LAYTON AVENUE & BEAR TRIBUTARY CROSSING STRUCTURE & STORM DRAIN

PREPARED FOR

CITY OF ALBUQUERQUE
P.O. BOX 1293
ALBUQUERQUE, NM 87103

JULY 5, 1989

PREPARED BY

BOHANNAN-HUSTON, INC.
COURTYARD I
7500 JEFFERSON ST., N.E.
ALBUQUERQUE, NM 87109

(505) 823-1000

JOB NO. 89182.01

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1.0 SCOPE OF REPORT

This analysis and report is prepared under the authorization of A/E Services Agreement No. 88-PWD-70, Layton & Bear Tributary Crossing Structure & Storm Drain. The Agreement includes three phases:

1. Preliminary Design Phase
2. Final Design Phase
3. Construction Phase

This report presents the hydrologic and hydraulic data and analysis that will serve as basis for the design as called for in Exhibit I, Section 1.g. of the contract, and represents partial completion of the Preliminary Design Phase.

2.0 INTRODUCTION

Layton Avenue, designated a Minor Arterial in the City of Albuquerque Long Range Major Street Plan, traverses the Bear Tributary Arroyo via a low flow crossing. Depending on the arroyo flow, vehicles crossing through water must risk nuisance splashing, reduction in traction or possibly danger of being carried downstream. Homeowners living adjacent to the crossing have complained of the continual splashing noise. Finally, the dip section and adjoining crest severely reduce line of sight for traffic in both directions.

To address these concerns, the City has contracted with Bohannan-Huston, Inc. (BHI) to design a structure to carry the Bear Tributary flow underneath Layton. Such a structure must carry the 100-year discharge safely while correcting the traffic site distance problem.

Construction of the crossing presents an opportunity to address other drainage problems one block east at the intersection of Eubank and Layton. Trickle flows from landscape irrigation of the Mountain Run Apartments frequently cross Eubank and continue down Layton to the Bear

Tributary, causing nuisance splashing from vehicles crossing Layton on Eubank. Larger rainfall runoff from the Apartments and the Mountain Run Shopping Center also cross Eubank, a major arterial, and continue west on Layton. An underground storm sewer is proposed to pick up these flows upstream of the Eubank/Layton intersection and carry them underneath Layton to the Bear Tributary.

3.0 HYDROLOGY

Extensive hydrologic analysis has been performed on the study area in the past. BHI was instructed in the contract to use this existing information after reviewing it for adequacy and accuracy. Table 1 presents this information for points of interest in the study area. Figure 1 shows the subbasins and points of interest and summarizes the data. Flowrates for both the 10-year and 100-year storms are needed to provide for flood protection as required by the City criteria.

The hydrology presented in Table 1 is a summary from the various reports and letters generated during development of the area (see Sources, Table 1). These sources are reproduced in Appendix I. Since two detention basins and one retention/detention basin are used to decrease peak runoff, Table 1 differentiates between the (fully) developed discharges and the discharges after retention/detention.

BHI, while not conducting an exhaustive check of these analyses, has at least reviewed the important variables to see if they are within acceptable ranges and has verified that all areas draining to Layton have been included. Typically, the subbasins within the Layton basin have a (rational formula) runoff coefficient C = .82 for 86% impermeable surface, a time of concentration of 10 minutes and a 6 hour, 100-year rainfall of 2.5 inches. The time to peak runoff for the Bear Tributary is 15 minutes, according to the Far Northeast Heights Master Drainage Plan. If time to peak for the Layton basin is assumed to be 2/3 of the time of concentration, then the Layton Tp = 2/3 x 10 = 7 minutes.

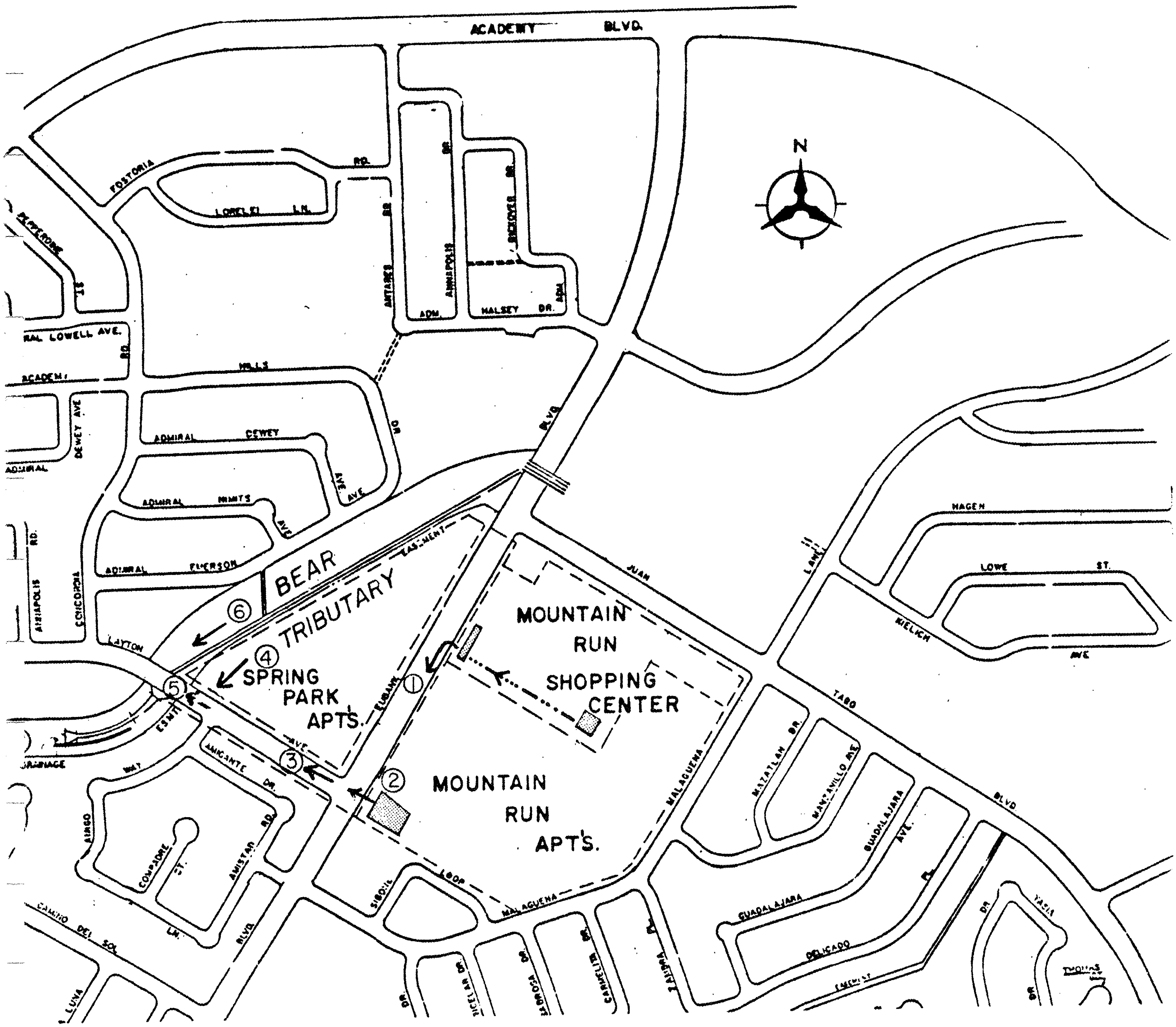
TABLE 1

LAYTON/BEAR TRIBUTARY AREA FLOW RATES (cfs)

POINT NO.	LOCATION	FULLY DEVELOPED NO DETENTION Q_{10}	FULLY DEVELOPED NO DETENTION Q_{100}	FULLY DEVELOPED DETENTION/RETENTION Q_{10}	FULLY DEVELOPED DETENTION/RETENTION Q_{100}
	N. END OF COMM. SITE	16.8[7]	25.4[2]	COMBINED WITH S. END	
	S. END OF COMM. SITE	15.5[7]	23.5[2]	17.2[7]	49.0[6]
	BANK SITE		5.0[2]		3.2[9]
	E. EDGE, SPR PK APTS.				5.4[12]
1	EUBANK N. OF LAYTON		53.9[2]	17.2[7]	49.0[6]
	FLOW GENERATED ON EUBANK	5.7[2]	8.7[2]	5.7[2]	8.7[2]
2	MT. RUN APTS. + REST.	51.9[1]	78.6[1]	28.0[2]	78.6[1]
3	LAYTON W. OF EUBANK	121.0[2]		64.6[2]	146.7[3]
4	SPRING PARK APTS.		43.0[4]	21.0[10]	43.0[4]
5	LAYTON @ BEAR TRIB.			85.6[11]	189.7[8]
6	BEAR TRIB ABOVE LAYT.	480.0[5]	992.0[5]	480.0[5]	992.0[5]
7	BEAR TRIB BELOW LAYT.	575.0[5]	1196.0[5]	575.0[5]	1196.0[5]

SOURCES OF INFORMATION

1. Mountain Run Apartments Drainage and Grading Plan, BHI, 8-83
2. Letter to B. Goolsby from BHI, 10-4-83 amending Drainage and Grading Plan.
3. 78.6[1] cfs + 49[6] cfs + 8.7[2] cfs + 5.0[2] cfs + 5.4[12] cfs = 146.7 cfs
4. Letter to B. Goolsby from Wilson & Co. Engineers, 12-6-84
5. Far Northeast Heights Master Drainage Plan, Weston, 1-88
6. Mountain Run Shopping Center Drainage Report, BHI, 9-83
7. Letters to B. Goolsby from BHI, 11-29-83 and 2-21-84
8. 146.7[3] cfs + 43.0[4] cfs = 189.7 cfs
9. Mountain Run parcel III Drainage and Grading Plan, BHI, 4-84
10. Estimated at 50% of Q_{100}, BHI, 6-89
11. 64.6[2] cfs + 21.0[10] cfs = 85.6 cfs
12. Revised Engineer's Report, Spring Hill Apartments, Wilson and Co., 3-17-85



DISCHARGES in CUBIC FT. per SEC.

	10-YEAR	100-YEAR
1	17.2	49.0
2	28.0	78.6
3	64.6	146.7
4	21.0	43.0
5	85.6	189.7
6	480.0	992.0
7	575.0	1196.0

LEGEND

DIRECTION of FLOW

DRAINAGE BASIN BOUNDARIES

DETENTION PONDS

LAYTON AVE. / BEAR TRIBUTARY CROSSING HYDROLOGY

FIGURE 1

It should be noted that the Far Northeast Heights Master Drainage Plan offers a flowrate at Point 5 much larger than that presented by the subdivision reports. It is our opinion, as well as the opinion of City hydrologists, that the subdivision reports are more accurate because of the greater detail included in the analyses. Appendix II contains correspondence concerning this disparity of discharges at Point 5.

4.0 STREET FLOW

The following City of Albuquerque criteria was used to determine adequacy of street capacity in Eubank and Layton:

A. For 10-year flows, the allowable depth of flow is the top of curb.

B. For 100-year flows, the allowable depth of flow is .2 feet above the top of the curb.

C. For arterials, one dry lane is required during the 10-year discharge.

D. 10-year flows or less cannot cross an arterial at an intersection.

Eubank is classified a major arterial and Layton a minor arterial by the City of Albuquerque Long Range Major Street Plan.

CRITERION

Since both Layton and Eubank have slopes in excess of 1.5%, these streets satisfy criteria A. and B. However, neither street satisfies Criterium C and all flows discharging from the Mountain Run Apartments currently cross Eubank at Layton. These flows are quantified and solutions presented in the following sections. Calculations for street hydraulics are found in Appendix III.

4.1 Eubank Street Flows. For the 10-year storm, discharges from the Mountain Run Shopping Center leave the retention/detention pond at the southwest corner of the shopping center and enter Eubank at a peak rate of 17.2 cubic feet per second (cfs). This entire flow is carried by the northbound lane. Using Criterium C., the allowable street flow is 4.5 cfs. Thus, 17.2-4.5 = 12.7 cfs must be picked up by a storm sewer. Plate I shows the installation of three catch basins just downstream of the southern outlet of the shopping center detention pond and a 24" RCP running southward from the catch basins towards Layton.

4.2 10-Year Flows Crossing Eubank. There are two sources of flows currently crossing Eubank at Layton in violation of Criterium D., the remaining Eubank street flows not captured by the Section 4.1 catch basins (4.5 cfs) and the flows leaving the Mountain Run Apartments (28.0 cfs). The street flows must be captured upstream of Layton, as the Eubank flowline warps from the east side to the west side as it approaches Layton. For this report, we show a wide grating to pick up the flows from the detention pond (which presently are spread wide under the sidewalk). However, less expensive alternates, such as installing a new pipe from the pond to the catch basin, will be considered during the design effort. Plate 1 shows the single catch basin upstream of Layton and the wide grated catch basin at the outlet of the Mountain Run Apartments detention pond. Downstream of the wide grating, the storm sewer is sized as a 30" RCP carrying 45.2 cfs.

4.3 Layton Street Flows. Criterium C calls for one dry lane down Layton during the 10-year storm. Calculations show that the carrying capacity of Layton is 30 cfs with one dry lane. The 10-year discharge at Point 3 is 64.6 cfs (see Figure 1). However, the storm sewer is already carrying 45.2 cfs, so the remaining street flow (64.6-45.2 = 19.4 cfs) is less than the maximum allowable 30 cfs.

At Point 5, a 10-year discharge of 21 cfs (see Point 4) enters Layton from the Spring Park Apartments. This swells the 10-year flow to 85.6 cfs. Since the allowable street flow is 30 cfs, an additional 10.4 cfs must be diverted into the storm sewer (85.6-30.0-45.2 = 10.4 cfs).

This is accomplished by adding two catch basins just downstream of the Spring Park discharge, as shown on Plate 1. The storm sewer downstream of this junction must then carry 45.2+10.4 = 55.6 cfs.

4.4 100-Year Street Flow. As stated in Section 1, Criterium B. is met on both Layton and Eubank even without the storm sewer. The only 100-year consideration is discharging the Layton flows into the Bear Tributary. This report proposes a street-wide grating and catch basin at the bottom of a designed sag in the vertical alignment of the roadway. The flows would drop vertically into a box culvert running parallel to seven 3'x6' concrete box culverts which will be proposed in Section 5. This eight box culvert, which would also accept flows from the proposed Layton storm sewer, would be dedicated exclusively to the Layton basin flows. Plate 1 shows this box culvert, grating and the storm sewer connecting to the box.

It is our opinion that separation of the Bear Tributary and Layton Avenue flows is superior to outletting the Layton Avenue flows into one of the Bear Tributary culverts. If the street flows were dropped into one of the arroyo boxes from above, much hydraulic turbulence would be induced, causing an increase in the box headwater depth during the 100-year flow.

