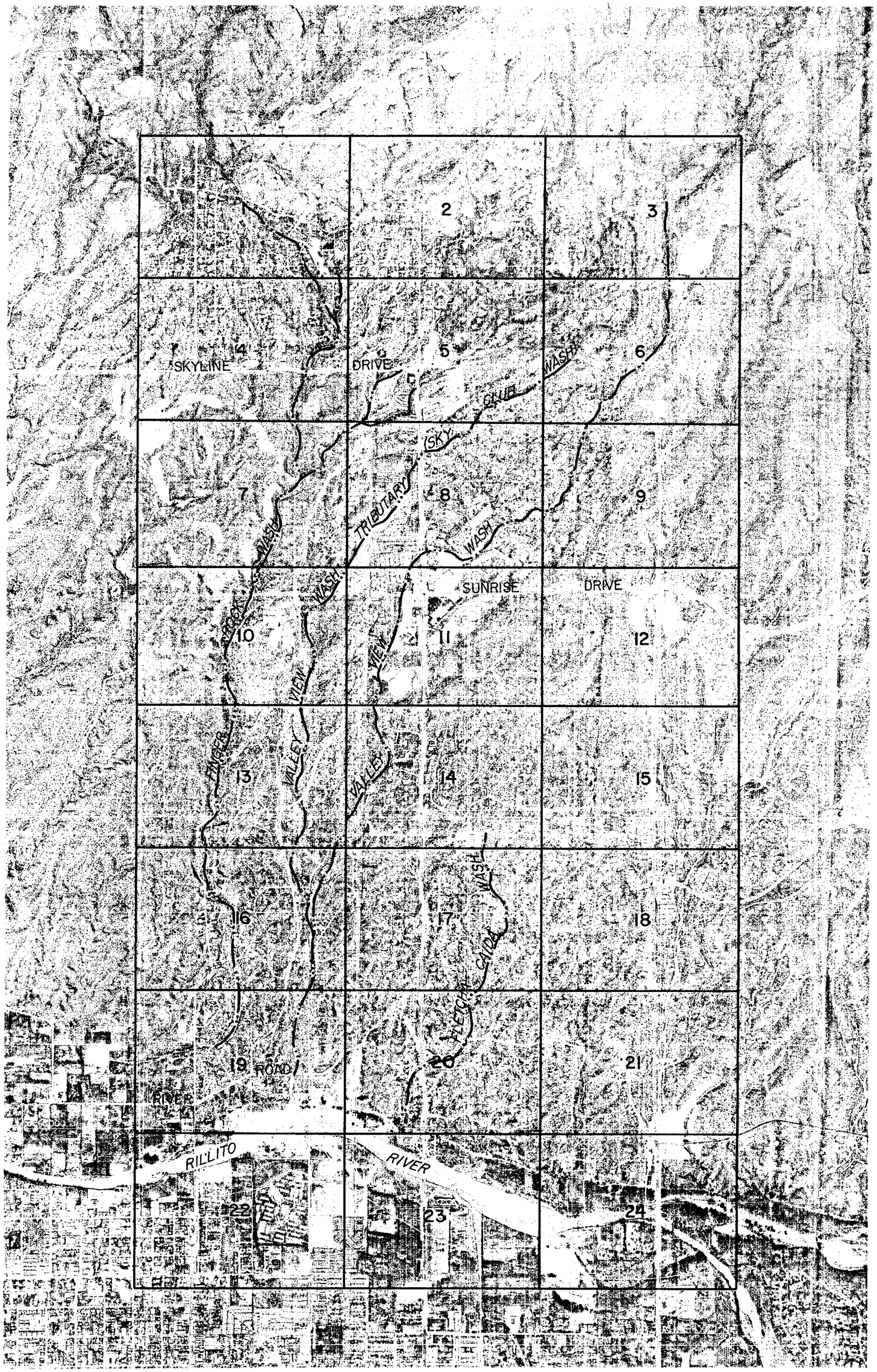
(SEPARATE DOCUMENT)



CROSS SECTION REFERENCE TABLE

WASH	SECTION NUMBER	SHEET NUMBER
	-------FINGER ROCK WASH------	
Main Watercourse:	1 - 3	19
	4 - 10	16
	11 - 17	13
	18 - 26	10
	26 - 34	7
	35 - 44	4
	44 - 48	1
Tributaries:	1.1 - 3.1	7
	4.1 - 8.1	5
	1.11- 4.11	5
	1.2	4
	2.2 - 3.2	1
	-------VALLEY VIEW WASH------	
Main Watercourse:	1 - 5	19
	6 - 12	16
	12	17
	13	13
	13 - 24	14
	24 - 34	11
	35 - 43	8
	43 - 49	9
	49 - 57	6
	58 - 62	3
Tributary: (Sky Club Wash)	1.1 - 4.1	16
	4.1 - 10.1	13
	11.1 - 18.1	10
	19.1	11
	20.1 - 29.1	8
	30.1 - 33.1	5
	-------FLECHA CAIDA WASH------	
Main Watercourse:	1 - 9	20
	10 - 18	17
	19 - 20	14

SCALE 1" = 100'

100 50 0 100 200

CONTOUR INTERVAL 1 FOOT

INDEX MAP

100-YEAR FLOOD PLAINS FOR THE FLECHA CAIDA FLOOD IMPROVEMENT STUDY

1" = 100' sla SIMONS, LI & ASSOCIATES, INC. SHEET 1 OF 1

"FLECHA CAIDA"

PHOTO DATES: 10-13-84 & 8-3-84

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

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

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ARIZONA, U.S.A.

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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

a 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.

I.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.

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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 I). 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 removed will be incorporated in providing for the 27 volume.

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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	2649.5 (0.6)	2648.9	2648.7	2648.5	2648.0
2	2640.8	2640.8	2640.5	2640.3	2640.2	2639.8
3	2636.3	2636.7 (0.4)	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	2630.8 (0.6)	2630.5	2630.3	2630.0	2629.7
7	2628.5	2628.8 (0.3)	2628.5	2628.3	2628.1	2627.8
8	2619.5	2618.6	2617.9	2617.4	2617.0	2616.4
9	2598.7	2599.3 (0.6)	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

$V_{50} = 500 A C_p = 500(1023)(.91) = 465{,}465\ FT^3$

$V_{5D} = 11$ 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

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

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

$V_s = 27$ ACRE FT + 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.

FIGURE 4
SHEET 4 OF 4

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 = 7.65 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 #	Q_{100}	Q_{25}	Q_{10}	Q_5	Q_2
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

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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

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

RESIDENCE	% DAMAGE 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 #	SCA #	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

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COMPUTATION OF AVERAGE ANNUAL RISK

FLOOD RETURN PERIOD (YEARS)	FLOOD DAMAGE ($)	PROBABILITY	AVERAGE DAMAGE ($)	Δ PROBABILITY	AVERAGE ANNUAL RISK ($)
0	0	1			
2	0	0.5	0	0.50	0
5	0	0.20	0	0.30	0
10	24,170	0.10	13,085	0.10	1,209
25	50,768	0.04	37,469	0.06	2,248
100	73,682	0.01	62,225	0.03	1,867

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" x 44" 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" x 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"×44" 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 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 I.

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 I) 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 @ 116 MIN.

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

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

CP #2 & CP #3 → CP #4 (SEC. 27 — 30): 4.85 fps (SUB)
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 — 34
HYDROGRAPH ANALYSIS SUMMARY SHEETS
(SHEET 1 OF 15)

2-YEAR
COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (CFS)	Q@CP243 (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

FIGURE 13
SHEET 2 OF 15

COMPOSITE HYDROGRAPH LISTING

METHOD B

TIME (MIN)	Q@CP3A (CFS)	Q@CP213 (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

* 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 $Q_P/2$ 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@CP243 (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

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	209.2	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	↓ 592.7	↓ 608.1 *
70	3.9	8.6	536.2	548.7

* PEAK VALUE ASSUMING SWAN RD. CROSSING IS UPGRADED

FIGURE 13

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 (SUPER)

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

FIGURE 13
SHEET 7 OF 15

10-YEAR

COMPOSITE HYDROGRAPH LISTING

TIME (MIN)	Q@CP3A (cfs)	Q@CP213 (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

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@CP243 (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	790.7
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

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

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

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

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

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.

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.

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

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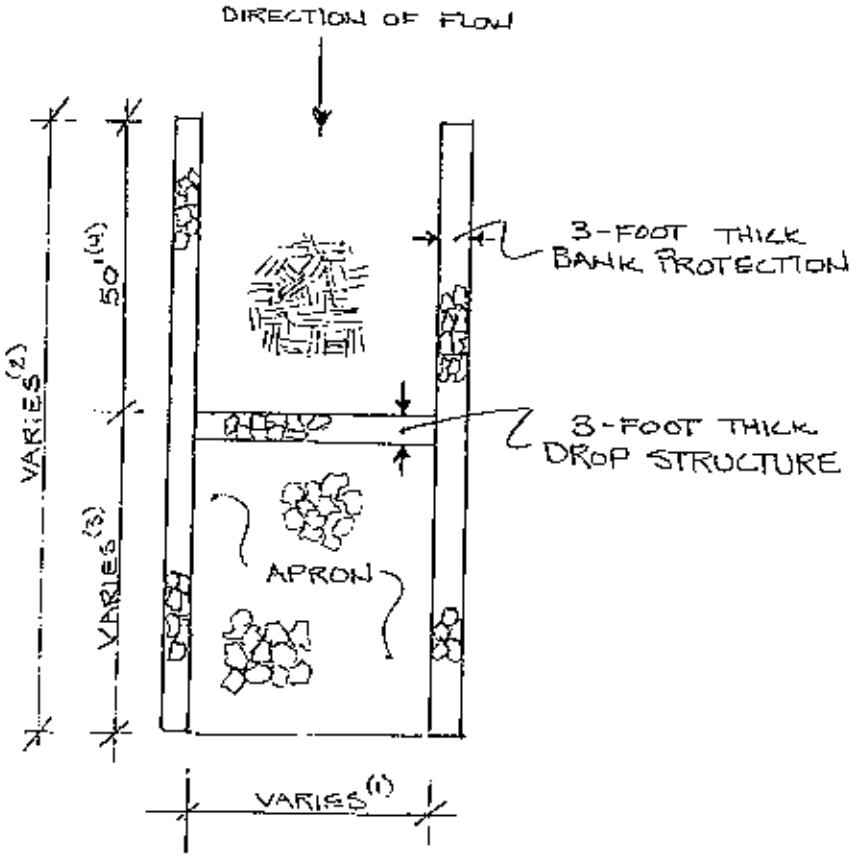
PROJECT DATE CHECKED DATE 11-10-88
DETAIL CHECKED BY COMPUTED BY RLS

TYPICAL PLAN VIEW
OF
GABION GRADE-CONTROL/DROP STRUCTURE



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

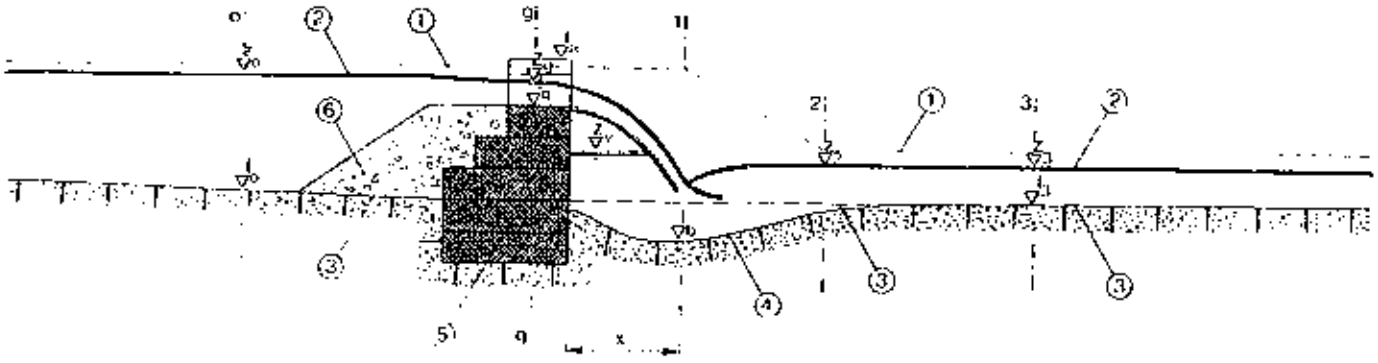
(2) OVERALL LENGTH FOR SINGLE DROP WOULD BE APPROX. 103 FT.; FOR STEPPED STRUCTURE LENGTH WOULD BE APPROX. 139 FT

(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.

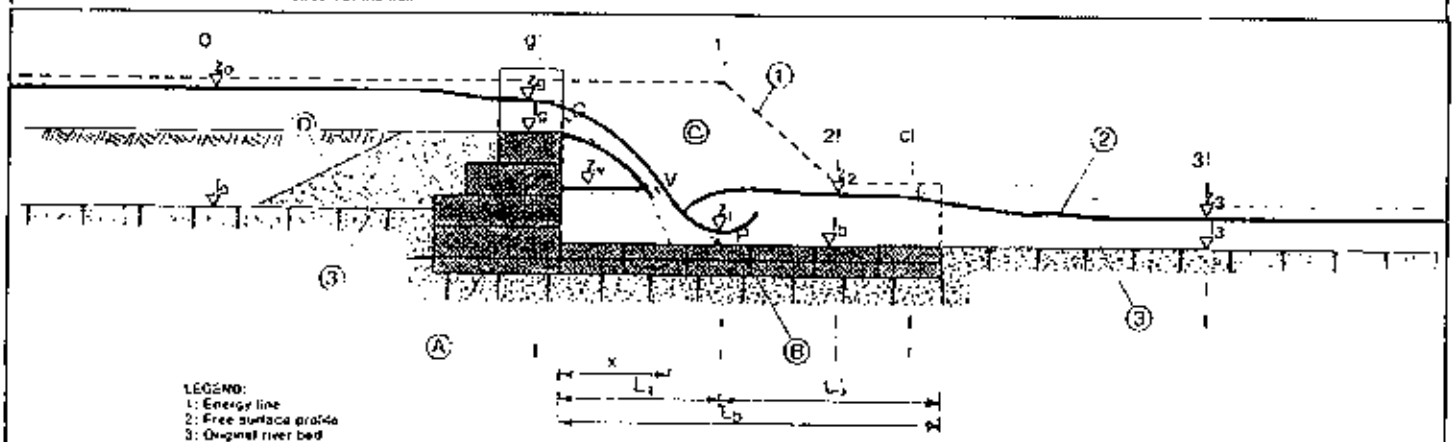
FIGURE 16

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
z': Weir wings elevation
SECTIONS:
0: Section upstream of the weir
c: 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
z': Elevation of wings of weir
SECTIONS:
0: Section upstream of the weir
c: Section at crest
1: Section at max. bed scour
2: Section at sequent depth
c': Section at cavitation
3: Section downstream of the weir

TYPICAL SECTION OF GRADE-CONTROL STRUCTURE WITH DOWNSTREAM APRON

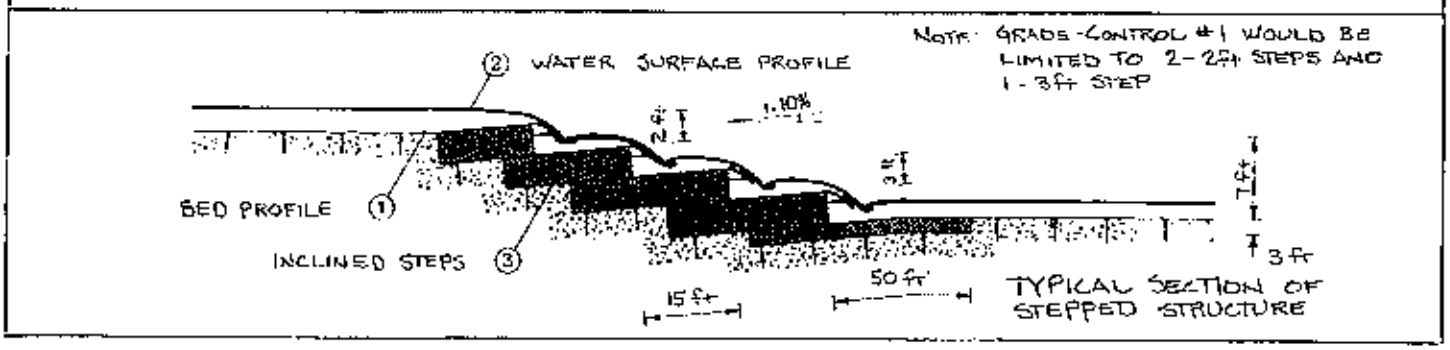


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FIGURE 17

CROSS SECTION OF PROPOSED GABION STRUCTURES

5X

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

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,

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

1. Simons, Li & Associates, "Flecha Caida Flood Improvement Study - Phase I 100-year Peak Discharge Magnitudes and Floodplain Mapping," January 28, 1986.

2. Pima County, Department of Transportation and Flood Control District, "Hydrology Manual for Engineering Design and Floodplain Management within Pima County, Arizona," September, 1979.

3. U.S. Army Corps of Engineers, "HEC-2: Water Surface Profiles, Users Manual," September,1982.

4. Pima County Department of Transportation and Flood Control District/City of Tucson, "Stormwater Detention/Retention Manual," July, 1987.

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.

6. Federal Highway Administration, U.S. Department of Transportation, "Hydraulic Design of Energy Dissipators for Culverts and Channels," Hydraulic Engineering Circular No. 14, September, 1983.

7. United States Department of the Interior, Bureau of Reclamation, "Hydraulic Design of Stilling Basins and Energy Dissipators," Engineering Monograph No. 25, January 1978.

8. Simons, Li and Associates, Inc., "Project Report: Hydraulic Model Study of Local Scour Downstream of Rigid Grade Control Structures," February 1986.

9. Pima County, Arizona, "Floodplain and Erosion Hazard Management Ordinance No. 1988-FC2", passed and adopted by the Board of Supervisors sitting as the Board of Directors of the Pima County Flood Control District, December 6, 1988.

AZPDOT07/TC/sR01.WP5
(PAZ-PDOT-07)

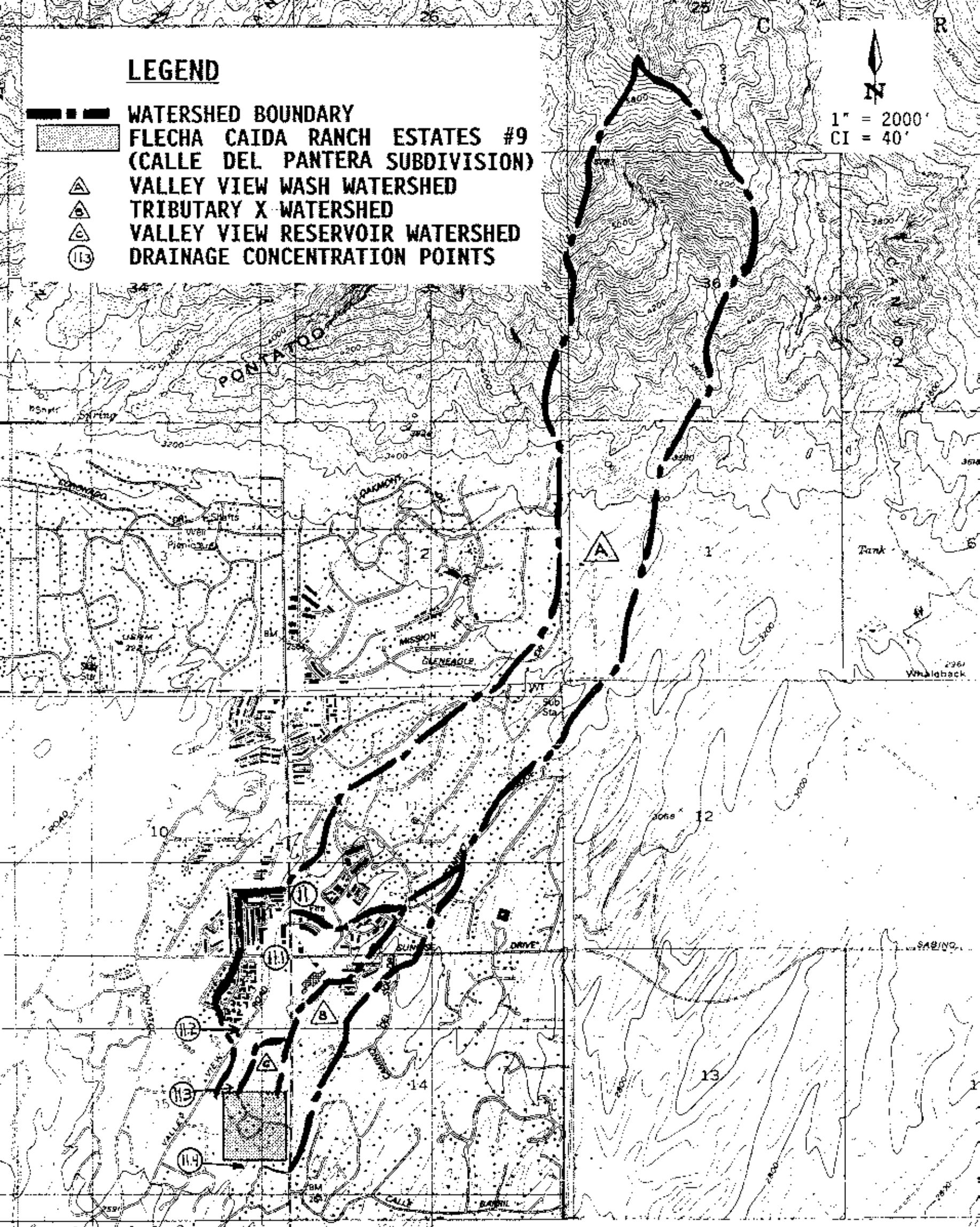
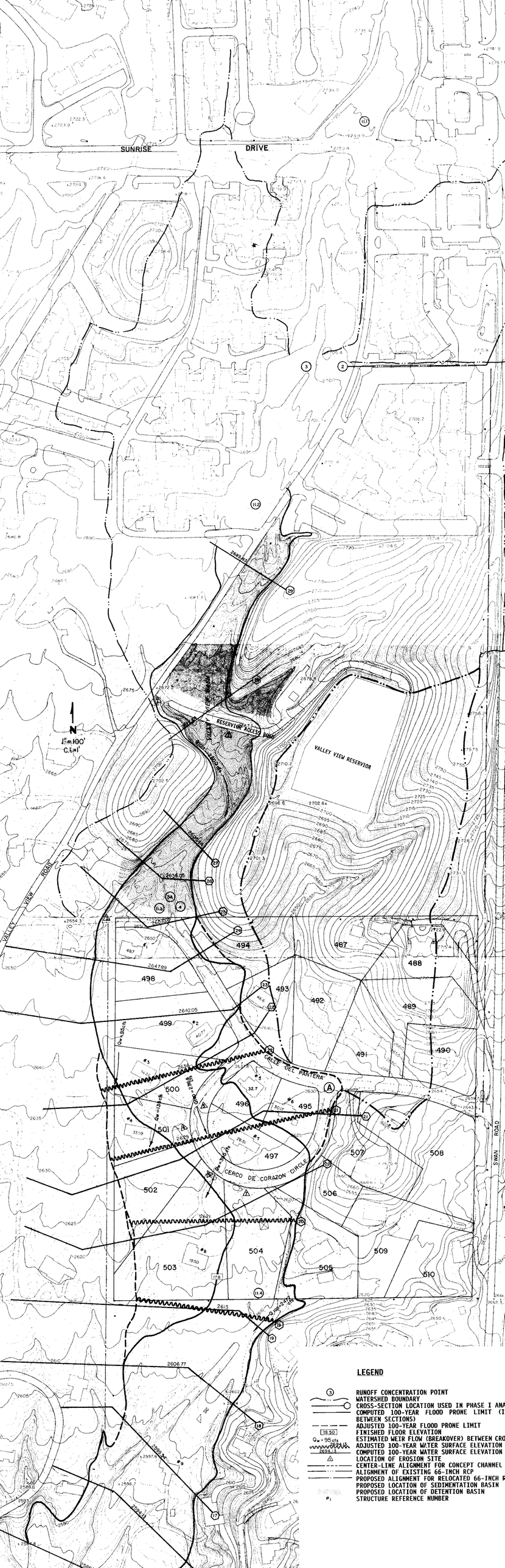


FIGURE 1

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

Swan Rd

sla SIMONS, LI & ASSOCIATES, INC.


