RESOURCE TECHNOLOGY, INCORPORATED

~~7800 MARBLE AVENUE NE, SUITE 5~~
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ALBUQUERQUE, NM 87110
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F-10/D6

April 8, 1986

Mr. Roger Green, P.E.
CE, Design Hydrology
Design Hydrology Section
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

RE: DRAINAGE REPORT FOR SANTA FE VILLAGE UNIT III, PHASES I AND II

Dear Roger:

In response to your letter dated February 26, 1986 we are pleased to submit the following response to your concerns regarding the sizing of sidewalk culverts.

You may recall from one of our meetings in early March that we discussed removal of sidewalk culverts at Vulcan Parkway (at the South Branch of San Antonio Arroyo), Butte Place, Cliffrose Road and Ja Court. The Design Review Committee requested the removal of the sidewalk culverts at Butte Place, Cliffrose Road and Ja Court and preferred a depressed sidewalk entrance to the concrete rundowns to allow access for maintenance vehicles.

The three concrete rundowns have a 9-foot bottom width; however, only the depressed sidewalk at Butte Place will have a 9-foot bottom width. The depressed sidewalk width at Cliffrose Road and Ja Court are 10.4 feet due to a 60 degree angle of the rundowns with the right-of-way boundary. Figure 1 is a copy of the typical depressed sidewalk-rundown section as submitted on the construction drawings.

The capacity for each depressed sidewalk entrance was determined by the formula Q = 0.67 (L)(2(32.2)**0.5)(H**1.5)(King and Brater, p. 4-5). The capacity of the 9-foot opening with a 0.67-foot curb = 26 cfs. and the capacity of a 10.4-foot opening with a 0.67-foot curb = 30.67 cfs. However, due to the large 100-yr. peak discharge at Cliffrose Road of 67.72 cfs., (assuming that the storm sewer, with 27 cfs. capacity, is not operating) a modification of the typical rundown header curb is necessary to contain the peak discharge as discussed below and shown on Figure 1.

Using the above equation assuming Q = 67.72 cfs. and L = 10.4 feet then H = 1.14 feet (approximately 1.2 feet). To accomodate a depth of 1.20 feet, the header curb at the back of sidewalk will be the same elevation as the rundown wall at Point D (see Figure 1) and the header curb will be extended out until reaching the sidewalk elevation equal to Point C plus 1.2 feet. Therefore, for the 100-yr. peak discharge of 67.72 cfs. (assuming no storm sewer) the water may pond up to the header curb for a few minutes during the peak discharge. The header curb will protect the adjacent lots, and the house slabs adjacent to the rundown will also be raised (from approved grading plan) to add extra safety from flooding.

The header curb discussed above was sized assuming the storm sewer was not operational. Assuming the storm sewer was able to accept 27 cfs. as determined in the drainage report, the 100-yr. peak discharge reaching the rundown at Cliffrose Road equals 67.72 cfs. - 27 cfs. = 40.72 cfs. Using the equation given above and solving for H, H = 0.81 feet. Therefore the header curb will protect the lots adjacent to the rundown at Cliffrose Road in both cases discussed above.

The following table is a summary of the 100-yr. peak discharges and entrance capacities for the depressed sidewalk entrances.

	L (ft.)	H Avail. (ft.)	Capacity Qp (cfs.)	100-yr. Qp (cfs.)	H Reqd. (ft.)
Cliffrose Road*	10.4	1.20	67.72	67.72	1.20
Cliffrose Road **	10.4	1.20	67.72	40.72	0.81
Ja Court	10.4	1.00	30.67	7.95	0.27
Butte Place	9	1.00	26	12.60	0.41

* Assuming storm sewer is non-operational
** Assuming storm sewer is operating at maximum capacity of 27 cfs.

At the Vulcan Parkway crossing, large depressed sidewalks will be constructed on both sides of the street as shown on Figure 2. The 100-yr. peak discharges reaching the Vulcan Parkway crossing from Rincon and Santa Fe Village Unit III are 81.57 cfs. and 25.72 cfs., respectively for a total of 107.29 cfs. The depressed sidewalk lengths required were determined using the formula given above. The length required to pass 53.65 cfs. (0.5*107.29 cfs.) through the depressed sidewalk at each flow line (assuming H = 0.67 feet) is 18.20 feet.

However, due to the large 100-yr. Qp of 107.29 cfs., the depressed sidewalk entrance capacaity was checked using a weir formula Q = CLH**3/2. The assumed C = 3.13 (Table 5-13, Figure 5-23, King and Brater) and H = 0.67 feet. The maximum length at the east flowline was limited by the concrete channel width and the length of the 12-foot sidewalk transition. The available length = 25 feet and the maximum capacity is 43 cfs. at a depth of 0.67 feet. Therefore 107.29 cfs. - 43 cfs. = 64.29 cfs. which must pass through the depressed sidewalk section at the west flowline. The length of the west depressed sidewalk section is 38 feet. Therefore, the 100-yr. peak discharge will be safely discharged into the channel below Vulcan Parkway.

The capacity of the three 2-foot wide sidewalk culverts located on Vulcan Parkway at the Type C storm inlet were resized using the weir formula given above (C = 3.13, H = 0.67) because the flow will enter the culverts similar to a weir due to an inclined approach. Therefore, with a length of 6 feet the maximum capacity of the 3 culverts = 10.3 cfs. The 100-yr. peak Q reaching the sidewalk culverts is 21.47 cfs., with the storm sewer operating, and the 10-yr. is 4.9 cfs. Therefore the 10-yr. peak Q will pass through the culverts and any flows exceeding the capacity of the culverts will flow over the steel plate tops and discharge into Vulcan Parkway. Thus 11.17 cfs. (21.47 cfs. - 10.3 cfs. = 11.17) will flow over the steel plate tops in the 100-yr. peak flow. The flow exceeding the culvert capacity will discharge across the 9-foot long right-of-way as overland flow to Vulcan Parkway.

The capacity of the remaining sidewalk culverts located along the south flow line of Vulcan Parkway were resized using the equation Q = 0.67 (L) (2(32.2)**0.5)(H**3/2) as discussed previously. The capacity of a 1-foot wide sidewalk culvert is 2.9 cfs. and a 2-foot wide culvert is 5.9 cfs. (assuming H = 0.67 feet). The following Table is a summary of the culvert locations, 10-yr. peak discharges and proposed number of culverts as given in the drainage plan.

Sub-basin	10-yr. Qp (cfs.)	No. & Size of Sidewalk Culverts Planned	Capacity (cfs.)	H Avail (ft.)	H Reqd. (ft.)
B	1.76	1 1-ft.	2.9	0.58	0.32
K	1.32	2 1-ft.	5.8	0.58	0.12
N	0.77	1 1-ft.	2.9	0.58	0.14
O	0.56	1 1-ft.	2.9	0.58	0.10
Q	4.12	2 2-ft.	11.8	0.58	0.19

The sidewalk culverts which were planned at the Bogart Street crossing have been eliminated because the low point in Bogart Street now lies north of the channel. A storm sewer with accompanying storm inlets will be constructed in Santa Fe Village Unit I at a later date to accomodate these flows. A berm will be constructed across Bogart Street north of the channel as a temporary solution. Ponding against the berm will be eliminated by use of a slotted man hole cover. This is shown on the construction drawings.

In summary, the sidewalk culverts have been eliminated and replaced with depressed sidewalks at Vulcan Parkway, Butte Place, Cliffrose Road and Ja Court. The sidewalk culverts at Bogart Street have been eliminated and a storm sewer will discharge flows into the concrete channel. The remaining sidewalk culverts located along the south flow line of Vulcan Parkway will adequately carry the 10-yr. peak discharge. Should flow exceed the culvert capacity, the flow will overtop the steel plate and discharge into Vulcan Parkway which is the desired drainage location.

Very truly yours,

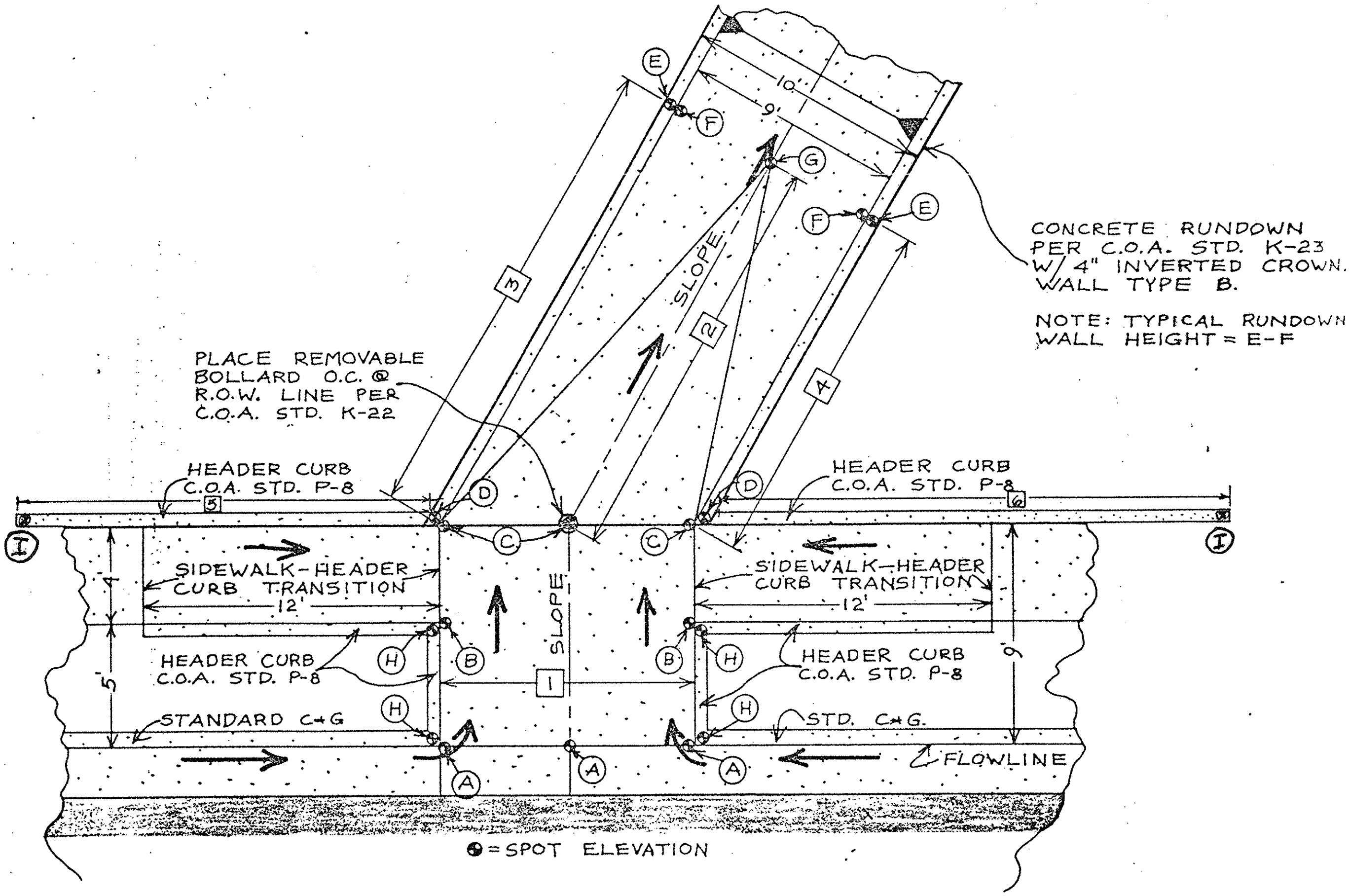
Pat Stovall

Pat Stovall
Hydrologist

PS/ec

		CLIFFROSE ROAD	J.A. COURT	BUTTE PLACE
ELEVATIONS (FT.)	A	36.04	34.57	45.57
	B	36.00	34.40	45.50
	C	35.93	34.26	45.44
	D	37.43	35.58	46.58
	E	37.23	35.00	46.25
	F	35.73	34.00	45.24
	G	35.40	33.67	44.91
	H	36.73	35.25	46.25
	I*	37.13	—	—
DIMEN. (FT.)	1	10.40	10.40	9.00
	2	45.00	17.40	38.00
	3	47.26	20.00	38.00
	4	42.40	14.80	38.00
	5	19.20	12.00	12.00
	6	24.00	12.00	12.00
SLOPE		1.19%	3.39%	1.39%
RUNDOWN BEARING		N 30° 13' 46" E	N 30° 13' 46" E	N 20° 36' 57" E

* SIDEWALK ELEVATION



DEPRESSED SIDEWALK AT RUNDOWN (TYPICAL)

NOT TO SCALE

FIGURE 1

X.C.