5.0 BEAR TRIBUTARY CROSSING

The 100-year discharge for the Bear Tributary upstream of the Layton crossing is 992 cfs (see Figure 1). The entrance configuration is a wide grassy swale with a low flow concrete ribbon at the bottom to carry nuisance flows. The downstream side of the Layton crossing begins with a 118' wide bottom and sloped sides and rapidly narrows to a 31' wide bottom with sloping sides as it continues downstream.

From a design standpoint, it is preferable to place the new invert near or above its present vertically location, as a 12" cast iron waterline, a 24" concrete cylinder well collector water line and an 8" vitrified clay sanitary sewer line currently run under Layton at the crossing (see Plate 1) and could conflict with a deeply buried drainage structure. Opposing this vertical location is the need to provide for a deeper headwater at the inlet in order to push through the 100-year flow under inlet control conditions. To best meet these conditions, a wide, shallow crossing structure is indicated with the conduit placed as deep as possible without conflicting with the existing utilities.

This investigation looked at a variety of options using different materials, widths and depths, as well as the use of an earthen berm at the inlet to raise the water surface. A long span bridge was not investigated, as the cost was not considered effective compared to the other options considered. Four viable alternatives are presented in the following sections.

5.1 Alternative 1-A. This alternative includes the construction of a berm at the inlet and installation of 16 side-by-side reinforced concrete pipes (RCP's) 36" in diameter. The required headwater at the inlet is 4.5'. The width of the crossing is approximately 93'.

5.2 Alternative 1-B. Similar to Alternative 1-A, this alternative would require construction of an inlet berm and installation of 13 RCP's with a 42" diameter. 4.5' of head is required and a total width of 82'.

5.3 Alternative 2. This alternative does not include an inlet berm, so the headwater is limited to 3'. 14 concrete box culverts (CBC's), each 3' high by 5' wide, would be installed side-by-side. The total width would be approximately 84'.

5.4 Alternative 3. Alternative 3 includes the construction of the inlet berm to allow 4.5' of headwater. The conduits required are seven 3'x6' CBC's with a total width of 47'.

5.5 Costs. Looking only at the installed conduits and the berm where required, the following costs were estimated for the alternatives:

Alternative	Cost
1-A	$98,480.00
1-B	$107,580.00
2	$130,800.00
3	$117,100.00

5.6 Preferred Alternative. No clear choice is indicated by the cost differences shown in Section 5.5 above, as the total price difference between the four alternatives is only $30,000. Several considerations were taken into account by this report when choosing a preferred alternative:

a. The wide crossings (Alternatives 1-A, 1-B and 2) could cause meandering of the low flows at the inlet, jeopardizing the park sod at the inlet.

b. The 36" and 42" circular conduits are more prone to plugging than the concrete box culverts.

c. Installation of circular conduits invariably results in differential settlement of the roadway, as the compacted earth directly on top of the conduits has a different final settlement than the earth between the conduits. A "washboard" effect above the conduits is often the result. In contrast, the CBC's abut each other, and the settlement in the roadway above can be expected to be much more uniform.

These considerations lead to a preference for the box culverts, if price differentials are not great. Since Alternative 2 CBC's have no advantage over the Alternative 3 CBC's and since Alternative 3 is less expensive, it is the recommendation of this report that Alternative

3, with its seven 3'x6' CBC's, be used for the crossing design. This alternative is shown on Plate 1. Note that the eighth box culvert shown on Plate 1 is dedicated to Layton Avenue flows, and carries no Bear Tributary discharge.

6.0 CONCLUSION

This report defines the hydrology to be considered in design of hydraulic structures by summarizing various reports and letters submitted to and approved by the City of Albuquerque during development of the study area. The flowrates are summarized on Figure 1 for the 10-year and 100-year storms.

A 24" underground storm sewer is recommended, beginning on Eubank 345' north of Layton Avenue and running to Layton. At this point the storm sewer turns west and increases in size to 30" and finally to 36" before outletting into the Bear Tributary Arroyo (see Plate 1).

For the Bear Tributary crossing structure, seven 3'x6' concrete box culverts are recommended (see Alternative 3 above). This conduit and the storm sewer described in the preceding paragraph will carry the 10-year and 100-year storm runoff for the study area in compliance with City of Albuquerque standards.

DRAINAGE AND GRADING PLAN
FOR
MOUNTAIN RUN APARTMENTS

AUGUST, 1983

PREPARED FOR

CAMPBELL & COMPANY
5 POST OAK PARK, SUITE 1300
HOUSTON, TEXAS 77027

PREPARED BY

BOHANNAN–HUSTON, INC.
4125 CARLISLE BOULEVARD, N.E.
ALBUQUERQUE, NEW MEXICO 87107
505/881-2000

DAVID M. Y. MILLIKAN
NEW MEXICO
7547
REGISTERED PROFESSIONAL ENGINEER
8-19-83

David M Y Millikan

DAVID M. Y. MILLIKAN, P.E.
N.M.P.E. NO. 7547

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INTRODUCTION

The Mountain Run Apartment complex will contain 472 housing units in 31 building "clusters". The complex is to be constructed on a 15.85 acre site of presently undeveloped land.

The purpose of this report is to describe existing drainage conditions on and adjacent to the site, and to present a grading and drainage plan which provides a workable means for treating all flows impacting or generated on the site.

SITE LOCATION AND DESCRIPTION

The Mountain Run site is located on a portion of Tracts I and H of the Academy Place Subdivision in the City of Albuquerque, New Mexico. Tracts I and H are located at the southeast corner of Eubank and Juan Tabo Blvds. In addition to the Mountain Run Apartment Complex, an 11.73 acre commercial business site and a 2.30 acre restaurant site will also be located on Tracts I and H. Figure 1 is a vicinity map of the area.

The site slopes at approximately 4% from the east to the west. Soils on the site are classified as Embudo–Tijeras association (Etc). This soil type is described as level to moderately sloping, well drained loamy and gravelly soils. It is classified as hydrologic soil type "B" by the U.S. Soil Conservation Service. Figure 2 is a copy of the soils map for the area.

METHOD OF ANALYSIS

All methods of analysis and computations are done in accordance with Chapter 22 of the City of Albuquerque's Development Process Manual (DPM). Allowable discharge rates and volumes were determined in accordance with the "Drainage Ordinance".

For existing condition calculations, a runoff coefficient of .34 is used (type "B" soil, 0% impervious). A runoff coefficient of .82 is used assuming the site to be 86% impervious when developed.

Rainfall intensity calculations are based on a 6 hr. rainfall volume of 2.5 in., (as obtained from Plate 22.2 D–1 of the DPM). For existing 10-year and 100-year rainfall intensity calculations, a time of concentration of 25.0 min. is used. For all developed condition calculations, rainfall intensity is found assuming the time of concentration to be 10.0 min. Time of concentration calculations are included in the Appendix.

The Rational Formula has been applied to determine 10-year and 100-year peak flow rates and runoff volumes.

EXISTING DRAINAGE CONDITIONS

The only flows which impact the site are generated on the remainder of Tracts I & H. The Flood Hazard Boundary Map (Figure 3) indicates that flows in Juan Tabo and Eubank are contained within the street right-of-way.

Under present conditions, all of Tracts I & H including the Mountain Run Site drain to the southwest corner of the Mountain Run Apartment site. Therefore, flow rate and runoff volume values have been computed for all of Tracts I & H. The portion attributable to the Mountain Run Apartment site can be found by proportioning its area to the remainder of the 29.88 acre tract. The 10-year peak flow rate for the Mountain Run site is 12.0 cfs. The 100-year peak flow rate is 18.2 cfs. The 10-year and 100-year runoff volumes are 0.737 ac.-ft. and 1.124 ac.-ft., respectively. Computations are included in the Appendix.

DEVELOPED DRAINAGE CONDITIONS

All flows impacting the site will be generated on the Mountain Run Apartment site or the restaurant site. Runoff from the commercial site will discharge directly to Eubank without crossing this site. Internal street capacities are sufficient to allow discharge from the adjacent restaurant site to be conveyed through the site.

Runoff from this site is discharged to Eubank Blvd. The flows are then discharged from Eubank Blvd. to Layton Avenue. From this point the flows are conveyed in Layton Avenue to the Bear Tributary Arroyo. Page 4 of the Appendix shows calculations determining the capacity of Layton Ave. It has been assumed that all basins which drain to Layton Avenue under present conditions will drain to Layton Avenue under developed conditions. The boundaries of this basin are shown on Figure 3. The capacity of Layton just upstream of the Bear Tributary Arroyo was calculated as 260 cfs. Assuming the entire basin contributing to Layton is developed with a runoff coefficient of 0.82 results in a peak flow rate in Layton of 181 cfs. (41.4 ac.) This is less than the capacity of Layton, therefore, uncontrolled discharge from the site is allowable.

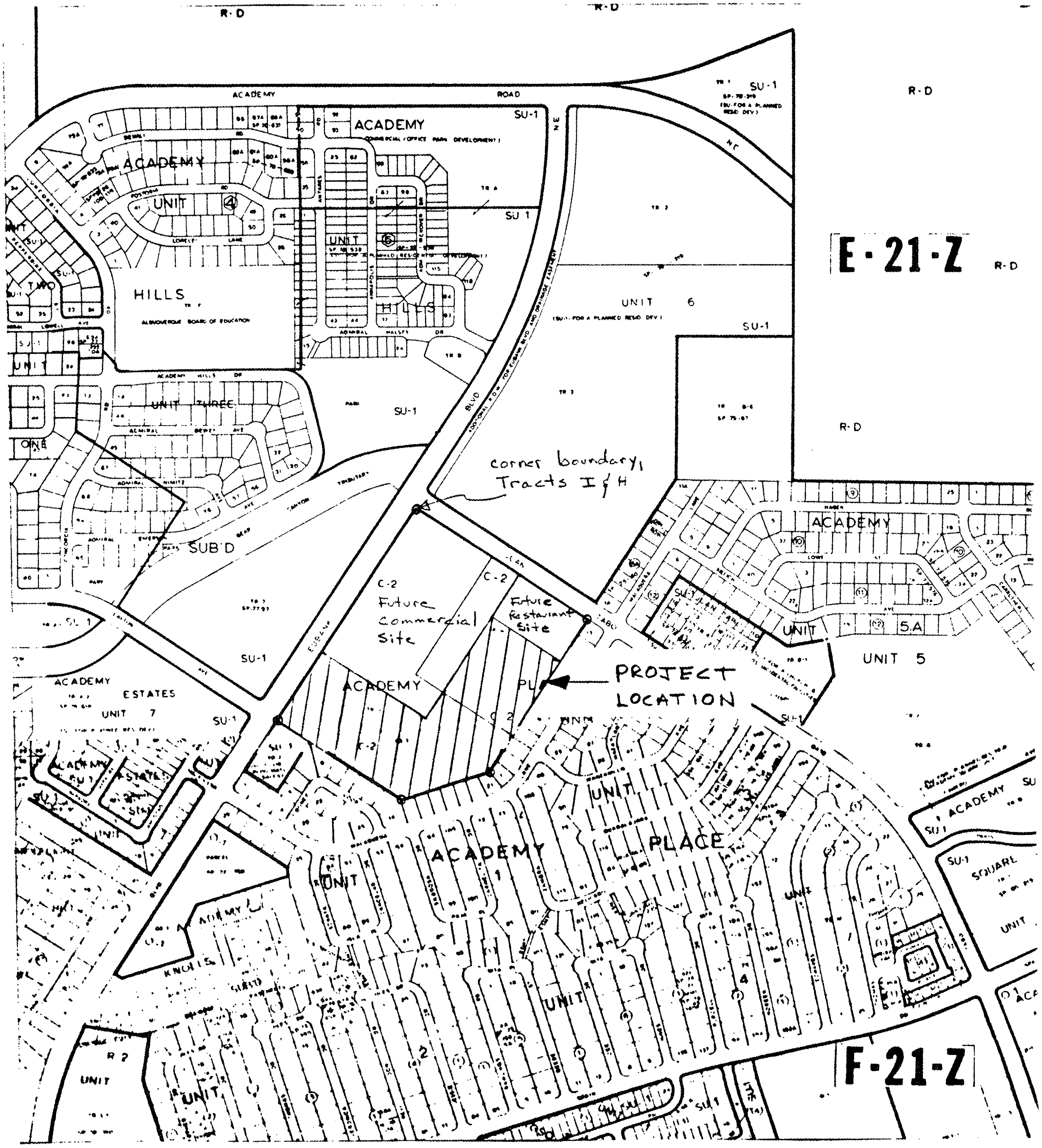
All runoff from the site will be discharged to Eubank at the southwest corner of the site. A small retention facility will be constructed at this corner of the site to prevent nuisance flows from entering Eubank. This is in accordance with the "Drainage Ordinance". Flows which exceed the capacity of the small retention facility will be discharged over a turfed weir at the end of the facility. They will be conveyed across Eubank Blvd. and routed down Layton Avenue to the Bear Tributary Arroyo. Based on a 100-year storm, a peak flow rate of approximately 79 cfs will be discharged from the site.

The Drainage and Grading Plan (Plates 1 and 2) indicate the drainage basins, flow directions and flow quantities (100-year storm) corresponding to the developed grading of the site. Pages 2 and 3 of the Appendix show individual basin flow rates and runoff volume values. Curb openings will be used in parking area islands in Basins B and J. These openings will be 2 ft. in width. Flows generated within Basins E and F will be collected by a curb opening inlet,routed through an 18" concrete pipe and discharged through an expansion box into Basin J. All flows, excluding those from Basin L, are conveyed through the small retention facility. Flows from Basin L will be discharged directly onto Eubank Blvd. Nuisance flows from Basin L should be minimal, since the only impervious areas in this area are the roofs of the buildings adjacent to Eubank.

EROSION CONTROL PLAN

Slopes on the site at locations where there are concentrated flows are mild (1%–5%). During the construction phase, erosion in these areas should be minimal. Upon completion of the paving and landscaping no erosion should occur in these areas. Slopes in landscaped areas are 3:1 or flatter. No concentrated flows are directed over these slopes. Therefore, erosion upon completion of the project should not occur in these areas.

All runoff from the site (except Basin L) is being routed through the small retention facility in the southwest corner. Therefore, any erosion that occurs on the site during construction should be prevented from leaving the site.