SUNRISE DRIVE
VALLEY VIEW ROAD
SWAN ROAD
VALLEY VIEW RESERVOIR
RESERVOIR ACCESS ROAD
CALLE DEL PANTERA
CERCO DE CORAZON CIRCLE
N
1" = 100'
C.I. = 1'
LEGEND
RUNOFF CONCENTRATION POINT
WATERSHED BOUNDARY
CROSS-SECTION LOCATION USED IN PHASE I ANA
COMPUTED 100-YEAR FLOOD PRONE LIMIT (I
BETWEEN SECTIONS)
ADJUSTED 100-YEAR FLOOD PRONE LIMIT
FINISHED FLOOR ELEVATION
ESTIMATED WEIR FLOW (BREAKOVER) BETWEEN CRO
ADJUSTED 100-YEAR WATER SURFACE ELEVATION
COMPUTED 100-YEAR WATER SURFACE ELEVATION
LOCATION OF EROSION SITE
CENTER-LINE ALIGNMENT FOR CONCEPT CHANNEL
ALIGNMENT OF EXISTING 66-INCH RCP
PROPOSED ALIGNMENT FOR RELOCATED 66-INCH R
PROPOSED LOCATION OF SEDIMENTATION BASIN
PROPOSED LOCATION OF DETENTION BASIN
STRUCTURE REFERENCE NUMBER

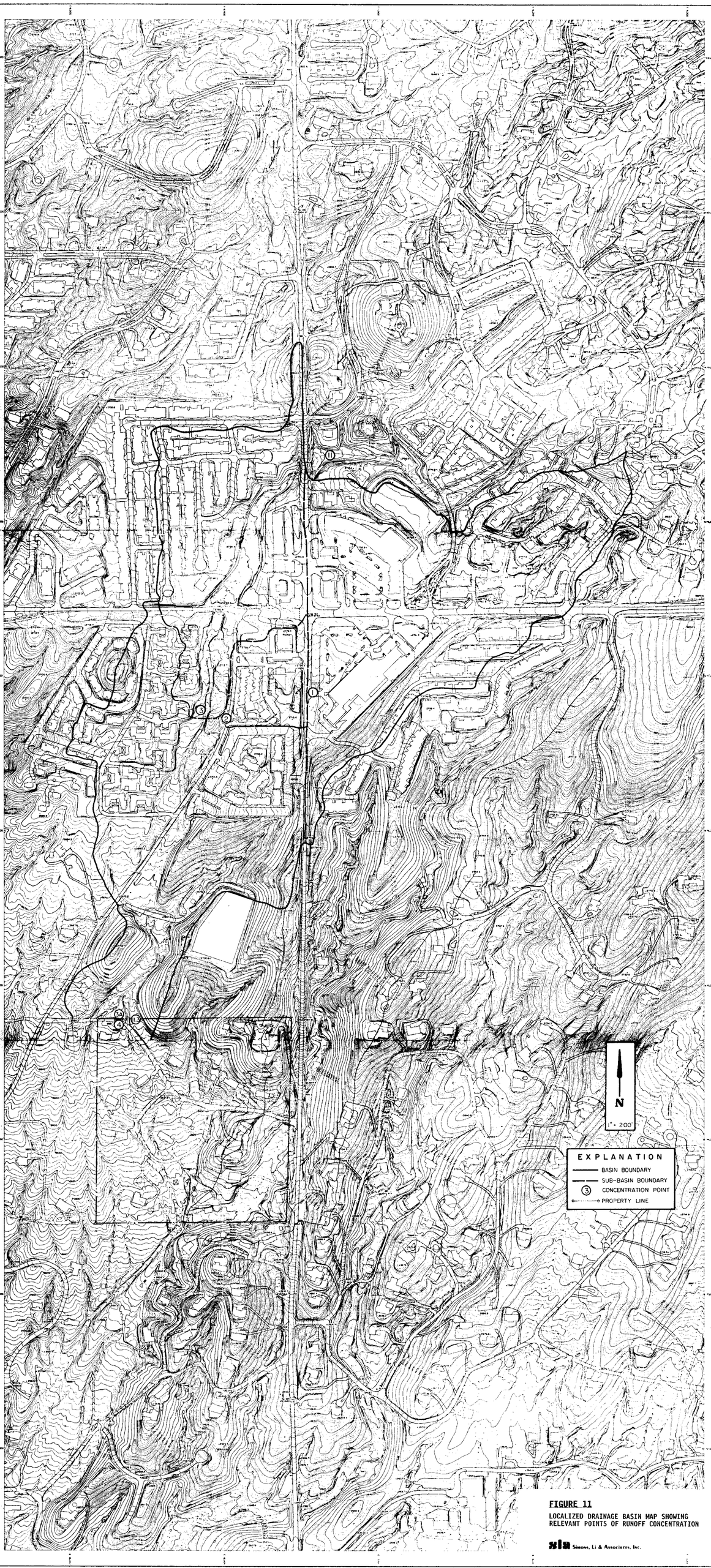
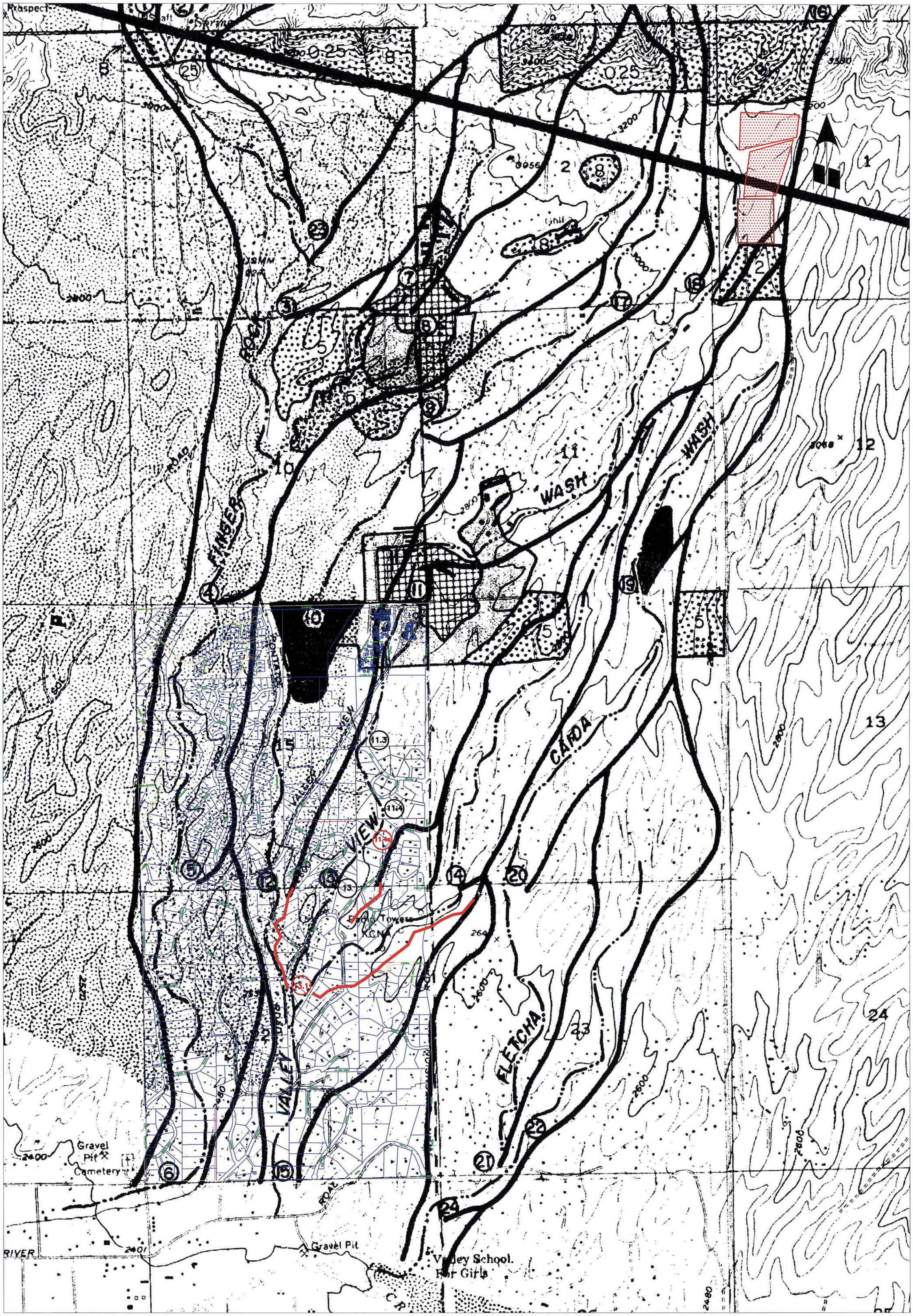


FIGURE 11

LOCALIZED DRAINAGE BASIN MAP SHOWING RELEVANT POINTS OF RUNOFF CONCENTRATION

Simons, Li & Associates, Inc.

p131415E-PC.dwg



800' 0' 800' 1600'
SCALE: 1 INCH = 800 FEET

Legend

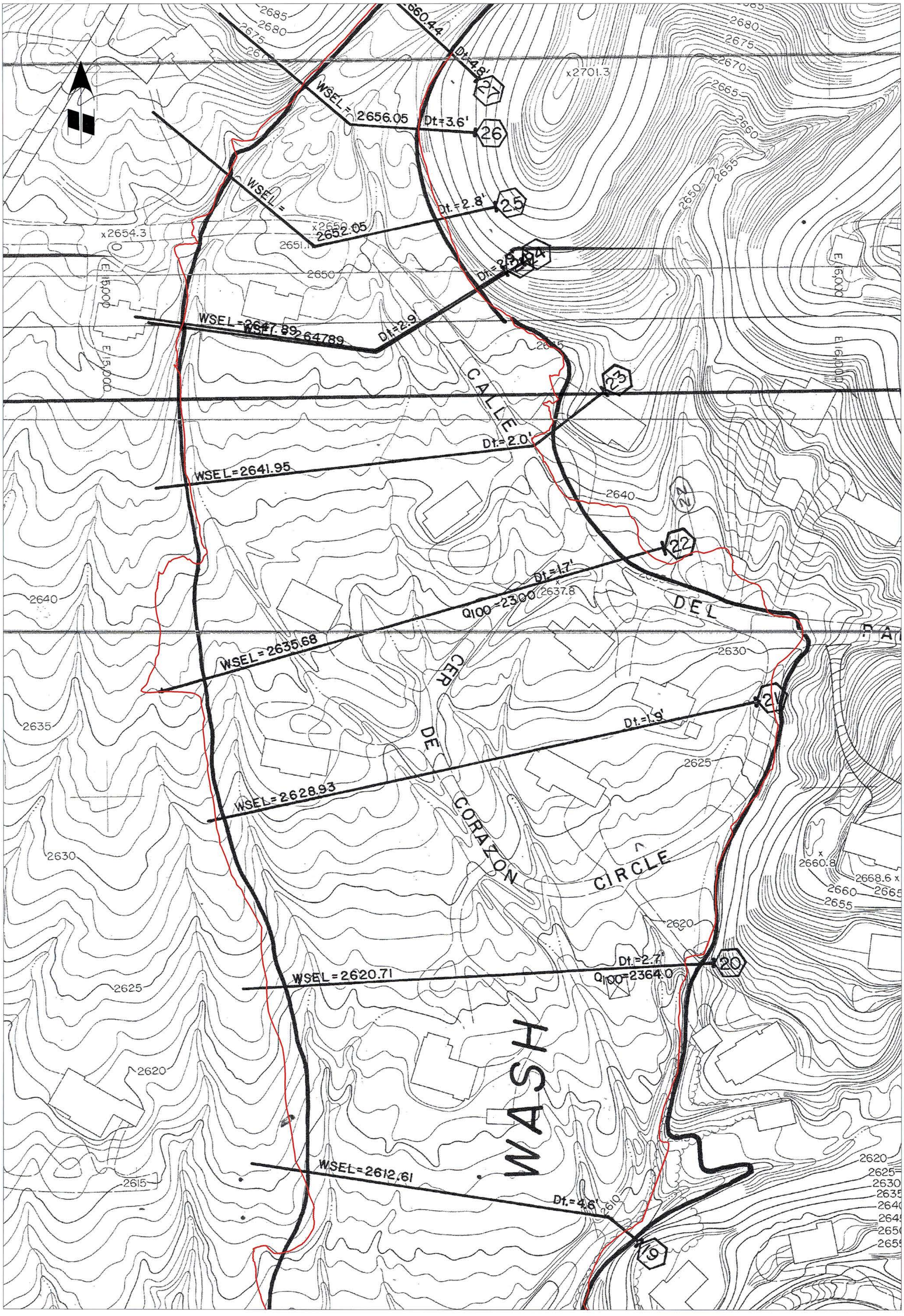
- ⑥ 1984 and 1989 Drainage Concentration Points
- ⑥ 2007 Drainage Concentration Point Additions
- 1984 and 1989 Drainage Basin Boundaries
- 2007 Drainage Basin Boundary/Impervious Revisions

Revised Drainage Basin Exhibit

1984 and 1989 Drainage Basin Boundaries
with 2007 Additions/Revisions

JE Fuller / Hydrology & Geomorphology, Inc.

FP comparison.dwg



60' 0' 60' 120'

SCALE: 1 INCH = 60 FEET
Contour Interval (NGVD 29) = 1 FEET

Legend

- 6 — 1986 HEC-2 Cross Section with WSEL (NGVD 29)
- 1986 100-Year Floodplain Boundary
- 2007 100-Year Floodplain Boundary

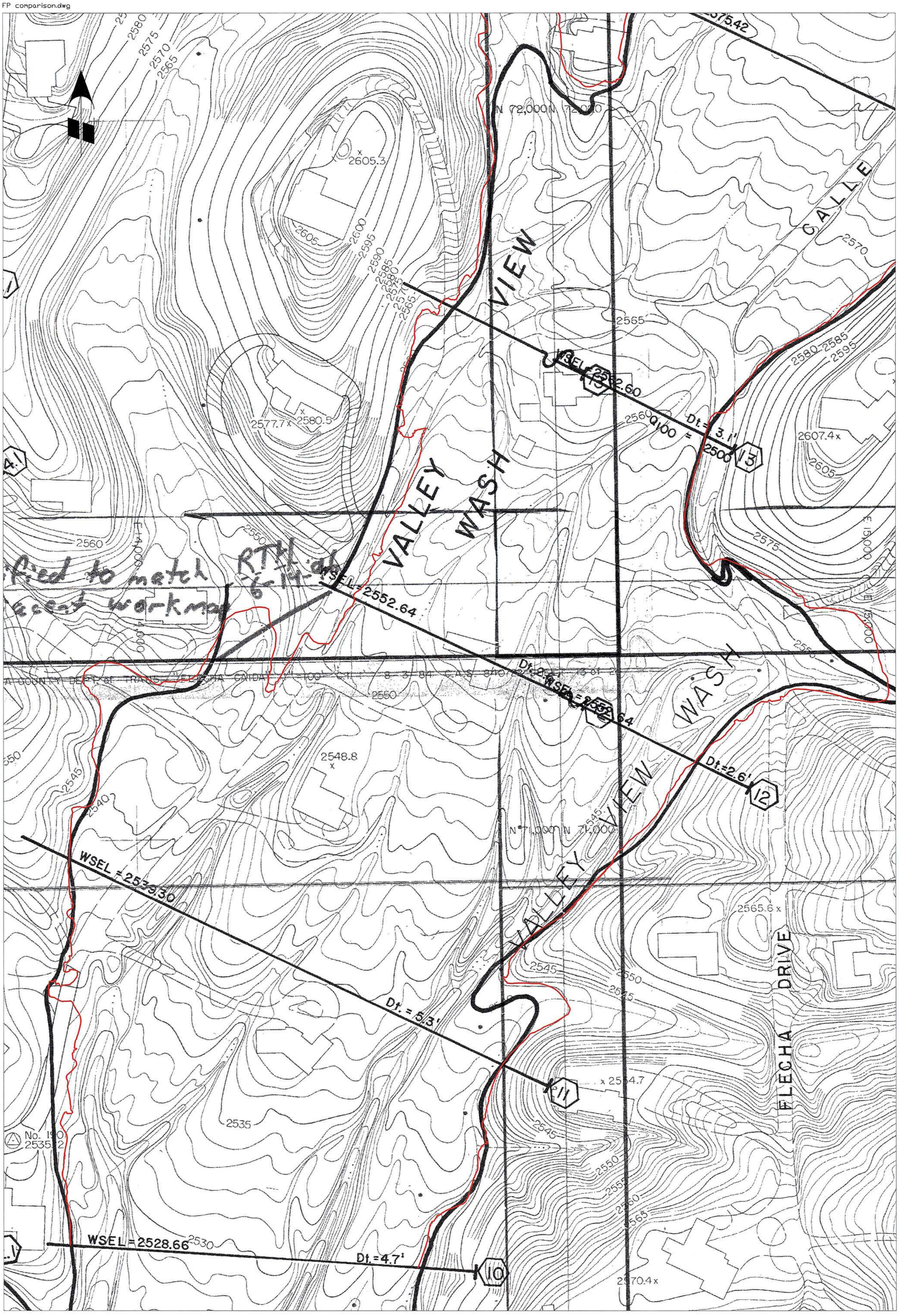
Flood Plain Comparison Exhibit

1986 versus 2007 (1984 base map)

Sheet 1 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

FP comparison.dwg



60' 0' 60' 120'

SCALE: 1 INCH = 60 FEET
Contour Interval (NGVD 29) = 1 FEET

Legend

- 6 — 1986 HEC-2 Cross Section with WSEL (NGVD 29)
- 1986 100-Year Floodplain Boundary
- 2007 100-Year Floodplain Boundary

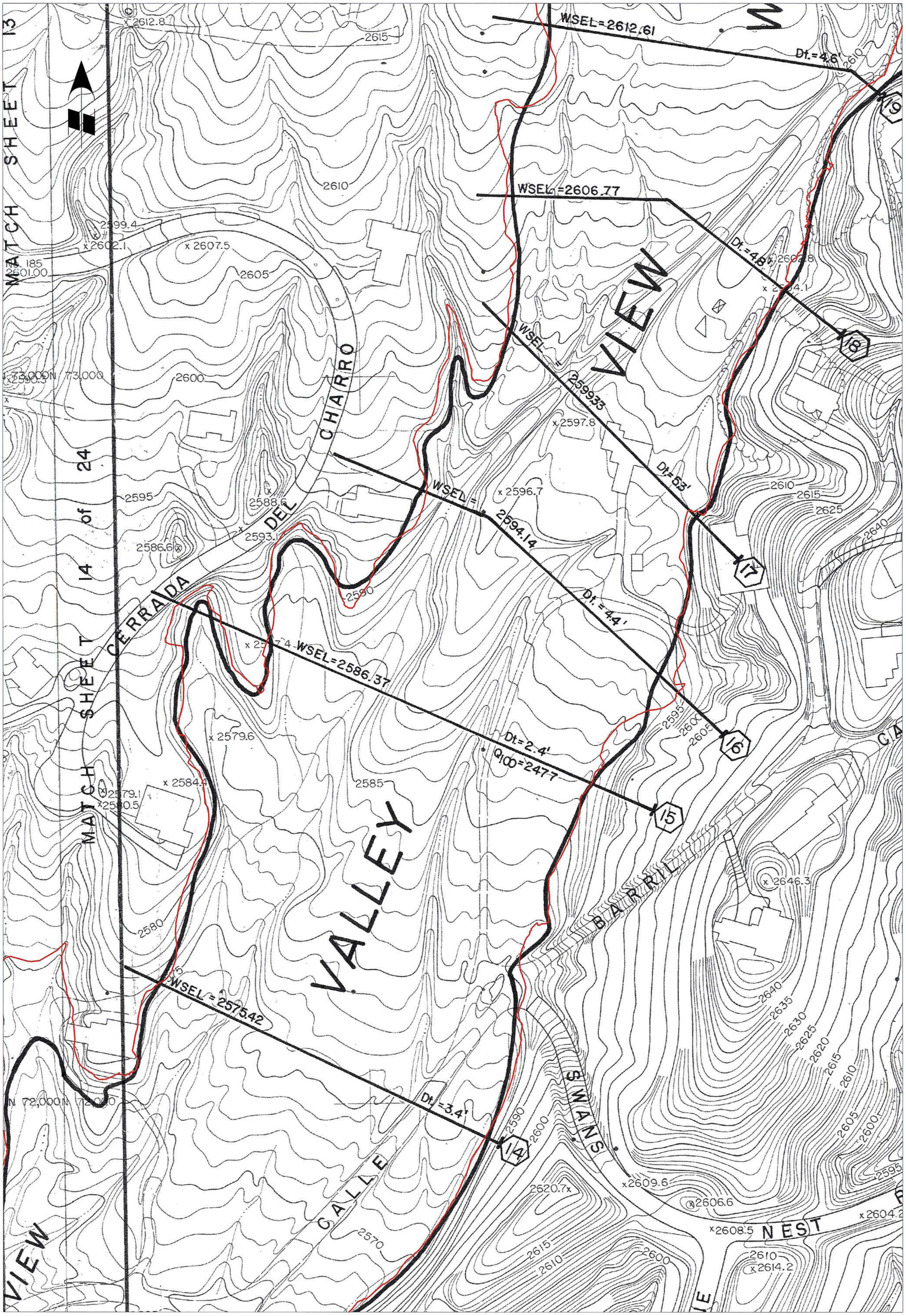
Flood Plain Comparison Exhibit

1986 versus 2007 (1984 base map)

Sheet 3 of 3

JE Fuller / Hydrology & Geomorphology, Inc.