ALBUQUERQUE NEW MEXICO 1706

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

DESIGN HYDROLOGY SECTION
123 Central NW, Albuquerque, NM 87102
(505) 766-7644

May 9, 1986

Elvidio Diniz, P.E.
Resource Technology, Inc.
2620 San Mateo Blvd., NE
Albuquerque, New Mexico 87119

RE: DRAINAGE REPORT SUBMITTAL OF PRUDENT LINE ANALYSIS OF THE SOUTH BRANCH OF THE SAN ANTONIO ARROYO RECEIVED APRIL 3, 1986 FOR PRELIMINARY PLAT APPROVAL OF SANTA FE VILLAGE UNIT III, PHASE II (F-10/D6)

Dear Elvidio:

I have no adverse comments on the above referenced submittal dated April 3, 1986, but it cannot be approved until City Open Space Division either accepts or rejects the maintenance costs and responsibilities identified by your report. Table 6 was forwarded to Rex Funk of Open Space on April 15, 1986, for his response, which I as yet have not received.

As discussed previously, Rex would like to investigate the possibility of watershed treatment upstream in lieu of downstream maintenance or structural channel improvements.

I will again address approval of the drainage report after these maintenance and treatment questions have been resolved.

Cordially,

Roger A. Green, PE

Roger A. Green, P.E.
C.E./Hydrology Section

cc: Rex Funk, Open Space Division
Coda Roberson

RAG/bsj

F10/D6A

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

ALBUQUERQUE NEW MEXICO 1706

KEN SCHULTZ
MAYOR

RECEIVED
MAY 15 1989
HYDROLOGY SECTION

May 15, 1989

CERTIFICATE OF COMPLETION AND ACCEPTANCE

Coda Roberson
Roberson Construction
6001 Atrisco Road N.W.
Albuquerque, NM 87120

RE: TURNKEY PROJECT NO. 2794, SANTA FE VILLAGE, UNIT III, PHASE I, (MAP NO. F-10

Dear Mr. Roberson:

The above referenced project has been completed according to the plans and specifications. The project consisted of concrete lining and all preparatory and storm drainage connections on the South Branch of San Antonio Arroyo.

The City of Albuquerque accepts the referenced project as a whole and the contractual correction period began on March 18, 1988. The correction period on this project is for three (3) years.

Sincerely,

Russell B. Givler

Russell B. Givler, P.E.
Chief Construction Engineer
Construction Management Division
Engineering Group
Public Works Department

SE 1/4 T81

RG:tjp
2706

City of Albuquerque
P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

DESIGN HYDROLOGY SECTION
123 Central NW, Albuquerque, NM 87102
(505) 766-7644

January 2, 1984

Mr. Gary Tibljas
Denney-Gross & Associates
2400 Comanche Road NE
Albuquerque, NM 87108

REF: SANTA FE VILLAGE UNIT III, DATED OCTOBER 1984 (F10-D6)

Dear Gary:

The proposed discharge scheme of draining developed flows and off-site flows to the South San Antonio Arroyo appears to be appropriate; however, this concept will be contingent on downstream capacity and channel stability. Also the flow calculated for the South San Antonio Arroyo is not consistant with the adopted drainage management plan by Matotan and Associates.

Your request for site development plan approval cannot be granted at this time without an approved report per the DPM that addresses those issues above.

In addition, the proposed channelization of the arroyo will require a map revision in accordance with FEMA guidelines (see attached FEMA guidelines).

If you have any questions, please feel free to call me at 766-7644.

Sincerely yours,

Fred J. Aguirre
Design Hydrologist

FJA:mrk

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

DESIGN HYDROLOGY SECTION
123 Central NW, Albuquerque, NM 87102
(505) 766-7644

September 24, 1985

Mr. Elvidio Diniz
Resource Technology, Inc.
2620 San Mateo NE, Suite, B
Albuquerque, NM 87102

REF: REVISED CONCEPTUAL DRAINAGE REPORT FOR SANTA FE VILLAGE UNIT III (F10-D6) RECEIVED AUGUST 30, 1985 & SEPTEMBER 23, 1985

Dear Mr. Diniz:

The information presented today is in compliance with our meeting on September 20, 1985 regarding the previous submittal for conceptual plan approval. Consequently, conceptual drainage plan approval is hereby granted and the Site Development Plan can be signed-off at DRB.

Please be advised that as per our discussion, this date, a complete comprehensive drainage report is to be submitted for review and approval prior to application for preliminary plat approval at DRB.

Should you have any questions or comments, please call this office at 766-7644.

Cordially,

Billy J. Goolsby

Billy J. Goolsby
CE/Design Hydrology

BJG/cl

City of Albuquerque
P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

DESIGN HYDROLOGY SECTION
123 Central NW, Albuquerque, NM 87102
(505) 766-7644

January 9, 1986

Elvidio Diniz, P.E.
Resource Technology, Inc.
2620 San Mateo Blvd., NE Suite B
Albuquerque, New Mexico 87110

RE: DRAINAGE REPORT FOR SANTA FE VILLAGE, UNIT III
PHASE I & II SUBMITTED JANUARY 7, 1986 FOR PRELIMINARY PLAT
APPROVAL (F-10/D6)

Dear Elvidio:

The referenced submittal dated January 6, 1986, is approved for Preliminary Plat, Phase I only.

As discussed with your office today, some lot elevations along retaining walls are in error and should be revised on grading plan. Also, a retaining wall was omitted between Lotss 45 and 46.

Preliminary Plat approved for Phase II cannot be granted until arroyo treatment is resolved above Vulcan Parkway.

If you have any questions, call me at 766-7644.

Cordially,

Roger A. Green, PE

Roger A. Green, P.E.
C.E./Design Hydrology

cc: Coda Roberson

RAG/bsj

FILE COPY

City of Albuquerque

P.O. BOX 1293   ALBUQUERQUE, NEW MEXICO 87103

DESIGN HYDROLOGY SECTION
123 Central NW, Albuquerque, NM 87102
(505) 766-7644

March 28, 1986

Elvidio Diniz, P.E.
Resource Technology, Inc.
2620 San Mateo Blvd., NE Suite B
Albuquerque, New Mexico 87110

RE: GRADING PLAN OF SANTA FE VILLAGE, UNIT III, PHASE I
(F-10/D6)

Dear Elvidio:

The above referenced Grading Plan, dated March 24, 1986, is approved for Rough Grading Permit as indicated by approval signature on mylar drawings.

Please provide this office with a blue line copy for our files.

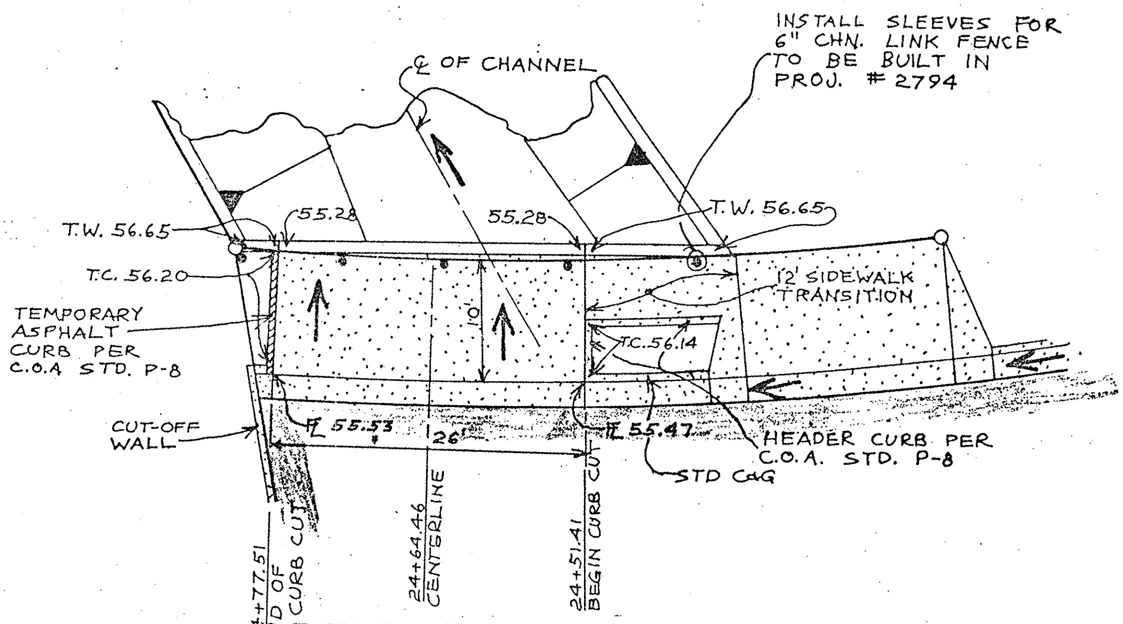
If you have any questions, call me at 766-7644.