VICINITY MAP

FIGURE 1

see commercial site report (#31860) for pages 4+5

I + H, EXISTING DRAINAGE CONDITIONS

per. (c = .34 , "B" soil)

.ainfall = 2.5"

Tracts I + H (total area):

$s = .022$ $V = 1.1$ ft/s $T_c = \frac{1650}{1.1(60)} = 25.0$ min

$A = 29.88$ ac

$I/6 = 1.35$ in/hr

$I_{10} = (2.5 \times .657)(1.35) = 2.2$ in/hr

$I_{100} = (2.5)(1.35) = 3.4$ in/hr

$Q_{10} = (.34)(2.2)(29.88) = 22.6$ cfs

$Q_{100} = (.34)(3.4)(29.88) = 34.3$ cfs

$V_{10} = (.34)(2.5 \times .657)(29.88)/12 = 1.39$ ac-ft

$V_{100} = (.34)(2.5)(29.88)/12 = 2.12$ ac-ft

Proportioning by area:

apartment site = $\frac{15.85}{29.88} \times 100 = 53\%$

future shopping center = $\frac{11.73}{29.88} \times 100 = 39\%$

future restaurant = $\frac{2.30}{29.88} \times 100 = 8\%$

∴

	restaurant	shp. center	apart. site
Q_{10} (cfs)	1.8	8.8	12.0
Q_{100} (cfs)	2.8	13.4	18.2
V_{10} (ac-ft)	.111	.542	.737
V_{100} (ac-ft)	.170	.827	1.124

PROJECT NAME Mtn. Run Apts. SHEET 1 OF 8

PROJECT NO. 31543 BY [initials] DATE 8/15/83

SUBJECT Existing Drainage Calcs. CH'D DATE

BASIN DISCHARGE FLOWRATES AND RUNOFF VOLUMES

Analyzing a typical section of the site:

$A_{total} = .494$ ac
$A_{imper} = .250$ ac

$\therefore \frac{.250}{.494} = .51$ or 51% impervious

Analyzing Basin A for instance:

$A_{total} = 1.76$ ac
$A_{street} = 1.25$ ac (100% imper.) , $A_{other} = .51$ ac (51% imper.)

$\therefore \frac{.51}{1.76}(.51) + \frac{1.25}{1.76}(1.00) = .86$ or 86% imper.

Use 86% imper. for developed condition

note- per DPM Plate 22.2 C-1, runoff coeffecient C = .82 for 86% imper., type "B" soil. Also, 6hr rainfall, 100yr frequency = 2.5 in

Sample Calc.

Basin A: $s = .015$ $\therefore V = 3.3$ ft/s (Plate 22.2 B-2)
$A = 1.70$ ac

$T_c = \frac{L}{V(60)} = \frac{950}{3.3(60)} = 4.~~8~~ ~~min~~ \rightarrow$ assume 10.0 min (use 10.0 min for all basin calcs)

$I/6 = 2.11$ in/hr (Plate 22.2 D-2)

$I_{10} = (2.11)(2.5 \times .657) = 3.5$ in/hr $\quad I_{100} = 2.11(2.5) = 5.3$ in/hr

$Q_{10} = CI_{10}A = (.82)(3.5)(1.83) = \underline{5.25 \text{ cfs}}$ $\quad Q_{100} = CI_{100}A = (.82)(5.3)(1.83) = \underline{7.95 \text{ cfs}}$

$V_{10} = (.82)(2.5 \times .657)(1.83)/12 = \underline{0.21 \text{ ac-ft}}$ $\quad V_{100} = (.82)(2.5)(1.83)/12 = \underline{0.31 \text{ ac-ft}}$

PROJECT NAME Mtn. Run Apts. SHEET 2 OF 8
PROJECT NO. 31543 BY LJ DATE 8/15/83
SUBJECT Developed Drainage Calcs. CH'D DATE

SUMMARY OF BASIN FLOWRATES AND RUNOFF VOLUMES

Basin	A (ac)	Q_{10} (cfs)	Q_{100} (cfs)	V_{10} (ac-ft)	V_{100} (ac-ft)
Restaurant site	2.40	6.89	10.43	0.27	0.41
A	1.83	5.25	7.95	0.21	0.31
B	2.18	6.26	9.47	0.24	0.37
C	1.03	2.96	4.48	0.12	0.18
D	0.93	2.67	4.04	0.10	0.16
E	1.77	5.02	7.61	0.20	0.30
F	0.88	2.53	3.82	0.10	0.15
G	2.20	6.31	9.56	0.25	0.38
H	0.85	2.44	3.69	0.10	0.15
J	2.49	7.15	10.82	0.28	0.42
K	1.13	3.24	4.91	0.13	0.19
L	0.42	1.21	1.83	0.47	0.07
		51.93	78.61		

note - All basins contribute to retention facility at southwest corner of site except for Basin L. Total Q_{100} = 76.78 cfs at facility.

PROJECT NAME Mtn. Run Apts. SHEET 3 OF 8
PROJECT NO. 31543 BY KJ DATE 8/15/83
SUBJECT Developed Drainage Calcs. CH'D DATE

Size SD pipe to carry flows Basins E+F to J

$$\overset{11.4}{\cancel{Q}} = \overset{.6}{\cancel{c}} A\sqrt{2gh} \quad \text{or} \quad h = \frac{1}{2g}\left(\frac{Q}{CA}\right)^2$$

available slope of hydr. g.l. (℄ minus top of pipe at outlet) -

$$s = \frac{60.5' - 56.5'}{120} = .031$$

cap. of 18" pipe flowing full is ≈ 19.0 cfs (from design manual) ∴ OK.

$$A = \pi r^2 = \pi\left[\frac{18}{2(12)}\right]^2 = 1.77\ ft^2$$

$$h = \frac{1}{2g}\left[\frac{11.4}{.6(1.77)}\right]^2 = 1.79\ ft$$ (center of pipe to ℄ of inlet)

add .5 ft additional clearance plus 1/2 pipe dia.

$$1.79 + 0.50 + 0.75 = \cancel{3.04'}\ 3.0'$$ (min. elev. diff. between inlet grade elev. + invert of pipe)

inlet ℄ elev. 60.5' minus 3.04' = 57.5'

$$\frac{57.5' - 55.0'}{120'} = .019$$ (18" pipe at 1.9% has cap. of ≈ 15 cfs) ✓

Size curb opening inlet

$$\overset{}{\underset{11.4}{\cancel{Q}}} = \underset{3.0}{\cancel{C}} L H^{3/2} \quad \text{or} \quad L = \frac{Q}{CH^{3/2}}$$

try H = 6"

$$L = \frac{11.4}{(3.0)(.5)^{3/2}} = 10.7'$$

try H = 8"

$$L = \frac{11.4}{(3.0)(.67)^{3/2}} = \cancel{6.9} \rightarrow \text{use } 7.0'$$

∴ use 8" high curb opening - extend 8" high curb 5' out from each side of inlet.

STORM DRAIN OUTLET DESIGN
(Expansion Box)

Normal depth of outlet:

try 6' width @ 2% slope -

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$D = .32' = 3.89''$ (ht. of outlet = 4.00")

Near pressure flow, check orifice eqn.:

$$Q = cA\sqrt{2gh} \Rightarrow h = \frac{1}{2g}\left(\frac{Q}{cA}\right)^2 = \frac{1}{2g}\left(\frac{11.4}{.6(1.77)}\right)^2 = 1.31' = 15.78''$$

Add 2" for 1/2 depth of opening (17.78")

Top of pipe = 1'6" = 18"

Therefore, OK as flow in pipe is not under pressure flow.

Use 6' wide, 4" deep outlet at 2% slope.

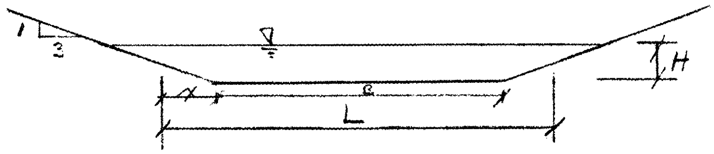
PROJECT NAME Mtn. Run Apts. SHEET 7 OF 8
PROJECT NO
BY DMYM DATE 8/16/83
SUBJECT SD Outlet Sizing CH'D [illegible] DATE 8/16/83

Retention Facility Weir Design

$Q = CLH^{3/2}$

Let $B = 15'$ w/ 3:1 side slopes, $C = 3.0$

$Q = 63.0$ cfs (contributions from all basins but K)



$B = L - 2x$, $H = 2/3\,x$

$x = 3/2\,H$

$\therefore$ $B = L - 3/2\,H$

$L = 3/2\,H + 15$

$Q = CLH^{3/2}$

$Q/C = (3/2\,H + 15)(H^{3/2}) = 3/2\,H^{5/2} + 15\,H^{3/2} = \frac{78.6}{3.0} = 26.2$

$H = 1.35'$ ✓

Construct 3:1 side sloped turfed weir w/ 15' bottom width

PROJECT NAME Mtn. Run Apts. SHEET 8 OF 8

PROJECT NO. 31543 BY KJ DATE 8/16/83

SUBJECT Retention Facility Weir Design CH'D DATE

BOHANNAN-HUSTON INC.
4125 CARLISLE BLVD., N.E. ALBUQUERQUE, NEW MEXICO 87107 2000

October 4, 1983

Mr. Billy J. Goolsby, P.E.
Civil Engineer/Hydrology
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

Re: Drainage and Grading Plan for Mountain Run Apartments (E21--D22)

Dear Billy:

Please consider this letter as an amendment to the Mountain Run Apartments Drainage and Grading Plan. The purpose of this plan is to define the required drainage improvements and demonstrate the feasibility of the proposed improvements. A detailed grading plan at a scale of 1"=30' will be prepared for construction. For this reason the grading plan was not prepared with all of the details required for construction. The detailed grading plan will be submitted to you for review and approval before construction begins.

A review of the "Drainage Ordinance" and the Development Process Manual indicates that flows within the streets must be designed in accordance with the folowing criteria:

a. Manning's roughness coefficient is 0.017.
b. Flow depths in the event of the 100-year design discharge may not exceed 0.2 feet above curb height or 0.87 feet at any location.
c. Flow depths in the event of the 10-year design discharge may not exceed 0.5 feet in any collector or arterial street. One lane free of flowing or standing water in each traffic direction must be preserved on arterial streets.
d. The product of depth times velocity shall not exceed 6.5 in any location in any street in the event of a 10-year design storm (with velocity calculated as the average velocity measured in feet per second and depth measured at the gutter flowline in feet.)
e. The discharge of nuisance waters to public streets shall be discouraged.

With this criteria in mind, our responses to your comments are as follows:

1. In our report, it indicates that in the undeveloped state, all of the runoff from Tracts I and H is discharged at the southwest corner of the site. When developed, as shown in our report, only runoff from the apartment site and possibly the restaurant site will occur at the southwest corner. Runoff from the commercial site will be discharged at the southwest corner of the commercial site, not the aprtment site. Therefore, the maximum discharge at any one point to Eubank is approximately 79 cfs.

Combining the flow rates from this report, and the shopping center report results in the following flow rates.

Apartment site + restaurant site	78.6 cfs
South end of commercial site	23.5 cfs
North end of commercial site	25.4 cfs
Bank site	5.0 cfs
Total	132.5 cfs

The total flows discharged to Eubank are not concentrated at one location, but at four separate locations. Of these flows, the 78.6 cfs does not have to flow along the curbline in Eubank. It is discharged in such a location that it may flow directly across Eubank to Layton Avenue. Therefore the maximum flow rate which must be conveyed in the east half of Eubank is 53.9 cfs. This is well within the capacity of the street.

2. Since no criteria is given concerning flows entering a street from the side, the following assumption was made. The flows entering at one location are very similar to flows entering at a side street. The only criteria for flows entering at a side street are as listed above. Upon review of this criteria, it is our interpretation that the flows entering from the side may not exceed a depth of 0.5 feet or have a depth times velocity in excess of 6.5 due to a 10-year design discharge.

   We are enclosing sketches of the proposed outlet to Eubank. The flows enter Eubank in compliance with these criteria. Since the discharge rate of 78.6 cfs does not have to change directions and flow to the south in Eubank, the formula given in your letter for computing depth of flow at this point in Eubank does not appear applicable.

   Since the manner in which these flows are introduced into Eubank meet the criteria for a side street, we feel that it is satisfactory.

3. The additional flows leaving this site due to development will not affect the peak flow rate in the Bear Tributary Arroyo at its crossing of Layton Avenue. Therefore, whether or not the crossing is in place or not, should not affect the allowable discharge from this site.

4. A review of the long range major street plan does indicate that Layton Avenue is designated as a minor arterial. A significant amount of discussion occurred along with review of the S.A.D. No. 210 drawings concerning criteria which states that one lane in each direction must be free of flowing water. Our understanding of the outcome of those discussions was that for six-lane arterials that criteria was satisfactory. However, for arterials which only consist of four lanes,

the 0.5 foot depth criteria is more applicable. The 0.5 foot depth criteria is equivalent on a four-lane road to the one lane in each direction free of flowing water criteria for six-lane roads, if the depression within the gutter is excluded. We have calculated the allowable flow rate in Layton to comply with this criteria (64.6 cfs). The estimated 10-year developed discharge from the basins (which contribute to Layton Avenue under present conditions) is 121.0 cfs (see enclosed computations). Therefore, in order to meet the 10-year criteria, some ponding must be provide on each site. As indicated in our report, ponding is not required to meet the 100-year criteria.

We have revised the grading at the southwest corner of the site to provide the required amount of ponding. The peak discharge from the apartment site during a 10-year storm will be 28 cfs. This peak discharge rate requires a ponding volume of 0.40 acre-feet.

We have enclosed hydrographs, computations, and diagrams indicating the proposed outlet configuration. The drainage solution outlined in these enclosures meets all of the applicable 10-year and 100-year criteria contained in the "Drainage Ordinance" and the DPM.

5.a. Spot elevations are shown at all property corners on the outside of the existing block wall around the east and south sides of the site. Since the property on the outside of the wall is under separate ownership, is relatively inaccessible, and is not likely to change, spot elevations should be adequate.

b. All curb within City rights-of-way is 8" high. All curb within the site is 6" high. Locations where there is depressed curb or unusual conditions will be shown in more detail on the final grading plan.

c. There are no drainage easements proposed within this site.

d. Additional information will be provided on the final grading plan. In no cases will the finished floor elevation be beneath the adjacent finished ground elevation.

6. The drainage solution proposed in the original report did not propose any detention. Therefore, hydrographs were not necessary. Hydrographs are provided for the revised solution outlined in this report.

7. The drainage report for the commercial site was submitted to your office on September 1,1983. The restaurant site will be submitted as a separate report. The bank site at the corner of Eubank and Juan Tabo will also be submitted as a separate report. Each of these reports should be approved as a separate document.

8. Additional notes will be provided on the final grading plan which outline the information provided in the end of the original report. It is our belief that this meets the requirements of an erosion control plan.

If you have any additional comments or questions, please contact Dave Millikan or me.