FP comparison.dwg



Legend

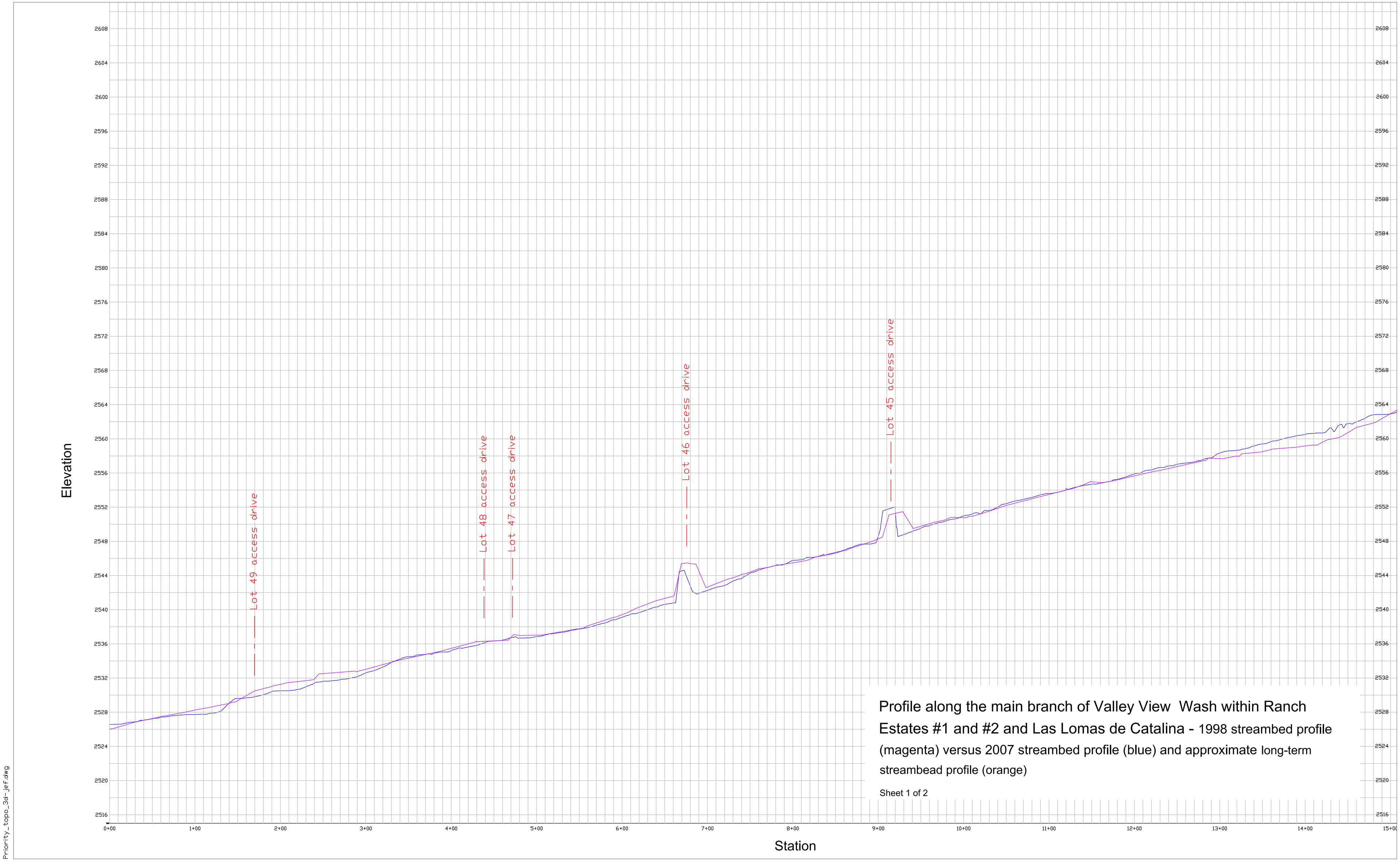
- 6 — 1986 HEC-2 Cross Section with WSEL (NGVD 29)
- 1986 100-Year Floodplain Boundary
- 2007 100-Year Floodplain Boundary

Flood Plain Comparison Exhibit

1986 versus 2007 (1984 base map)

Sheet 2 of 3

JE Fuller / Hydrology & Geomorphology, Inc.



Profile along the main branch of Valley View Wash within Ranch Estates #1 and #2 and Las Lomas de Catalina - 1998 streambed profile (magenta) versus 2007 streambed profile (blue) and approximate long-term streambead profile (orange)

Sheet 1 of 2

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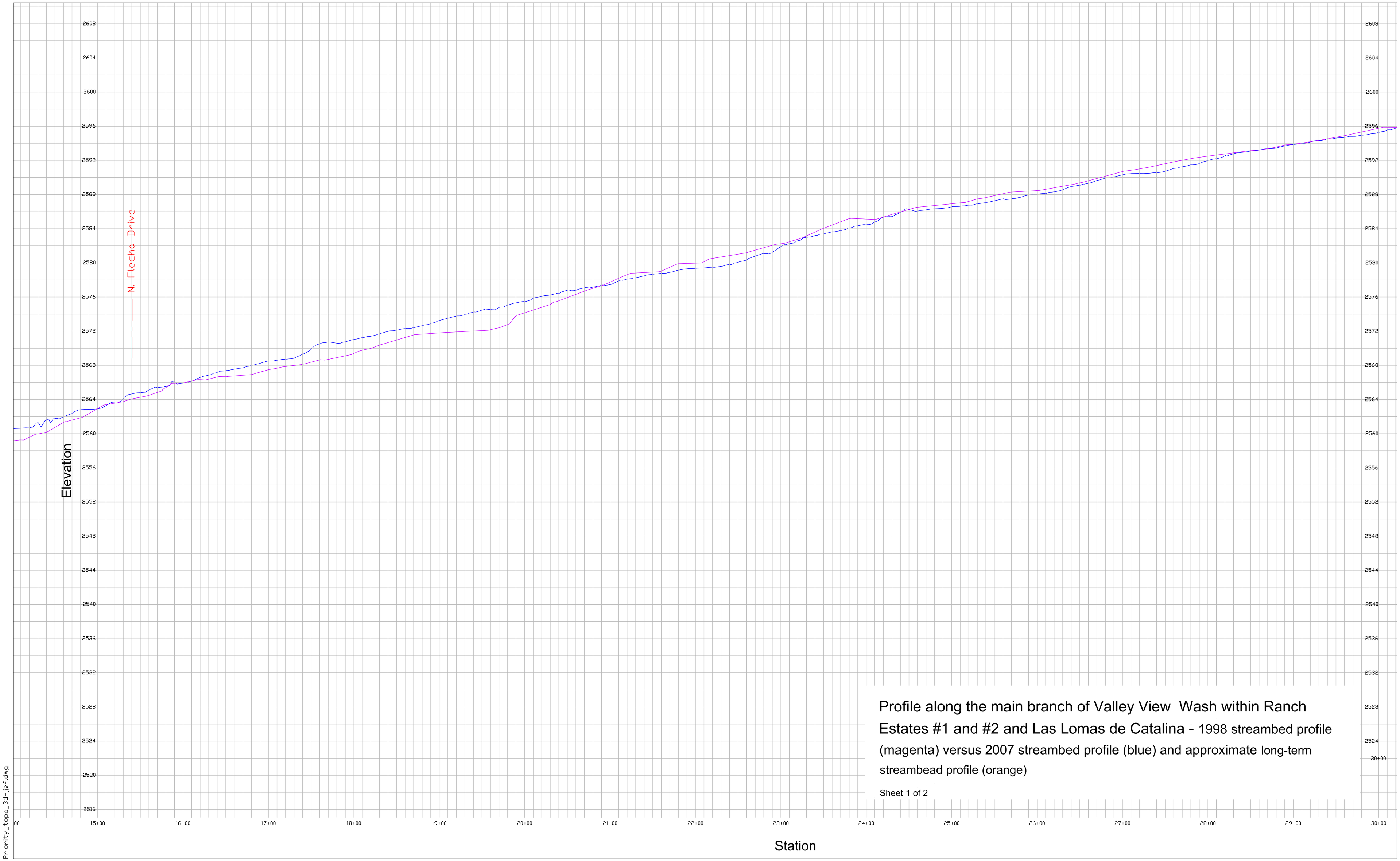
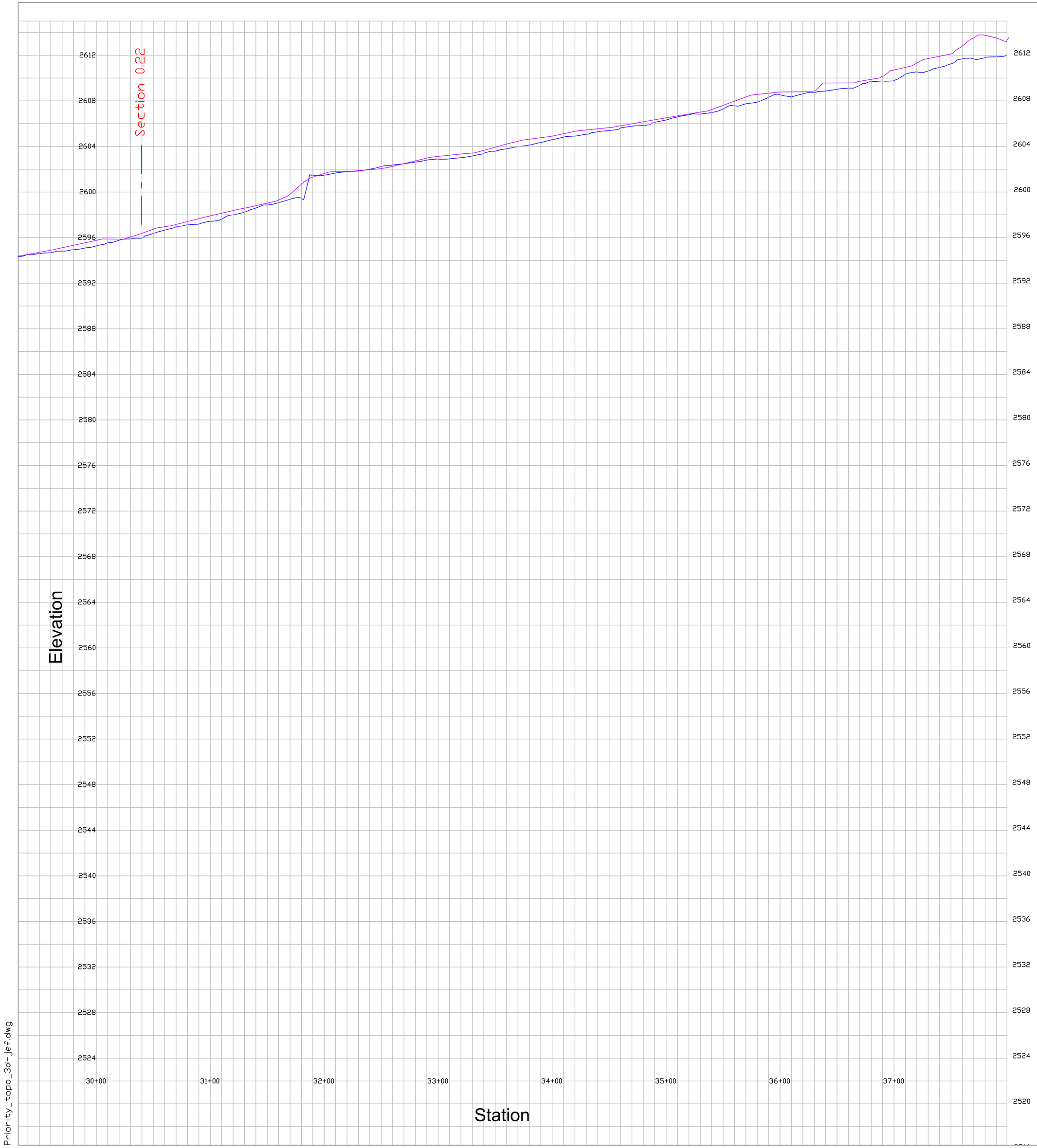


Profile along the main branch of Valley View Wash within Ranch Estates #1 and #2 and Las Lomas de Catalina - 1998 streambed profile (magenta) versus 2007 streambed profile (blue) and approximate long-term streambead profile (orange)

Sheet 1 of 2

JE Fuller / Hydrology & Geomorphology, Inc.



Profile along the main branch of Valley View Wash within Ranch Estates #1 and #2 and Las Lomas de Catalina - 1998 streambed profile (magenta) versus 2007 streambed profile (blue) and approximate long-term streambead profile (orange)

Sheet 1 of 2

Appendix B

NOAA Atlas 2, Volume VIII (1973) Hydrologic Data Sheets

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11 (1986)

Watershed Area (A): 908 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.047
5600	3800	1800	4000	0.450	0.047
3800	3400	400	2200	0.182	0.047
3400	3200	200	1900	0.105	0.047
3200	2740	460	12500	0.037	0.047
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 21700 ft

Length to Center of Gravity (Lca): 10850 ft

Mean Slope (Sc): 0.0670 ft/ft

Weighted Basin Factor (nb): 0.047

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.69	0%	2.69

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%

Impervious Surfaces: Percent 8%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	38	83	87.06	0.55
C	13	82	86.28	0.53
D	49	90	92.52	0.71

Weighted Runoff Coefficient (Cw): 0.652

100-year Peak Discharge (Q100): **2217** **cfs**

Time of Concentration (Tc): 38 min.

Rainfall Intensity (i) at Tc: 3.71 in/hr

Runoff Supply Rate (q) at Tc: 2.42 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.1 (1986)

Watershed Area (A): 927 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.047
5600	3800	1800	4000	0.450	0.047
3800	3400	400	2200	0.182	0.047
3400	3200	200	1900	0.105	0.047
3200	2740	460	12500	0.037	0.047
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 21700 ft

Length to Center of Gravity (Lca): 10850 ft

Mean Slope (Sc): 0.0670 ft/ft

Weighted Basin Factor (nb): 0.047

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.69	0%	2.69

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 8%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	38	83	87.06	0.55
C	13	82	86.28	0.53
D	49	90	92.52	0.71

Weighted Runoff Coefficient (Cw): 0.652

100-year Peak Discharge (Q100): **2263** **cfs**

Time of Concentration (Tc): 38 min.

Rainfall Intensity (i) at Tc: 3.71 in/hr

Runoff Supply Rate (q) at Tc: 2.42 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 13 (1986)

Watershed Area (A): 1239 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.045
5600	3800	1800	4000	0.450	0.045
3800	3400	400	2200	0.182	0.045
3400	3200	200	1900	0.105	0.045
3200	2740	460	12500	0.037	0.045
2740	2580	160	5600	0.029	0.045
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 27300 ft

Length to Center of Gravity (Lca): 13650 ft

Mean Slope (Sc): 0.0545 ft/ft

Weighted Basin Factor (nb): 0.045

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.68	0%	2.68

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	53	83	87.04	0.55
C	10	82	86.26	0.53
D	37	90	92.50	0.71

Weighted Runoff Coefficient (Cw): 0.648

100-year Peak Discharge (Q100): **2512** **cfs**

Time of Concentration (Tc): 49 min.

Rainfall Intensity (i) at Tc: 3.11 in/hr

Runoff Supply Rate (q) at Tc: 2.01 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 18 (1986)

Watershed Area (A): 610 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.055
5600	3800	1800	4000	0.450	0.055
3800	3400	400	2200	0.182	0.055
3400	3200	200	1900	0.105	0.055
3200	2980	220	4100	0.054	0.055
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 13300 ft

Length to Center of Gravity (Lca): 7000 ft

Mean Slope (Sc): 0.1339 ft/ft

Weighted Basin Factor (nb): 0.055

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.72	0%	2.72

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 3%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	8	83	87.12	0.55
C	19	82	86.34	0.53
D	73	90	92.57	0.72

Weighted Runoff Coefficient (Cw): 0.677

100-year Peak Discharge (Q100): **2175** **cfs**

Time of Concentration (Tc): 22 min.

Rainfall Intensity (i) at Tc: 5.22 in/hr

Runoff Supply Rate (q) at Tc: 3.54 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.1 (1986 updated)

Watershed Area (A): 927 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.047
5600	3800	1800	4000	0.450	0.047
3800	3400	400	2200	0.182	0.047
3400	3200	200	1900	0.105	0.047
3200	2740	460	12500	0.037	0.047
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 21700 ft

Length to Center of Gravity (Lca): 10850 ft

Mean Slope (Sc): 0.0670 ft/ft

Weighted Basin Factor (nb): 0.047

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.69	0%	2.69

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 9.0%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	38	83	87.06	0.55
C	13	82	86.28	0.53
D	49	90	92.52	0.71

Weighted Runoff Coefficient (Cw): 0.656

100-year Peak Discharge (Q100): **2279** **cfs**

Time of Concentration (Tc): 38 min.

Rainfall Intensity (i) at Tc: 3.72 in/hr

Runoff Supply Rate (q) at Tc: 2.44 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 13 (1986 updated)

Watershed Area (A): 1239 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.045
5600	3800	1800	4000	0.450	0.045
3800	3400	400	2200	0.182	0.045
3400	3200	200	1900	0.105	0.045
3200	2740	460	12500	0.037	0.045
2740	2580	160	5600	0.029	0.045
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 27300 ft

Length to Center of Gravity (Lca): 13650 ft

Mean Slope (Sc): 0.0545 ft/ft

Weighted Basin Factor (nb): 0.045

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.68	0%	2.68

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%

Impervious Surfaces: Percent 12.8%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	53	83	87.04	0.55
C	10	82	86.26	0.53
D	37	90	92.50	0.71

Weighted Runoff Coefficient (Cw): 0.650

100-year Peak Discharge (Q100): **2527** **cfs**

Time of Concentration (Tc): 49 min.

Rainfall Intensity (i) at Tc: 3.11 in/hr

Runoff Supply Rate (q) at Tc: 2.02 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 18 (1986 updated)

Watershed Area (A): 610 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.055
5600	3800	1800	4000	0.450	0.055
3800	3400	400	2200	0.182	0.055
3400	3200	200	1900	0.105	0.055
3200	2980	220	4100	0.054	0.055
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 13300 ft

Length to Center of Gravity (Lca): 7000 ft

Mean Slope (Sc): 0.1339 ft/ft

Weighted Basin Factor (nb): 0.055

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.25	0%	3.25
2-hour	n/a	3.03	0%	3.03
1-hour	n/a	2.72	0%	2.72

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 4.5%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	8	83	87.12	0.55
C	19	82	86.34	0.53
D	73	90	92.57	0.72

Weighted Runoff Coefficient (Cw): 0.681

100-year Peak Discharge (Q100): **2192** **cfs**

Time of Concentration (Tc): 22 min.

Rainfall Intensity (i) at Tc: 5.23 in/hr

Runoff Supply Rate (q) at Tc: 3.57 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 14 (1986 duplicate effective)

Watershed Area (A): 239 acres

Watershed Type: Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s limit	d/s limit	Hi (ft)	Li (ft)	Si (ft/ft)	Factor nb
3000	2800	200	4200	0.048	0.035
2800	2610	190	6600	0.029	0.035
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 10800 ft

Length to Center of Gravity (Lca): 5000 ft

Mean Slope (Sc): 0.0345 ft/ft

Weighted Basin Factor (nb): 0.035

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.17	0%	3.17
2-hour	n/a	2.96	0%	2.96
1-hour	n/a	2.63	0%	2.63

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 20%

Impervious Surfaces: Percent 15.0%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic Group	Group %	Curve Number Normal	Curve Number Adjusted	Runoff Coefficient
A				
B	100	83	86.93	0.54
C		82		
D		90		

Weighted Runoff Coefficient (Cw): 0.601

100-year Peak Discharge (Q100): **737** **cfs**

Time of Concentration (Tc): 21 min.