Cordially,

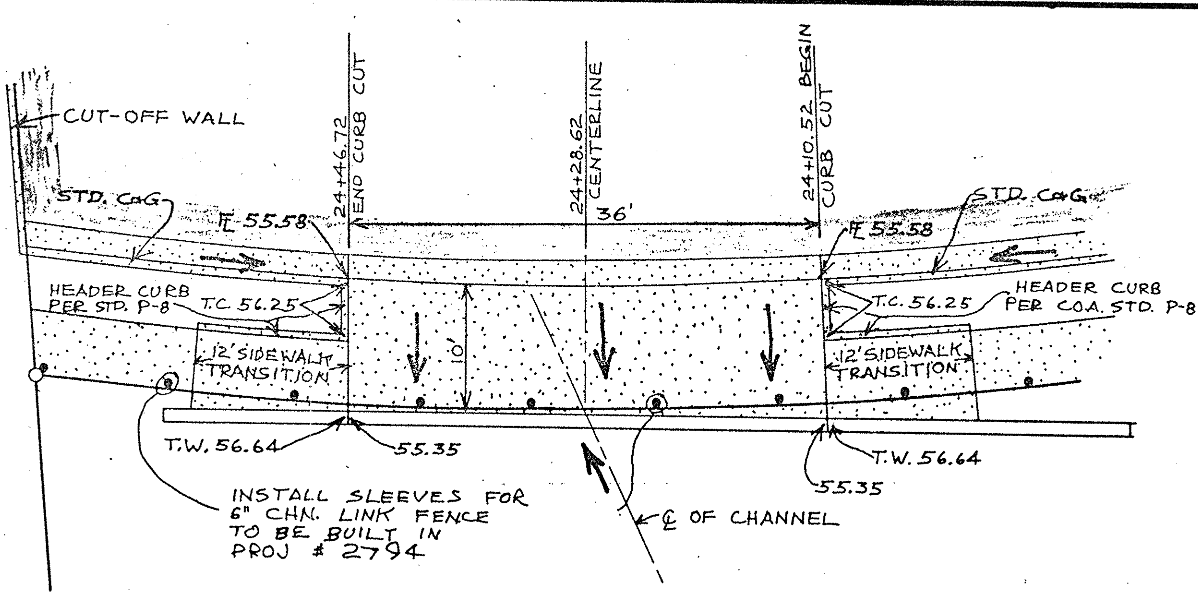
Roger A. Green, PE

Roger A. Green, P.E.
C.E./Design Hydrology

RAG/bsj



DEPRESSED SIDEWALK AT VULCAN ROAD CROSSING– EAST SIDE



DEPRESSED SIDEWALK AT VULCAN ROAD CROSSING– WEST SIDE

FIGURE 2

DRAINAGE REPORT FOR

SANTA FE VILLAGE UNIT III,

PHASES I AND II

ALBUQUERQUE, NEW MEXICO

prepared for

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

by

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

October 21, 1985

DRAINAGE REPORT FOR
SANTA FE VILLAGE UNIT III
PHASE I AND II

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INTRODUCTION

Santa Fe Village Unit III will be a 70-acre subdivision which has been divided into Phases 1 through 4 for development. The subdivision is located on Zone Atlas Page F-10. The developer is Roberson Construction Company; the planner is Richard Elliott Architects with topographic mapping by Denney-Gross Associates and boundary surveying by Southwest Surveying Company. Appendix A is the plat for Santa Fe Village Unit III. This drainage report covers Santa Fe Village Unit III, Phase I and Phase II. Phase I will be comprised of townhouses (20 acres) and Phase II will be comprised of apartments (9 acres).

This report generally conforms to and details the Conceptual Drainage Plan for Santa Fe Village Unit III, which was submitted to and approved by the City Design Hydrology Section on 9-24-85.

SITE DESCRIPTION

Location

Figure 1 shows the location of the proposed Santa Fe Village Unit III, Phase I and Phase II. Phase I, which is approximately 20 acres in area, is bounded on the east by Unser Blvd. NW (Atrisco Rd. NW), on the north by the South Branch of San Antonio Arroyo and on the west and south by Vulcan Parkway (a proposed street leading into Santa Fe Village Unit III). Phase II, which is approximately 9 acres in area, is located south of Vulcan Parkway and extends to the base of the volcanic escarpment. The mesa area above the escarpment is part of Boca Negra Park, which is an open space preserve of the City of Albuquerque.

The subdivision site includes parts of the present 100-yr. flood plain along the South Branch of San Antonio Arroyo; however, as discussed in the Conceptual Drainage Plan, this arroyo will be confined in a concrete channel within an Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) Right-of-Way (R.O.W.). The present AMAFCA R.O.W. is 70 feet wide and is only dedicated from the AMAFCA gabion grade control structure immediately upstream from the Unser Blvd. (Atrisco Rd. NW) crossing to the southwest boundary of Santa Fe Village Unit I which is located north of Phase I (see Figure 1).

In conformance with the Conceptual Drainage Plan, this report will reduce the 70-foot R.O.W. to 50 feet to accomodate the concrete channel which will confine the South Branch of San Antonio Arroyo, and will also extend and define the new AMAFCA R.O.W. along the arroyo to the base of the volcanic escarpment. The reduction of R.O.W. width has been discussed with AMAFCA, and a request for the new R.O.W. has been submitted to the AMAFCA Board of Directors.

Soils

Figure 2 shows the on- and off-site soils for the site as presented in the U.S. Soil Conservation Service report entitled "Soil Survey for Bernalillo County and Parts of Sandoval and Valencia Counties, New Mexico". The project site is located on the Bluepoint-Kokan association (BKD). The off-site soils include the Bluepoint-Kokan association, Kokan-Rock outcrop association (KR) and the Alemeda sandy loam (AmB). The following descriptions of each soil type are summarized from the Soil Conservation Service report.

> Bluepoint-Kokan association (BKD) - About 50% Bluepoint loamy fine sand and 40% Kokan gravely sand. Hydrologic Soil Group A.
>
> Kokan-Rock outcrop association (KR) - About 75% Kokan gravelly sand on 25% to 45% slopes, with about 40% of the surface covered with large basalt boulders. This soil is located off-site on the volcanic escarpment. Hydrologic Soil Group A.
>
> Alemeda sandy loam (AmB) - About 10% to 30% of Alemeda sandy loam consists of basalt rock outcrop and Akela soils. This soil is located on top of the volcanic escarpment. Hydrologic Soil Group C.

Land Use

The proposed development area is presently undeveloped with no structures; however, AMAFCA has a large gabion grade control structure located in San Antonio Arroyo. Grading has begun on Santa Fe Village Unit I which is located adjacent to the north boundary of the 50-foot AMAFCA R.O.W..

The proposed Phase 1 development will be comprised of townhouses and is located between Vulcan Parkway and the 50-foot AMAFCA R.O.W. as shown on

Figures 3A and 3B (in pocket at end of this report) which also show the proposed drainage plan. Phase II lies between Vulcan Parkway and the volcanic escarpment, as also shown in Figures 3A and 3B, and will consist of apartments.

At present, Unser Blvd. NW (Atrisco Road NW) is a paved two lane road with no curbs or gutters. Within the next two years, this roadway will be improved to a 4-lane divided arterial with curbs and gutters. The proposed intersection of Vulcan Parkway with Unser Blvd. NW will be a full intersection; however, left turns out of Vulcan Parkway will not be allowed. The area between Unser Blvd. NW and Bogart Street is part of Phase III, a future commercial development phase of Santa Fe Village Unit III. Drainage conditions for Phase III will be considered under a separate drainage report.

According to the Federal Emergency Management Agency flood hazard maps, most of the development area is in Flood Hazard Zone C with the areas near the South Branch of San Antonio Arroyo in Flood Hazard Zone AO. However, a trapezoidal shaped concrete lined channel will confine the arroyo flow within the 50-foot AMAFCA R.O.W. from the Vulcan Parkway crossing to the AMAFCA gabion grade control structure. Therefore, the proposed development area will be protected from flooding caused by the 100-yr. and lesser magnitude floods on the South Branch of San Antonio Arroyo.

OFF-SITE FLOWS

South Branch of San Antonio Arroyo

Most of the area located above the escarpment and west of the point where the South Branch of San Antonio Arroyo flows down the escarpment, drains through the South Branch of San Antonio Arroyo. The existing condition 100-yr. peak discharge at Unser Blvd. NW for the 855-acre watershed is 762 cfs. with a time to peak of 1.5 hours as determined previously by Fred Denney and Associates Inc. in "Hydrologic Report San Antonio - Mariposa and Boca Negra Arroyos" (October 1980). The City of Albuquerque Engineering staff and AMAFCA have approved the report, and the hydrology presented in that report was used in this drainage report.

Based on flow proportioning by means of a drainage area ratio, the 100-yr. peak discharge at the Vulcan Parkway crossing of the arroyo was estimated to be 651 cfs. Developed conditon runoff rates and channel hydraulics are discussed in a later section of this report.

Tributary Flows

The local off-site areas contributing flow to the proposed development area are located west of Phase I and south of Phase II. Off-site flows originate from on top of the escarpment and flow through Phase II and then Phase I to the South Branch of San Antonio Arroyo. Boca Negra Park is located on top of the escarpment and therefore no development will occur above the escarpment.

At present several small tributaries to the South Branch of San Antonio Arroyo drain from the mesa to the arroyo from the south side. Within the development area, flow from these tributaries will be diverted down the subdivision streets except in the case of Black Loop - West which will also have a storm sewer system.

The escarpment area is presently owned by the developer of Santa Fe Village Unit III, but it will remain undeveloped in accordance with the Albuquerque Comprehensive Plan and may eventually be deeded to the City of Albuquerque for open space uses. Off-site flows were considered for Phase I and Phase II as follows:

Phase I - Assume no future development south of Vulcan Parkway (Phase II). The undeveloped off-site areas were used to determine the discharge at Vulcan Parkway.

Phase II - Assume future development of Santa Fe Village Unit III Phase II, which will consist of apartments between Vulcan Parkway and the volcanic escarpment. The undeveloped off-site areas were used to determine the discharge at the top apartment drive.

Phase I

Figure 4 shows the off-site sub-basins for Phase I, which will be constructed before Phase II. In Figure 4, off-site flows from sub-basins S, T, W and X will reach Vulcan Parkway as overland flow. These flows will be collected into sidewalk culverts to cross under the sidewalk and drain into Vulcan Parkway.

Sub-basin V is the largest sub-basin with an area of 17.4 acres and a time of concentration (Tc) of 9.1 minutes; Tc is assumed to be 10 minutes according to DPM criteria for Tc. The data used in this computation are shown on Table 1 which also lists the corresponding off-site flow data. All other off-site sub-basins are much smaller than Sub-basin V, therefore, they will have a Tc

ON-SITE FLOWS

Basis of Plan

The general plan for on-site drainage is to carry the combined on- and off-site flows in the street gutters, where possible, and then discharge into the concrete lined channel through several rundowns located in drainage easements; however, a 24-inch storm sewer down Black Loop-West will also be used to convey runoff from off-site sub-basins C,D,E,F,K,L,M and N. These sub-basins and the alignment of the storm sewer are shown on Figures 3A and 3B. The storm sewer will be discussed in detail in a later section of this report.

The on-site area was divided into sub-basins (as shown on Figures 3A and 3B) based on several factors which are as follows:

1. The street profiles.
2. The location of natural and proposed drainage divides.
3. The lot elevations required along the concrete channel based on the depth with adequate freeboard needed to contain the South Branch of San Antonio Arroyo within the 50-foot R.O.W..
4. The orientation of the lots to the streets and the proposed drainage easements leading into the concrete channel.

Figures 3A and 3B show the street and apartment drive slopes, drainage divides, sub-basins, flow directions and the locations of drainage structures which will be discussed in the following sections. Figures 3A and 3B also show the location of all analysis points which are used to determine flow characteristics in the streets and at important drainage locations.

Figure 7 shows a typical cross section through the Phase II area from the base of the escarpment to Vulcan Parkway.

Sidewalk culverts are proposed at several locations in the study area to provide drainage into the streets or from streets to the South Branch of San Antonio Arroyo. In all cases the sidewalk culverts are located at low points in the sidewalk. Whenever possible the 100-yr. flow is diverted through these culverts; however, in some cases the culverts were only sized for the 10-yr. flow and the 100-yr. flow will drain over the culverts but remain within the low point in the sidewalk.