Sincerely yours,

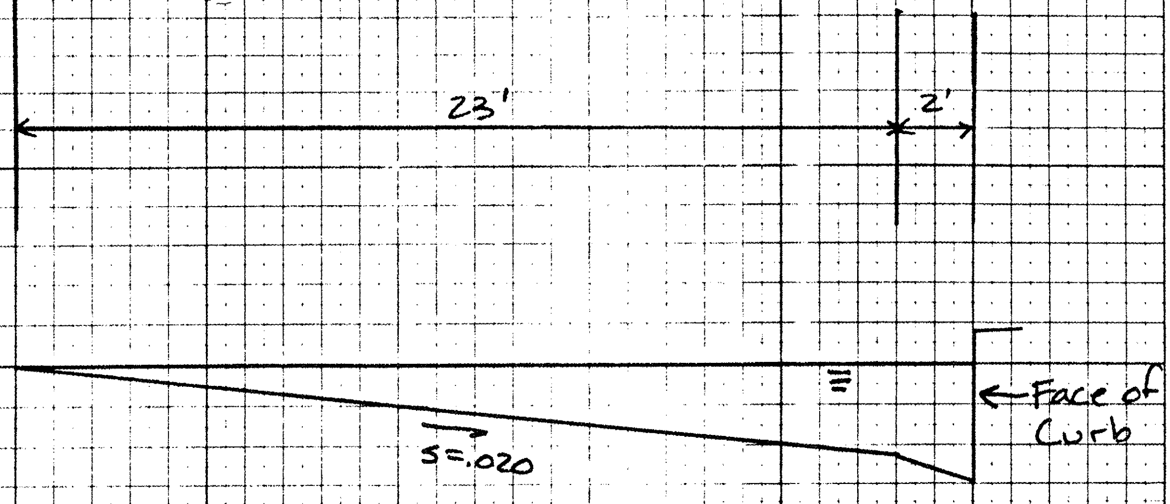
David M y Millikan

for Michial M. Emery, P.E.
Vice President

Enclosures

cc: Mr. Glenn Gronnerud

DMYM/plm
Job No. 3 154 3



$A = .125 + 1.17 + 5.29 = 6.585$ sf

$P = 2.004 + 23.005 + .585 = 25.594$

$R = .2573$

$$Q = \frac{1.486}{n} A \; R^{2/3} \; S^{1/2}$$

$$Q = \frac{1.486}{.017} (6.585)(.2573)^{2/3}(.0192)^{1/2}$$

$Q = 32.3$ cfs / half street

$2Q = 64.6$ cfs / full street

PROJECT NAME Mountain Run Apts SHEET ____ OF ____

PROJECT NO. 31543 BY Dmym DATE 10-4-83

SUBJECT Layton Ave 10-yr Capacity CH'D ____ DATE ____

Flows Impacting Layton Ave. Vs. 10-yr Capacity of Layton Ave.

Undeveloped land bounded by Layton, Eubank & Bear Tributary Arroyo

Area = 9.9 ac   assume "c" = 0.82

Assume T_c = 10 min

$I_{100} = 5.3$

$I_{10} = (.657)(5.3) = 3.48$

$Q_{10} = (0.82)(3.48)(9.90) = \underline{28.25\ cfs}$

Eubank R/W

Area = 1.65 ac   assume "c" = 1.00

Assume T_c = 10 min

$I_{10} = 3.48$

$Q_{10} = (1.0)(3.48)(1.65) = \underline{5.74\ cfs}$

$Q_{100} = (1.0)(5.3)(1.65) = 8.7\ cfs$

Future Shopping Center

from Report

$Q_{10} = \underline{32.3}\ cfs$

Developed Apartment site

$Q_{10} = \underline{51.9}\ cfs$

Future Bank Site

Area = 1.0 ac.

$I_{10} = 3.48$

assume "c" = 0.82

$Q = (0.82)(3.48)(1.0) = \underline{2.85\ cfs}$ Total = 121.0 cfs

PROJECT NAME Mountain Run Apts   SHEET 1 OF 2

PROJECT NO. 31543   BY Dmym   DATE 10-4-83

SUBJECT Flows Impacting Layton Ave.   CH'D   DATE

INC

Allowable Discharges (10-yr Storm)

Undeveloped Land Bounded by Layton, Eubank & Bear Tributary Arroyo

$$28.25 \times \frac{64.6}{121.0} = 15.1 \text{ cfs}$$

Future Shopping Center

$$32.3 \times \frac{64.6}{121.0} = 17.2 \text{ cfs}$$

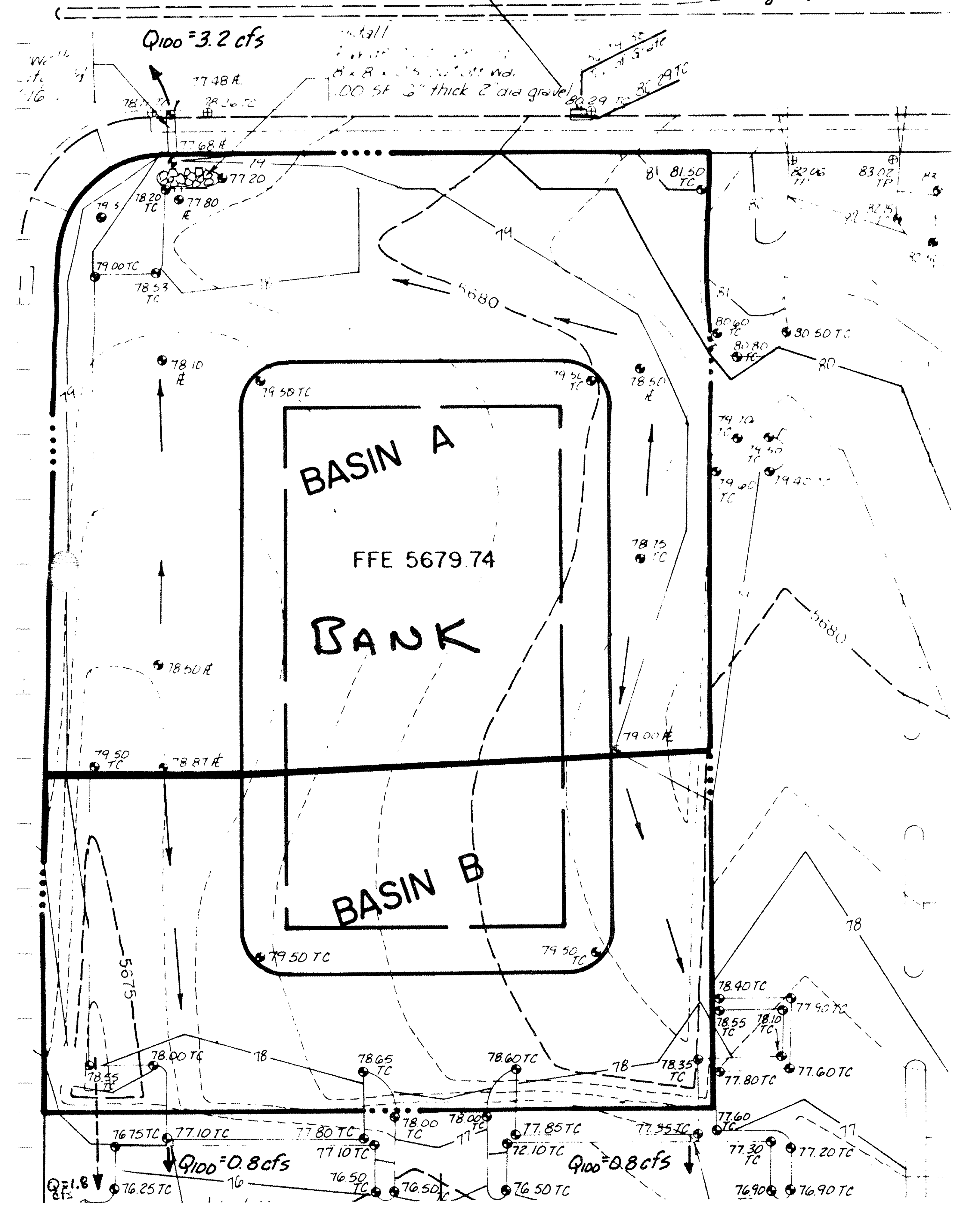
Developed Apartment Site

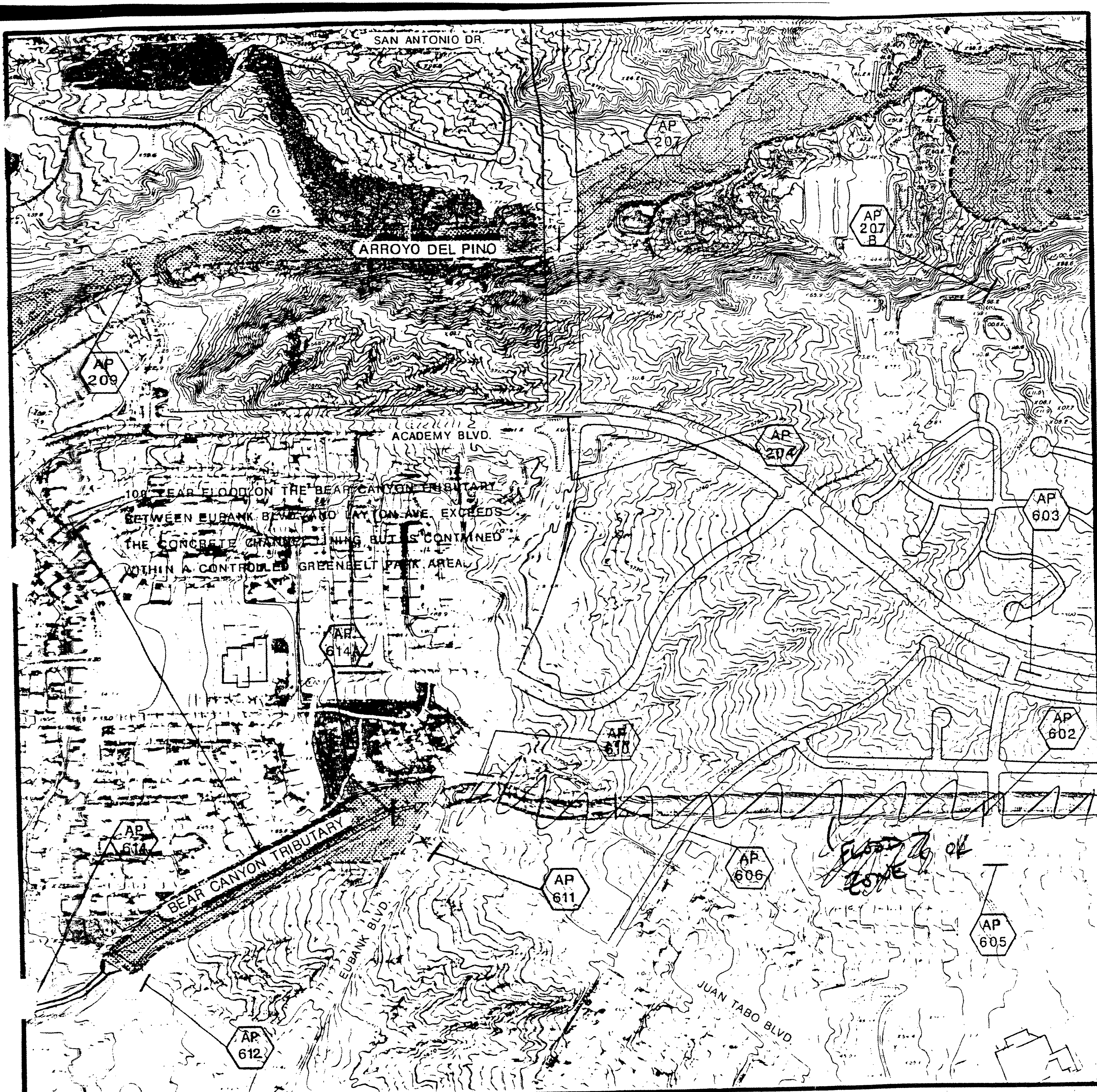
$$51.9 \times \frac{64.6}{121.0} = 27.7 \text{ cfs}$$

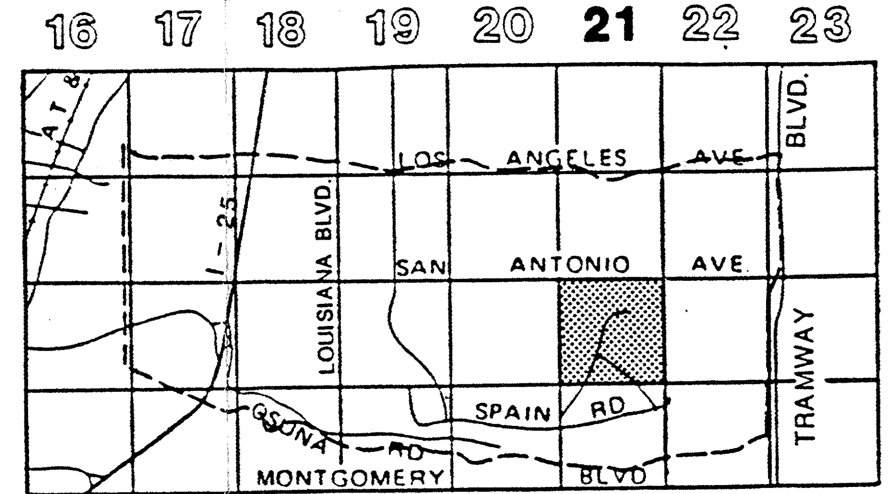
Future Bank Site

$$2.85 \times \frac{64.6}{121.0} = 1.5 \text{ cfs}$$

MT. RUN PARCEL III DRAINAGE & GRADING PLAN 4-84







MAP KEY

WESTON
MANAGERS DESIGNERS/CONSULTANTS
5301 CENTRAL AVENUE, N.E.
ALBUQUERQUE, NEW MEXICO 87108
PHONE: (505) 255-1445

ALBUQUERQUE NEW MEXICO

N

SCALE: 1" = 500'

1000 500 250 0 500 1000

SCALE IN FEET

LEGEND

- STUDY AREA BOUNDARY
- 10 YEAR FLOOD HAZARD AREA
- 100 YEAR FLOOD HAZARD AREA
- AP 102 — ANALYSIS POINTS
- PSD-102 RP 24" — STORM DRAIN IMPROVEMENT, RP= REPLACE PIPE, ADD = NEW CONSTRUCTION

E-21

APPENDIX F (CONTINUED)

NOTE: FUTURE CONDITIONS ARE SHOWN IN BRACKETS [ ]