Rainfall Intensity (i) at Tc: 5.09 in/hr

Runoff Supply Rate (q) at Tc: 3.06 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

Appendix C

NOAA Atlas 14, Volume I (2006) Hydrologic Data Sheets

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11

Watershed Area (A): 908 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.047
5600	3800	1800	4000	0.450	0.047
3800	3400	400	2200	0.182	0.047
3400	3200	200	1900	0.105	0.047
3200	2740	460	12500	0.037	0.047
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 21700 ft

Length to Center of Gravity (Lca): 10850 ft

Mean Slope (Sc): 0.0670 ft/ft

Weighted Basin Factor (nb): 0.047

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%

Impervious Surfaces: Percent 9%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	38	83	87.63	0.60
C	13	82	86.87	0.58
D	49	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.695

100-year Peak Discharge (Q100): **2802** **cfs**

Time of Concentration (Tc): 35 min.

Rainfall Intensity (i) at Tc: 4.40 in/hr

Runoff Supply Rate (q) at Tc: 3.06 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.1

Watershed Area (A): 927 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.047
5600	3800	1800	4000	0.450	0.047
3800	3400	400	2200	0.182	0.047
3400	3200	200	1900	0.105	0.047
3200	2740	460	12500	0.037	0.047
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 21700 ft

Length to Center of Gravity (Lca): 10850 ft

Mean Slope (Sc): 0.0670 ft/ft

Weighted Basin Factor (nb): 0.047

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 9%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	38	83	87.63	0.60
C	13	82	86.87	0.58
D	49	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.695

100-year Peak Discharge (Q100): **2861** **cfs**

Time of Concentration (Tc): 35 min.

Rainfall Intensity (i) at Tc: 4.40 in/hr

Runoff Supply Rate (q) at Tc: 3.06 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3

Watershed Area (A): 1034 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 24600 ft

Length to Center of Gravity (Lca): 12300 ft

Mean Slope (Sc): 0.0590 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	44	83	87.63	0.60
C	12	82	86.87	0.58
D	44	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.697

100-year Peak Discharge (Q100): **2916** **cfs**

Time of Concentration (Tc): 40 min.

Rainfall Intensity (i) at Tc: 4.01 in/hr

Runoff Supply Rate (q) at Tc: 2.80 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3a

Watershed Area (A): 1045 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
2660	2640	20	770	0.026	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 25370 ft

Length to Center of Gravity (Lca): 12685 ft

Mean Slope (Sc): 0.0572 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	45	83	87.63	0.60
C	12	82	86.87	0.58
D	43	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.696

100-year Peak Discharge (Q100): **2855** **cfs**

Time of Concentration (Tc): 42 min.

Rainfall Intensity (i) at Tc: 3.89 in/hr

Runoff Supply Rate (q) at Tc: 2.71 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3b

Watershed Area (A): 1055 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
2660	2640	20	770	0.026	0.046
2640	2620	20	770	0.026	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 26140 ft

Length to Center of Gravity (Lca): 13070 ft

Mean Slope (Sc): 0.0556 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	46	83	87.63	0.60
C	11	82	86.87	0.58
D	43	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.696

100-year Peak Discharge (Q100): **2804** **cfs**

Time of Concentration (Tc): 43 min.

Rainfall Intensity (i) at Tc: 3.79 in/hr

Runoff Supply Rate (q) at Tc: 2.64 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.4

Watershed Area (A): 1073 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
2660	2640	20	770	0.026	0.046
2640	2620	20	770	0.026	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 26140 ft

Length to Center of Gravity (Lca): 13070 ft

Mean Slope (Sc): 0.0556 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	47	83	87.63	0.60
C	11	82	86.87	0.58
D	42	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.695

100-year Peak Discharge (Q100): **2844** **cfs**

Time of Concentration (Tc): 43 min.

Rainfall Intensity (i) at Tc: 3.78 in/hr

Runoff Supply Rate (q) at Tc: 2.63 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.4a

Watershed Area (A): 1189 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
2660	2640	20	770	0.026	0.046
2640	2620	20	770	0.026	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 26140 ft

Length to Center of Gravity (Lca): 13070 ft

Mean Slope (Sc): 0.0556 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	52	83	87.63	0.60
C	10	82	86.87	0.58
D	38	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.690

100-year Peak Discharge (Q100): **3119** **cfs**

Time of Concentration (Tc): 44 min.

Rainfall Intensity (i) at Tc: 3.77 in/hr

Runoff Supply Rate (q) at Tc: 2.60 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 13

Watershed Area (A): 1239 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.045
5600	3800	1800	4000	0.450	0.045
3800	3400	400	2200	0.182	0.045
3400	3200	200	1900	0.105	0.045
3200	2740	460	12500	0.037	0.045
2740	2580	160	5600	0.029	0.045
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 27300 ft

Length to Center of Gravity (Lca): 13650 ft

Mean Slope (Sc): 0.0545 ft/ft

Weighted Basin Factor (nb): 0.045

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%

Impervious Surfaces: Percent 12.8%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	53	83	87.63	0.60
C	10	82	86.87	0.58
D	37	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.691

100-year Peak Discharge (Q100): **3219** **cfs**

Time of Concentration (Tc): 44 min.

Rainfall Intensity (i) at Tc: 3.73 in/hr

Runoff Supply Rate (q) at Tc: 2.58 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 18

Watershed Area (A): 610 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.055
5600	3800	1800	4000	0.450	0.055
3800	3400	400	2200	0.182	0.055
3400	3200	200	1900	0.105	0.055
3200	2980	220	4100	0.054	0.055
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 13300 ft

Length to Center of Gravity (Lca): 7000 ft

Mean Slope (Sc): 0.1339 ft/ft

Weighted Basin Factor (nb): 0.055

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.70	0%	3.70
2-hour	n/a	3.50	0%	3.50
1-hour	n/a	3.11	0%	3.11

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 4.5%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	8	83	87.81	0.61
C	19	82	87.06	0.59
D	73	90	93.10	0.76

Weighted Runoff Coefficient (Cw): 0.728

100-year Peak Discharge (Q100): **2823** **cfs**

Time of Concentration (Tc): 19 min.

Rainfall Intensity (i) at Tc: 6.30 in/hr

Runoff Supply Rate (q) at Tc: 4.59 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3 (2-year)

Watershed Area (A): 1034 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 24600 ft

Length to Center of Gravity (Lca): 12300 ft

Mean Slope (Sc): 0.0590 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	1.53	0%	1.53
2-hour	n/a	1.46	0%	1.46
1-hour	n/a	1.28	0%	1.28

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.91 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	44	83	80.93	0.16
C	12	82	79.95	0.14
D	44	90	87.82	0.33

Weighted Runoff Coefficient (Cw): 0.314

2-year Peak Discharge (Q2): **268** **cfs**

Time of Concentration (Tc): 104 min.

Rainfall Intensity (i) at Tc: 0.82 in/hr

Runoff Supply Rate (q) at Tc: 0.26 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3 (5-year)

Watershed Area (A): 1034 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 24600 ft

Length to Center of Gravity (Lca): 12300 ft

Mean Slope (Sc): 0.0590 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	1.97	0%	1.97
2-hour	n/a	1.89	0%	1.89
1-hour	n/a	1.68	0%	1.68

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.93 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	44	83	83.71	0.31
C	12	82	82.82	0.29
D	44	90	89.95	0.49

Weighted Runoff Coefficient (Cw): 0.451

5-year Peak Discharge (Q5): **680** **cfs**

Time of Concentration (Tc): 71 min.

Rainfall Intensity (i) at Tc: 1.45 in/hr

Runoff Supply Rate (q) at Tc: 0.65 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3 (10-year)

Watershed Area (A): 1034 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 24600 ft

Length to Center of Gravity (Lca): 12300 ft

Mean Slope (Sc): 0.0590 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	2.31	0%	2.31
2-hour	n/a	2.22	0%	2.22
1-hour	n/a	1.97	0%	1.97

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.94 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	44	83	85.02	0.39
C	12	82	84.18	0.37
D	44	90	90.96	0.58

Weighted Runoff Coefficient (Cw): 0.527

10-year Peak Discharge (Q10): **1092** **cfs**

Time of Concentration (Tc): 59 min.

Rainfall Intensity (i) at Tc: 1.99 in/hr

Runoff Supply Rate (q) at Tc: 1.05 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 11.3 (25-year)

Watershed Area (A): 1034 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.046
5600	3800	1800	4000	0.450	0.046
3800	3400	400	2200	0.182	0.046
3400	3200	200	1900	0.105	0.046
3200	2740	460	12500	0.037	0.046
2740	2660	80	2900	0.028	0.046
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 24600 ft

Length to Center of Gravity (Lca): 12300 ft

Mean Slope (Sc): 0.0590 ft/ft

Weighted Basin Factor (nb): 0.046

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	2.78	0%	2.78
2-hour	n/a	2.67	0%	2.67
1-hour	n/a	2.37	0%	2.37

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%/20%

Impervious Surfaces: Percent 12%

Runoff Coefficient: 0.95 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	44	83	86.31	0.49
C	12	82	85.50	0.47
D	44	90	91.94	0.66

Weighted Runoff Coefficient (Cw): 0.609

25-year Peak Discharge (Q25): **1740** **cfs**

Time of Concentration (Tc): 49 min.

Rainfall Intensity (i) at Tc: 2.74 in/hr

Runoff Supply Rate (q) at Tc: 1.67 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 13.1

Watershed Area (A): 1608 acres

Watershed Type: Mtn (dev)/Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s	d/s	Hi	Li	Si	Factor
limit	limit	(ft)	(ft)	(ft/ft)	nb
6080	5600	480	1100	0.436	0.045
5600	3800	1800	4000	0.450	0.045
3800	3400	400	2200	0.182	0.045
3400	3200	200	1900	0.105	0.045
3200	2740	460	12500	0.037	0.045
2740	2580	160	5600	0.029	0.045
2580	2540	40	2150	0.019	0.045
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 29450 ft

Length to Center of Gravity (Lca): 14725 ft

Mean Slope (Sc): 0.0493 ft/ft

Weighted Basin Factor (nb): 0.045

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 30%

Impervious Surfaces: Percent 12.9%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic	Group	Curve Number		Runoff
Group	%	Normal	Adjusted	Coefficient
A				
B	64	83	87.63	0.60
C	8	82	86.87	0.58
D	29	90	92.96	0.75

Weighted Runoff Coefficient (Cw): 0.686

100-year Peak Discharge (Q100): **3797** **cfs**

Time of Concentration (Tc): 50 min.

Rainfall Intensity (i) at Tc: 3.41 in/hr

Runoff Supply Rate (q) at Tc: 2.34 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

HYDROLOGIC DATA SHEET

Watercourse or Project Name: Flecha Caida Flood Improvement Study

Drainage Concentration Point: 14 (NOAA14)

Watershed Area (A): 239 acres

Watershed Type: Foothills(dev)

Incremental Changes along Primary Watercourse by Reach					
Reach Elevations		Height	Length	Slope	Basin
u/s limit	d/s limit	Hi (ft)	Li (ft)	Si (ft/ft)	Factor nb
3000	2800	200	4200	0.048	0.035
2800	2610	190	6600	0.029	0.035
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000
0	0	0	0	0.000	0.000

Length of Watercourse (Lc): 10800 ft

Length to Center of Gravity (Lca): 5000 ft

Mean Slope (Sc): 0.0345 ft/ft

Weighted Basin Factor (nb): 0.035

Rainfall Data				
Storm Event	Mapped Values (in)	Computed Values (in)	Areal Reduction %	Reduced Values (in)
3-hour	n/a	3.57	0%	3.57
2-hour	n/a	3.39	0%	3.39
1-hour	n/a	3.00	0%	3.00

Cover Type(s): Mtn./Desert Brush Mix

Cover Density (pervious areas): 20%

Impervious Surfaces: Percent 15.0%

Runoff Coefficient: 0.96 (CN constant at 99)

Soils Data				
Hydrologic Group	Group %	Curve Number Normal	Curve Number Adjusted	Runoff Coefficient
A				
B	100	83	87.63	0.60
C		82		
D		90		

Weighted Runoff Coefficient (Cw): 0.651

100-year Peak Discharge (Q100): **962** **cfs**

Time of Concentration (Tc): 19 min.

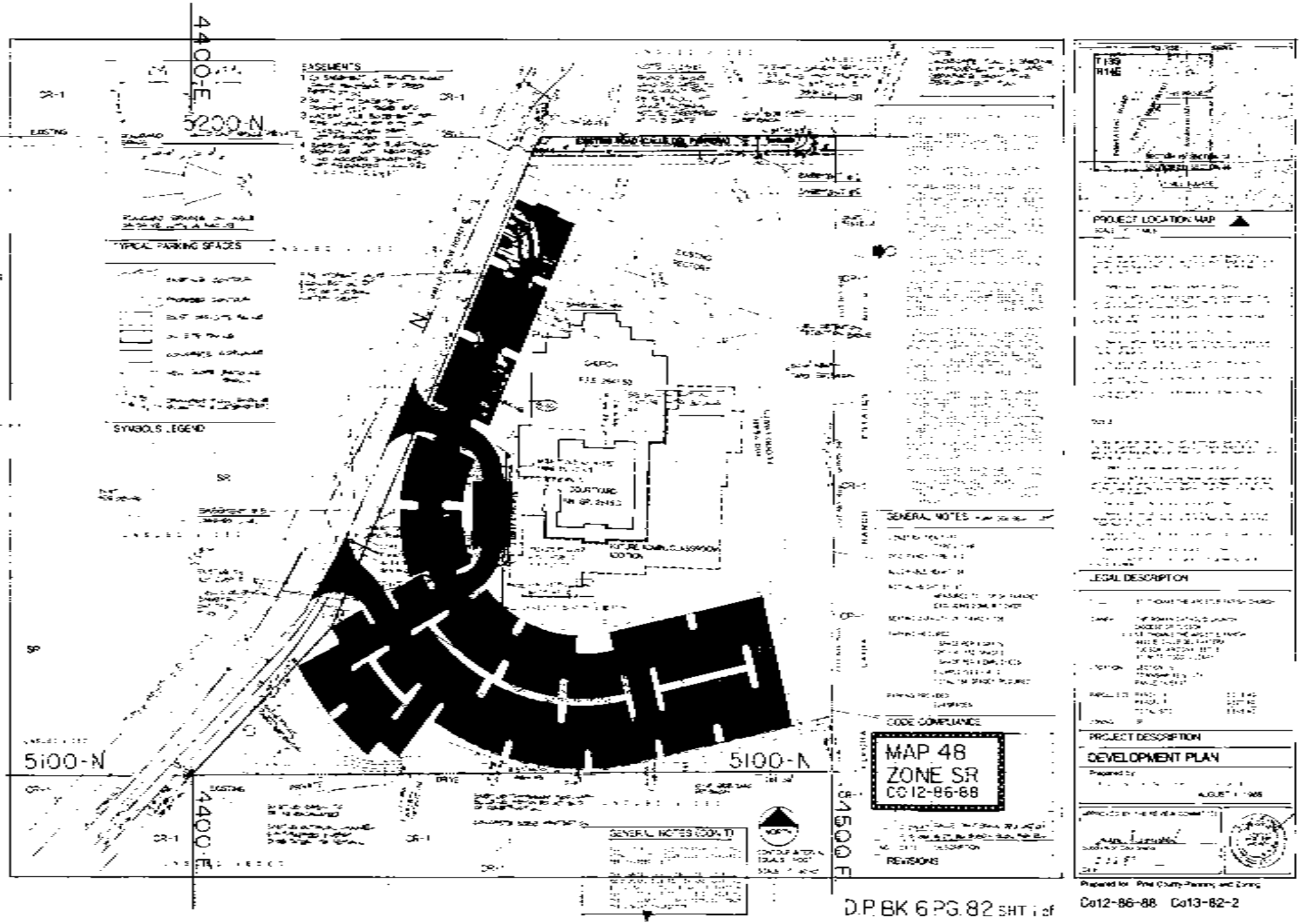
Rainfall Intensity (i) at Tc: 6.13 in/hr

Runoff Supply Rate (q) at Tc: 3.99 in/hr

For Return Intervals Other Than Q100		
25-year =	n/c	cfs
10-year =	n/c	cfs
5-year =	n/c	cfs
2-year =	n/c	cfs

Appendix D

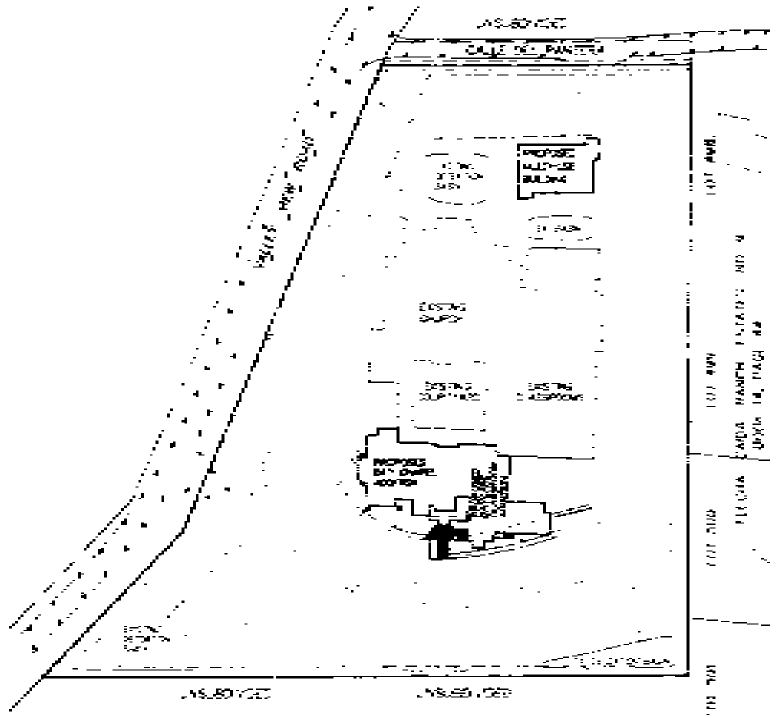
Development Plans for St. Thomas Church


4400-E
5200-N
EASEMENTS
TYPICAL PARKING SPACES
SYMBOLS LEGEND
5100-N
5100-N
4400-E
4500-E
GENERAL NOTES (CONT'D)
GENERAL NOTES
CODE COMPLIANCE
MAP 48
ZONE SR
CO12-86-88
REVISIONS
PROJECT LOCATION MAP
LEGAL DESCRIPTION
PROJECT DESCRIPTION
DEVELOPMENT PLAN
D.P. BK 6 PG. 82 SHT 1 of
Co12-86-88 Co13-82-2

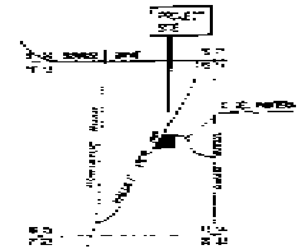
DEVELOPMENT PLAN
FOR
ST. THOMAS THE APOSTLE PARISH CHURCH
DAY CHAPEL & CLASSROOMS

GENERAL NOTES

PERMITTING NOTES



SITE PLAN



LOCATION MAP

LEGEND

PARKING CALCULATIONS

BUILDING SQUARE FOOTAGE

CODE COMPLIANCE

OWNER/DEVELOPER

MAP 48
ZONE SR
5150 N. Valley View Rd.
C012-96-90

Co 12-96-90
TAX CODE No. 109-13-0099
Co12-88-89
Co12-82-2
Co DW - 85-27
Co DW - 89-9

DEVELOPMENT PLAN
FOR
ST. THOMAS THE APOSTLE PARISH CHURCH
DAY CHAPEL & CLASSROOMS

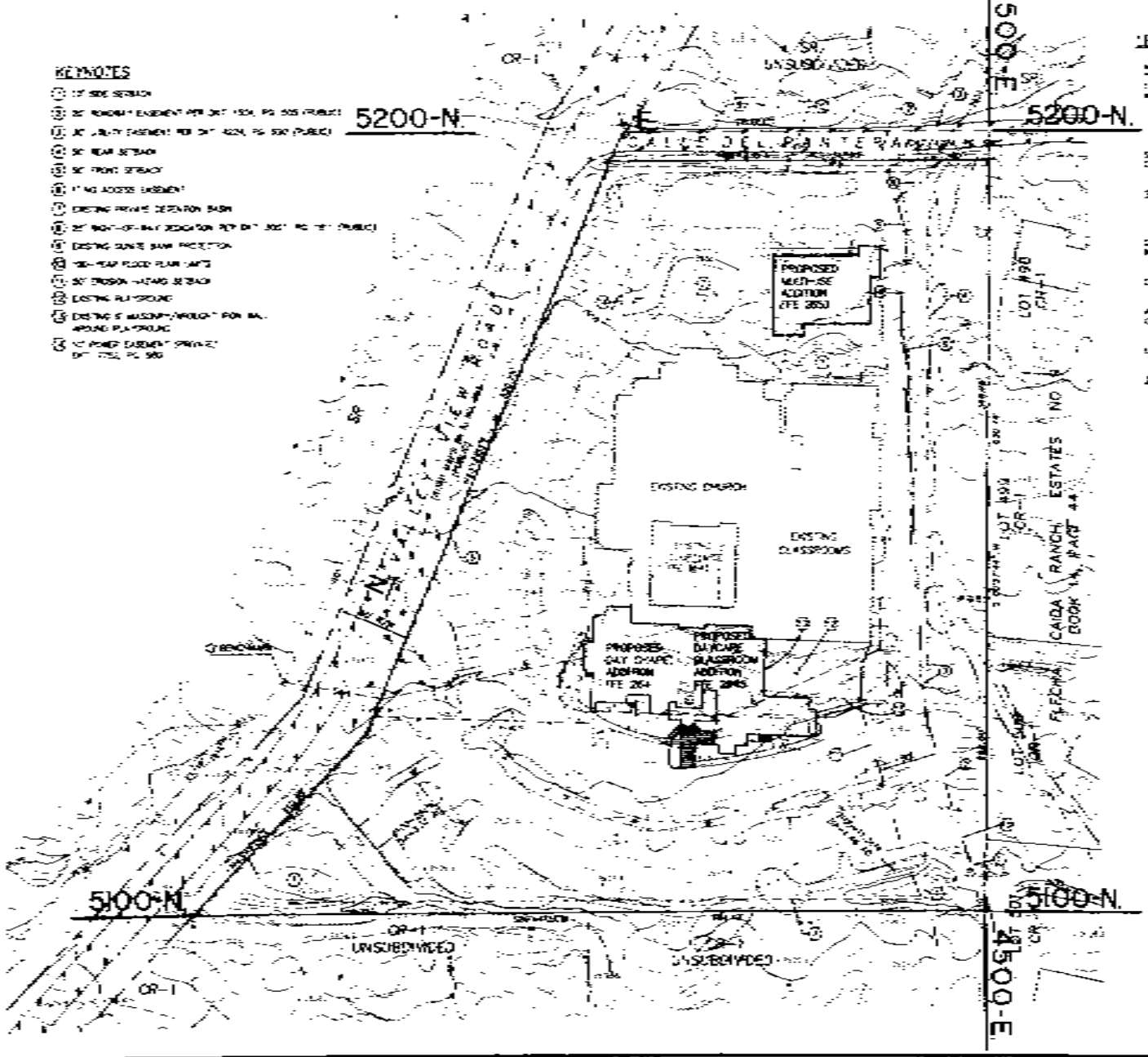
ST. THOMAS THE APOSTLE PARISH CHURCH
DAY CHAPEL & CLASSROOMS

1 2

D.P. BK 14 PG. 97 Sht 1 of 2

KEYNOTES

1. 15' SIDE SETBACK
2. 30' DRAINAGE EASEMENT PER DKT 1534, PG 305 (PUBLIC)
3. 30' UTILITY EASEMENT PER DKT 1534, PG 305 (PUBLIC)
4. 50' REAR SETBACK
5. 50' FRONT SETBACK
6. 1' NO ACCESS EASEMENT
7. EXISTING PRIVATE DETENTION BASIN
8. 30' RIGHT-OF-WAY DEDICATION PER DKT 3021, PG 1011 (PUBLIC)
9. EXISTING SANITARY SEWER PROJECTION
10. 100-YEAR FLOOD PLAIN LIMITS
11. 50' EROSION HAZARD SETBACK
12. EXISTING PLAYGROUND
13. EXISTING 5' MASONRY/WROUGHT IRON WALL AROUND PLAYGROUND
14. 10' POWER EASEMENT (PRIVATE) DKT 1534, PG 305