OC

Phase II Drainage

Phase II drainage is considered first because most flow from Phase II will drain through Phase I. The Time of Concentration calculation for Sub-basin E (the largest sub-basin) is shown on Table 5. The Runoff Coefficient "C" was determined as an area weighted "C" factor of the impervious and pervious areas within each sub-basin. The impervious and pervious areas were measured from Figures 3A and 3B which show the site plan for Phase II. Table 5 lists the developed condition flows for Phase II sub-basins as shown on Figures 3A and 3B. The sub-basin areas in Table 5 and Figures 3A and 3B are slightly larger than the areas in Table 2 and Figure 6 because of the inclusion of portions of the apartment drive areas

Apartment Drives and Parking Areas - The apartment drives are labeled Drive A, Drive B and Drive C and most parking areas will drain to these drives. The apartment drives will have no crown and will have a downhill cross slope of 2%. The flow line will be located at the curb on the downhill side, as shown on

Figure 8. The flow line between parking areas on the downhill side and the drives will be constructed in concrete valley gutters; and curbs and gutters will be used in all other areas. Figure 8 also shows a typical apartment drive cross section with the flow line across parking areas.

The uphill (southern) side of Drive A will have parking areas in some locations as shown in Figures 3A and 3B. All pavement on the south side will be bordered by a concrete header curb with a maximum height of 12 inches. All excavated slopes in excess of 1V:3H will be lined with architectural paving blocks. No retaining walls are planned.

The Phase II area was divided into five drainage areas with each area consisting of one or more sub-basins. Each drainage area, with its respective sub-basins and drainage structures, is discussed in the following sub-sections.

PHASE II - DRAINAGE

Drainage Area 1

This drainage area is comprised of Sub-basin A which will remain undeveloped (except for 1/2 of Vulcan Parkway). The drainage divide between drainage areas 1 and 2 begins on Vulcan Parkway approximately half of the distance between Black Loop - East and Ja Court. Storm runoff from this area will be carried east in the southern gutter of Vulcan Parkway.

A previous drainage plan has been prepared for the commercial area of Santa Fe Village Unit III titled Drainage Plan for Santa Fe Village Unit III - Commercial Area, Albuquerque, New Mexico. The runoff from Drainage Area 1 (Sub-basin A) was considered in that plan. Flows from the commercial area and from Drainage Area 1 will discharge into a 3.5-foot wide trench drain located across Vulcan Parkway, 130 feet west of Unser Blvd. NW. (Figure 3A). This trench drain will discharge directly into the AMAFCA gabion grade control structure at Unser Blvd. NW.. This drainage plan assumes approval of the Commercial Area Drainage Plan including the 3.5 ft. wide trench drain across Vulcan Parkway.

PHASE II - DRAINAGE
Drainage Area 2

This drainage area is comprised of Sub-basin B. Flows from Sub-basin B will be carried down the flow line of Apartment Drive A and will cross Vulcan Parkway in a small dip section with a 4-foot wide concrete valley gutter. The hydraulic characteristics of the valley gutter are as follows:

Centerline slope (across Vulcan Parkway) = 1.1%
West cross slope (along Vulcan Parkway) = 0.35%
East cross slope (along Vulcan Parkway) = 0.45%

	Flow Top Width (ft.)	Flow Depth (ft.)	Velocity (ft./sec.)
10-yr. Q = 5.70 cfs.	59.0	0.24	1.68
100-yr. Q = 8.66 cfs.	69.0	0.26	1.83

This dip in Vulcan Parkway will carry all flows to Black Loop - East.

Most flows from Sub-basin B are from the escarpment and will reach Apartment Drive A as overland flow; however, some discharge is expected from the apartment building and the escarpment behind this building, in the eastern portion of Sub-basin B. This flow will not reach Drive A but will discharge to Vulcan Parkway through a 1-ft. wide sidewalk culvert as shown on Figure 3A. The flow depth in the culvert for a peak 10-yr. discharge of 1.76 cfs. will be 0.38 ft. with a velocity of 5.09 ft./sec.

PHASE II- DRAINAGE

Drainage Area 3

This drainage area is comprised of Sub-basins C,D,E,F,K,L,M and N. The storm runoff from these sub-basins (except sub-basin N) will be carried down Swale E (Figures 3A and 3B).

Sub-basin K - This sub-basin will have two 1-ft. sidewalk culverts and the flow directions are shown on Figures 3A and 3B. The hydraulics of these culverts are approximately the same and are as follows. The 10-yr. peak Q through each culvert is 0.66 cfs. with a flow depth of 0.18 ft. and a velocity of 3.69 ft./sec..

Locate on Fig 3B

Sub-basin N - This sub-basin will discharge into Vulcan Parkway through a 1-ft. wide sidewalk culvert with a flow depth of 0.2 ft. and a velocity of 3.87 ft./sec. for the 10-yr. peak discharge of 0.77 cfs.

Sub-basins C, D, F and M - Flows from sub-basins C and D will drain west along Drive A to Swale E and sub-basins F and M will drain east along Drive A to Swale E.

Sub-basin E - A Type Double D storm inlet will be located at the outlet of Sub-basin E on the south side of the parking lot as shown on Figure 3B. This inlet will be located 5 feet behind the header curb. The grate elevation will be 0.5 ft. below an overflow weir located in the curb. This grate will also be 1.0 ft. above the flow line, thereby creating a small detention pond behind the curb. This pond will also act as a minor sediment control basin. The inlet will be lined with rip-rap, 5 feet wide on all four sides (Figure 5A) to provide erosion protection to the inlet from the large flows expected from

Sub-basin E.

The Type Double D storm inlet will discharge into a 24-inch storm sewer which begins at this inlet. The storm sewer continues down along Swale E, across Vulcan Parkway, and then down Black Loop - West to its outlet in the concrete channel for the South Branch of San Antonio Arroyo. Details on this storm sewer are provided later in this report.

To accomodate flows in excess of the inlet capacity (with 15% clogging) of 19.4 cfs. at a ponding depth of 0.5 ft. or if the inlet should become completely clogged, a weir will be located within an adjacent 12-inch high header curb. Using the broad crested weir discharge formula Q = CLH**3/2 for the 100-yr. peak discharge of 22.89 cfs. with a C of 3.0 and H = 0.5 ft., the weir length (L) = 22 ft.. Figure 3B shows the location of the Type Double D inlet and of the weir.

Swale E - All flow from Sub-basin E will be controlled by the 24-inch storm sewer and a large swale (Swale E) which starts at the overflow weir on the south side of Drive A. The swale continues down between sub-basins K and L to Vulcan Parkway. Because the central portions of Drive A drain to Swale E (Figure 3A and 3B), this swale will carry flows from sub-basins C, D, E, F, K, L and M. Swale E will be a dip across Drive A and through the parking lot; and will be a rip-rap lined trapezoidal shaped channel from the end of the parking lot to the south side of Vulcan Parkway (see Figure 3B).

Swale E hydraulics will be computed for two cases as follows:

CASE NO. 1 Assume the top Type Double D inlet is completely clogged and has no capacity. Therefore Swale E must carry the full 100-yr. and 10-yr. peak discharge

7.66 3.42 29.84 6.68 0.87

from sub-~~basins C,~~ D, E, F, and M. The 100-yr. peak Q is 48.47 cfs. and the 10-yr. peak Q is 31.90 cfs.

CASE NO. 2 Assume that the ponding depth is 0.5 feet above the inlet and that the top Type Double D inlet has 85% operating capacity due to 15% clogging of inlet and can intake 19.4 cfs. of the 100-yr. peak Q of 22.89 cfs. from Sub-basin E and will intake the full 10-yr. peak Q of 15.06 cfs. from Sub-basin E. Therefore the 100-yr. and 10-yr. peak discharge in Swale E would be equal to the remainder of flow from Sub-basin E = 22.89 cfs. - 19.4 cfs. = 3.49 cfs. and street area of Sub-basin E = 29.84 cfs. (Table 5) - 22.84 cfs. (Table 2) = 6.95 cfs. plus the 100-yr. peak flows from sub-basins C, D, F, and M = 18.63 cfs; therefore the 100-yr. peak ~~flow~~ = 3.49 cfs. + 6.95 cfs. + 18.63 cfs. = 29.07 cfs. The 10-yr. peak flow in Swale E would be equal to the 10-yr. peak flow from sub-basin C, D, F and M excluding Sub-basin E because the Type Double D inlet would intake the 10-yr. peak discharge from Sub-basin E; however the street area of Sub-basin E = 19.64 (Table 5) - 15.06 (Table 2) = 4.58 cfs.; therefore, the 10-yr. peak discharge = 4.58 cfs. + 12.26 cfs. = 16.84 cfs. at Apartment Drive A.

The following paragraphs discuss the different segments of Swale E.

Parking Lot Section

Location and Description - Beginning of Swale E at overflow weir at top of parking lot (south) to bottom of parking lot (north) (Figure 5A). Because this swale will only be used for local runoff and when the storm sewer capacity is exceeded, it will be a concrete valley gutter across the parking lot with a 5% slope down the flowline and 1V:20H side slopes. The swale is designed to carry the total 100-yr. peak flow to preclude any problems if the storm sewer is not operating.

This section of Swale E will have the following hydraulic characteristics.

Concrete valley gutter length = 81 ft.
Top swale elevation at flowline of
Apartment Drive A = 5153.3 ft.
Bottom swale elevation at the bottom corner of the
parking lot = 5149.2
Concrete valley gutter slope = 5%
n = 0.015
Side Slopes 1V:20H

	CASE NO. 1		CASE NO. 2	
	100-yr.	10-yr.	100-yr.	10-yr.
Peak discharge (cfs.)	48.47	31.90	29.07	16.84
Flow top width (ft.)	20.8	18.0	17.2	14.0
Flow depth (ft.)	0.52	0.45	0.43	0.35
Velocity (ft./sec.)	9.10	8.27	8.03	7.0

Rip-Rap Section

Location and Description - Beginning at the north end of the parking lot and continuing to the concrete lined transition channel on the south side of Vulcan Parkway (Figures 5A and 5B). This section of Swale E will be trapezoidal shaped rip-rap lined swale with a 28% slope ? and side slopes of 1V:2H and a total depth of 1.7 ft. 2.8

This section of the swale will have the following hydraulic characteristics.

Rip-rap swale length = 80 ft.
Top swale elevation = 5149.2
Bottom swale elevation = 5,146.6 ft.
Rip-rap swale slope = 3.25% ? ~~should match above~~
2.8
n = 0.04
Trapezoidal shape -
Bottom width = 5 ft.
Side slopes = 1V:2H

	CASE NO. 1		CASE NO. 2	
	100-yr.	10-yr.	100-yr.	10-yr.
Peak discharge (cfs.)	48.47	31.90	29.07	16.84

Flow top width (ft.)	9.56	8.64	8.44	7.56
Flow depth (ft.)	1.14	0.91	0.86	0.64
Velocity (ft./sec.)	5.88	5.21	5.06	4.30

Freeboard = 0.56 ft. with a maximim capacity = 100 cfs.

The rip-rap size required to resist movement from the velocity of 5.88 ft./sec. due to the 100-yr. peak Q of 48.47 cfs. was determined from Engineering Monograph No. 25 published by the U.S. Bureau of Reclamation in July 1963 and titled "Hydraulic Design of Stilling Basins and Energy Dissipators." The median stone diameter required for a velocity of 5.88 ft./sec. is 5 inches; however, 6-inch diameter rip-rap will be used in Swale E.

Type Double D Storm Inlet

Location and Description - The Type Double D inlet grate used in the Phase I sediment control basin (discussed previously) will be located in the concrete transition channel 5 feet back from the 3 sidewalk culverts as shown in Figure 5B. A concrete transition channel will improve hydraulics from the end of the trapezoidal rip-rap channel to the rectangular openings of the 3 steel plate sidewalk culverts which will discharge to the dip in Vulcan Parkway. The top of the inlet grate will be set 0.5 ft. below the invert of the sidewalk culverts; therefore, flow exceeding the capacity of the inlet will flow through the sidewalk culverts.