ANALYSIS POINT ID NO.	ZONE ATLAS MAP NO.	10 YEAR FLOOD Q10 (cfs)	10 YEAR FLOOD OVERLAND FLOW (cfs)	10 YEAR FLOOD OVERLAND DEPTH (ft)	10 YEAR FLOOD OVERLAND VELOCITY (fps)	100 YEAR FLOOD Q100 (cfs)	100 YEAR FLOOD OVERLAND FLOW (cfs)	100 YEAR FLOOD OVERLAND DEPTH (ft)	100 YEAR FLOOD OVERLAND VELOCITY (fps)	REMARKS AND CHANNEL OR DAM LOCATION (IF APPLICABLE)
508	F20	81 [81]	81 [34]	0.72 [0.52]	5.0 [3.7]	164 [164]	164 [117]	0.97 [0.83]	8.0 [6.3]	
* 509	F20	249				517				BEAR ARROYO
* 511	F20	292				611				BEAR ARROYO
603	F21	57	0	0.00	0.0	138	0	0.00	0.0	
605	E21	57	57	0.63	4.5	119	119	0.87	5.4	
602	E21	196 [225]	0	0.00	0.0	453 [487]	0	0.00	0.0	
606	E21	260 [271]	0	0.00	0.0	583 [583]	0	0.00	0.0	
610	E21	30 [80]	0 [0]	0.00 [0.00]	0.0 [0.0]	85 [167]	0 [77]	0.00 [0.50]	0.0 [6.1]	[EXPANDED PRINCIPAL ART.]
611	E21	140	90	0.65	8.0	265	214	0.87	8.0	
* 614-A	E21	420 [480]				908 [992]				BEAR ARROYO TRIBUTORY
* 614	E21	518 [575]				1123 [1196]				BEAR ARROYO TRIBUTORY
623	E20	157 [157]	157 [14]	0.80 [0.36]	7.2 [3.4]	324 [324]	324 [180]	1.10 [0.87]	8.5 [7.5]	
* 618	E20	691 [735]				1495 [1546]				BEAR ARROYO TRIBUTORY
* 630	E20/F20	794 [830]				1715 [1768]				BEAR ARROYO TRIBUTORY
* 631	F19	816 [923]				1802 [1979]				BEAR ARROYO TRIBUTORY
* 632	F19	808 [944]				1810 [2031]				BEAR ARROYO TRIBUTORY
701	E18	99	99	0.68	8.0	204	204	0.87	8.6	
* 703	E18	157 [174]				343 [375]				BOREALIS ARROYO
705	E18	96 [96]	96 [0]	0.77 [0.00]	5.0 [0.0]	199 [199]	199 [88]	0.95 [0.79]	6.0 [3.2]	
* 704	E18	232 [250]				517 [552]				BOREALIS ARROYO
* 709	E18	282 [296]				618 [652]				BOREALIS ARROYO
710	E18	83	0	0.00	0.0	176	86	0.61	6.0	
* 718	E17	503 [643]				1060 [1319]				BOREALIS ARROYO
723	E17	130	114	**	**	256	240	**	**	PONDING/UNDERSIZED CULVERT
* 724-A	E17	593 [725]				1244 [1493]				BOREALIS ARROYO
* 724	E17	534 [1356]				2363 [2847]				BOREALIS ARROYO
801	E17	127 [244]	127 [0]	0.66 [0.00]	6.2 [0.0]	279 [467]	279 [166]	0.93 [0.75]	7.5 [6.6]	
802	E17	183 [411]	183 [0]	0.79 [0.00]	6.6 [0.0]	416 [800]	416 [142]	1.10 [0.80]	8.1 [5.0]	

APPENDIX E (CONTINUED)

NOTE: FUTURE CONDITIONS ARE SHOWN IN BRACKETS [ ]

SUBBASIN ID NO.	AREA (SQ.MI.)	CURVE NO.	PERCENT IMP.	TP (HRS.)	K (HRS.)	Q10 (cfs)	Q100 (cfs)
406	0.0430	70	10	0.119	0.060	15	45
		[75]	[40]			[56]	[118]
408	0.1973	70	10	0.287	0.143	34	85
		[75]	[40]			[123]	[257]
410	0.1614	74	41	0.218	0.109	127	264
414	0.1507	78	45	0.393	0.197	84	171
416	0.1363	75	40	0.216	0.108	108	226
419	0.0933	70	10	0.156	0.078	26	64
		[80]	[50]			[128]	[260]
421	0.0933	70	10	0.278	0.139	17	41
		[80]	[50]			[79]	[158]
425	0.1220	75	40	0.215	0.107	97	203
427	0.1076	80	50	0.158	0.079	146	296
432	0.0933	74	31	0.334	0.167	35	77
434	0.1327	86	70	0.219	0.109	165	316
436	0.0466	80	90	0.390	0.195	40	73
437	0.1650	79	41	0.194	0.097	137	292
		[82]	[56]			[184]	[368]
442	0.1435	79	41	0.190	0.095	122	259
		[88]	[72]			[212]	[403]
443	0.1363	88	40	0.153	0.076	193	395
		[88]	[72]			[244]	[465]
445	0.1220	90	82	0.257	0.128	151	280
446	0.0650	88	72	0.129	0.064	135	258
447	0.0310	88	72	0.148	0.074	56	108
448	0.0430	88	72	0.138	0.069	83	160
450	0.0717	79	75	0.170	0.085	108	205
		[88]	[72]			[117]	[222]
451	0.0359	88	72	0.320	0.160	34	64
500	0.0646	75	40	0.153	0.076	69	145
503	0.0574	79	49	0.111	0.056	102	208
508	0.0861	79	49	0.233	0.116	81	164
509	0.0717	75	40	0.142	0.071	81	170
511	0.0430	75	40	0.141	0.071	48	102
600	0.0789	72	12	0.135	0.068	32	92
		[72]	[30]			[68]	[151]
602	0.0861	75	40	0.154	0.077	91	191
603	0.1363	73	21	0.236	0.118	57	138
605	0.0753	75	40	0.228	0.114	57	119
606	0.0825	80	51	0.118	0.059	146	296
		[75]	[40]			[109]	[230]
610	0.1148	72	13	0.251	0.126	30	85
		[75]	[40]			[80]	[167]
611	0.1004	87	69	0.225	0.113	140	265
612	0.0717	90	78	0.102	0.051	219	410
614	0.1004	72	27	0.214	0.107	54	122
618	0.0538	75	40	0.134	0.067	64	135
623	0.1578	77	44	0.191	0.095	157	324
630	0.1255	76	42	0.193	0.097	115	241

INFORMATION SHEET

PROJECT TITLE Mountain Run Shopping Center **TYPE OF SUBMITTAL** Drainage Report

ZONE ATLAS PAGE NO. E-21, F-21 **CITY ADDRESS**

LEGAL DESCRIPTION 10.73 acre site of Tracts I & H, Academy Place Subdivision

ENGINEERING FIRM Bohannan–Huston, Inc. **CONTACT** Dave Millikan

ADDRESS 4125 Carlisle Blvd., NE **PHONE** 881-2000

OWNER The Dawn Company **CONTACT** Mr. Bob Spooner

ADDRESS 6401 Skillman, Suite 300 Dallas, TX 75231 **PHONE** 214–340-1500

ARCHITECT R & A Architects **CONTACT** Hy Applebaum

ADDRESS 10101 Fondren, Suite 554 Houston, TX 77096 **PHONE** 713–981-7315

SURVEYOR Bohannan–Huston, Inc. **CONTACT** Dwain Weaver

ADDRESS 4125 Carlisle Blvd., NE **PHONE** 881-2000

CONTRACTOR N/A **CONTACT**

ADDRESS **PHONE**

DATE SUBMITTED 9-1-83

BY David M. Y. Millikan

DRAINAGE REPORT
FOR
MOUNTAIN RUN SHOPPING CENTER

SEPTEMBER, 1983

PREPARED FOR

THE DAWN COMPANY
6401 SKILLMAN, SUITE 300
DALLAS, TEXAS 75231

PREPARED BY

BOHANNAN—HUSTON, INC.
4125 CARLISLE BOULEVARD, N.E.
ALBUQUERQUE, NEW MEXICO 87107
505/881-2000

DAVID M.Y. MILLIKAN
NEW MEXICO
7547
REGISTERED PROFESSIONAL ENGINEER

DAVID M.Y. MILLIKAN, P.E.
N.M.P.E. NO. 7547

TABLE OF CONTENTS

FIGURES

APPENDIX

PLATE

INTRODUCTION

The 10.73 acre Mountain Run Shopping Center will be built at the intersection of Eubank and Juan Tabo Blvds. in the City of Albuquerque, New Mexico. The planned construction site is undeveloped in its present state.

The purpose of this report is to describe existing drainage conditions on and adjacent to the site, and to present a grading and drainage plan which provides a workable means for treating all flows impacting or generated on the site.

SITE LOCATION AND DESCRIPTION

The site is located at the southeast corner of Eubank and Juan Tabo Blvds. and is a portion of Tracts I and H of the Academy Place Subdivision. In addition to this commercial site, a 15.85 acre apartment complex, 1.0 acre bank site and a 2.30 acre restaurant site will also be located on Tracts I and H. Figure 1 is a vicinity map of the area.

The site slopes at approximately 3½% from northeast to southwest. Soils are classified as Embudo—Tijeras association (Etc) and Tijeras series (TgB). Embudo—Tijeras is described as level to moderately sloping, well drained loamy and gravelly soils. Tijeras is described as nearly level to gently sloping gravelly fine sandy loam. Both soil groups are classified as hydrologic soil type "B" by the U.S. Soil Conservation Service. Figure 2 is a copy of the soils map for the area.

METHOD OF ANALYSIS

All methods of analysis and computations are done in accordance with Chapter 22 of the City of Albuquerque's Development Process Manual (DPM). Allowable discharge rates and volumes were determined in accordance with the "Drainage Ordinance".

For existing condition calculations, a runoff coefficient of .34 is used (type "B" soil, 0% impervious). A runoff coefficient of 0.86 is used assuming the site to be 90% impervious when developed.

Rainfall intensity calculations are based on a 6 hr. rainfall volume of 2.5 in., (as obtained from Plate 22.2 D–1 of the DPM). For existing 10-year and 100-year rainfall intensity calculations, a time of concentration of 25.0 min. is used. For all developed condition calculations, rainfall intensity is found assuming the time of concentration to be 10.0 min. Time of concentration calculations are included in the Appendix.

The Rational Formula has been applied to determine 10-year and 100-year peak flow rates and runoff volumes.

EXISTING DRAINAGE CONDITIONS

Besides flows generated on site, the only flows which impact the shopping center site are generated on the remainder of Tracts I and H. A drainage report has been submitted for the remainder of Tracts I and H. It was titled Drainage and Grading Plan for Mountain Run Apartments, August, 1983. The Flood Hazard Boundary Map (Figure 3) indicates that flows in Juan Tabo and Eubank are contained within the street right-of-way.

Under present conditions, all of Tracts I and H including the commercial site drain to the southwest corner of the apartment complex. Therefore, flow rate and runoff volume values have been computed for all of Tracts I and H. The portion attributable to the shopping center site can be found by proportioning its area to the remainder of the 29.88 acre tract. The 10-year peak flow rate for the site is 8.8 cfs. The 100-year peak flow rate is 13.4 cfs. The 10-year and 100-year runoff volumes are 0.542 ac.-ft. and 0.827 ac.-ft., respectively. Computations are included in the Appendix.

DEVELOPED DRAINAGE CONDITIONS

All flows impacting the shopping center site will be generated on site. Adequate flow capacity has been provided through the site should flows generated on the adjacent restaurant site require conveyance through the shopping center site. Provisions were also made in the drainage report for the apartment complex to accept runoff from restaurant site. Therefore, upon development, runoff from the restaurant can be discharged at an uncontrolled rate to either the apartments or the shopping center.

Runoff from this site will be discharged to Eubank Blvd. The flows will be conveyed within Eubank Blvd. to Layton Avenue. From this point the flows are conveyed in Layton Avenue to the Bear Tributary Arroyo. Page 3 of the Appendix shows calculations determining the capacity of Layton Ave. It has been assumed that all basins which drain to Layton Avenue under present conditions will drain to Layton Avenue under developed conditions. The boundaries of this basin are shown on Figure 3. The capacity of Layton just upstream of the Bear Tributary Arroyo was calculated as 260 cfs. The peak flow rate in Layton was determined using appropriate runoff coefficients for the contributing basins (see sheet 4 of Appendix). The combined peak flow rate is estimated to be 183.9 cfs. This is less than the capacity of Layton, therefore, uncontrolled discharge from the site is allowable.

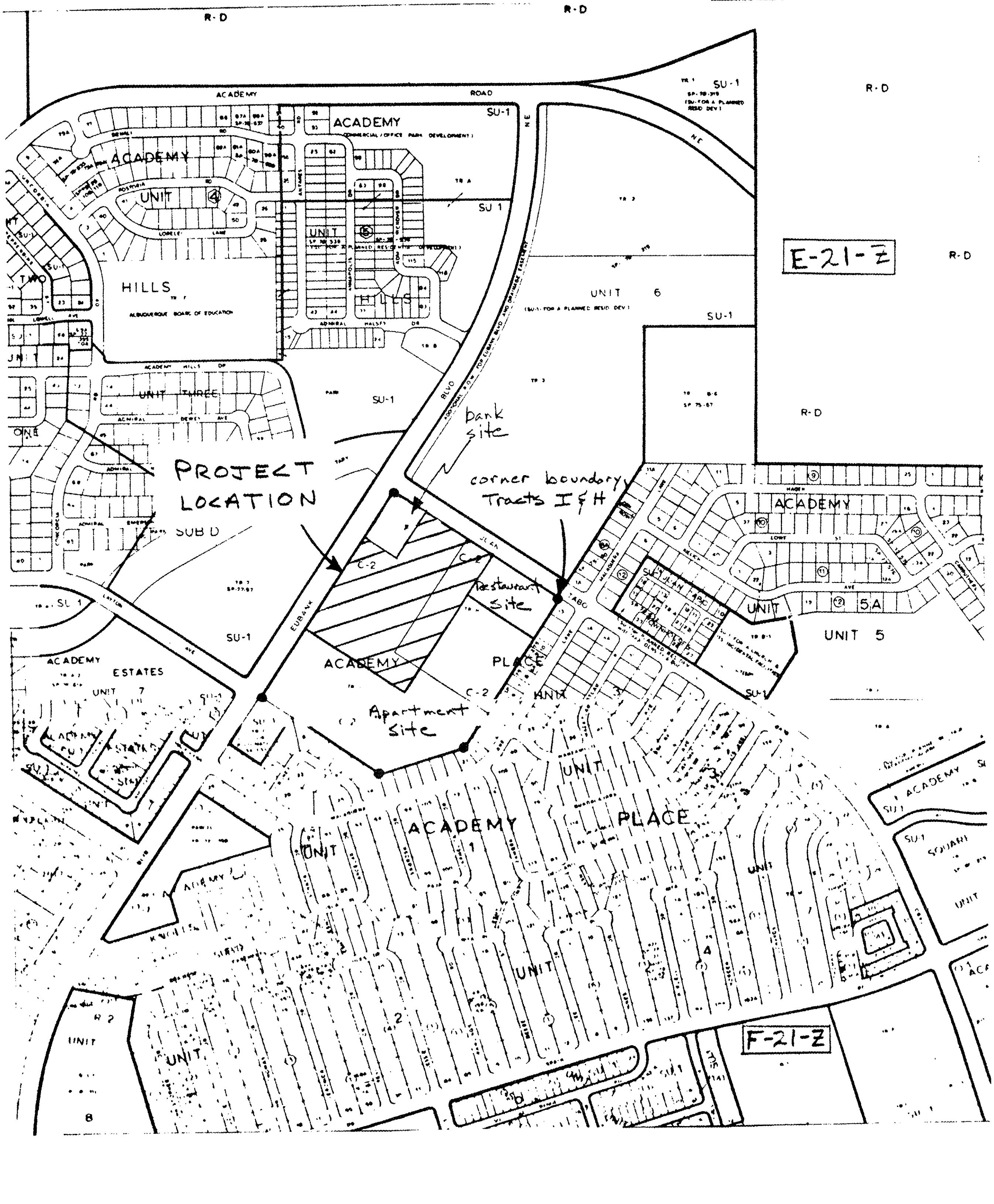
All runoff from the site will be discharged to Eubank at the southwest corner of the site. A small retention facility will be constructed between the most southwestern building on the site and Eubank to prevent nuisance flows from entering Eubank. This is in accordance with the "Drainage Ordinance". Flows which exceed the capacity of the small retention facility will be discharged over a turfed weir at the end of the facility. They will be conveyed down Eubank and Layton to the Bear Tributary Arroyo. Based on a 100-year storm, a peak flow rate of approximately 49 cfs will be discharged from the site.