LEGAL DESCRIPTION

ALL THAT PORTION OF THE NORTH HALF OF THE NORTHWEST QUARTER OF THE SOUTHEAST QUARTER OF SECTION 15, TOWNSHIP 13 SOUTH, RANGE 15 EAST, GILA AND SALT RIVER BASE AND MERIDIAN, PIMA COUNTY, ARIZONA, MORE PARTICULARLY DESCRIBED AS FOLLOWS:

COMMENCING AT THE EAST QUARTER CORNER OF SAID SECTION 15;

THENCE S 89°59'51" W, ALONG THE EAST-WEST QUARTER LINE OF SAID SECTION 15, A DISTANCE OF 1,313.20 FEET TO THE NORTHEAST CORNER OF THE NORTHWEST QUARTER OF THE SOUTHEAST QUARTER;

THENCE S 00°17'44" W, A DISTANCE OF 1,317.14 FEET TO THE **TRUE POINT OF BEGINNING**;

THENCE S 00°17'44" W, A DISTANCE OF 260.00 FEET;

THENCE S 89°59'51" W, A DISTANCE OF 684.40 FEET TO A POINT ON THE EAST RIGHT-OF-WAY LINE OF VALLEY VIEW ROAD AS RECORDED IN BOOK 5 OF ROAD MAPS AT PAGE 56;

THENCE N 00°45'07" E, ALONG THE EASTERLY RIGHT-OF-WAY LINE OF VALLEY VIEW ROAD, A DISTANCE OF 260.00 FEET;

THENCE N 89°59'51" E, ALONG SAID EASTERLY RIGHT-OF-WAY LINE, A DISTANCE OF 692.75 FEET TO A POINT ON THE SOUTH RIGHT-OF-WAY OF CAIDA DEL PUENTE AS RECORDED IN DKT 10314 AT PAGE 1882;

THENCE N 89°41'57" E, ALONG THE SOUTHERLY RIGHT-OF-WAY LINE OF CAIDA DEL PUENTE, A DISTANCE OF 31.50 FEET;

THENCE S 00°17'44" W, A DISTANCE OF 308.11 FEET TO THE **TRUE POINT OF BEGINNING**.

100-YEAR FLOOD PLAIN LIMITS

1. N 27°45'18" W
2. N 77°00'18" E
3. N 64°06'18" W
4. N 64°06'18" E
5. N 37°06'18" W
6. N 43°06'18" E

Ca12-96-96

DEVELOPMENT PLAN

ST. THOMAS THE APOSTLE PARISH CHURCH
DAY CHAPEL & CLASSROOMS
LOCATED within SECTION 15, Township 13 South, Range 15 East, Gila and Salt River Base and Meridian, Pima County, Arizona

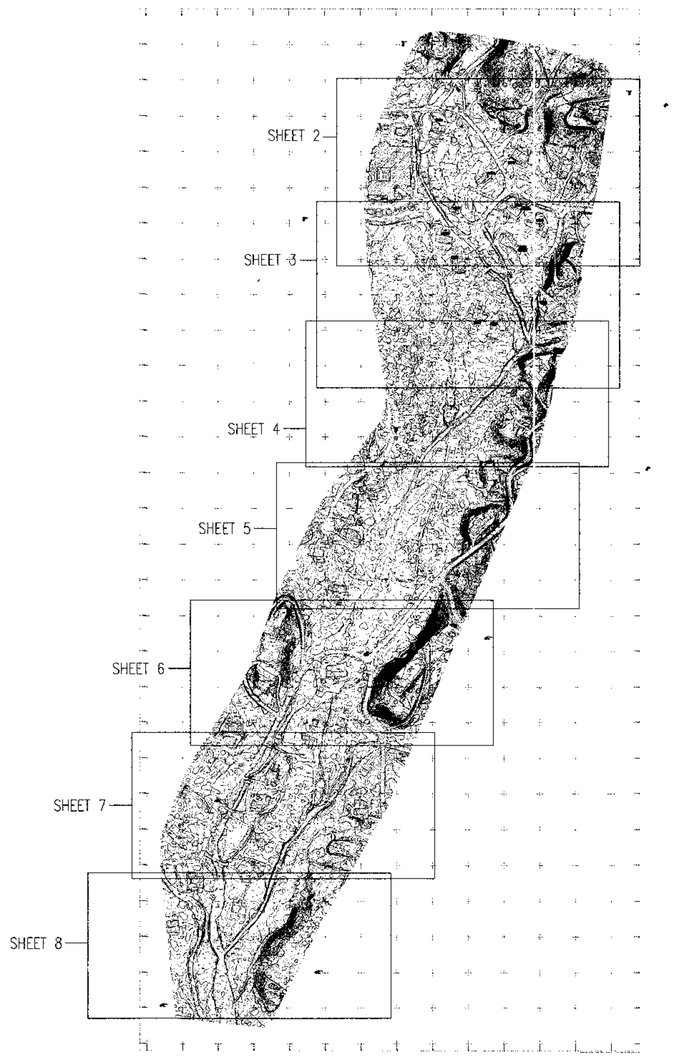
Greenlaw Engineers, Inc.

DATE: JULY 1996
JOB NO. E92-206
SHEET 2 | 2

D.P.BK.14 PG.97 SH 2 of 2

Appendix E

2007 Topographic Survey Sheets (Cardinal Land Surveying, Inc.)



N

FEET
0 400 800 1200 1600

1"=400 FEET
CONTOUR INTERVAL = 1 FOOT

Legend

CPT △ 2631.27	Control point with elevation
○ MON	Survey pin
⊠ SM	Survey monument
(11)	Point number – see Control Table

Notes

1. BASIS OF BEARING – is the Arizona State Plane Grid, Central Zone. State Plane coordinates were derived by a single unconstrained adjustment of Locus L1 static occupations of two existing PAG GIS points and the photogrammetric control. State Plane coordinates do not meet Arizona Statute standards.

   PAG GIS – Pima County Association of Government Geographic Information System
   RTK – Real Time Kinematic (dual frequency) positioning

   Basis of coordinates is Pima County Association of Government Geographic Information System Point NN–23 – 472781.099N, 1016294.820E (NAD83) The Control Table lists GROUND coordinates which were produced by using a single combined Scale Factor of 1.0001067 on surveyed control points.

2. Basis of Elevations is a stainless steel rod in casing at north end of bullnose at the intersection of Calle Barril and Swan Road (NN–23). Elevation = 2643.80 (NAVD88)

   Elevations in the Control Table with "(levels)" are by differential leveling, remaining elevations by static GPS observations.

   Elevations in the Control Table are NAVD88 per Pima County GIS. Three RM benchmarks were recovered as described in adjacent FIRM panels 1643 and 1644. GPS observations of these 3 RM's average 2.00 feet lower to equal NGVD29.

3. The purpose of this survey is to provide topographic mapping for a hydrology study within the mapping limits. The FFE (finish floor elevation) shown at selected homes were surveyed by differential leveling.

4. Photogrammetric control provided by Cardinal Land Surveying. Aerial mapping done by Sun Mapping at 1"=40', 1' CI (Job #071012). Information is available in Autocad R14 .DWG file format. Flight date 11/01/2007.

   This is an electronic copy of final document, original sealed document is with Cardinal Land Surveying, Michael D. Owens – RLS #31030.

Control Table

	N	E	EL	DESCRIPTION
1	474867.264	1016505.224	2689.14	BCSM "PCDOT R/W CONSTR" GIS L17 (levels)
2	472831.545	1016403.259	2643.80	FOUND STAINLESS STEEL ROD IN CASING(NN23)
3	472994.826	1015711.094	2640.30	SET BRIDGE SPIKE RANDOM (levels)
4	471883.568	1015505.901	2610.53	BCSM IN CONCRETE "RLS 4527"(levels)
5	470010.886	1014554.551	2537.83	SET MAG NAIL RANDOM IN AC(levels)
6	469819.386	1013688.345	2521.78	SET BRIDGE SPIKE RANDOM
7	470967.857	1013999.514	2539.88	SET MAG NAIL/SHINER RANDOM
8	473043.074	1014992.306	2602.85	BCSM WITH PUNCH, "PE 1322"
9	474231.029	1014477.484	2635.91	SET BRIDGE SPIKE RANDOM
10	475215.975	1015030.796	2666.99	SET BRIDGE SPIKE RANDOM
11	474941.103	1016310.890	2732.39	SET BRIDGE SPIKE RANDOM
12	473918.361	1015631.613	2624.05	SET MAG NAIL/WASHER "CONTROL POINT RLS31030" (levels)
13	471792.225	1014831.485	2566.78	SET BRIDGE SPIKE RANDOM(levels)

(Divide all coordinates by Scale Factor to equal PAG GIS "State Plane")

FFE Elevations

ELEVATION (NAVD88)	ADDRESS
2651.46	4510 Calle del Pantera
2648.78	4525 Calle del Pantera
2643.30	4520 Calle del Pantera
2635.54	4540 Calle del Pantera
2635.17	" Sunken living room
2632.26	4560 Calle del Pantera
2630.73	4565 Cerco del Corazon
2629.55	" Bedroom addition
2622.98	4580 Cerco del Corazon
2621.78	4560 Cerco del Corazon
2617.9	" Carport
2630.75	4550 Cerco del Corazon
2736.19	4540 Cerco del Corazon
2639.71	4530 Cerco del Corazon

REGISTERED LAND SURVEYOR
CERTIFICATE NO.
31030
MICHAEL D.
OWENS
ARIZONA U.S.A.

TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

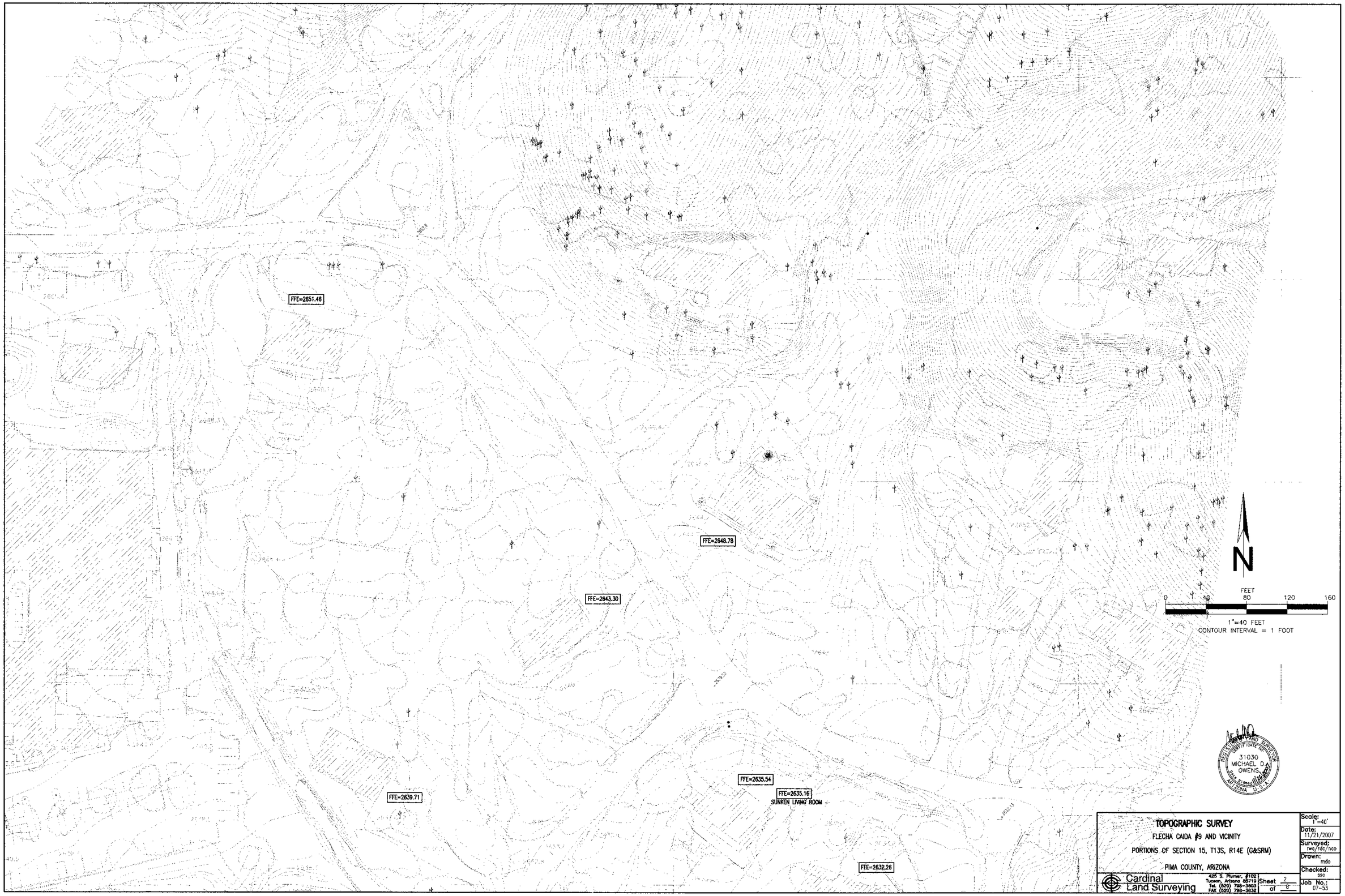
PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

Cardinal Land Surveying
425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 798-3603
FAX (520) 798-3632

Sheet 1 of 8

Scale: 1"=400'
Date: 11/21/2007
Surveyed: rwg/tdc/mao
Drawn: mdo
Checked: sso
Job No.: 07-53



TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

Cardinal Land Surveying

425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 798-3603
FAX (520) 798-3632

Sheet 2 of 8

Scale:	1"=40'
Date:	11/21/2007
Surveyed:	rwq/rdc/ngo
Drawn:	mdo
Checked:	sso
Job No.:	07-53

FFE=2639.71

FFE=2635.54

FFE=2635.16
SUNKEN LIVING ROOM

FFE=2632.26

FFE=2636.19

FFE=2630.73

FFE=2629.55
BEDROOM ADDITION

FFE=2630.75

FFE=2622.98
GARAGE

FFE=2621.78

FFE=2617.9
CARPORT

N

FEET
0 40 80 120 160

1"=40 FEET
CONTOUR INTERVAL = 1 FOOT

REGISTERED LAND SURVEYOR
CERTIFICATE NO.
31030
MICHAEL D.
OWENS
Date Signed 11-26-07
ARIZONA U.S.A.

TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

Cardinal Land Surveying
425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 798-3603
FAX (520) 798-3632

Sheet 3 of 8

Scale:	1"=40'
Date:	11/21/2007
Surveyed:	rwa/rdc/mdo
Drawn:	mdo
Checked:	sso
Job No.:	07-53

TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

Cardinal Land Surveying
425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 798-3603
FAX (520) 798-3832

Sheet 4 of 8

Scale: 1"=40'
Date: 11/21/2007
Surveyed: rwo/rdc/mdo
Drawn: mdo
Checked: SSO
Job No.: 07-53

N
FEET
0 40 80 120 160
1"=40 FEET
CONTOUR INTERVAL = 1 FOOT
REGISTERED LAND SURVEYOR
CERTIFICATE NO.
31030
MICHAEL D.
OWENS
ARIZONA U.S.A.
TOPOGRAPHIC SURVEY
FLECHA CAIDA #9 AND VICINITY
PORTIONS OF SECTION 15, T13S, R14E (G&SRM)
PIMA COUNTY, ARIZONA
Cardinal Land Surveying
425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 798-3603
FAX (520) 798-3832
Sheet 5 of 8
Scale: 1"=400'
Date: 11/21/2007
Surveyed: rwa/rdc/nao
Drawn: mdo
Checked: SSO
Job No.: 07-53

TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

Cardinal Land Surveying

425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 795-3403
FAX (520) 795-3832

Sheet 6 of 8

Scale:	1"=40'
Date:	11/21/2007
Surveyed:	rwo/rdc/nao
Drawn:	mdo
Checked:	sso
Job No.:	07-53

TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

Cardinal Land Surveying

425 S. Plumer, #102
Tucson, Arizona 85719
Tel. (520) 798-3003
FAX (520) 798-3832

Sheet 7 of 8

Scale: 1"=400'

Date: 11/21/2007

Surveyed: rwo/rfc/nco

Drawn: mdo

Checked: sso

Job No.: 07-53

TOPOGRAPHIC SURVEY

FLECHA CAIDA #9 AND VICINITY

PORTIONS OF SECTION 15, T13S, R14E (G&SRM)

PIMA COUNTY, ARIZONA

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Tucson, Arizona 85719
Tel. (520) 798-3603
FAX (520) 798-3632

Sheet 8 of 8

Scale: 1"=40'

Date: 11/21/2007

Surveyed: rwo/rdc/noo

Drawn: mdo

Checked: sso

Job No.: 07-33

Appendix F

Scour Computation Sheets

Summary of Scour Analysis - Input Parameters and Results

Flecha Caida Ranch Estates #1 and #2 and Las Lomas de Catalina (Sections .01-0.10)

GENERAL INPUT PARAMETERS

This section summarizes the general input parameters that were used in the scour analysis. Additional input parameters (if applicable) are provided on the individual computation sheets.

The scour equations are from the "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona", City of Tucson, 1989.

Hydraulic Data

	100-yr		
Q_{total} =	3797		total discharge
Q_{ch} =	1210	cfs	channel discharge
A_{ch} =	131.10	sq ft	channel area of flow
T_{ch} =	55.77	feet	channel topwidth of flow
Ymax =	5.03	feet	maximum depth of flow
S_e =	0.0181	ft/ft	energy slope or channel bed slope
TW =	0.00	feet	tailwater depth (if different than Y_{max}; otherwise 0)
n-value =	0.040		Manning's n-value for channel

Note: The energy slope is typically used when the basic hydraulic data is obtained from a HEC-2 or HEC-RAS analysis of the watercourse, and the channel bed slope is typically used when the data is obtained from a normal-depth analysis of the channel section.

For a straight reach, the radius of curvature (r_c) should be equal to or greater than: 557.66 feet

Applied r_c =	350	feet	radius of curvature of channel centerline

DESIGN SCOUR DEPTH

Per COT Drainage Design Manual, Equation 6.3:

Z_{gs} =	0.00	feet	(general scour)
Z_a =	1.16	feet	(anti-dune trough)
Z_{ls} =	0.00	feet	(controlling drop scour not applicable)
Z_{bs} =	0.49	feet	(bend scour)
Z_{lft} =	1.00	feet	(low-flow thalweg)
	2.66	feet	
	1.30		(safety factor)
Z_t =	3.45	feet	(minimum recommended design scour depth)

Depth of Scour

Flecha Caida Ranch Estates #1 and #2 and Las Lomas de Catalina (Sections .01-0.10)

GENERAL SCOUR

Per COT Drainage Design Manual, Equation 6.4:

V_m =	9.23	fps	
Y_{max} =	5.03	feet	
Y_h =	2.35	feet	
S_e =	0.0181	ft/ft	
Z_{gs} =	0.00	feet	(if a negative value is calculated, result appears as 0)

ANTI-DUNE TROUGH DEPTH

Per COT Drainage Design Manual, Equation 6.5:

V_m =	9.23	fps
g =	32.20	accelaration due to gravity in ft/s^2
Z_a =	1.16	feet

LOW-FLOW THALWEG

Per COT Drainage Design Manual:

Assume thalweg depth (Z_{lft}) is 2.0 feet for regional watercourses and 1.0 feet for all other watercourses, unless field observations dictate otherwise.

Z_{lft} =	1.00	feet

BEND SCOUR

Per COT Drainage Design Manual, Equation 6.6:

V_m =	9.23	fps	
Y_{max} =	5.03	feet	
Y_h =	2.35	feet	
S_e =	0.0181	ft/ft	
r_c =	350.00	feet	
T =	55.77	feet	
r_c/T =	6.28		(calculated r_c/T is limited to $0.5 < r_c/T < 10$)
Z_{bs} =	0.49	feet	(using r_c/T and substituting Eqn. 6.7 into Eqn. 6.6)

Summary of Scour Analysis - Input Parameters and Results

Flecha Caida Ranch Estates # 1 and #2 and Las Lomas de Catalina (Sections 0.11-0.21)

GENERAL INPUT PARAMETERS

This section summarizes the general input parameters that were used in the scour analysis. Additional input parameters (if applicable) are provided on the individual computation sheets.