Case No. 1 as discussed previously will now change to Case No. 1A and will assume that the Type Double D inlet at Vulcan Parkway is clogged and has no capacity. Therefore, the 100-yr. peak Q and 10-yr. peak Q are 48.47 cfs. and OK 31.90 cfs., respectively, and this is the discharge that would reach the three sidewalk culverts at Vulcan Parkway.

Case No. 2 as discussed previously will now change to Case No. 2A and will include 85% operating capacity of the Type Double D inlet near Vulcan Parkway. The capacity of the inlet with 0.5 ft. of head and 85% grate opening (assuming 15 % clogging) = 19.4 cfs.. Therefore, the 100-yr. peak discharge remaining as surface flow through the sidewalk culverts assuming that the top Type Double D inlet near Drive A is also operating at 85% capacity (Case No. 2) is as follows. The 100-yr. peak flow that would enter the sidewalk culverts = 29.07 (Case No. 2) - 19.4 (Type Double D inlet capacity) = 9.67 cfs. However, as shown later in this report the maximum capcity of the storm sewer will be 27 cfs.; therefore, the 100- yr. peak flow entering the sidewalk culverts will be 48.47 - 27 = 21.47 cfs. if both Double-D inlets on 24" RCP are working; if not Q_{100} = 48.47 cfs
Q_{10} = 31.90 cfs

Sidewalk Culverts

Location and Description - Beginning at the back of the sidewalk at Vulcan Parkway and continuing to the south flow line at Vulcan Parkway. Under normal operating conditions three 24-inch steel plate sidewalk culverts will have to carry the flow exceeding the capacity of the Type Double D storm inlet. The number of sidewalk culverts needed were sized to carry the 10-yr. peak flow assuming no capacity in the storm sewer. The 10-yr. peak flow at the sidewalk culverts is 31.90 cfs. and the hydraulics in the culverts are as follows:

	100-yr.	10-yr.
Peak discharge (cfs.)	48.47	31.90

n = 0.015		
Sidewalk culvert invert slope (%)	2.0	2.0
Flow depth (ft.)	0.79	0.60
Velocity (ft./sec.)	10.27	8.87

Type C Storm Inlet at Vulcan Parkway

Location and Description - This inlet will be located adjacent to the exit of the sidewalk culverts on the south side of the dip in Vulcan Parkway, across from Black Loop-West. The sub-basins contributing flow to this inlet are K 3.00, L 0.43, N 1.17 and any flow that may pass through the sidewalk culverts. This catch basin will discharge into the Type Double D inlet near Vulcan Parkway through an 18-inch R.C.P.. The Type Double D catch basin will then discharge into the 24-inch storm drain through another 18-inch R.C.P..

The capacity of the Type C inlet was determined by a chart from the Neenah Foundry Company as discussed previously. The capacity assuming 0.25 ft. of head and 85% open grate area (assuming 15% clogging) is 8.11 cfs.; with 0.4 ft. of head and 85% open area, the capacity is 11.74 cfs.. Q_{100} = 4.6 + excess from above.

Swale E at Vulcan Parkway

Vulcan Parkway will have a reduced crown at this location of 3 inches (0.25 ft.) instead of 5 inches; therefore, flows exceeding the capacity of the Type C inlet or the storm sewer will flow over the crown and down Black Loop-West. The following hydraulic characteristics are based on the storm sewer not being in operation (Case 1A), and representing the flow as weir flow over the roadway crown (C = 3.09 and level weir).

	100-yr.	10-yr.
Peak discharge (cfs.)	53.07	34.93
Flow top width (ft.)	48.50	45.57
Flow depth (ft.)	0.50	0.40
Velocity (ft./sec.)	2.0	1.7

Drainage Area 4

This drainage area is comprised of Sub-basin Q. This sub-basin will discharge into a small swale adjacent and parallel to the development boundary along the top of the bank of South Branch of San Antonio Arroyo; therefore no developed flow will enter the natural arroyo channel below Sub-basin Q. The swale will have a 1 ft. bottom width and side slopes of 1V:2H.

This swale will outlet through two 24-inch sidewalk culverts at Vulcan Parkway (see Figure 3B). Most of Swale Q will have a slope of 0.7%; however, the slope will be increased to 2% for a distance of 10 feet back from the sidewalk culverts. The slope will be increased to reduce the flow depth to be carried through the two sidewalk culverts; and the bottom width and side slopes will be transitioned to join the back of the sidewalk culverts.

The hydraulic characteristics are as follows:

	Bottom Width (ft.)	Top Width (ft.)	Flow Depth (ft.)	Velocity (ft./sec.)
Typical Swale				
Side slopes 1V:2H				
10-yr. peak Q = 4.12 cfs.	1.0	4.3	0.8	1.8
100-yr. peak Q = 6.26 cfs.	1.0	5.0	1.0	2.1
Swale at sidewalk culverts				
Side slopes at culvert entrance 1V:1H				
10-yr. peak Q = 4.12 cfs.	2.5	3.5	0.5	2.8
100-yr. peak Q = 6.26 cfs.	2.5	3.8	0.6	3.2

Drainage Area 5

This drainage area is comprised of sub-basins G, H, I, O and P. Sub-basins G, H, I and P will flow down Apartment Drive C and the flow from these sub-basins will join the flow of Sub-basin O at the intersection of Apartment Drive C and Vulcan Parkway.

The small arroyo within Sub-basin I will be filled to accomodate Apartment Drive A. Flows discharging from Sub-basin I will cross the drive and drain down Apartment Drive A. Figure 9 shows the profile of Apartment Drive A at Arroyo I and J. Figure 10 shows a cross section through Apartment Drive A at Arroyo I.

Sub-basins I and H will flow from the west and Sub-basin G will flow from the east down Apartment Drive A. These three sub-basins and Sub-basin P will drain down Apartment Drive C, which discharges into Vulcan Parkway.

Because the high point of Vulcan Parkway lies between sub-basins O and N, as shown on Figure 3B, Sub-basin O will flow west down Vulcan Parkway and Sub-basin N will drain to Black Loop-West. Vulcan Parkway will be a dip at the box culverts at the South Branch of San Antonio Arroyo. Sub-basins G, H, I, O, P and Q will also discharge down Vulcan Parkway to the dip in Vulcan Parkway at the box culverts. Flows from the future development of Sub-basins KK and LL are also assumed to drain down to the dip in Vulcan Parkway.

Steel top sidewalk culverts will be located at the dip in Vulcan Parkway at the box culverts and will carry flow from the gutter to the back of the sidewalk, and the flow will free fall to the channel below. Two 24-inch sidewalk culverts located on the southwest side of the dip in Vulcan Parkway will have the capacity for the 10-yr. discharge from sub-basins G, H, I, O, P, Q and KK.

One 24-inch sidewalk culvert will be located on the northeast side of the dip in Vulcan Parkway at the box culverts. This culvert will have the capacity for the 10-yr. discharge from the future development of Sub-basins LL and 18 (Phase I) as shown on Figure 3B.

The culverts were sized to carry the 10-yr. peak discharge and discharges exceeding the capacity of the sidewalk culverts will flow over the top of the culverts. Because the culverts will be at the bottom of the dip in Vulcan Parkway, discharges exceeding the capacity of the culverts will be forced to flow over the culverts and then into the lined channel below. The hydraulics of these sidewalk culverts are as follows:

Southwest side of Vulcan Parkway Dip	Northeast side of Vulcan Parkway Dip
Two 24-inch sidewalk culverts	One 24-inch sidewalk culvert
10-yr. Q = 30.11 cfs.	10-yr. Q = 9.49 cfs.
n = 0.015	n = 0.015
slope = 4.5%	slope = 4.5%
depth = 0.60 ft.	depth = 0.48 ft.
velocity = 12.62 ft./sec.	velocity = 10.00 ft./sec.

Sub-Basin J - The arroyo at the outlet of Sub-basin J will be left in its natural state. If necessary, the eastern bank above the 100-yr. flood elevation will be protected from lateral migration of the South Branch of San Antonio Arroyo.

Phase I Drainage

Table 6 lists the developed flows from each Phase I sub-basin shown on Figures 3A and 3B. A description of the longest flow path and the Tc for this

TABLE 6

ON-SITE FLOW
PHASE I SUB-BASIN HYDROLOGY

Time of Concentration (Tc)

The flow path used in the Tc calculation begins at the high point in Vulcan Parkway immediately west of Black Loop-East. The flow path continues from this high point west down Vulcan Parkway then above Black Loop-West and then down the east-west segment of Black Loop to the concrete rundown structure at Black Loop-East.

Length = 1,195 ft.
Elevation Difference = 9.69 ft.

Tc = 11.7 min.
Average Velocity = 1.7 ft./sec.
Assume Velocity adjusted according to DPM for velocity = 2.4 ft./sec.
Tc adjusted according to DPM for Tc = 8.3 min.
Assume Tc = 10 min.

DESIGN RAINFALL

100-yr. 6-hr. Duration
rainfall depth = 2.20 in.
peak intensity = 4.65 in./hr.

10-yr. 6-hr. Duration
rainfall depth = 1.45 in.
peak intensity = 3.06 in./hr.

Area Name *	Lot #	C	Total Basin Area (ac.)	100-yr. Frequency Q (cfs.)	100-yr. Frequency V (cu. ft.)	10-yr. Frequency Q (cfs.)	10-yr. Frequency V (cu. ft.)
1	63-75	0.47	1.758	3.84	6,559	2.53	4,350
2	76-79	0.52	0.830	2.01	3,449	1.32	2,273
3	80-84	0.60	1.024	2.86	4,908	1.88	3,235
4	51-62	0.50	1.674	3.89	6,686	2.56	4,407
5	38-50	0.54	1.945	4.88	8,388	3.21	5,528
6	85-91	0.62	1.180	3.40	5,847	2.24	3,853
6.1	92	0.60	0.149	0.42	714	0.27	471

path is given in Table 6. The Tc for all on-site sub-basins is less than 10 minutes and therefore assumed to be 10 minutes according to DPM criteria for Tc.

The impervious area determination for a typical lot in Phase I is as follows:

Average sidewalk area on property	= 3 ft. x 20 ft.	= 60 sq. ft.
Average townhouse area		= 1,200 sq. ft.
Average garage area	= 20 ft. x 18 ft.	= 360 sq. ft.
Average driveway area	= 14 ft. x 16 ft.	= 224 sq. ft.
Total impervious area/lot		= 1,844 sq. ft.
Average lot area = 46.19 ft. x 94.11 ft.		= 4,347 sq. ft.
% imperviousness/lot = 1,844 sq. ft/4,347 sq. ft. = 42%		

Phase I is located in Hydrologic Soil Group A and therefore the developed condition Runoff Coefficient "C" is 0.38. The lot areas within each sub-basin were determined from the Plat (Appendix A). The street area within each sub-basin was measured from the Plat and assumed to be 98% impervious with a Runoff Coefficient of 0.92. The Runoff Coefficient was determined as an area weighted average of the lot area and street area per basin using the respective Runoff Coefficients for each. The "C" factors vary because some sub-basins have more or less street area than others. Runoff will be carried in all streets.