The Drainage and Grading Plan (Plate 1) indicates the drainage basins, flow directions and flow quantities (100-year storm) corresponding to the developed grading of the site. Page 2 of the Appendix shows individual basin flow rates and runoff volume values.

EROSION CONTROL PLAN

Slopes on the site at locations where there are concentrated flows are moderate (1%–7%). During the construction phase, erosion in these areas may occur. Upon completion of the rough grading on the site, provisions should be made to ensure that all runoff from the site goes through the small retention pond. This facility should trap any sediment before it leaves the site.

Upon completion of the paving and landscaping no erosion should occur in these areas. Slopes in landscaped areas are 3:1 or flatter. No concentrated flows are directed over these slopes. Therefore, erosion upon completion of the project should not occur.



VICINITY MAP

FIGURE 1

TRACTS I & H, EXISTING DRAINAGE CONDITIONS

assume 0% imper. ($C = .34$, type "B" soil)
6-hr, 100-yr rainfall = 2.5"

Tracts I + H (total area):

$S = .033$ $A = 29.88$ ac

$V = 1.1$ ft/s $T_c = \frac{1650}{1.1(60)} = 25.0$ min

$I/6 = 1.35$ in/hr

$I_{10} = (2.5 \times .657)(1.35) = 2.2$ in/hr

$I_{100} = (2.5)(1.35) = 3.4$ in/hr

$\therefore$ $Q_{10} = (.34)(2.2)(29.88) = 22.6$ cfs

$Q_{100} = (.34)(3.4)(29.88) = 34.3$ cfs

$V_{10} = (.34)(2.5 \times .657)(29.88)/12 = 1.39$ ac-ft

$V_{100} = (.34)(2.5)(29.88)/12 = 2.12$ ac-ft

Proportioning by area:

shopping center (includes 1-ac bank site) $= \frac{11.73}{29.88} \times 100 = 39\%$

apartment site $= \frac{15.85}{29.88} \times 100 = 53\%$

restaurant site $= \frac{2.30}{29.88} \times 100 = 8\%$

$\therefore$

	shopping center	apartment	restaurant
Q_{10} (cfs)	8.8	12.0	1.8
Q_{100} (cfs)	13.4	18.2	2.8
V_{10} (ac-ft)	0.542	0.737	0.111
V_{100} (ac-ft)	0.827	1.124	0.170

PROJECT NAME Mtn. Run Shop. Ctr. SHEET 1 OF 4
PROJECT NO 31860 BY [illegible] DATE 8/22/83
SUBJECT Existing Drainage Calcs. CHD DATE

Developed Drainage Conditions

assume 90% impervious ($C=0.86$ for type "B" soil)
6-hr rainfall, 100-yr frequency = 2.5" (Plate 22.2 D-1 of DPM)

Basin A: (see Plate 1)

$S=.025 \therefore V=2.5$ ft/s (Plate 22.2 B-1)

$A=5.15$ ac

$T_c = \frac{L}{V(60)} = \frac{1180}{2.5(60)} = \cancel{7.81 \text{ min}}$ assume 10.0 min per LHM

$I/6 = 2.11$ in/hr (Plate 22.2 D-2)

$I_{10} = (2.11)(2.5 \times .657) = 3.5$ in/hr

$I_{100} = (2.11)(2.5) = 5.3$ in/hr

$Q_{10} = C I_{10} A = (0.86)(3.5)(5.15) = \underline{15.5 \text{ cfs}}$

$Q_{100} = C I_{100} A = (0.86)(5.3)(5.15) = \underline{23.5 \text{ cfs}}$

$\forall_{10} = (0.86)(2.5 \times .657)(5.15)/12 = \underline{0.60 \text{ ac-ft}}$

$\forall_{100} = (0.86)(2.5)(5.15)/12 = \underline{0.92 \text{ ac-ft}}$

Basin B: $A = 5.58$ ac

assume T_c also equals 10.0 min, $I_{10} = 3.5$, $I_{100} = 5.3$

$Q_{10} = (0.86)(3.5)(5.58) = \underline{16.8 \text{ cfs}}$

$Q_{100} = (0.86)(5.3)(5.58) = \underline{25.4 \text{ cfs}}$

$\forall_{10} = (0.86)(2.5 \times .657)(5.58)/12 = \underline{0.66 \text{ ac-ft}}$

$\forall_{100} = (0.86)(2.5)(5.58)/12 = \underline{1.00 \text{ ac-ft}}$

CAPACITY OF LAYTON AVE.

$$Q = \frac{1.486}{n} A R^{2/3} S^{1/2}$$

$n = .017$, $S = .0192$

$A_1 = \frac{1}{2}(.2)\left(\frac{.2}{\tan\theta}\right) = .780$

$A_2 = 30(.87 - .60) = 8.10$

$A_3 = \frac{1}{2}(.60)(30) = 9.00$

$17.88\ ft^2 \times 2 = 35.76\ ft^2 = A$

wetted perimeter

$P = [7.80 + .67 + 30.0]\,2$

$P = 76.94\ ft$

$\therefore R = A/P$

$R = \frac{35.76}{76.94} = .465$

$$Q = \frac{1.486}{.017}(35.76)(.465)^{2/3}(.0192)^{1/2}$$

$Q = 260\ cfs$

total street capacity

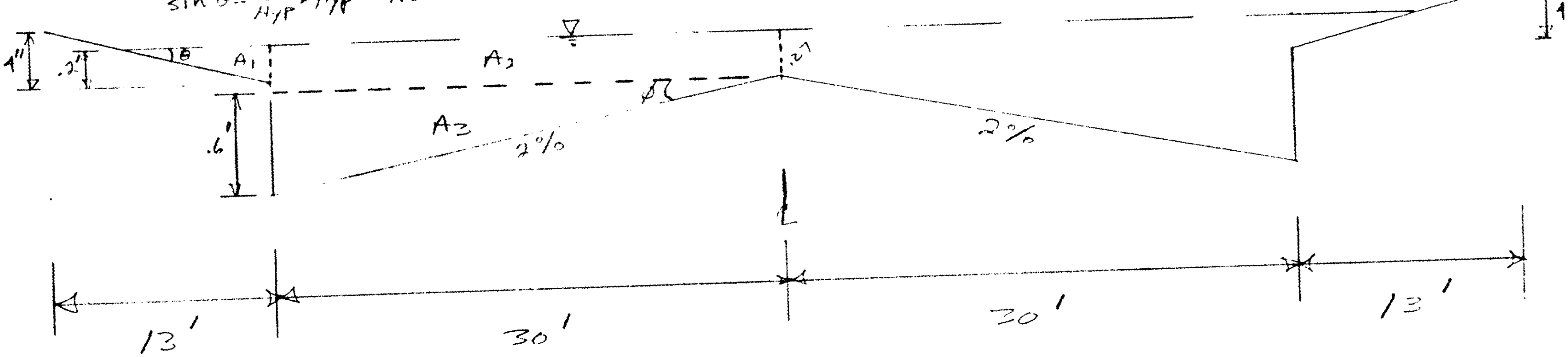
$\tan\alpha = \frac{.3333}{13}$

$\alpha = 1.4687 = \theta$

$\sin\theta = \frac{.2}{Hyp} \Rightarrow Hyp = 7.80'$

$\tan\phi = \frac{.6}{30} \Rightarrow \phi = 1.146$

$\sin\phi = \frac{.6}{Hyp} \Rightarrow Hyp = 30.0'$



86' R/W

Flows Impacting Layton Ave. vs. Capacity of Layton Ave.

Undevel. land bounded by Layton, Eubank and Bear Trib. -

$s = \frac{72.0 - 24.0}{1300} = .037$, $V = 1.2$

$T_c = \frac{1300}{1.2(60)} = 18.1$ $\quad I/6 = 1.65$

$I_{100} = 2.5(1.65) = 4.1$ $\quad Q_{100_{UD}} = (.34)(4.1)(9.90) = \underline{13.8\ cfs}$

assume developed: $c = 0.82$, $T_c = 10.0$ min $\quad Q_{100_D} = (.82)(5.3)(9.90) = \underline{\underline{43.0\ cfs}}$

On Eubank -

$s = \frac{78.5 - 42.7}{1200} = .030$ $\quad V = 4.6$

$T_c = \cancel{4.35} \rightarrow 10.0$ $\quad I/6 = 2.11$

$I_{100} = 2.5(2.11) = 5.3$ $\quad Q_{100} = (1.0)(5.3)(1.653) = \underline{8.8\ cfs}$

Shopping area (includes adjacent bank site) -

$Q_{100_{UD}} = \underline{13.4\ cfs}$ (see pg. 1 of Appendix)

assume developed: $c = 0.82$, $T_c = 10.0$ min

$Q_{100_D} = (.86)(5.3)(11.73) = \underline{\underline{53.5\ cfs}}$

Apartment site (from drainage report) -

$Q_{100_D} = \underline{\underline{78.61\ cfs}}$

$\therefore Q_{100\,(total)}$ when developed $= 183.9 < 260$ cfs ✓

therefore adequate capacity in Layton for uncontrolled discharge from all contributing basins.

PROJECT NAME		SHEET 4	OF 4
PROJECT NO. 31860		BY [illegible]	DATE 8/83
SUBJECT Flows Impacting Layton Ave.		CH'D	DATE

November 29, 1983

Mr. Billy Goolsby
Civil Engineer - Hydrology
City of Albuquerque
P.O. Box 1293
Albuquerque, NM 87103

Re: Drainage and Grading Plan for Mountain Run Shopping Center (E21 – D23)

Dear Billy:

In order to ensure that the discharge from this site conforms to the requirements agreed upon (as outlined in my letter to you dated November 14, 1983), we have performed a more detailed analysis of the flows leaving this site after development.

In order to ensure compliance with the criteria outlined in my letter dated November 14, 1983, we propose the following method of discharging runoff from this site. The allowable peak discharge rate due to a 10-year storm is 17.2 cfs (see letter dated October 4, 1983).

The uncontrolled discharge rate from the site due to a 10-year storm is:

a) 15.6 cfs from the upper basin (Basin A)
b) 16.8 cfs from the lower basin (Basin B)

We propose rooftop detention within Basin B that will reduce the peak discharge rate to 12.0 cfs from this basin due to a 10-year storm. We also propose to limit the 10-year peak discharge rate from Basin A to 5.2 cfs. These controlled discharge rates result in on-site detention requirements as follows:

a) 5,112 cu.ft. on roofs in Basin B
b) 11,326 cu.ft. in pond in Basin A

Roof top ponding will be accomplished by designing the rundowns so that a limited number of rundowns operate during runoff from a 10-year storm. Additional rundowns will be provided which will only function when the volume of water on the roof exceeds the required detention volume of 5112 cu.ft.

Runoff from Basin A will be routed through a detention pond which will limit the discharge to 5.2 cfs until the required detention volume of 11,326 cu.ft. is exceeded. Enclosed is a sketch showing the proposed pond in Basin A. All runoff from this basin will be collected in a storm drain inlet within the parking lot and conveyed to the pond. The storm drain will be sized to accept the flows generated by a 100-year storm. The 5.2 cfs will be discharged through an appropriately sized opening in the block retaining wall. The opening will discharge through a sidewalk culvert (as being used for the Mountain Run Apartments) to Eubank Boulevard. The overflow will discharge through three sidewalk culverts as shown on the enclosed sketch. Runoff entering Eubank at these locations will meet the criteria outlined in the November 14, 1983 letter. Since the depth within the pond exceeds 18", a 30" high wall will be placed along the public sidewalk along Eubank Boulevard. The pond is designed to retain nuisance flows.

Runoff in Basin B will be discharged through the drivepad at the southwest corner of the site. In order to prevent nuisance flows from entering Eubank, a trench drain will be installed to intercept these flows. The trench drain will discharge these flows into a small retention pond immediately south of this driveway. When the volume of the retention pond is exceeded, the runoff intercepted by the trench drain will be redirected back into the driveway. The trench drain has been designed to intercept nuisance flows only. This will prevent runoff from storms being unnecessarily circulated through the retention pond.

We are enclosing computation sheets which show the method used in determining detention pond volumes. Please review these along with the sketch of the detention pond in Basin A and the retention pond in Basin B.

A detailed grading plan will be submitted to you for approval prior to issuance of a building permit. The sidewalk culverts will be constructed along with the median improvements to Eubank Boulevard.

If you have any questions or comments, please contact Dave Millikan or me. We are proceeding with the assumption that the methods outlined in this report are acceptable.