The scour equations are from the "Standards Manual for Drainage Design and Floodplain Management in Tucson, Arizona", City of Tucson, 1989.

Hydraulic Data

	100-yr		
Q_{total} =	3219		total discharge
Q_{ch} =	1600	cfs	channel discharge
A_{ch} =	200.29	sq ft	channel area of flow
T_{ch} =	97.25	feet	channel topwidth of flow
Ymax =	3.13	feet	maximum depth of flow
S_e =	0.0180	ft/ft	energy slope or channel bed slope
TW =	0.00	feet	tailwater depth (if different than Y_{max}; otherwise 0)
n-value =	0.040		Manning's n-value for channel

Note: The energy slope is typically used when the basic hydraulic data is obtained from a HEC-2 or HEC-RAS analysis of the watercourse, and the channel bed slope is typically used when the data is obtained from a normal-depth analysis of the channel section.

For a straight reach, the radius of curvature (r_c) should be equal to or greater than: 972.47273 feet

Applied r_c =	350	feet	radius of curvature of channel centerline

DESIGN SCOUR DEPTH

Per COT Drainage Design Manual, Equation 6.3:

Z_{gs} =	0.00	feet	(general scour)
Z_a =	0.87	feet	(anti-dune trough)
Z_{ls} =	0.00	feet	(controlling drop scour not applicable)
Z_{bs} =	0.66	feet	(bend scour)
Z_{lft} =	1.00	feet	(low-flow thalweg)
	2.53	feet	
	1.30		(safety factor)
Z_t =	3.29	feet	(minimum recommended design scour depth)

Depth of Scour

Flecha Caida Ranch Estates # 1 and #2 and Las Lomas de Catalina (Sections 0.11-0.21)

GENERAL SCOUR

Per COT Drainage Design Manual, Equation 6.4:

V_m =	7.99	fps	
Y_{max} =	3.13	feet	
Y_h =	2.06	feet	
S_e =	0.0180	ft/ft	
Z_{gs} =	0.00	feet	(if a negative value is calculated, result appears as 0)

ANTI-DUNE TROUGH DEPTH

Per COT Drainage Design Manual, Equation 6.5:

V_m =	7.99	fps
g =	32.20	accelaration due to gravity in ft/s^2
Z_a =	0.87	feet

LOW-FLOW THALWEG

Per COT Drainage Design Manual:

Assume thalweg depth (Z_{lft}) is 2.0 feet for regional watercourses and 1.0 feet for all other watercourses, unless field observations dictate otherwise.

Z_{lft} =	1.00	feet

BEND SCOUR

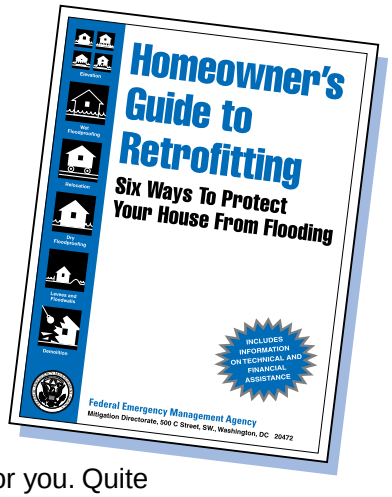
Per COT Drainage Design Manual, Equation 6.6:

V_m =	7.99	fps	
Y_{max} =	3.13	feet	
Y_h =	2.06	feet	
S_e =	0.0180	ft/ft	
r_c =	350.00	feet	
T =	97.25	feet	
r_c/T =	3.60		(calculated r_c/T is limited to $0.5 < r_c/T < 10$)
Z_{bs} =	0.66	feet	(using r_c/T and substituting Eqn. 6.7 into Eqn. 6.6)

Appendix G

Guides to Retrofitting Flood Prone Structures

Who The Guide Is For



As a homeowner, you need clear information about the options that are available to reduce flood damage to your home – and straightforward guidance on selecting the option that is best for you. Quite often this is a difficult task. The publication described here is for readers who have little or no knowledge of flood protection methods or building construction techniques.

You should take action to avoid repetitive flood damage to your house. First, you need to know what damage-reduction methods are available, the degree to which they work, how much they cost, and whether they meet your needs. All of these questions are answered by the guide. In addition, the guide explains how the degree of flood risk varies from one location to another. By knowing the basic questions to ask, you are guided towards the investment in retrofitting that is appropriate for you.

You can download FEMA 312, or parts of it, from FEMA's web site –
http://www.fema.gov/mit/rfit/

Some retrofitting techniques may not be used in certain circumstances under state or local laws, ordinances, or regulations.

Want To Learn More?

Homeowner's Guide to Retrofitting:

Six Ways To Protect Your House From Flooding is FEMA publication 312.
Call 1-800-737-8669 to get a copy of this important guide. For copies of other FEMA publications, including those listed below, call 1-800-480-2520.

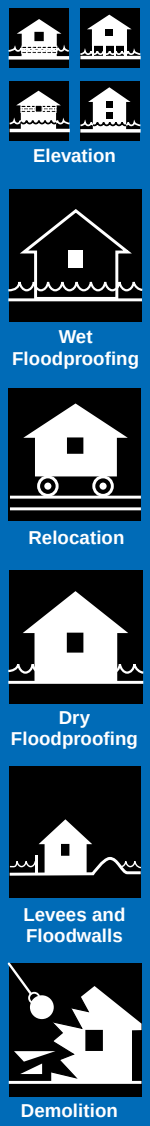
Related Publications

- **FEMA 55**
  Coastal Construction Manual
- **FEMA 257**
  Mitigation of Flood and Erosion Damage
- **FEMA 102**
  Floodproofing Non-Residential Structures

Recommended for Architects and Engineers —

- **FEMA 259**
  Engineering Principles and Practices for Retrofitting Flood Prone Residential Buildings

State and local representatives of emergency management, emergency services, floodplain management, building code, and planning and zoning agencies may have copies of FEMA 312 for immediate distribution.



PROJECT IMPACT

FEMA-L235

Homeowner's Guide to Retrofitting

Six Ways To Protect Your House From Flooding

Federal Emergency Management Agency
Mitigation Directorate
Washington, DC 20472
www.fema.gov

What Is "Retrofitting"?

Retrofitting means making changes to an existing building to protect it from flooding or other hazards such as high winds and earthquakes. FEMA publication 312, ***Homeowner's Guide to Retrofitting:*** *Six Ways To Protect Your House From Flooding*, provides information that will help you decide whether your house is a candidate for retrofitting.

The guide helps by describing six retrofitting methods that protect your house from flooding.

Elevation is raising your house so that the lowest floor is above the flood level. This is the most common way to avoid flood damage.

Wet floodproofing makes uninhabited parts of your house resistant to flood damage when water is allowed to enter during flooding.

Relocation means moving your house to higher ground where the exposure to flooding is eliminated altogether.

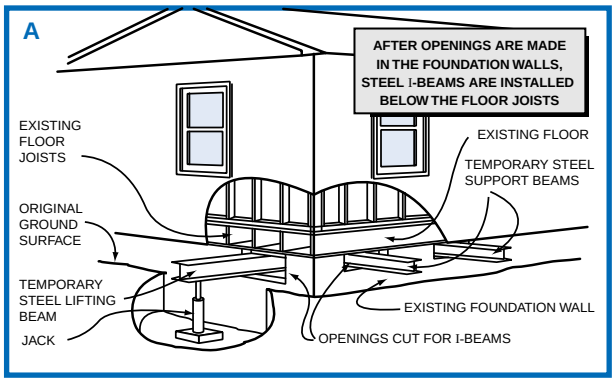
Dry floodproofing is sealing your house to prevent flood waters from entering.

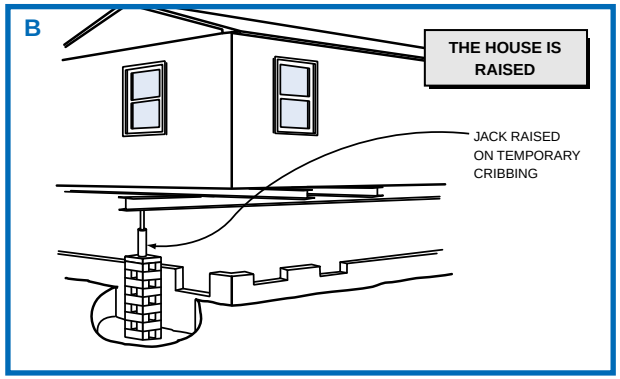
Levee and floodwall protection means constructing barriers to prevent flood waters from entering your house.

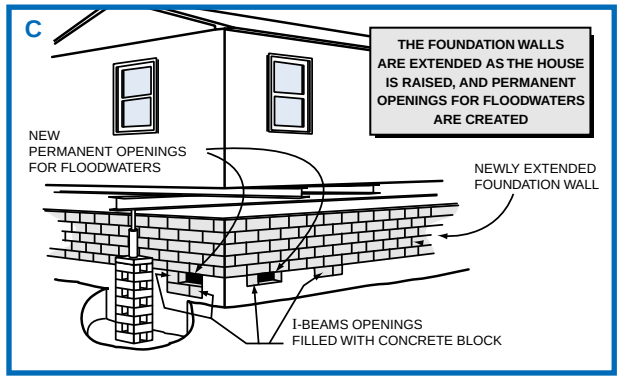
Demolition means razing your house and rebuilding properly on the same property or buying a house elsewhere.

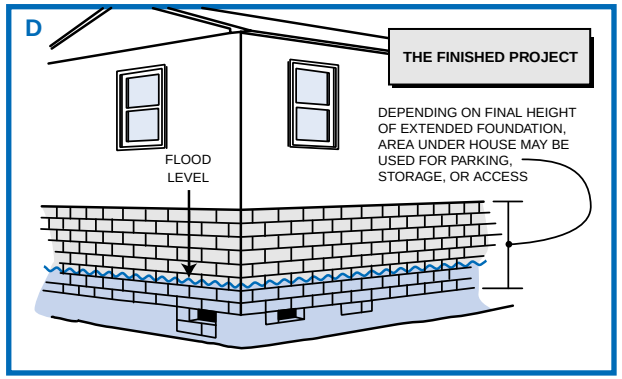
The guide uses photographs and illustrations to help explain how each of the six retrofitting methods works.

For example, this series of figures from the guide shows how a house on a basement or crawlspace foundation can be elevated above the flood level on extended foundation walls.









The Next Step

Whether or not your house has been damaged by flooding, contact your local floodplain administrator or building official before retrofitting. This contact is the critical next step in reducing your potential flood losses. Local officials know the retrofitting methods that meet state and local government requirements.

Financial Assistance

The guide provides information on government and non-government financial assistance that can help homeowners with retrofitting projects. Financial assistance means loans, grants, and insurance payments. The assistance goes to individual property owners, communities, and states.

For example, under FEMA's National Flood Insurance Program, a policy holder may qualify for Increased Cost of Compliance (ICC) coverage. If your house is substantially damaged by flooding, ICC coverage may help pay for some types of retrofitting. Other programs, such as the Hazard Mitigation Grant Program and the Flood Mitigation Assistance Program are designed to help financially. The guide describes many government and non-government programs, and it explains how you might qualify for assistance.

Protecting Your Business From Flooding

ARE YOU AT RISK?

If you aren't sure whether your business is at risk from flooding, check with your local floodplain manager, building official, city engineer, or planning and zoning administrator. They can tell you whether you are in a flood hazard area, and they also can tell you how to protect your business from flooding.

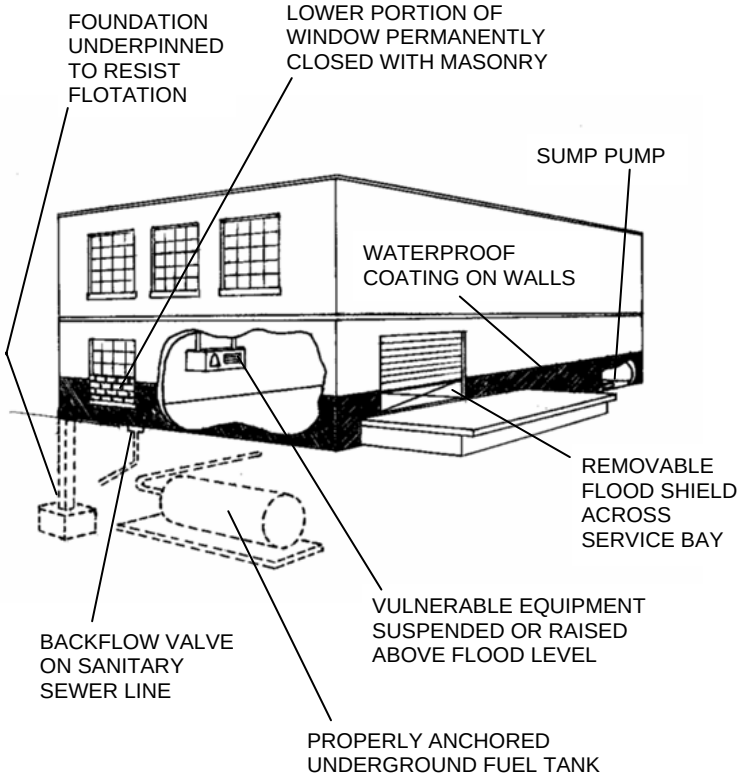
WHAT YOU CAN DO

Protecting your business from flooding can involve a variety of actions, from inspecting and maintaining your buildings to installing protective devices. Most of these actions, especially those that affect the structure of your buildings or their utility systems, should be carried out by qualified maintenance staff or professional contractors licensed to work in your state, county, or city. One example of flood protection is using dry floodproofing techniques to protect buildings in flood hazard areas.

DRY FLOODPROOF YOUR BUILDING

One way to protect a building and its contents from flood damage is to seal the building so that flood waters cannot enter. This method, referred to as "dry floodproofing," encompasses a variety of measures (some of which are covered by separate fact sheets – see back of this sheet):

- applying a waterproof coating or membrane to the exterior walls of the building
- installing watertight shields over doors, windows, and other openings
- anchoring the building as necessary so that it can resist floatation
- installing backflow valves in sanitary and storm sewer lines
- raising utility system components, machinery, and other pieces of equipment so that they are above the flood level
- anchoring fuel tanks and other storage tanks to prevent flotation
- installing a sump pump and foundation drain system
- strengthening walls so that they can withstand the pressures of flood waters and the impacts of floodborne debris



Dry Floodproof Your Building

TIPS

Keep these points in mind when you dry floodproof a building:

- ✓ Dry floodproofing is appropriate primarily for slab-on-grade buildings with concrete or solid masonry walls. Concrete and masonry are easier to seal, more resistant to flood damage, and stronger than other conventional construction materials.

- ✓ If you dry floodproof a “substantially damaged” or “substantially improved” building (as defined by the National Flood Insurance Program regulations) or a newly constructed building, and if the building’s lowest floor (including any basement) is below the Base Flood Elevation (BFE) shown on the Flood Insurance Rate Map (FIRM) for your community, your dry floodproofing must be certified as providing protection from the BFE. To obtain this certification, you must floodproof your building to a height at least 1 foot above the BFE. Check with your local floodplain manager or building official for more information.

- ✓ The height of your dry floodproofing should not exceed 3 feet. The pressures exerted by deeper water can cause walls to buckle or collapse. Before you use dry floodproofing to protect against greater flood depths, have a structural engineer evaluate the strength of your walls.

- ✓ If your dry floodproofing measures require human intervention, such as placing shields over doors and windows before flood waters arrive, you should have an operations and maintenance plan that describes all the actions that must be taken and lists the persons who are responsible. It must also include a schedule of periodic maintenance that states how often the dry floodproofing measures will be inspected and who will perform the inspections.

ESTIMATED COST

The cost of individual dry floodproofing measures will vary with the size, condition, and use of your building; the dry floodproofing height; and the extent to which you use contractors and engineers.

OTHER SOURCES OF INFORMATION

Install Sewer Backflow Valves, Protecting Your Property from Flooding, FEMA Hazard Mitigation Fact Sheet, 1998

Anchor Fuel Tanks, Protecting Your Property from Flooding, FEMA Hazard Mitigation Fact Sheet, 1998

Non-Residential Floodproofing – Requirements and Certification for Buildings Located in Special Flood Hazard Areas, FEMA Technical Bulletin 3-93, April 1993

Floodproofing Non-Residential Structures, FEMA 102, 1986

To obtain copies of FEMA documents, call FEMA Publications at 1-800-480-2520. Information is also available on the World Wide Web at **http//:www.fema.gov.**

Protecting Your Property From Flooding

ARE YOU AT RISK?

If you aren't sure whether your house is at risk from flooding, check with your local floodplain manager, building official, city engineer, or planning and zoning administrator. They can tell you whether you are in a flood hazard area. Also, they usually can tell you how to protect yourself and your house and property from flooding.

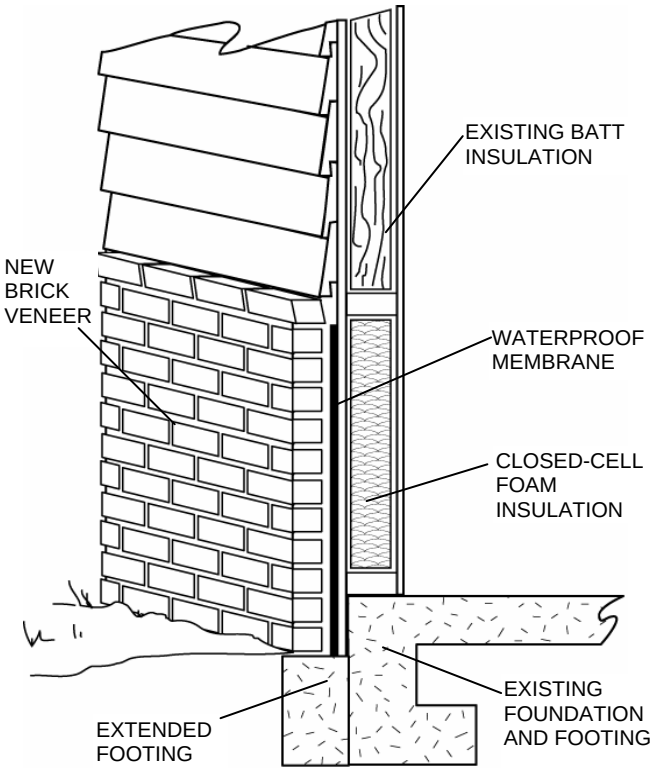
WHAT YOU CAN DO

Flood protection can involve a variety of changes to your house and property – changes that can vary in complexity and cost. You may be able to make some types of changes yourself; however, complicated or large-scale changes and those that affect the structure of your house or its electrical wiring and plumbing should be carried out only by a professional contractor licensed to work in your state, county, or city. One example of flood protection is adding a waterproof veneer to the exterior walls of your house. This is something that only a licensed contractor should do.

ADD WATERPROOF VENEER TO EXTERIOR WALLS

Even in areas where flood waters are less than 2 feet deep, a house can be severely damaged if water reaches the interior. The damage to walls and floors can be expensive to repair, and the house may be uninhabitable while repairs are underway.

One way to protect a house from shallow flooding is to add a waterproof veneer to the exterior walls and seal all openings, including doors, to prevent the entry of water. As shown in the figure, the veneer can consist of a layer of brick backed by a waterproof membrane. Before the veneer is applied, the siding is removed and replaced with exterior grade plywood sheathing. If necessary, the existing foundation footing is extended to support the brick. Also, because the wall will be exposed to flood water, changes are made to the interior walls as well so that they will resist moisture damage. In the area below the flood level, standard batt insulation is replaced with washable closed-cell foam insulation, and any wood blocking added inside the wall cavity is made of exterior grade lumber.



Add Waterproof Veneer to Exterior Walls

TIPS

Keep these points in mind if you plan to have a waterproof veneer added to the exterior walls of your house:

- ✓ Adding a waterproof veneer is appropriate in areas where the flood depth is less than 2 feet. When flood depths exceed 2 feet, the pressure on waterproofed walls increases greatly, usually beyond the strength of the walls. If greater flood depths are expected, consult with a licensed civil or structural engineer before using this method.
- ✓ Changes to the foundation of your house must be done by a licensed contractor, who will ensure that the work is done correctly and according to all applicable codes. This is important for your safety.
- ✓ If your house is being remodeled or repaired, consider having the veneer added as part of the remodeling or repair work. It will probably be cheaper to combine these projects than to carry them out separately.
- ✓ If your house has brick walls, you can still use this method. The new brick veneer and waterproof membrane are added over the existing brick.
- ✓ If your house is flooded by groundwater entering through the floor, this method will not be effective.

ESTIMATED COST

If you have a contractor add a waterproof brick veneer to your house, you can expect to pay about $10 per square foot of exterior wall. For example, a 3-foot-high brick veneer on a house measuring 60 feet by 30 feet would cover about 540 square feet and would cost about $5,400. This figure does not include the cost of sealing doors and other openings or extending the foundation.

OTHER SOURCES OF INFORMATION

Protecting Your Home from Flooding, FEMA, 1994

Repairing Your Flooded Home, FEMA-234, 1992

Flood Emergency and Residential Repair Handbook, FIA-13, 1986

Retrofitting Flood-Prone Residential Structures, FEMA-114, 1986

To obtain copies of these and other FEMA documents, call FEMA Publications at 1-800-480-2520. Information is also available on the World Wide Web at **http//:www.fema.gov.**

Protecting Your Property From Flooding

ARE YOU AT RISK?

If you aren't sure whether your house is at risk from flooding, check with your local floodplain manager, building official, city engineer, or planning and zoning administrator. They can tell you whether you are in a flood hazard area. Also, they usually can tell you how to protect yourself and your house and property from flooding.