Bogart Street

Bogart Street will be an all weather crossing over the South Branch of San Antonio Arroyo. The street will be located on two 8-ft. wide box culverts as shown in Figure 11. Bogart Street will have a dip at the box culverts and a 1-ft. wide sidewalk culvert will be required on each side. Both culverts will have 2% slopes and will have a free fall discharge into the concrete channel on both sides of the box culvert.

The west side of the dip will require one 1-ft. wide sidewalk culvert to carry the 100-yr. peak flow. The 100-yr. peak Q is 4.03 cfs. and would have a flow depth of 0.66 ft. with a velocity of 6.11 ft./sec. on a 2% slope through the sidewalk culvert.

The east side of the dip will also require one 1-ft. wide sidewalk culvert to carry the flows expected at that location, which were determined as follows:

Assume Tc = 10 minutes because of a very short flow path of 250 ft. and because all on-site Tcs previously calculated are less than 10 minutes.

DESIGN RAINFALL

100-yr. 6-hr. duration	10-yr. 6-hr. duration
peak intensity 4.65 in./hr.	peak intensity = 3.06 in./hr.
rainfall depth = 2.20 in.	rainfall depth = 1.45 in.

Soil type is Bluepoint - Kokan association which is in Hydrologic Soil Group A.

Percent imperviousness = 98%; Runoff Coefficient C = 0.92.

The area draining to the east dip in Bogart Street is all paved and is 1/2 (60 ft. street R.O.W. width) x 200 ft. from Santa Fe Village Unit III plus 1/2 (60 ft. street R.O.W. width) x 250 ft. from Santa Fe Village Unit I) = 13,500 sq. ft. = 0.31 ac.

	100-yr.	10-yr.
Peak discharge (cfs.)	1.32	0.87
Runoff volume (cu. ft.)	2,278	1,501

Drainage Rundowns

Three drainage rundowns are needed to carry the on - and off-site flows from the streets of Phase I to the concrete channel. These rundowns are located on Butte Court, at Analysis Point 31A; on Ja Court at Analysis Point 14A; and on Black Loop at Analysis Point 19A as shown on Figures 3A and 3B.. These rundown structures are discussed in Table 7 along with their hydraulic

2/25/86
Hydraulic design
checks by RAG.

TABLE 7

DRAINAGE RUNDOWN HYDRAULICS

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1. BUTTE COURT

A. Description - Located at the northern cul-de-sac in Butte Court rectangular concrete lined channel and a 0.6 depth and 3.75 ft. bottom width and a 5-ft. easement width. (See City of Albuquerque Standard Specification Drawing No. K-23 for details).

TOO NARROW for CITY STDS.

Required 100-yr. Q	=	12.60 cfs. ok
Slope	=	3.86 %
n	=	0.015
Flow Depth	=	0.38 ft.
Velocity	=	9.09 ft./sec.
Freeboard	=	0.22 ft.
Maximum Capacity	=	25 cfs.

$Q = CLH^{3/2}$

find min. H w/ C = 3.0

$12.6 = (3.0)(3.75) H^{3/2}$

$1.12 = H^{3/2}$

$(1.12)^{2/3} = H = 1.08'$

1. ~~BLACK LOOP~~ — CLIFFROSE RD. ?ok?

Description - Located at the eastern cul-de-sac in Black Loop 1-ft. deep concrete lined channel with a 9-ft. bottom width and 10-ft. easement width. (See City of Albuqurerque Standard Specification Drawing No. K-23 for details).

Required 100-yr. Q	=	67.72 cfs.
Slope	=	1 %
n	=	0.013
Flow Depth	=	0.84 ft.
Velocity	=	9.10 ft./sec.
Freeboard	=	0.16 ft.
Maximum Capacity	=	90 cfs.

3. JA COURT

A. Description - Located at Analysis Point 14-A (Figure 3A), trapezoidal shaped rip-rap lined channel with side slopes of 2H:1V and a 0.75-ft. depth and 2-ft. bottom width and 6-ft. easement width

Too Narrow for City STDS.

Required 100-yr. Q	=	7.95 cfs
Slope	=	4.3 %
n	=	0.033

see DPM 22.6 pg 99
DPM 22.8 pg 166-167
STD DRWG K-23

24" dia S.D.

$W = \left[2 \times \underset{5\,min}{depth}\right] + 2 + 4$

$10 + 2 + 2 + 4$

$= 18'$ MIN.

characteristics. The number and characteristics of the sidewalk culverts required to carry flow from the streets into the rundown structures are discussed below.

✓ Butte Court - Two sidewalk culverts will be required to carry the 100-yr. flow from the street into the concrete rundown structure; one 2-ft. wide sidewalk culvert and a one 1-ft. wide sidewalk culvert side by side (each at 2% slope). The 100-yr. peak Q of 12.60 cfs. would have a flow depth 0.55 ft. with a velocity of 7.68 ft./sec.

Check $Q = CLH^{3/2}$ w/ $C = 3.0$

$12.6 = (3.0)(3)(H^{3/2}) \Rightarrow H = 1.25'$

CLIFFROSE ROAD

✓ ~~Black Loop~~ - The Tc at this drainage rundown is given in Table 8 along with the corresponding design rainfall and 100-yr. peak discharge is 67.72 cfs. (Case 1A). Four 2-ft. sidewalk culverts will be required to carry the flow from the street into the concrete rundown structure. The concrete sidewalk culverts will be designed to have 1/2 inch steel supports instead of the normal concrete supports between culverts. The 100-yr. Q of 67.72 cfs. would have a flow depth of 0.73 ft. and a velocity of 11.76 ft./sec.; the 10-yr. Q would have a flow depth of 0.56 ft. and a velocity of 10.11 ft./sec..

$Q = CLH^{3/2}$

✓ Ja Court - One 2-ft. wide sidewalk culvert would be required to carry the flow from the street into the rip-rap trapezoidal shaped rundown structure. The 100-yr. Q of 7.95 cfs. would have a flow depth of 0.56 ft. and a velocity of 7.12 ft./sec.. The rip-rap rundown structure is discussed in Table 7.

Use $Q = CLH^{3/2}$

General Drainage Considerations For Phase I and Phase II

Analysis Points

Analysis points are points of interest in terms of peak discharge and flow characteristics. Table 8 shows the on-site flows for the analysis points shown on Figures 3A and 3B. The time of concentration at each analysis point was assumed to be 10 minutes (except for Analysis Point 19A) and therefore the peak discharge at each analysis point was determined by adding the peak discharge from each contributing sub-basin. The Tc for Analysis Point 19A is 14.5 minutes and the data for this calculation are given in Table 9.

The peak discharges at all analysis points assume no depletion by the storm sewer.

Storm Sewer (24-inch R.C.P.)

The storm sewer will be constructed in conjunction with Phase I and Phase II.

Phase 1 - In Phase I the storm sewer will begin at the concrete channel and continue up Black Loop - West and will be terminated 40 feet south of Vulcan Parkway (see Figure 5A). There will be two manholes in Black Loop - West (each at a bend) and one manhole in Vulcan Parkway (at a bend) as shown in Figures 3A and 3B. As discussed previously, the Type Double D and Type C storm inlets will be located south of Vulcan Parkway and in Vulcan Parkway, respectively. The Type C inlet will discharge into the Type Double D inlet through an 18-inch R.C.P.. This catch basin will discharge into the 24-inch storm sewer through a second 18-inch R.C.P. with a junction angle of 30 degrees.

Phase II - In Phase II the storm sewer will be extended from the Phase I termination point (discussed above) to the Type Double D inlet near Apartment Drive A. There will be one manhole in the apartment area at a bend as shown in Figures 3A and 3B, and then the storm sewer will continue up to the Type Double D inlet behind the parking lot (discussed previously).

TABLE 9

TOTAL FLOW AT RUNDOWN ON BLACK LOOP

Tc Calculation from top of Basin E to the rundown structure at east Black Loop.

Tc = 8.5 minutes from Sub-basin E to top of apartment drive.

Tc from top of apartment drive to the rundown at east Black Loop-East

Length	= 1,195 ft.
Elevation Difference	= 18.3 ft.
Tc	= 9.13 min.
Average Velocity	= 2.18 ft./sec.
Adjusted Average Velocity According to DPm Criteria for Velocity	= 3.3 ft./sec.
Tc	= 6.0 min.

Tc = 8.5 + 6.0
Total Tc = 14.5 min.

DESIGN RAINFALL

100-yr. 6-hr. Duration
rainfall depth = 2.20 in.
peak intensity = 3.85

10-yr. 6-hr. Duration
rainfall depth = 1.45 in.
peak intensity = 2.54

RUNOFF COEFFICIENT "C" -

The runoff coefficient is an area weighted average of all developed "C" factors for sub-basins B, C, D, E, F, K, L, 1/2 M, 5, 6, 6.1, 7, 8, 9, 10, 11, 12

Total drainage area up to the rundown structure at Black Loop-East = 35.10 ac.

Weighted Runoff Coefficient "C" = 0.50

	Frequency 100-yr.	10-yr.
Peak Discharge (cfs.)	67.72 ✓	44.68
Runoff Volume (cu. ft.)	140,474	92,585

Figure 12 shows the plan and profile for the storm sewer. Flow depths in the concrete channel for the South Branch of San Antonio Arroyo were used to compute the hydraulic grade line (H.G.L) in the storm sewer. The 100-yr. flow depth of 2.73 ft. was used as the outlet control elevation for 100-yr. conditions. The 10-yr. flow depth in the concrete channel is 1.83 ft.; however, the soffit elevation was used as the 10-yr. control elevation according to DPM criteria for control elevation in H.G.L. calculations.

The non-pressurized capacity of the storm sewer section at a 0.1% grade (most downstream section) using Manning's formula is about 7 cfs.; when flows exceed 7 cfs. the storm sewer will flow under pressure. The capacity of the storm sewer was determined by calculating the H.G.L. for various discharges. The maximum capacity of the storm sewer is 27 cfs. and is in part determined when the H.G.L. reaches the top of the Type Double D inlet grate south of Vulcan Parkway. Any possible increase in capacity of the storm sewer due to increased head beyond this inlet elevation is lost because the H.G.L. reaches the top of this inlet.

Therefore, the storm sewer will carry almost half of the 100-yr. peak flow which would be expected at Analysis Point 19A. ✓

Figure 12 shows the 100-yr. H.G.L. and 10-yr. H.G.L.. The H.G.L. is approximately the same for the 100-yr. and 10-yr. outlet control depths. The H.G.L. calculations are included in Appendix B.

✓ Street and Apartment Drive Hydraulics

The street and apartment drive hydraulics at the analysis points shown on

Figures 3A and 3B are given in Table 10.

Total Development Impact on Arroyo

A comparision of flow characteristics at the AMAFCA drop structure near Unser Blvd. NW was determined for the total southern tributary area including Phase I and Phase II for undeveloped and developed conditions as follows:

Undeveloped Conditions - Table 11 lists the Tc calculation and the 100 - and 10-yr. peak discharges for the undeveloped condition from the combined drainage area of Phase I and Phase II (plus other contributing off-site areas). The Runoff Coefficient "C" was determined as a weighted "C" of all sub-basins for undeveloped conditions, and the peak discharge was then determined using the Rational Formula (Q=CIA).

Developed Conditions - As shown on Table 12, the Tc was determined for the total drainage area of Phase I and Phase II (plus other contributing off-site areas) for developed conditions. Table 12 also shows the 100 - and 10-yr. peak discharge. The "C" factor was determined as an area weighted "C" from all developed sub-basins.