Sincerely yours,

Michial M. Emery, P.E.
Vice President

Enclosures

cc: Mr. Bob Spooner

DMYM/rms
Job No. 31660

DRAINAGE INFORMATION

UNDEVELOPED

C = .35 AREA = 1.0 acre I_{100} = 5.3 inches Q_{100} = 1.8 cfs

DEVELOPED

BASIN A

AREA = 0.64 acres Q_{100} = 3.2 cfs

BASIN B

AREA = 0.36 acres Q_{100} = 1.8 cfs

REVISIONS

NO.	DATE	REMARKS	B

PREPARED UNDER THE DIRECTION OF

APPROVED FOR ROUGH GRADING

CITY ENGINEER DATE

Job No. 31868

SCALE 1" = 30'

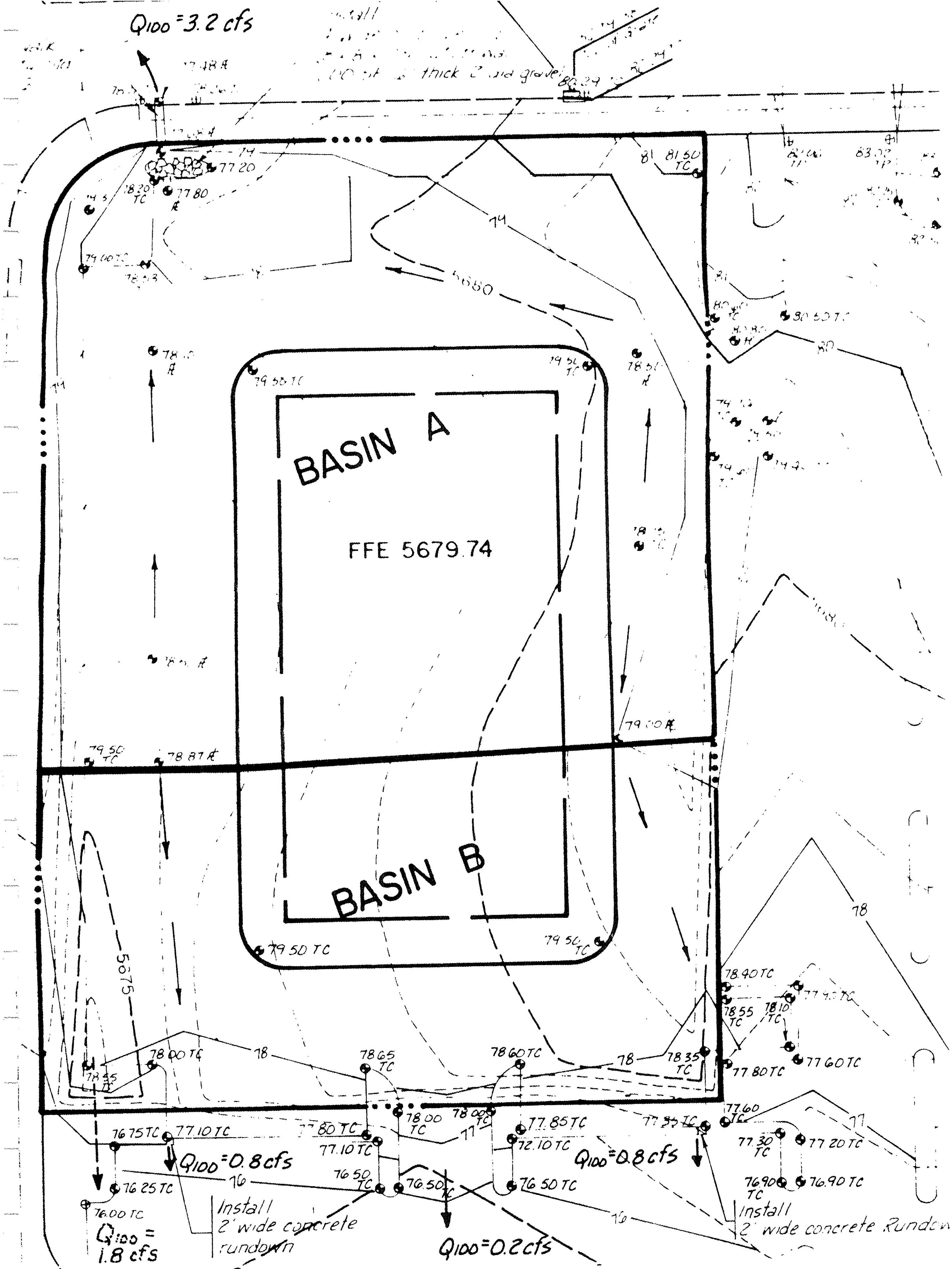
CITY OF ALBUQUERQUE
MUNICIPAL DEVELOPMENT DEPARTMENT
ENGINEERING DIVISION

TITLE: **MOUNTAIN RUN PARCEL III**
DRAINAGE & GRADING PLAN

APPROVALS	ENGINEER	DATE	APPROVALS	ENGIN
City Engineer			W.R.D.	
A.C.E.-Design			Traffic	
A.C.E.-Hydrology				

DRAWING NO.

SHEET 4



Q100=3.2 cfs
BASIN A
FFE 5679.74
BASIN B
5680
5675
79.50 TC
78.87 ft
79.00 ft
78.40 TC
78.55 TC
78.10 TC
77.60 TC
77.80 TC
78.35 TC
78.60 TC
78.65 TC
78.00 TC
78.55 TC
76.75 TC
77.10 TC
76.25 TC
76.00 TC
77.80 TC
77.10 TC
76.50 TC
76.50 TC
78.00 TC
78.00 TC
77.85 TC
72.10 TC
76.50 TC
77.35 TC
77.60 TC
77.30 TC
77.20 TC
76.90 TC
76.90 TC
Q100=0.8 cfs
Q100=0.8 cfs
Q100 = 1.8 cfs
Q100=0.2 cfs
Install 2' wide concrete rundown
Install 2' wide concrete Rundown
77.20
77.80 ft
81.50 TC

EES — WILSON & COMPANY — Albuquerque — 85-?? — Spring Park — 1 — 2nd Rev Dr. Report

30 April 1985

Undeveloped Flow Condition

6 hr - 100 yr rainfall = 2.5 in

Area = 13.23 Ac

C = 0.40

t_c = 10 min

i_{100} = 2.5 x 2.2 = 5.50

Q = CIA = 0.40 x 5.50 x 13.23 = 29.11 cfs

6 hr Volume = 2.5/12 x 13.23 x 43,560 x .40 = 48,025 c.f.

Developed Conditions

% Imper. = 286,920 / 576,300 = 49.7%

Hydrologic Soil Type = B

From Plate 22.2 C-1 C = 0.58

Area 1 Discharge to Eubank

Area = 1.15 Ac C = 0.86 (90% Impervious)

i = 5.50

Q_{100} = CiA = 0.86 x 5.50 x 1.15 = 5.44 cfs

Area 2 Discharge to Layton

Area = 10.62

C = 0.58

i = 5.50

Q_{100} = CiA = 0.58 x 5.50 x 10.62 = 33.9 cfs.

Capacity Layton

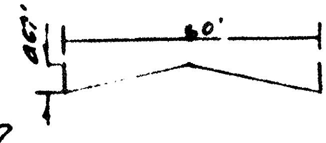
℄ Street Top Curb

Area = 60 x .67 x .5 = 20.10 ft²

WP = 61.33 ft

R = Hyd Rad = A/WP = 20.1/61.33 = .3277

Slope = 4.12% n = 0.017



Qcap (Curb Full) = AV = A $\frac{1.486}{n} R^{2/3} S^{1/2}$

= 20.1 x 87.41 x .4753 x .2030

Q = 169.5 cfs

Max rate from project = 33.9 cfs < 169.5 cfs

By RES
Ck
Date 17 Mar 85

WILSON & COMPANY ENGINEERS ARCHITECTS

Loc Albuquerque
Proj Spring Park
Rev. Dr. Plan
Job 83-518
Sheet 1

Undeveloped Flow Conditions
6hr-100yr rainfall = 2.5in
Area = 13.23Ac
C = 0.40
t_c = 10min
i_{100} = 2.5 x 2.2 = 5.50
Q = CiA = 0.40 x 5.50 x 13.23 = 29.11 cfs

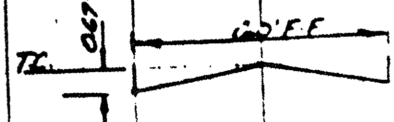
6hr Volume $\frac{2.5}{12}$ x 13.23 x 43,560 x .40 = 48,025 cf

Developed Condition
% Impv $\frac{286,280}{576,300}$ = 49.7%
From Plate 22.2 C1 C = 0.58

Area 1 Area = 0.72 Ac t_c < 10min i_{100} = 5.50 C = 0.76 (80% Impv)
Q_{100} = CiA = 0.76 x 5.50 x 0.72 = 3.0 cfs
Use Single C Inlet w/18" Outlet pipe

Area 2 Area = 1.02Ac t_c < 10min i_{100} = 5.50 C = 0.69 (70% Impv)
Q_{100} = CiA = 0.69 x 5.50 x 1.02 = 3.87 cfs
Use Single C Inlet w/18" Outlet pipe

Area 3 Area = 8.88 Ac t_c < 10min i_{100} = 5.50 C = 0.58 (50% Impv)
Q_{100} = CiA = 0.58 x 5.50 x 8.88 = 28.33 cfs
Drainages to Layton Ave
Capacity Layton Ave
Street width = 60
℄ St Elev = Top Curb Elev
A = $\frac{30 \times .67}{2}$ x 2 = 20.10 Sq Ft



Assume
℄ St elev = Top curb elev.

Area $\frac{30 \times .67}{2} \times 2 = 20.10$ Sq Ft.
Wet Per. = 61.33 Ft
Hyd Rad = A/WP = 20.10/61.33 = .3277

Slope = 4.12%   n = 0.017

$Q_{cap} = AV = A \frac{1.486}{n} \times R^{2/3} \times S^{1/2}$

$= 20.01 \times \frac{1.486}{.017} \times .3277^{2/3} \times .0412^{1/2}$

$= 20.01 \times 87.41 \times .4753 \times .2030$

Q_{cap} = 168.7 cfs

Vel = 8.43 ft/sec

Vel × depth = 8.43 × 0.67 = 5.65 < 6.5 max allowed

The max rate of discharge from Area 3 is 283.3 cfs
& capacity at Ceylon Ave is 168.7 cfs.

Area 4 discharges to Eubank Blvd
A = 1.15 Ac   C = 0.86 (90% Impervious)
$i = 5.50$
$Q_{100} = CiA = 0.86 \times 5.50 \times 1.15 = 5.44$ cfs

Drainage and Sedimentation Control

The proposed project is currently planned to be developed on a 13.23 acre parcel(Zone Map E21) of Tract 7 of the Academy Hills Subdivision. The land is bordered by the Bear Canyon Tributary Arroyo(Developed Channel) on the northwest, Eubank Ave. on the east and Layton Ave. on the south. In the current plan, surface waters are conducted to the Bear Canyon Tributary Arroyo and Eubank Ave. in the northern section of the property, and to Layton Ave. on the southern section of the property. Surface waters conducted to Layton Ave. are conducted to the intersection of Layton Ave. and the arroyo just offsite.

On October 7, 1983, a meeting was held between the chief municipal hydrologist, Mr. F. Aquirre, and a representative of Wilson and Company Engineers and Architects who have been retained by Pacific Realty Corporation. In this meeting, it was decided that there should be free drainage to the arroyo if sufficient downstream capacity exists. In addition, a concern was raised regarding bank protection, and the need for further evaluation of such protection.

On December 13,1983, the Engineers Drainage Report was filed with the City for the proposed project. The report consisted of one page and two full size drawings (one depicting the project, and the other depicting surface water hydrology and the surface water drainage system). The report concluded "that there are no off site drainage areas contributing flows onto the project site since flows are intercepted by Eubank Ave. on the east and the Bear Canyon Tributary Arroyo on the north and west. The report utilized the Rational Formula(Clark and Viessman,1970) for calculating surface water runoff for three drainage areas, and in the case of Layton Ave. compared runoff to street capacity flow.

On December 15, 1983, the environmental planning commission held a meeting on the proposed Spring Hill Apartment Project(Case# Z-75-71-1) and approved the project for 300 separate units. This approval was granted prior to approval by the Chief Design Hydrologist, Mr. F. A. Aquirre who I understand has set a date of December 28, 1983 for final approval of the drainage report, and in the face of issues raised on certain hydrological aspects of the project. Mr. John B. Case, a resident in the Academy Hills subdivision, and a professional engineer (NM No. 8049) had sent a letter to members of the commission stating a concern that the proposed project plan does not address flood control issues with regard to the adjacent Bear Canyon Tributary Arroyo nor potential erosion and sedimentation problems that might be created on and off site.

It is respectively requested that the city council considers the following aspects of the project:

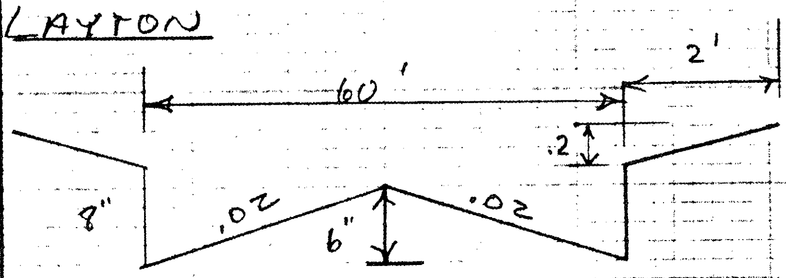
o Flood Control - It is requested that the drainage plan refer to documentation that supports the contention that the proposed project is outside the design flood plain. Such documentation should include calculations (flood hydrograph analysis, or evaluation of existing channel crossections) to substantiate the, at present, unsupported conclusion regarding off site surface runoff and downstream capacity presented in the drainage report.

o Sediment Control - The plan should provide for onsite storage of surface water runoff, and sediment retention by construction of several surface water retention ponds. Such ponds should provide onsite erosion control, and avoid offsite damage to adjacent streets or walkways.

These issues are fundamentally important in site planning and selection. As stated in Section 2.11 of Site Planning Standards (Dechira and Koppelman,1978) a development plan should demonstrate "Freedom from Surface Floods, and Suitability for Siting of Projected Buildings." Provision should be made for the best availiable routing of runoff water to assure that buildings or other important facilities will not be endangered by a major emergency flood runoff that would become active if the capacity of the storm drainage system were exceeded. Drainage swales should not carry runoff or sediment across walks or streets in quantities that will make them undesirable to use.

STREET CAPACITIES

LAYTON

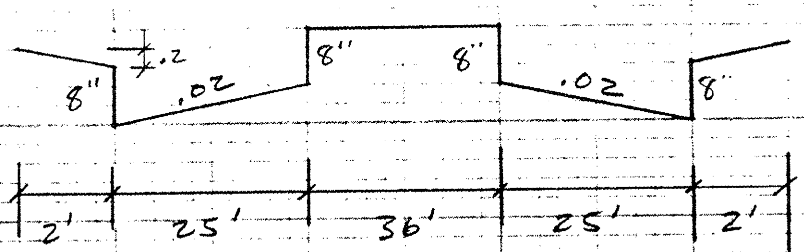


Roughness 'n' = .017

SLOPE = 0.018

DIST.	ELEV.
0	.87
2	.67
2.1	0
32	.5
62	0
62.1	.67
64	.87

EUBANK



Roughness 'n' = .017

SLOPES VARY:

$S_1 = .03233$

$S_2 = .0122$

DIST.	ELEV
-2	1.07
0	.87
2	.67
2.1	0
27	.5
27.1	1.17
63	1.17
63.1	.5
88	0
88.1	.67
90	.87
92	1.07

BOHANNAN-HUSTON INC.