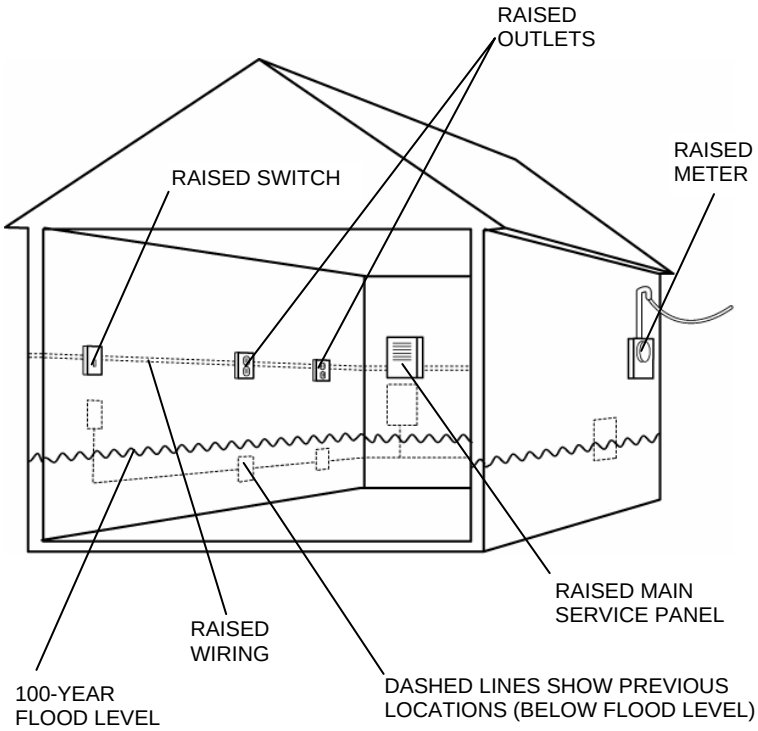
WHAT YOU CAN DO

Flood protection can involve a variety of changes to your house and property – changes that can vary in complexity and cost. You may be able to make some types of changes yourself; however, complicated or large-scale changes and those that affect the structure of your house or its electrical wiring and plumbing should be carried out only by a professional contractor licensed to work in your state, county, or city. One example of flood protection is raising the components of your electrical system above the level of the 100-year flood. This is something that only a licensed contractor should do.

RAISE ELECTRICAL SYSTEM COMPONENTS

Electrical system components, including service panels (fuse and circuit breaker boxes), meters, switches, and outlets, are easily damaged by flood water. If they are inundated for even short periods, they will probably have to be replaced. Another serious problem is the potential for fires caused by short circuits in flooded systems. Raising electrical system components helps you avoid those problems. Also, having an undamaged, operating electrical system after a flood will help you clean up, make repairs, and return to your home with fewer delays.

As shown in the figure, all components of the electrical system, including the wiring, should be raised at least 1 foot above the 100-year flood level. In an existing house, this work will require the removal of some interior wall sheathing (drywall, for example). If you are repairing a flood-damaged house or building a new house, elevating the electrical system will be easier.



Raise Electrical System Components

TIPS

Keep these points in mind when you have your electrical system components raised:

- ✓ Electrical system modifications must be done by a licensed contractor, who will ensure that the work is done correctly and according to all applicable codes. This is important for your safety.
- ✓ Your contractor should check with the local power company about the maximum height that the electric meter can be raised.
- ✓ If your house is equipped with an old-style fuse box or low-amperage service, you may want to consider upgrading to a modern circuit breaker system and higher-amperage service, especially if you have large appliances or other electrical equipment that draws a lot of power.

ESTIMATED COST

Raising the electrical service panel, meter, and all of the outlets, switches, and wiring in a 1,000-square-foot, single-floor house will cost about $1,500 to $2,000. If this work is performed during the repair of a damaged house or construction of a new house, the cost may be much lower.

OTHER SOURCES OF INFORMATION

Protecting Your Home from Flooding, FEMA, 1994

Repairing Your Flooded Home, FEMA-234, 1992

Flood Emergency and Residential Repair Handbook, FIA-13, 1986

Retrofitting Flood-Prone Residential Structures, FEMA-114, 1986

To obtain copies of these and other FEMA documents, call FEMA Publications at 1-800-480-2520. Information is also available on the World Wide Web at **http//:www.fema.gov.**

Protecting Your Property From Flooding

Are You at Risk?

If you aren't sure whether your house is at risk from flooding, check with your local floodplain manager, building official, city engineer, or planning and zoning administrator. They can tell you whether you are in a flood hazard area. Also, they usually can tell you how to protect yourself and your house and property from flooding.

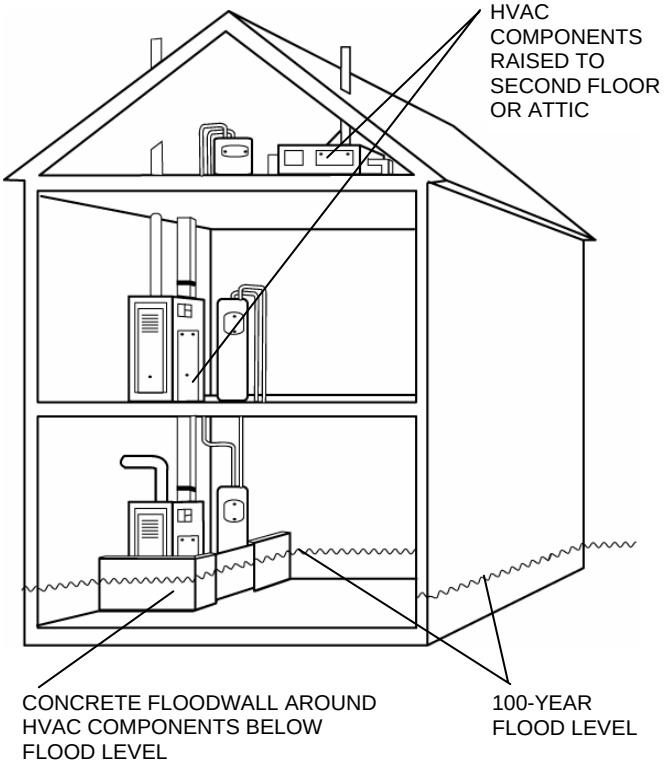
What You Can Do

Flood protection can involve a variety of changes to your house and property – changes that can vary in complexity and cost. You may be able to make some types of changes yourself; however, complicated or large-scale changes and those that affect the structure of your house or its electrical wiring and plumbing should be carried out only by a professional contractor licensed to work in your state, county, or city. One example of flood protection is raising the heating, ventilating, and cooling equipment in your house so that it is above the flood level, or surrounding it with a flood wall. These are things that only a licensed contractor should do.

Raise or Floodproof HVAC Equipment

Heating, ventilating, and cooling (HVAC) equipment, such as a furnace or hot water heater, can be damaged extensively if it is inundated by flood waters. The amount of damage will depend partly on the depth of flooding and the amount of time the equipment remains under water. Often, the damage is so great that the only solution is replacement.

In floodprone houses, a good way to protect HVAC equipment is to move it from the basement or lower level of the house to an upper floor or even to the attic. A less desirable method is to leave the equipment where it is and build a concrete or masonry block floodwall around it. Both of these methods require the skills of a professional contractor. Relocation can involve plumbing and electrical changes, and floodwalls must be adequately designed and constructed so that they are strong enough and high enough to provide the necessary level of protection.



Raise or Floodproof HVAC Equipment

TIPS

Keep these points in mind when you have your HVAC equipment raised or floodproofed:

- ✓ Changes to the plumbing, electrical system, and ventilating ductwork in your house must be done by a licensed contractor, who will ensure that the work is done correctly and according to all applicable codes. This is important for your safety.
- ✓ If you are having your existing furnace or hot water heater repaired or replaced, consider having it relocated at the same time. It will probably be cheaper to combine these projects than to carry them out at different times.
- ✓ Similarly, if you have decided to raise your HVAC equipment, consider upgrading to a more energy-efficient unit at the same time. Upgrading can not only save you money on your heating and cooling bills, it may also make you eligible for a rebate from your utility companies.
- ✓ If you decide to protect your HVAC equipment with a floodwall, remember that you will need enough space in the enclosed area for system repairs and routine maintenance. Also, depending on its height, the wall may have to be equipped with an opening that provides access to the enclosed area. Any opening will have to be equipped with a gate that can be closed to prevent flood waters from entering.

ESTIMATED COST

Having your furnace and hot water heater moved to a higher floor or to the attic will cost about $ 1,500. The cost of a floodwall will depend partly on its height and length. A 3-foot-high wall with a perimeter length of 35 feet would cost about $1,000.

OTHER SOURCES OF INFORMATION

Protecting Your Home from Flooding, FEMA, 1994

Repairing Your Flooded Home, FEMA-234, 1992

Flood Emergency and Residential Repair Handbook, FIA-13, 1986

Retrofitting Flood-Prone Residential Structures, FEMA-114, 1986

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Protecting Your Property From Flooding

ARE YOU AT RISK?

If you aren't sure whether your house is at risk from flooding, check with your local floodplain manager, building official, city engineer, or planning and zoning administrator. They can tell you whether you are in a flood hazard area. Also, they usually can tell you how to protect yourself and your house and property from flooding.

WHAT YOU CAN DO

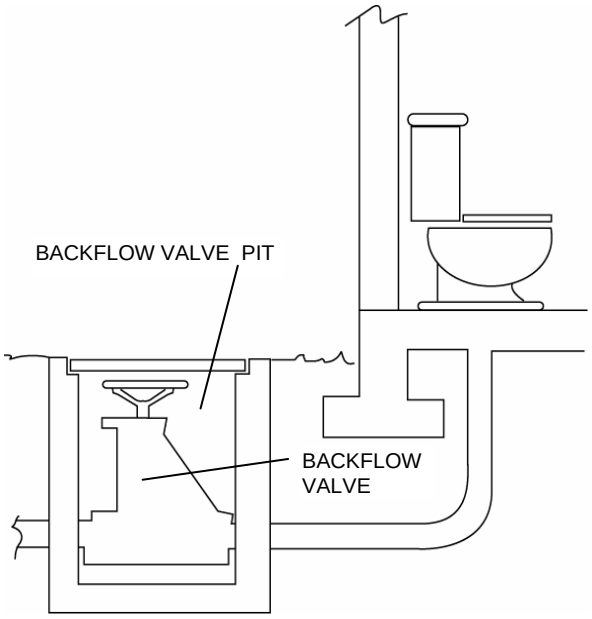
Flood protection can involve a variety of changes to your house and property – changes that can vary in complexity and cost. You may be able to make some types of changes yourself; however, complicated or large-scale changes and those that affect the structure of your house or its electrical wiring and plumbing should be carried out only by a professional contractor licensed to work in your state, county, or city. One example of flood protection is installing a backflow valve to prevent sewage from backing up into your house. This is something that only a licensed plumber or contractor should do.

INSTALL SEWER BACKFLOW VALVES

In some floodprone areas, flooding can cause sewage from sanitary sewer lines to back up into houses through drain pipes. These backups not only cause damage that is difficult to repair, but also create health hazards.

A good way to protect your house from sewage backups is to install backflow valves, which are designed to block drain pipes temporarily and prevent flow into the house. Backflow valves are available in a variety of designs that range from the simple to the complex. The figure shows a gate valve, one of the more complex designs. It provides a strong seal, but must be operated by hand. So the effectiveness of a gate valve will depend on how much warning you have of impending flooding. Among the simpler valves are a flap or check valves, which open to allow flow out of the house but close when the flow reverses. These valves operate automatically but do not provide as strong a seal as a gate valve.

TYPICAL INSTALLATION OF AN EXTERIOR BACKFLOW VALVE



← NORMAL DIRECTION OF FLOW (VALVE PREVENTS FLOW IN REVERSE DIRECTION)

Install Sewer Backflow Valves

TIPS

Keep these points in mind if you have backflow valves installed:

- ✓ Changes to the plumbing in your house must be done by a licensed plumber or contractor, who will ensure that the work is done correctly and according to all applicable codes. This is important for your safety.
- ✓ Some valves incorporate the advantages of both flap and gate valves into a single design. Your plumber or contractor can advise you on the relative advantages and disadvantages of the various types of backflow valves.
- ✓ Valves should be installed on all pipes that leave the house or that are connected to equipment that is below the potential flood level. So valves may be needed on washing machine drain lines, laundry sinks, fuel oil lines, rain downspouts, and sump pumps, as well as sewer/septic connections.
- ✓ If you have a sump pump, it may be connected to underground drain lines, which may be difficult to seal off.

ESTIMATED COST

Having a plumber or contractor install one backflow valve will cost you about $525 for a combined gate/flap valve or about $375 for a flap valve. These figures include the cost of excavation and backfilling.

OTHER SOURCES OF INFORMATION

Protecting Your Home from Flooding, FEMA, 1994

Repairing Your Flooded Home, FEMA-234, 1992

Flood Emergency and Residential Repair Handbook, FIA-13, 1986

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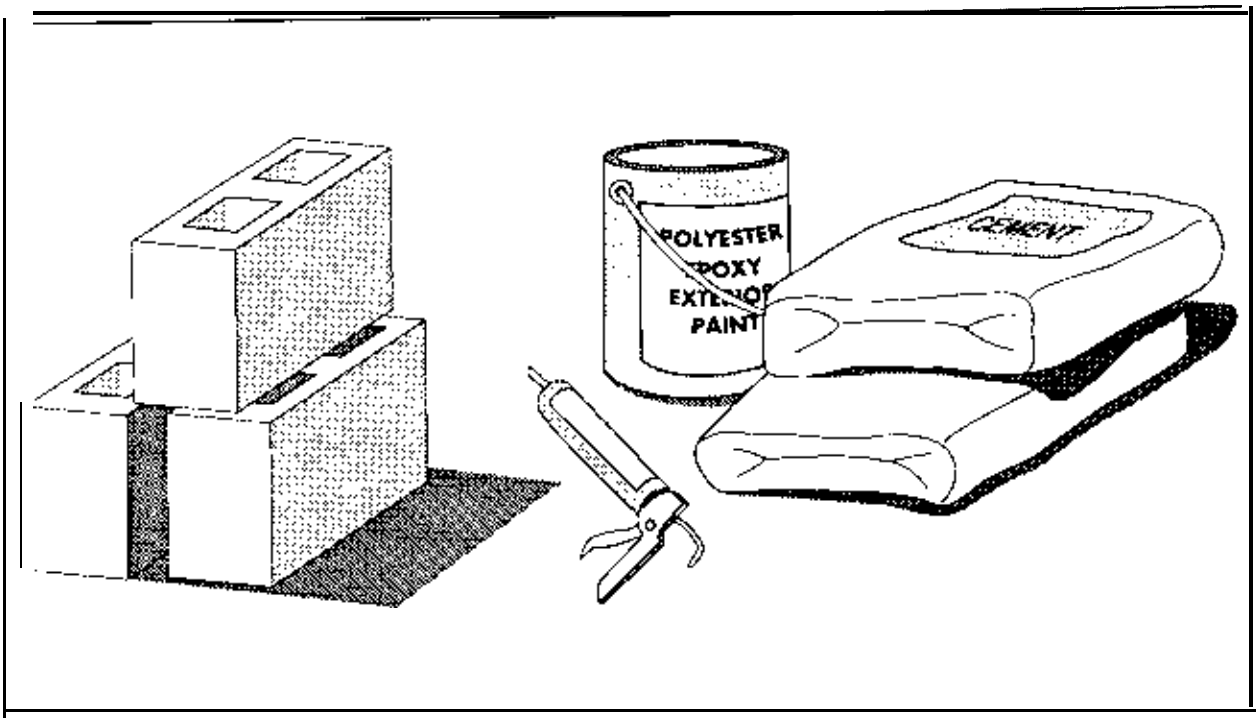
Technical Bulletin 2-93

Flood-Resistant Materials Requirements

for Buildings Located in Special Flood Hazard Areas in accordance with the National Flood Insurance Program



FEDERAL EMERGENCY MANAGEMENT AGENCY
FEDERAL INSURANCE ADMINISTRATION

FIA-TB-2
4/93

Key Word/Subject index:

This index allows the user to quickly locate key words and subjects in this Technical Bulletin. The Technical Bulletin User's Guide (printed separately) provides references to key words and subjects throughout the Technical Bulletins. For definitions of selected terms, refer to the Glossary at the end of this bulletin.

Key Word/Subject	Page
Breakaway wall materials in V zones, made of flood-resistant materials	12
Flood-resistant flooring materials	4
Flood-resistant material, definition of	1
Flood-resistant materials, classifications, use of	2
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U.S. Army Corps of Engineers "Flood Proofing Regulations"	2

Any comments on the Technical Bulletins should be directed to:

FEMA/FIA
Office of Loss Reduction
Technical Standards Division
500 C St., SW, Room 417
Washington, D.C. 20472

Technical Bulletin 2-93 replaces Technical Bulletin 88-2 (draft) "Flood-Resistant Materials."

Graphic design based on the Japanese print The Great Wave Off Kanagawa, by Katsushika Hokusai [1 760-1849), Asiatic collection, Museum of Fine Arts, Boston.

TECHNICAL BULLETIN 2-93

Flood-Resistant Materials Requirements for Buildings Located In Special Flood Hazard Areas in accordance with the National Flood Insurance Program

Introduction

The requirement to use construction and finishing materials that are resistant to flood damage in all new and substantially improved buildings in identified Special Flood Hazard Areas (SFHAs) is an important part of the National Flood Insurance Program's (NFIP's) flood-damage-resistant design and construction standards. A residential building's lowest floor is required to be elevated to or above the base flood elevation (BFE). All construction below the lowest floor is susceptible to flooding and must consist of flood-resistant materials. Uses of enclosed areas below the lowest floor in a residential building are limited to parking, building access, and limited storage—areas that can withstand inundation by floodwater without sustaining significant structural damage.

The purpose of this Technical Bulletin is to provide data and guidance on what constitute "materials resistant to flood damage" and how and when these materials must be used to improve a building's ability to withstand flooding.

NFIP Regulations

Section 60.3(a)(3) of the NFIP regulations requires that the community:

> ***"Review all permit applications to determine whether proposed building sites will be reasonably safe from flooding. If a proposed building site is in a floodprone area, all new construction and substantial improvements shall... be constructed with materials resistant to flood damage..."***

> It should be noted that Technical Bulletins provide guidance on the minimum requirements of the NFIP regulations. Community or State requirements that exceed those of the NFIP take precedence. Design professionals should contact the community to determine whether more restrictive local or State regulations apply to the building or site in question. All applicable standards of the State or local building code must also be met for any building in a flood hazard area.

Required Use of Flood-Resistant Materials

Flood-Resistant Material

"Flood-resistant material" is defined as any building material capable of withstanding direct and prolonged contact with floodwaters without sustaining significant damage. The term "prolonged contact" means at least 72 hours, and the term "significant damage" means any damage requiring more than low-cost cosmetic repair (such as painting).

As stated previously, **all structural and non-structural building materials at or below the BFE must be flood resistant.** This requirement applies regardless of the expected or historic flood duration. For example, buildings in coastal areas that experience relatively short-duration flooding (generally, flooding with a duration of less than 24 hours) must be constructed with flood-resistant materials below the BFE. As noted in the tables within this bulletin, **only Class** 4 **and Class 5 materials are acceptable for areas below the BFE in floodprone buildings.**

In some instances, Class 1,2, and 3 materials may be permitted below the BFE, when specifically required to meet local building code provisions concerning life-safety issues. In below-BFE applications, materials that meet life-safety code requirements and have maximum resistance to damage from flood inundation should be used. This applies to the flood-resistant requirements only. In Zones V, VE, and V 1 -V30, the installation of such materials may create an obstruction. Because obstructions in V zones could result in structural failure of the building, they represent a life-safety issue and shall therefore take precedence over local building codes. Refer to Technical Bulletin 5, "Free of Obstruction Requirements," for further information.

Lowest Floor

Under the NFIP, the term "lowest floor" is used to define the lowest level of a building that must be located at or above the BFE as required under Sections 60.3(c)(2) and (3) of the NFIP regulations. The floodplain management regulations, under Section 60.3(c)(5), limit the use of all areas below the lowest floor to parking of vehicles, storage, and building access. These reasonable uses below the BFE are permitted because the amount of damage caused by flooding to these areas can easily be kept to a minimum if design and construction requirements contained in the NFIP regulations are met. Failure to meet the requirements can increase the building's damage potential and result in the application of higher flood insurance premiums. The requirement to use flood-resistant materials means that all interior wall, floor, and ceiling materials located below the BFE be unfinished and resistant to flood damage. This is meant to exclude the use of materials and finishes normally associated with living areas constructed above the BFE.

Flood Insurance Implication

An NFIP flood insurance requirement regarding the use of materials in areas below the BFE must also be considered. Flood insurance will not pay a claim for finishing materials (such as clay floor tiles) located in basements or in enclosed areas below the lowest floor of an elevated building, even if such materials are considered to be flood resistant. The NFIP defines finishing materials as anything beyond basic wall construction.

Flood-Resistant Classification of Materials

The information in this Technical Bulletin is based primarily on the U.S. Army Corps of Engineers (COE) 1992 "Flood Proofing Regulations. " The following table (Table 1) classifies building materials according to their ability to resist flood damage.

Table 1 Flood-Resistant Classification of Materials		
NFIP	Class	Class Description
Acceptable	5	Highly resistant to floodwater damage. Materials within this class are permitted for partially enclosed or outside uses with essentially unmitigated flood exposure.
	4	Resistant to floodwater damage. Materials within this class may be exposed to and/or submerged in floodwaters in interior spaces and do not require special waterproofing protection.
unacceptable	3	Resistant to clean water damage. Materials within this class may be submerged in clean water during periods of intentional flooding.
	2	Not resistant to water damage. Materials within this class require essentially dry spaces that may be subject to water vapor and slight seepage.
	1	Not resistant to water damage. Materials within this class require conditions of dryness.