The Tc for the undeveloped condition is 14.4 minutes with a 100-yr. peak discharge of 72.58 cfs. and the Tc for developed condition is 19.6 minutes with a 100-yr. peak discharge of 106.99 cfs.. The difference in Tc is a result of major changes in existing and developed condition flow paths. Therefore, as expected there is an increase in flow due to development of Phase I and Phase II; however, the increase in flow due to development is not significant.

TABLE 11

TOTAL TRIBUTARY AREA FLOWS (UNDEVELOPED)

TIME OF CONCENTRATION (Tc)

The flow path used for the Tc calculation begins at the top of Sub-basin V (Figure 4) and ends at the AMAFCA gabion grade control structure at Unser Blvd. NW and Vulcan Parkway. Total flow length is measured along existing channels.

Flow Length	= 3,870 ft.
Elevation Difference	= 189 ft.
Average Velocity	= 4.5 ft./sec.
Tc	= 14.4 min.

DESIGN RAINFALL

100-yr. 6-hr. Duration
rainfall depth = 2.20 in.
peak intensity = 3.86 in./hr.

10-yr. 6-hr. Duration
rainfall depth = 1.45 in.
peak intensity = 2.54 in./hr.

RUNOFF COEFFICIENT "C" -

The runoff coefficient is an area weighted average of all undeveloped "C" factors for sub-basins A-Q (Figures 3A and 3B) and the undeveloped Phase I area is as follows.

Sub-basins A-Q

Soil Types - Bluepoint - Kokan association - Hydrologic Soil Group A,
Kokan-Rock outcrop association - Hydrologic Soil Group A,
Alemeda sandy loam - Hydrologic Soil Group C

Weighted "C" for sub-basins A-Q = 0.35
Total area of sub-basins A-Q = 44.84 ac.

Phase I

Soil Type - Bluepoint-Kokan association

Hydrologic Soil Group A

0% imperviousness; "C" = 0.16
Total Area = 20 ac.

Total area of sub-basins A-Q and Phase I = 64.84 ac.
Weighted "C" = 0.35 (44.84) + 0.16 (20) / 64.84 = 0.29

	Frequency	
	100-yr.	10-yr.
Peak Discharge (cfs.)	72.58	47.76
Runoff Volume (cu. ft.)	150,166	98,973

TABLE 12

TOTAL TRIBUTARY AREA FLOWS (DEVELOPED)

TIME OF CONCENTRATION (Tc)

The Tc calculation begins at the top of Sub-basin E (Figure 6) and continues down through the concrete/rip-rap lined, swale across Vulcan Parkway, down Black Loop-West to the concrete channel within the 50-foot AMAFCA R.O.W. to the AMAFCA gabion grade control structure.

Tc for Off-Site Area E = 8.5 min. (Table 5)

Tc from top of Apartment Drive A at outlet of Swale E to end of parking lot along Swale E.

Length	= 132 ft.
Elevation Difference	= 5.8 ft.
Tc	= 1.1 min.
Average Velocity	= 2.0 ft./sec.
Adjusted Average Velocity	= 2.8 ft./sec.
Adjusted Tc According to DPM Criteria for Tc	= 0.79 min.

Tc in rip-rap section from end of parking lot to Type Double D inlet at Vulcan Parkway along Swale E.

Length	= 82 ft.
Elevation Difference	= 2.6 ft.
Tc	= 0.88 ft.
Average Velocity	= 1.5 ft./sec.
Adjusted Average Velocity	= 1.1 ft./sec.
Adjusted Tc According to DPM Criteria for Tc	= 1.24 min.

Tc from sidewalk culverts at Vulcan Parkway to AMAFCA gabion grade control structure.

Length	= 1,850 ft.
Elevation Difference	= 29.6 ft
Tc	= 12.6 min.
Average Velocity	= 2.4 ft./sec.
Adjusted Average Velocity	= 3.4 ft./sec.
Adjusted Tc According to DPM Criteria for Tc	= 9.1 min.

Total Tc = 8.5 + 0.79 + 1.24 + 9.1 = 19.63 min.

DESIGN RAINFALL

100-yr. 6-hr. Duration	10-yr. 6-hr. Duration
rainfall depth = 2.20 in.	rainfall depth = 1.45 in.
peak intensity = 3.30 in./hr.	peak intensity = 2.17 in./hr.

RUNOFF COEFFICIENT "C" -

The runoff coefficient is an area weighted average of all developed "C" factors for sub-basins A-Q and the Phase I sub-basins as follows.

Sub-basins A-Q

Weighted "C" for sub-basins A-Q = 0.48
Total area of sub-basins A-Q = 44.84 ac.

Phase I Sub-basins

Soil Types - Bluepoint - Kokan association - Hydrologic Soil Group A,
Kokan - Rock outcrop association - Hydrologic Soil Group A,
Alemeda sandy loam - Hydrologic Soil Group C

Weighted "C" for sub-basins 1-18 = 0.54
Total area of sub-basins 1-18 = 20 ac.

Total area of sub-basins A-Q and 1-18 = 64.84
Weighted "C" = 0.48 (44.84) + 0.54 (20)/64.84 = 0.50

	Frequency	
	100-yr.	10-yr.
Peak Discharge (cfs.)	106.99	70.35
Runoff Volume (cu. ft.)	258,906	170,643

SOUTH BRANCH OF SAN ANTONIO ARROYO

Maximum Peak Discharges for South Branch of San Antonio Arroyo

The 100-yr. peak discharge for the undeveloped basin of the South Branch of San Antonio Arroyo is 762 cfs and the time to peak is 1.5 hours as discussed previously. The 100-yr. peak discharge for the fully developed basin will also be approximately 762 cfs. for the following reasons.

The largest Time of Concentration (Tc) determined in this report for development of Phase I and Phase II and the contributing off-site area is 19.6 minutes as determined in Table 12. The Tc for off-site areas KK and LL is 10 minutes (Table 3) and for area MM is 17.32 minutes. Areas KK, LL, MM and Santa Fe Village Unit III Phase I and II are all near the basin outlet as shown on Figure 13. The peak discharge from these areas will reach the basin outlet in less than 0.5 hrs. the beginning of rainfall and using a triangular distribution where Tc is a third of flow duration, the study area hydrograph will have receded to almost no flow in 1.5 hrs. When the peak for the total basin will reach the basin outlet. Therefore, the discharges from the development of these areas should not be added to the peak discharge for the total basin and consequently the 100-yr. peak discharge for the basin with full development will be 762 cfs.. (The 10-yr. peak discharge at the basin outlet is 380 cfs..) ✓

General Plan Through Phase I and II

The South Branch of the San Antonio Arroyo will remain in its natural channel from the base of the escarpment to within 50 feet of Vulcan Parkway. The extensive existing vegetation will remain and additional native and adapted plants will be introduced.

There will be no runoff from Phase II entering the natural portion of the South Branch of San Antonio Arroyo because Swale Q as discussed previously, will divert all developed condition runoff to Vulcan Parkway. The natural arroyo channel will be transitioned to the box culvert under Vulcan Parkway by means of a 50-foot long rip-rap and gabion structure. Figure 14 shows a plan view and details the of box culvert at Vulcan Parkway.

A concrete transition channel will begin at the outlet of the box culvert and continue for 50-feet until joining with the uniform trapezoidal concrete channel as shown in Figure 14. This typical uniform section continues to the large bend (Curve 1) and then changes slightly to accomodate superelevation requirements (see Figure 3B for typical and super-elevated sections). Downstream from Curve 1 the channel section is again typical until reaching the next bend (Curve 2) which also requires a superelevated bank. The channel section is typical from Curve 2 down to the Bogart Street box culvert. A trapezoidal concrete transition channel 27 feet long will join the typical channel section with the box culvert at Bogart Street. A concrete transition channel will begin at the downstream end of the box culvert and continue to the AMAFCA gabion grade control structure which has a notch in the gabions that is 3 ft. deep and 48 ft. long.

Figure 11 shows the Bogart Street Box Culvert plan view with details.

HEC-2 Analysis

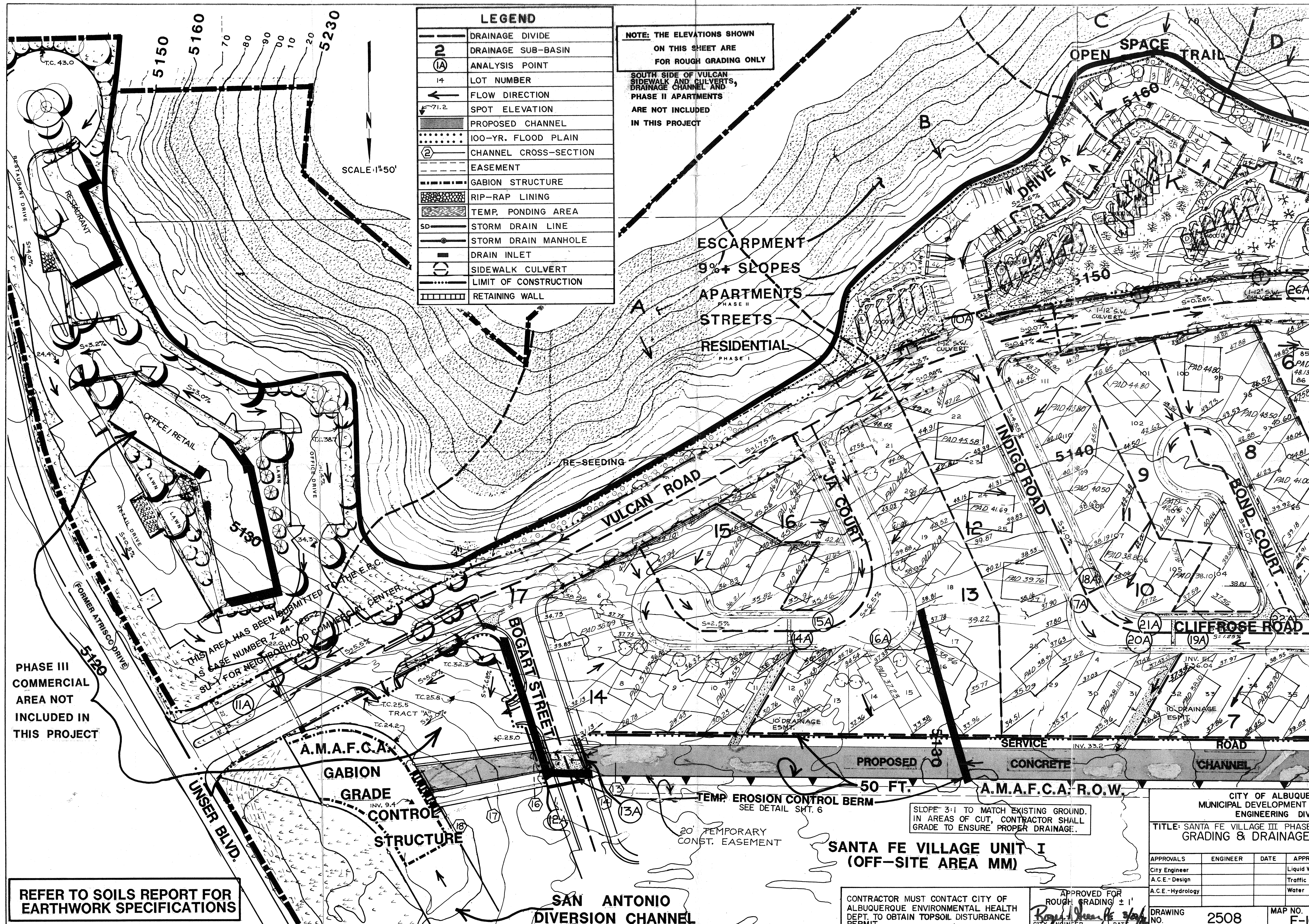
Backwater elevations for the South Branch of San Antonio Arroyo were computed using the U.S. Army Corps of Engineers HEC-2 program. The unlined (natural) arroyo upstream of Vulcan Parkway was computed as subcritical flow. The "n" values used were 0.045 for the rip-rap transition and 0.035 for the vegetated arroyo. Tables 13A and 13B are the summary printout from this subcritical run for the 10-yr. and 100-yr. peak discharge, respectively. Figures 3A and 3B show the locations of the cross sections in plan view and the 100-yr. floodplain based on the HEC-2 results.