PROJECT NAME ______ SHEET 1 OF 4

PROJECT NO. ______ BY BG DATE 6-22-89

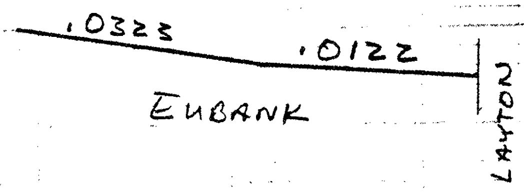
SUBJECT ______ CH'D ______ DATE ______

STREET CAPACITIES

POINT 1 EUBANK - MAJOR ARTERIAL

FIGURE 1 — $Q_{10} = 17.2$ cfs , $Q_{100} = 49.0$ cfs

All of this flow is in east half of Eubank. The street slopes are:



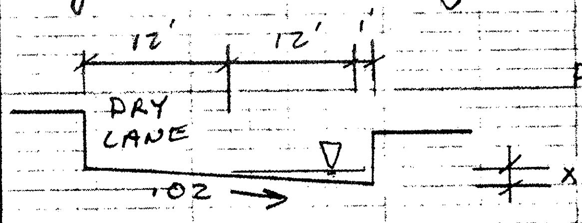
City Criteria: 10-YEAR DEPTH = Top of Curb
100-YEAR DEPTH = ToC + .2'

Curb HT = 8" = .67'

TABLE 1 — Q_{10} ALLOWABLE = 87 cfs for SL = .0323
TABLE 2 — = 55 cfs for SL = .0122
55 cfs > 17.2 OK

TABLE 1 — Q_{100} ALLOW = 169 for SL = .0323
TABLE 2 — = 104 for SL = .0323
104 > 49 cfs OK

City Criteria: 1 dry lane during 10-YR STORM

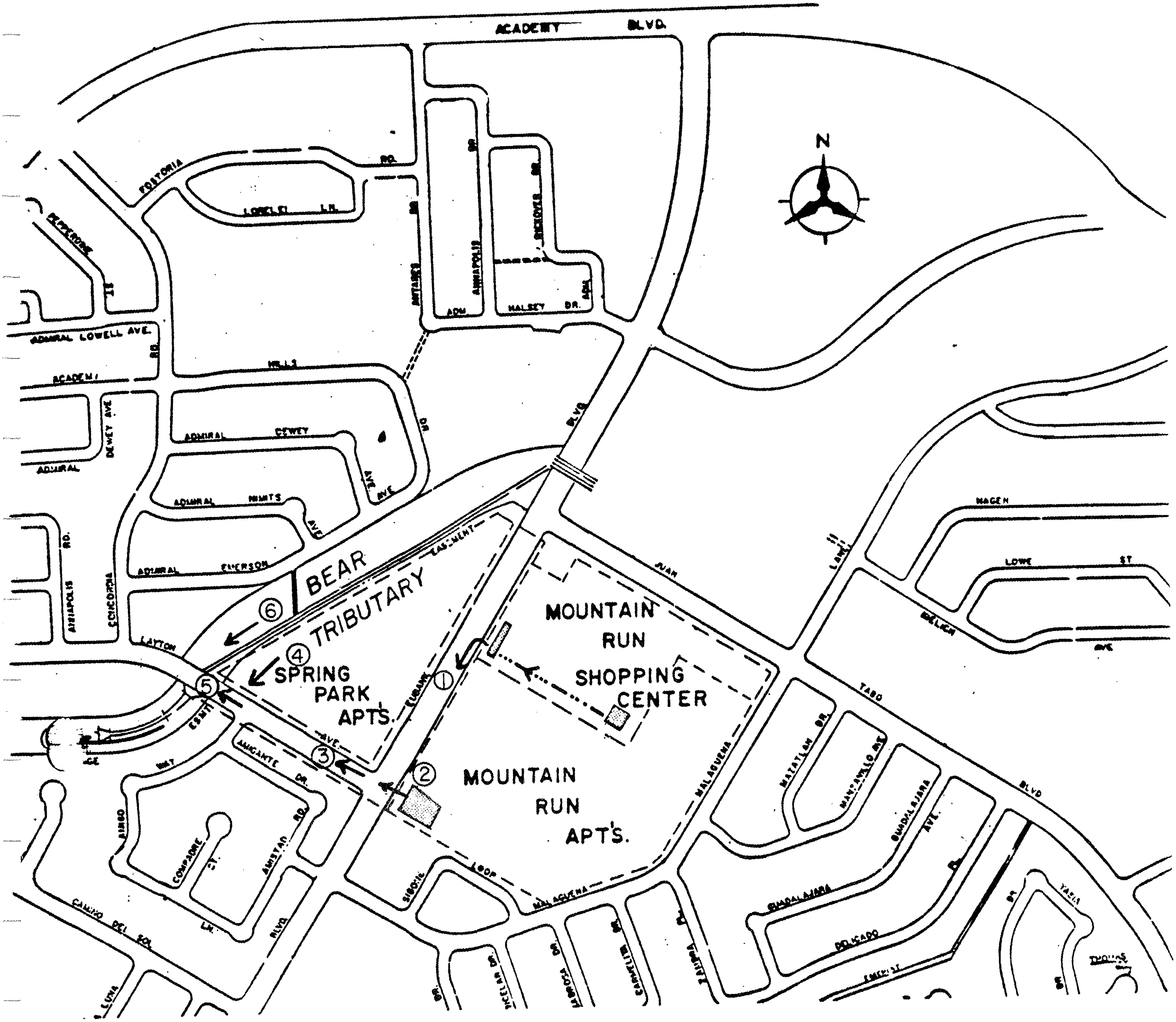


Flow Depth X = 13' x .02 = .26

TABLE 1 — Q_{10} ALL @ d = 0.26' = 7.3 cfs SL = .0323
for SL = .0122 = 4.5 cfs < 17.2 cfs
INSTALL CATCH BASINS JUST BELOW
CATCH BASINS TO CATCH 17.2 - 4.5 = 12.7 cfs

BOHANNAN-HUSTON INC.

PROJECT NAME ______ SHEET 2 OF 4
PROJECT NO. ______ BY BG DATE 6-26-89
SUBJECT ______ CH'D ______ DATE ______



DISCHARGES in CUBIC FT. per SEC.

	10-YEAR	100-YEAR
1	17.2	49.0
2	28.0	78.6
3	64.6	146.7
4	21.0	43.0
5	85.6	189.7
6	480.0	992.0
7	575.0	1196.0

LEGEND

DIRECTION of FLOW

DRAINAGE BASIN BOUNDARIES

DETENTION POND

LATON AVE. / BEAR TRIBUTARY CROSSING HYDROLOG

FIGURE 1

TABLE 1

STREET FLOW CAPACITY FOR HALF OF ELBANK NORTH OF LAYTON

MANNING'S N = .0170 SLOPE = .0323

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	-0.20	1.07	3	2.00	0.67	5	27.00	0.50
2	0.00	0.87	4	2.10	0.00	6	27.10	1.17

WSEL (FT)	DEPTH INC	FLOW AREA (SQ FT)	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOP WID
0.1	0.1	0.2	0.5	5.1	2.1	5.0
0.2	0.2	1.0	3.3	10.2	3.3	10.0
0.3	0.3	2.2	9.9	15.2	4.4	15.0
0.4	0.4	4.0	21.2	20.3	5.3	20.0
0.5	0.5	6.2	38.5	25.4	6.2	25.0
0.6	0.6	8.7	67.1	25.6	7.7	25.0
0.7	0.7	11.2	100.9	26.1	9.0	25.3
0.8	0.8	13.8	138.5	27.2	10.0	26.3
0.9	0.9	16.5	182.3	28.0	11.0	27.1
1.0	1.0	19.2	233.6	28.3	12.1	27.2
1.1	1.1	21.1	272.4	28.5	12.9	27.3

TABLE 2

STREET FLOW CAPACITY FOR HALF OF ELBANK NORTH OF LAYTON

MANNING'S N = .0170 SLOPE = .0122

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	-0.20	1.07	3	2.00	0.67	5	27.00	0.50
2	0.00	0.87	4	2.10	0.00	6	27.10	1.17

WSEL (FT)	DEPTH INC	FLOW AREA (SQ FT)	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOP WID
0.1	0.1	0.2	0.3	5.1	1.3	5.0
0.2	0.2	1.0	2.1	10.2	2.1	10.0
0.3	0.3	2.2	6.1	15.2	2.7	15.0
0.4	0.4	4.0	13.0	20.3	3.3	20.0
0.5	0.5	6.2	23.6	25.4	3.8	25.0
0.6	0.6	8.7	41.2	25.6	4.7	25.0
0.7	0.7	11.2	62.0	26.1	5.5	25.3
0.8	0.8	13.8	85.1	27.2	6.2	26.3
0.9	0.9	16.5	112.0	28.0	6.8	27.1
1.0	1.0	19.2	143.5	28.3	7.5	27.2
1.1	1.1	21.1	167.4	28.5	7.9	27.3

STREET CAPACITIES

POINT 3 LAYTON - MINOR ARTERIAL

FIGURE 1	$Q_{10} = 64.6$ cfs -
FIGURE 1	$Q_{100} = 146.7$ cfs
	STREET SL = .018
	10-YR CAPACITY → d = .67'
TABLE 3	Q_{10} ALLOW = 164 cfs >
	ONE DRY LANE: X = 18 x .02 = .36'

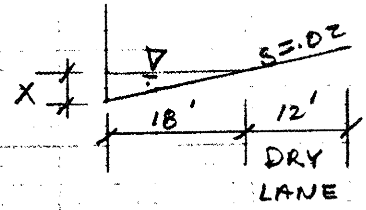


TABLE 3	Q_{10} at d = .36 = 30.0 cfs
	LIMIT 10-YR STREET FLOW TO 30 cfs
	100-YR CAPACITY → d = .87'
	Q_{100} ALLOW = 290 cfs > 146.7 OK

TABLE 3

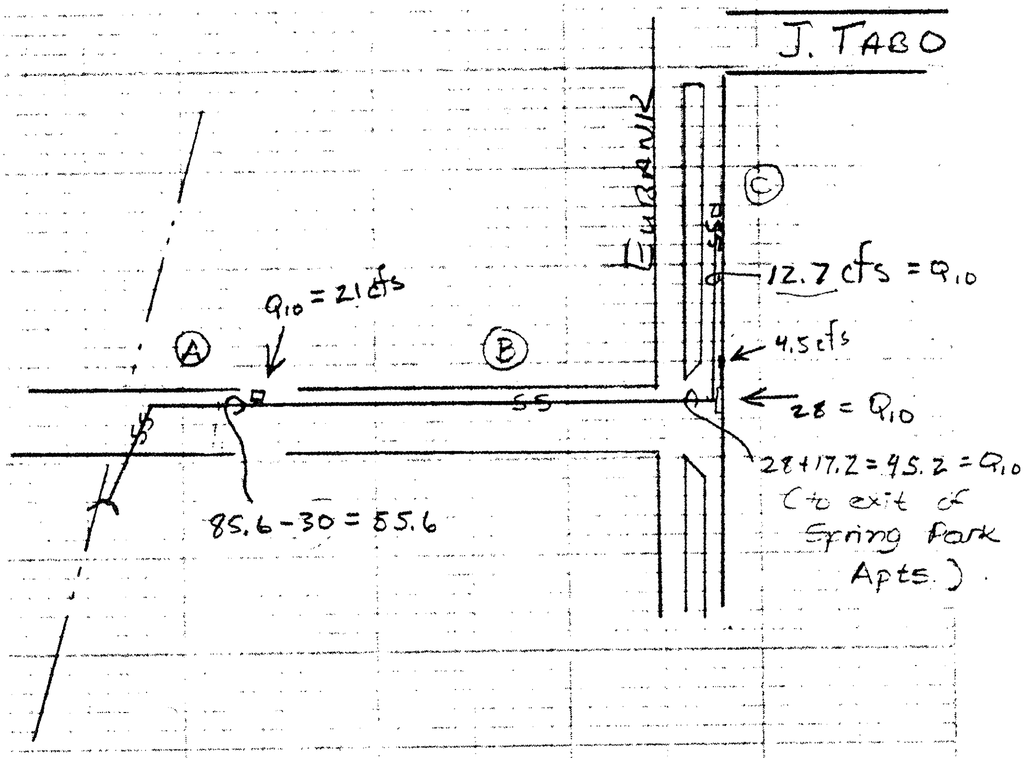
STREET FLOW CAPACITY FOR LAYTON WEST OF EUBANK

MANNING'S N = .0170 SLOPE = .0180

POINT	DIST	ELEV	POINT	DIST	ELEV	POINT	DIST	ELEV
1	0.00	0.87	4	32.00	0.50	7	64.00	0.87
2	2.00	0.67	5	62.00	0.00			
3	2.10	0.00	6	62.10	0.67			

WSEL (FT)	DEPTH INC	FLOW AREA (SQ FT)	FLOW RATE (CFS)	WETTED PER (FT)	FLOW VEL (FPS)	TOP WID
0.1	0.1	0.6	0.9	12.2	1.6	12.0
0.2	0.2	2.4	6.0	24.4	2.5	24.0
0.3	0.3	5.4	17.7	36.6	3.3	36.0
0.4	0.4	9.6	38.2	48.7	4.0	48.0
0.5	0.5	15.0	69.2	60.9	4.6	60.0
0.6	0.6	21.0	121.0	61.1	5.8	60.1
0.7	0.7	27.0	182.6	61.9	6.8	60.7
0.8	0.8	33.2	251.9	63.8	7.6	62.6
0.9	0.9	37.6	306.0	65.2	8.1	64.0

Summary of St. Sewer Flows



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PAPA ~~BEAR~~

Ⓒ $Q = 12.7$ cfs
$n = 0.013$

φ	S	Max Q	Full Q
21"	3%	29.5	27.4
18"	2%	16.0	
15"	2%	9.8	9.1
	3%	12.0	11.2
	3.5%	13.0	12.1

Ⓐ : $Q = 55.6$ cfs
$n = 0.013$

φ	S	Max Q	Full Q	
36"	1%	71.7	66.7	
	0.5%	50.7	47.2	
	0.65%	57.8	53.8	Min.

φ	S	Max Q	Full Q	
24"	5%	54.25	50.6	
	6%	59.42	55.4	
	6.1%	59.9	55.9	Min. Slope
30"	0.5%	31.2	29.0	
	0.6%	34.2	31.8	
	2.5%	69.7	64.9	
	1 %	44.1	41.02	
	1.5%	54.03	50.2	
	1.6%	55.80	51.88	Min Slope

Ⓑ : $Q = 45.2$ cfs
$n = 0.013$

φ	S	Max Q	Full Q	
24"	2%	34.3	32.0	
	2.5%	38.4	35.8	
	3.0%	42.0	39.2	Min Slope
30"	1%	44.1	41.0	Min Slope
	2%		58	

BOHANNAN-HUSTON INC.

PROJECT NAME ______ SHEET 4 OF 4
PROJECT NO. ______ BY OA DATE 6-28-89
SUBJECT ______ CH'D BG DATE 6-28-89

PLATE 1 SINGLE PLAN - PROFILE - DOTTED
CHARLES BRUNING COMPANY
MADE IN U.S.A.