Source: COE 1992 "Floodproofing Regulations"

Flooring Materials

Table 2 lists flooring materials commonly used in construction that fall within the five classes described in Table 1. Not all available construction and finishing materials are listed. For products not listed herein, manufacturers' literature should be reviewed for recommended uses. Such recommendations must be complied with fully. All masonry and wood products used in floodprone buildings must comply with the applicable materials standards of the nationally recognized standards organizations, such as the American Society for Testing and Materials (ASTM), the American Concrete Institute (ACI), and the American Wood Products Association (AWPA).

Basis for Classification of Flooring Materials

The classification of flooring materials is based on their vulnerability to damage from inundation by floodwaters. Class 1,2, and 3 flooring materials are not acceptable for below-BFE applications for one or more of the following reasons:

- Normal suspended-floor adhesives specified for above-grade use are water soluble or are not resistant to alkali or acid in water, including ground seepage and vapor.
- Flooring materials contain wood and wood products.
- Flooring materials are not resistant to alkali or acid in water,
- Sheet-type floor coverings (linoleum, rubber, and vinyl) restrict evaporation from below.
- Flooring materials are impervious but dimensionally unstable.

Table 2 Flooring Materials Classifications for Flood Resistance					
Types of Flooring Materials	Classes of Flooring				
	Acceptable		Unacceptable		
	5	4	3	2	1
Asphalt Tile[1]					●
With asphaltic adhesives			●		
Carpeting (glued down type)					●
Cement/bituminous, formed-in-place		●			
Cement/latex, formed-in-place		●			
Ceramic tile[1]					●
With acid-and alkali-resistant grout			●		
Chipboard					●
Clay tile	●				
Concrete, precast or in-situ	●				
Concrete tile	●				
Cork					●
Enamel felt-base floor coverings					●
Epoxy, formed-in-place	●				
Linoleum					●
Magnesite (magnesium oxychloride)					●
Mastic felt-base floor covering					●
Mastic flooring, formed-in-place	●				
Polyurethane, formed-in-place	●				
PVA emulsion cement					●
Rubber sheets[1]					●
With chemical-set adhesives[2,3]	●				
Rubber tile[1]					●
With chemical-set adhesives		●			
Silicone floor, formed-in-place	●				

Table 2 Flooring Materials Classifications for Flood Resistance					
Types of Flooring Materials	Classes of Flooring				
	Acceptable		Unacceptable		
	5	4	3	2	1
Terrazo		●			
Vinyl sheets (homogeneous) [1]					●
With chemical-set adhesives [2,3]	●				
Vinyl tile (homogeneous) [1]					●
With chemical-set adhesives		●			
Vinyl tile or sheets (coated on cork or wood product backings)					●
Vinyl-asbestos tile (semi-flexible vinyl) [1]					●
With asphaltic adhesives		●			
Wood flooring or underlay merits					●
Wood composition blocks, laid in cement mortar				●	
Wood composition blocks, dipped and laid in hot pitch or bitumen				●	
Pressure-treated lumber, .40 CCA [4]	●				
Naturally decay-resistant lumber [4,5]	●				

Notes:

1 Using normally specified suspended flooring (i.e., above-grade) adhesives, including sulfite liquor (lignin or "linoleum paste"), rubber/asphaltic dispersions, or "alcohol" type resinous adhesives (culmar, oleoresin)

2 Not permitted as Class 2 flooring

3 E.g., epoxy-polyamide adhesives or latex-hydraulic cement

4 Not in the COE list; added by FEMA

5 Refer to local building code for guidance

Wall and Ceiling Materials

Table 3 lists wall and ceiling materials commonly used in construction that fall within the five classes described in Table 1. Not all available construction and finishing materials are listed. For products not listed herein, manufacturers' literature should be reviewed for recommended uses. Such recommendations must be complied with fully. All masonry and wood products used in floodprone buildings must comply with the applicable materials standards of the nationally recognized standards organizations, such as the American Society for Testing and Materials (ASTM), the American Concrete Institute (ACI), and the American Wood Products Association (AWPA).

Basis for Classification of Wall and Ceiling Materials

The classification of wall and ceiling materials is based on their vulnerability to damage from inundation by floodwaters. Class 1, 2, and 3 wall and ceiling materials are not acceptable for below-BFE applications for one or more of the following reasons:

- Normal adhesives specified for above-grade use are water soluble or are not resistant to alkali or acid in water, including ground seepage and vapor.
- Wall and ceiling material contains wood, wood products, gypsum products, or other material that dissolves or deteriorates, loses structural integrity, or is adversely affected by water.
- Wall or ceiling material is not resistant to alkali or acid in water.
- Wall or ceiling material is impervious but is dimensionally unstable.
- Wall or ceiling materials absorb or retain water excessively after submergence.

Table 3 Walls and Ceiling Materials Classifications for Flood Resistance					
Types of Wall and Ceiling Materials	Classes of Walls and Ceilings				
	Acceptable		Unacceptable		
	5	4	3	2	1
Asbestos-cement board (and cement board[1])	●				
Brick, face or glazed	●				
Common				●	
Cabinets, built-in					
Wood				●	
Metal	●				
Cast stone (in waterproof mortar)	●				
Chalkboards					
Slate, porcelain glass, nucite glass	●				
Cement-asbestos				●	
Composition, painted				●	
Chipboard					●
Exterior sheathing grade				●	
Clay tile					
Structural glazed	●				
Ceramic veneer, ceramic wall tile-mortar set		●			
Ceramic veneer, organic adhesives				●	
Concrete	●				
Concrete block	●				
Corkboard				●	
Doors					
Wood hollow				●	
Wood, lightweight panel construction				●	
Wood, solid				●	
Metal, hollow	●				
Metal, Kalamein				●	

Table 3 Walls and Ceiling Materials Classifications for Flood Resistance					
Types of Wall and Ceiling Materials	Classes of Walls and Ceilings				
	Acceptable		Unacceptable;		
	5	4	3	2	1
Fiberboard panels, vegetable types					
Sheathing grade (asphalt coated or impregnated)				●	
Otherwise					●
Gypsum products					
Gypsum board (including greenboard[1])				●	
Keene's cement of plaster				●	
Plaster, otherwise, including acoustical				●	
Sheathing panels, exterior grade				●	
Glass (sheets, colored tiles, panels)		●			
Glass blocks	●				
Hardboard					
Tempered, enamel or plastic coated				●	
All other types				●	
Insulation					
Foam or closed-cell types		●			
Batt or blanket types					●
All other types				●	
Metals, non-ferrous (aluminum, copper, or zinc tiles)			●		
Metals, Ferrous	●				
Mineral fiberboard					●
Plastic wall tile (polystyrene, urea formaldehyde, etc.)					
Set in waterproof adhesives, pointed with waterproof grout			●		
Set in water-soluble adhesives				●	

Table 3 Walls and Ceiling Materials Classifications for Flood Resistance					
Types of Wall and Ceiling Materials	Classes of Walls and Ceilings				
	Acceptable		Unacceptable		
	5	4	3	2	1
Paint					
Polyester-epoxy and other waterproof types		●			
All other types					●
Paperboard					●
Partitions, folding					
Wood, pressure treated, .40 CCA minimum 1 (if not treated, then material is Class 2)	●				
Metal		●			
Fabric-covered					●
Partitions, stationary					
Wood, pressure treated, .40 CCA minimum 1 (if not treated, then material is Class 2)	●				
Metal	●				
Glass, unreinforced		●			
Glass, reinforced		●			
Gypsum, solid or block					●
Rubber, moldings and trim with epoxy polyamide adhesive or latex-hydraulic cement		●			
All other applications					●
Steel, (panels, trim, tile) with waterproof applications	●				
With non-waterproof adhesive				●	
Stone, natural solid or veneer, waterproof grout	●				
Stone, artificial non-absorbent solid or veneer, waterproof grout	●				
All other applications				●	

Table 3 Walls and Ceiling Materials Classifications for Flood Resistance

Types of Wall and Ceiling Materials	Classes of Walls and Ceilings				
	Acceptable		unacceptable		
	5	4	3	2	1
Strawboard					
Exterior grade (asphalt-impregnated kraft paper)				●	
All other types				●	
Wall covering					
Paper, burlap, cloth types					●
Wood					
Solid, standard				●	
Solid, naturally decay-resistant[1,2]	●				
Solid pressure treated, .40 CCA minimum[1]	●				
Plywood					
Marine Grade[1]	●				
Pressure treated, .40 CCA minimum[1]	●				
Exterior grade				●	
Otherwise					●

Note: 1 Not on the COE list; added by FEMA

2 Refer to local building code for guidance

Construction Examples

Flood-Resistant Materials in Buildings in Zones A, AE, A1-A30, AR, AO, and AH

Figure 1 illustrates a building elevated on solid foundation walls, over a crawlspace. The NFIP regulations require that the lowest floor be at or above the BFE. The construction method illustrated in Figure 1 meets this requirement. Note, however, that the flooring materials and supporting wood members are at or below the BFE. Therefore, in Figure 1, all materials supporting the lowest floor, including the flooring itself, must be made of flood-resistant materials.

To maximize the use of the area below the lowest floor, it is a common floodplain construction technique to elevate a building a full story (approximately 8 feet), even though the BFE may only be 4 or 5 feet above grade. In such cases, while the NFIP regulations require that Class 4 or 5 building materials be used below the BFE, FEMA strongly recommends that Class 4 or Class 5 materials also be used for the construction of the remainder of the building below the lowest floor. Flood damage from a greater-than-design flood event will thereby be reduced in the lower area.

Flood-Resistant Materials in Buildings in Zones V, VE, and V1-V30

All structural and non-structural materials installed below the BFE must be flood resistant . The NFIP regulations require that the bottom of the lowest horizontal structural member of the lowest floor (usually the floor beam or girder) of a building in Zone V, VE, or Vl-V30 be at or above the BFE. Therefore, all materials below the floor beam(s) must be flood resistant. This includes but is not limited to breakaway wall materials and open latticework. Breakaway walls will remain in place during low-level floods and must be flood resistant, so that they will not deteriorate over time after being soaked by floodwaters. Figure 2, on the next page, illustrates this requirement.

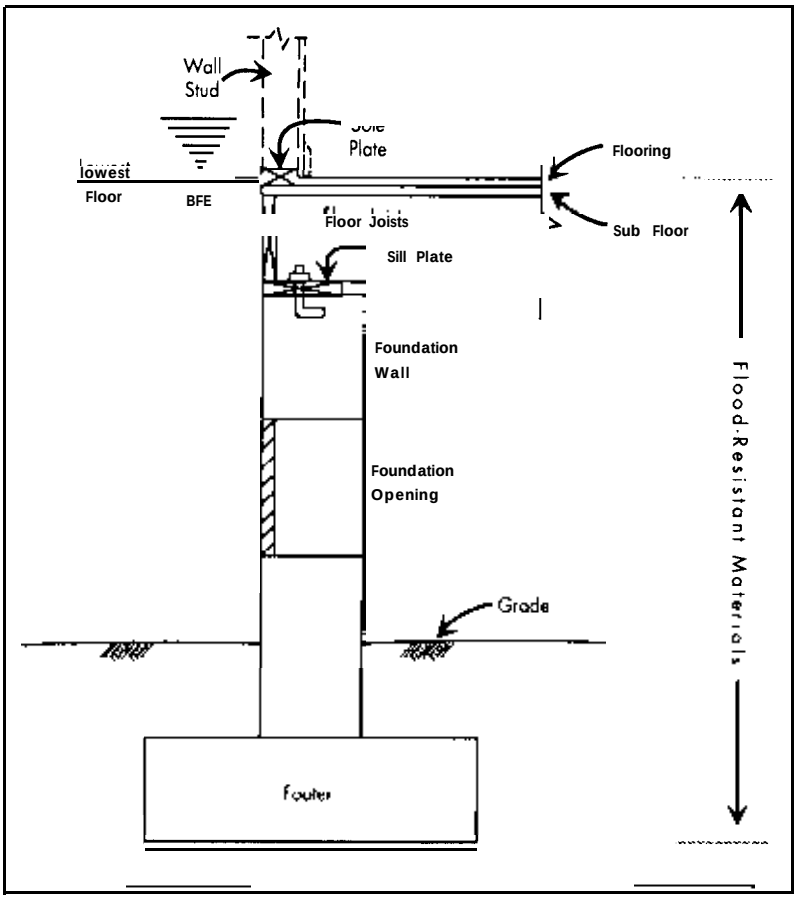

Figure 1. Building Elevated on Solid Foundation Walls Meeting the Minimum NFIP Requirements for Zones A, AE, A1-A30, AR, AO, and AH

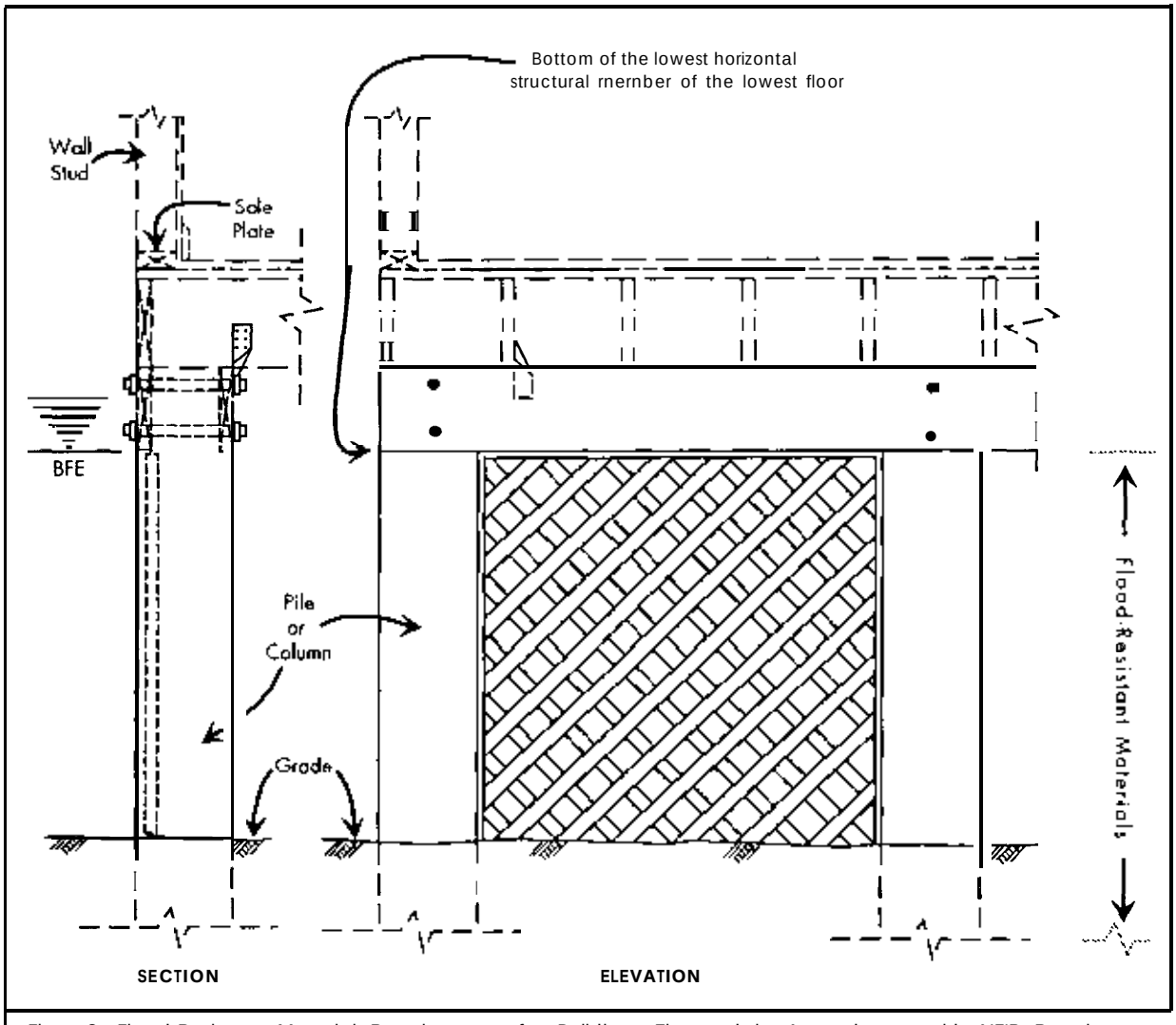

Figure2. Flood-Resistant Material Requirements for Buildings Elevated in Accordance with NFIP Requirements for Zones V, VE, and V1 -V30

Accessory Buildings

Some communities permit the construction of low-cost, small detached accessory buildings (e.g., garages, storage sheds) with a lowest floor elevation below the BFE (Technical Bulletin 5, "Free-of-Obstruction Requirements," provides definitions of "low-cost" and "small"). The below-BFE portions of such buildings must be constructed of flood-resistant materials so that flood damage will be minimized. Additional construction requirements for these buildings, such as the need to anchor the building to resist flotation, collapse, and lateral movement, also must be met before the building is permitted and built. For additional information about these requirements, contact the community that has permitting jurisdiction.

Wet Floodproofing

Wet floodproofing is designing a building to allow floodwaters to enter in order to equalize hydrostatic forces. The NFIP does not allow wet floodproofing in lieu of meeting the lowest

floor elevation requirements. However, in situations where the NFIP regulations do not apply, such as voluntary floodproofing of an existing (Pre-FIRM) building not in association with substantial improvements, the use of flood-resistant materials is advisable. Using flood-resistant materials will make cleanup and repair following a flood much easier and less costly than if the floodprone areas are constructed of non-flood-resistant materials.

The NFIP

The NFIP was created by Congress in 1968 to provide federally backed flood insurance coverage, because flood insurance was generally unavailable from private insurance companies. The NFIP is also intended to reduce future flood losses by identifying floodprone areas and ensuring that new development in these areas is adequately protected from flood damage. The NFIP is based on an agreement between the federal government and participating communities that have been identified as floodprone. FEMA, through the Federal Insurance Administration (FIA), makes flood insurance available to the residents of a participating community provided that the community adopts and enforces adequate floodplain management regulations that meet the minimum NFIP requirements. The NFIP encourages communities to adopt floodplain management ordinances that exceed the minimum NFIP criteria. Included in the NFIP requirements, found under Title 44 of the U.S. Code of the Federal Regulations, are minimum building design and construction standards for buildings located in SFHAs. Through their floodplain management ordinances, communities adopt the NFIP design performance standards for new and substantially improved buildings located in floodprone areas identified on FIA's FIRMs.

Technical Bulletins

This is one of a series of Technical Bulletins FEMA has produced to provide guidance concerning the building performance standards of the NFIP. These standards are contained in Title 44 of the U.S. Code of Federal Regulations at Section 60.3. The bulletins are intended for use primarily by State and local officials responsible for interpreting and enforcing NFIP regulations and by members of the development community, such as design professionals and builders. New bulletins, as well as updates of existing bulletins, are issued periodically, as necessary. The bulletins do not create regulations; rather they provide specific guidance for complying with the minimum requirements of existing NFIP regulations. Users of the Technical Bulletins who need additional guidance concerning NFIP regulatory requirements should contact the Natural Hazards Branch of the appropriate FEMA regional office. The "User's Guide to Technical Bulletins" lists the bulletins issued to date and provides a key word/subject index for the entire series.

Ordering Information

Copies of the Technical Bulletins can be obtained from the appropriate FEMA regional office. Technical Bulletins can also be ordered from the FEMA publications warehouse. Use of FEMA Form 60-8 will result in a more timely delivery from the warehouse — the form can be obtained from FEMA regional offices and your state's Office of Emergency Management. Send publication requests to FEMA Publications, P,O. Box 70274, Washington, D.C. 20024.

Further Information

The following publications provide further information concerning the use of flood-resistant materials:

1. "Answers to Questions About Substantially Damaged Buildings," FEMA, May 1991, FEMA-213.
2. "Floodproofing Non-Residential Structures," FEMA, May 1986, FEMA- 102.
3. "Flood Proofing Regulations", Chapters 9 and 10, U.S. Army Corps of Engineers, March 1992, EP 1165-2-314.
4. "Flood Proofing Systems and Techniques," U.S. Army Corps of Engineers, December, 1984.
5. "Repairing Your Flooded Home," FEMA and the American Red Cross, August 1992, FEMA-234, ARC 4477.
6. "Technical Notes for Brick Construction,''Brick Institute of America, McLean, Virginia, n.d.

Glossary

Base flood — The flood that has a 1-percent probability of being equaled or exceeded in any given year (also referred to as the 100-year flood).

Base Flood Elevation (BFE) — The height of the base flood, usually in feet, in relation to the National Geodetic Vertical Datum of 1929 or other datum as specified.

Basement — Any area of a building having its floor subgrade (below ground level) on all sides.

Coastal High Hazard Area — An area of special flood hazard extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area subject to high-velocity wave action from storms or seismic sources.

Federal Emergency Management Agency (FEMA) — The independent federal agency that, in addition to carrying out other activities, oversees the administration of the National Flood Insurance Program.

Federal Insurance Administration (FIA) — The component of FEMA directly responsible for administering the National Flood Insurance Program.

Flood Insurance Rate Map (FIRM) — The insurance and floodplain management map issued by FEMA that identifies, on the basis of detailed or approximate analyses, areas of 100- year flood hazard in a community.

Floodprone area — Any land area susceptible to being inundated by floodwater from any source.

Lowest floor — The lowest floor of the lowest enclosed area of a building, including a basement. Any NFIP-compliant unfinished or flood-resistant enclosure useable solely for parking of vehicles, building access, or storage (in an area other than a basement) is not considered a building's lowest floor.

Special Flood Hazard Area (SFHA) — Area delineated on a Flood Insurance Rate Map as being subject to inundation by the base flood and designated as Zone A, AE, A1-A30, AR, AO, AH, V, VE, or V1-V30.

Substantial damage — Damage of any origin sustained by a structure whereby the cost of restoring the structure to its before-damaged condition would equal or exceed 50 percent of the market value of the structure before the damage occurred.

Substantial improvement — Any reconstruction, rehabilitation, addition, or other improvement of a structure, the cost of which equals or exceeds 50 percent of the market value of the structure before the "start of construction" of the improvement. This term includes structures that have incurred "substantial damage," regardless of the actual repair work performed.