The proposed AMAFCA R.O.W. shown on Figures 3A and 3B was determined based on the HEC-2 analysis. Upstream from Vulcan Parkway the entire natural floodplain

No, need prudent line, or channel bank Treatment

will be dedicated to AMAFCA Figure 3B. Downstream of Vulcan Parkway the 100-yr. floodplain is confined to the concrete lined channel. The hydraulic conditions for the concrete channel computed as supercritical flow with an "n" value of 0.015. Tables 14A and 14B are the summary printout from this supercritical run for the 10-yr. and 100-yr. peak discharge, respectively. The 100-yr. water surface profile is shown in Figure 15 for the concrete and natural channel as reaches determined by the HEC-2 analysis.

Table 15 lists the 10-yr. and 100-yr. hydraulics at straight unobstructed uniform channel sections. Table 16 lists the hydraulic characteristics of the box culverts at Vulcan Parkway and Bogart Street and the transition from the box culvert at Bogart Street to the 48-ft. wide opening at the AMAFCA gabion grade control structure at Unser Blvd. NW..



LEGEND

— — —	DRAINAGE DIVIDE
2	DRAINAGE SUB-BASIN
(1A)	ANALYSIS POINT
14	LOT NUMBER
←	FLOW DIRECTION
71.2	SPOT ELEVATION
	PROPOSED CHANNEL
	100-YR. FLOOD PLAIN
(2)	CHANNEL CROSS-SECTION
	EASEMENT
	GABION STRUCTURE
	RIP-RAP LINING
	TEMP. PONDING AREA
SD	STORM DRAIN LINE
	STORM DRAIN MANHOLE
	DRAIN INLET
	SIDEWALK CULVERT
	LIMIT OF CONSTRUCTION
	RETAINING WALL

NOTE: THE ELEVATIONS SHOWN ON THIS SHEET ARE FOR ROUGH GRADING ONLY

SOUTH SIDE OF VULCAN SIDEWALK AND CULVERTS, DRAINAGE CHANNEL AND PHASE II APARTMENTS ARE NOT INCLUDED IN THIS PROJECT

SLOPE 3:1 TO MATCH EXISTING GROUND. IN AREAS OF CUT, CONTRACTOR SHALL GRADE TO ENSURE PROPER DRAINAGE.

REFER TO SOILS REPORT FOR EARTHWORK SPECIFICATIONS

CONTRACTOR MUST CONTACT CITY OF ALBUQUERQUE ENVIRONMENTAL HEALTH DEPT. TO OBTAIN TOPSOIL DISTURBANCE PERMIT.

APPROVED FOR ROUGH GRADING ± 1'
CITY ENGINEER DATE 3/20/86

AS BUILT INFORMATION: CONTRACTOR / WORK STARTED BY / INSPECTOR'S ACCEPTANCE BY / FIELD VERIFICATION BY / DRAWINGS CORRECTED BY — DATE
MICRO-FILM INFORMATION: RECORDED BY — DATE — NO.

BENCH MARKS
CONTROL: BENCH MARK 7-F11, WHICH BEARS N 26° 09' 08" E, 758.55' FROM THE S.E. CORNER OF SANTA FE VILLAGE III, A 3/4" PIPE. BENCH MARK IS A STANDARD A.C.S. BRASS CAP STAMPED "7-F11".
M.S.L. ELEVATION=5116.57'

SURVEY INFORMATION — FIELD NOTES — NO. — BY — DATE

ENGINEER'S SEAL

REVISIONS — NO. — DATE — REMARKS — BY

DESIGN
DESIGNED BY B.T. A.G. DATE 2-86
DRAWN BY C.M. A.P. M.S. R.G. B.T. DATE 2-86
CHECKED BY E.D.

CITY OF ALBUQUERQUE
MUNICIPAL DEVELOPMENT DEPARTMENT
ENGINEERING DIVISION

TITLE: SANTA FE VILLAGE III PHASE I
GRADING & DRAINAGE EAST

APPROVALS	ENGINEER	DATE	APPROVALS	ENGINEER	DATE
City Engineer			Liquid Waste		
A.C.E.-Design			Traffic		
A.C.E.-Hydrology			Water		

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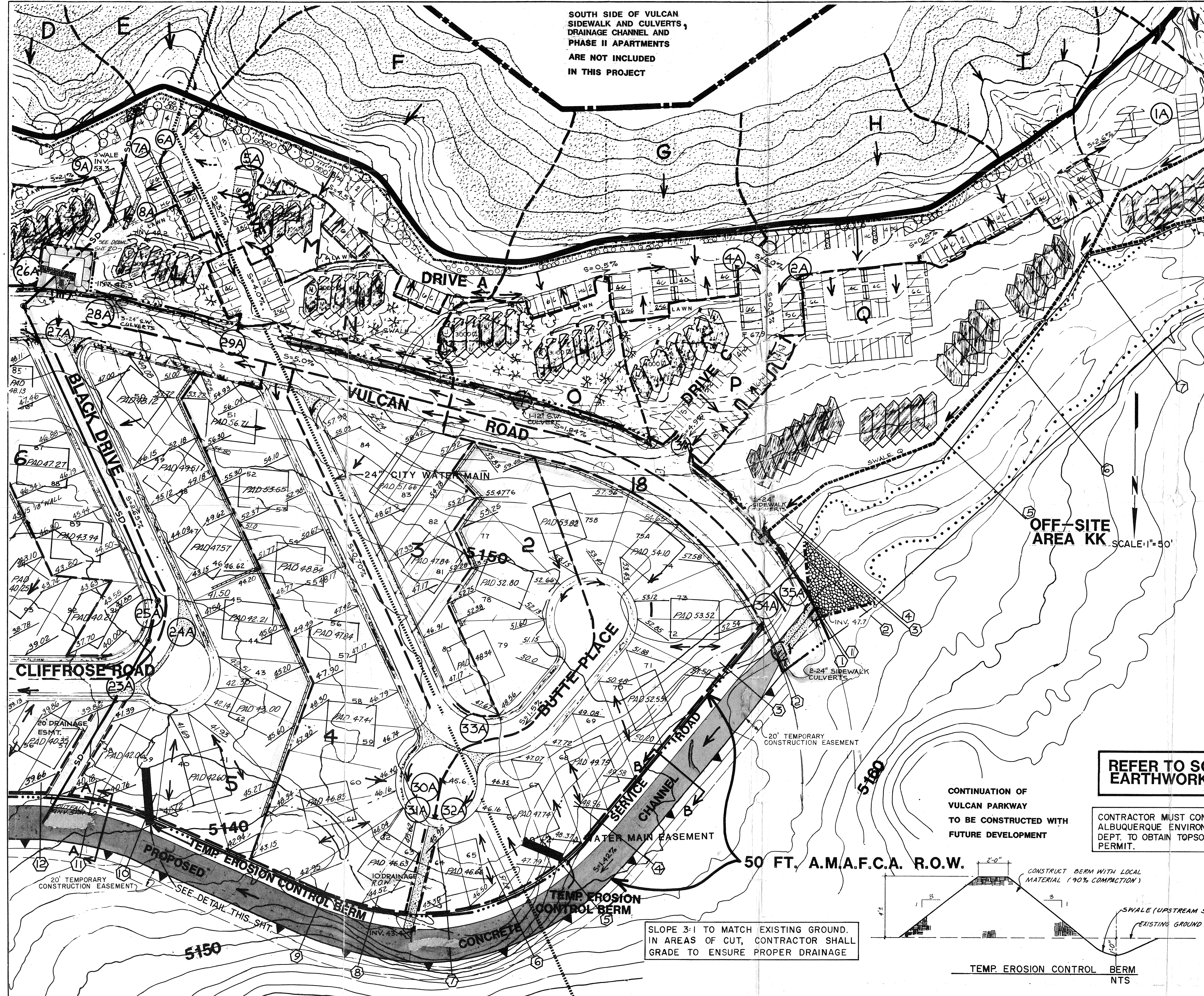
DRAWING NO. 2508 — MAP NO. F-10 — SHEET 5 OF 21



LEGEND	
	DRAINAGE DIVIDE
2	DRAINAGE SUB-BASIN
1A	ANALYSIS POINT
14	LOT NUMBER
	FLOW DIRECTION
71.2	SPOT ELEVATION
	PROPOSED CHANNEL
	100-YR. FLOOD PLAIN
2	CHANNEL CROSS-SECTION
	EASEMENT
	GABION STRUCTURE
	RIP-RAP LINING
	TEMP. PONDING AREA
SD	STORM DRAIN LINE
	STORM DRAIN MANHOLE
	DRAIN INLET
	SIDEWALK CULVERT
	LIMIT OF CONSTRUCTION
	RETAINING WALL

NOTE: THE ELEVATIONS SHOWN ON THIS SHEET ARE FOR ROUGH GRADING ONLY

REFER TO SOILS REPORT FOR EARTHWORK SPECIFICATIONS

CONTRACTOR MUST CONTACT CITY OF ALBUQUERQUE ENVIRONMENTAL HEALTH DEPT. TO OBTAIN TOPSOIL DISTURBANCE PERMIT.

APPROVED FOR ROUGH GRADING ± 1'
CITY ENGINEER DATE

SLOPE 3:1 TO MATCH EXISTING GROUND. IN AREAS OF CUT, CONTRACTOR SHALL GRADE TO ENSURE PROPER DRAINAGE

2'-0"
CONSTRUCT BERM WITH LOCAL MATERIAL (90% COMPACTION)
SWALE (UPSTREAM SIDE)
EXISTING GROUND

TEMP. EROSION CONTROL BERM
NTS

BENCH MARKS

CONTROL: BENCH MARK 7-F11, WHICH BEARS N 26° 09' 08" E, 758.55' FROM THE S.E. CORNER OF SANTA FE VILLAGE III. A 3/4" PIPE. BENCH MARK IS A STANDARD A.C.S. BRASS CAP STAMPED "7-F11".
M.S.L. ELEVATION=5116.57'

AS BUILT INFORMATION

SURVEY INFORMATION

ENGINEER'S SEAL

REVISIONS

DESIGN: DESIGNED BY S.T. RG DATE 2-86; DRAWN BY CM, AP, MS, RG, BT DATE 2-86; CHECKED BY E.O DATE 2-86

CITY OF ALBUQUERQUE
MUNICIPAL DEVELOPMENT DEPARTMENT
ENGINEERING DIVISION

TITLE: SANTA FE VILLAGE III PHASE I
GRADING & DRAINAGE WEST

APPROVALS	ENGINEER	DATE	APPROVALS	ENGINEER	DATE
City Engineer			Liquid Waste		
A.C.E.-Design			Traffic		
A.C.E.-Hydrology			Water		

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