ALBUQUERQUE NEW MEXICO 1706

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

August 6, 1992

Dennis Lorenz, P.E.
Brasher Engineering
11930 Menaul Boulevard, NE #111
Albuquerque, New Mexico 87112

RE: REVISED DRAINAGE PLAN FOR VENTURA ESTATES (D-20/D7)
ENGINEER'S STAMP DATED JULY 1, 1992

Dear Mr. Lorenz:

Based on the information provided on the revised plan which proposes to add a cul-de-sac at the east end of the project is acceptable for Final Plat. The plan was received on July 8, 1992.

Be advised that financial guarantees must be in place prior to the City Engineer signing-off on the plat.

If you should have any questions, please do not hesitate to call me at 768-2650.

Cordially,

Gilbert Aldaz

Gilbert Aldaz, P.E. P.S.
Civil Engineer/Hydrology

GA
wp+3338

May 8, 1992

Dennis Lorenz, P.E.
Brasher Engineering
11930 Menaul Boulevard, NE #111
Albuquerque, New Mexico 87112

RE: REVISED ROUGH GRADING PLAN FOR VENTURA ESTATES (D-20/D7)
ENGINEER'S STAMP DATED APRIL 1, 1992

Dear Mr. Lorenz:

Based on the information provided on the revised grading plan for the referenced subdivision received April 14, 1992, the plan is acceptable. In the future, could you please identify the grading changes.

If you should have any questions, please do not hesitate to call me at 768-2650.

Cordially,

Gilbert Aldaz

Gilbert Aldaz, P.E./P.S.
Civil Engineer/Hydrology

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DRAINAGE REPORT

FOR

VENTURA ESTATES

Prepared for:

B & R Land Developers, Inc.
P.O. Box 20100
Albuquerque, New Mexico 87109

Prepared By:

Brasher Engineering, Inc.
11930 Menaul N.E. Suite 113
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DENNIS A. LORENZ
NEW MEXICO
8647
REGISTERED PROFESSIONAL ENGINEER

7-1-92

UPDATED
July 1992

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LIST OF FIGURES AND EXHIBITS

PURPOSE AND SCOPE

The purpose of this Drainage Report is to establish the criteria for controlling surface storm runoff from **VENTURA ESTATES** and contributing upstream drainage areas in a manner which is acceptable to the City of Albuquerque and the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA). The report studies the existing and developed conditions of **VENTURA ESTATES** and analyzes both conditions at the 100-year and 10-year/6-hour duration storm events. The report outlines the drainage management criteria for the complete development of **VENTURA ESTATES**.

The scope of this report is to ensure that the **VENTURA ESTATES** project will be protected from storm runoff and that the development of this project will not increase the flooding potential of adjacent and downstream properties.

UPDATE NO.1

This drainage report is updated to identify the grading and drainage improvements required to construct a cul-de-sac at the east end of Redmont St. N.E. Approval of this report will facilitate plat approval for Lot 11A and 12A, Block B, and Lot 11A and 12A Block C, Ventura Estates, unit one, DRB 92-220.

SITE LOCATION AND DESCRIPTION

VENTURA ESTATES is located in northeast Albuquerque in North Albuquerque Acres (see Vicinity Map, Figure 1). **VENTURA ESTATES** is bounded by Ventura Street on the west; Holbrook Street on the east; San Antonio right-of-way (to be vacated) and Tanoan properties on the south; and by Heritage East on the north. **VENTURA ESTATES** is presently described as Ventura Estates, Blocks A, B, C and D, contains approximately 16.1 acres (including the vacated portion of San Antonio).

The project site is presently undeveloped and covered with native vegetation. Site topography slopes from east to west at approximately 3 percent. Small natural drainage swales cross the site which is typical for lands located in North Albuquerque Acres. The North Arroyo del Pino is located adjacent and north of the project site and serves as the major drainage outfall for the area. The arroyo is concrete lined which allows for development up to the right-of-way line. As shown by the Flood Hazard Zone Map for the area, **VENTURA ESTATES** does not lie within a designated Flood Hazard Zone (see Figure 3).

On-site soils consist mainly of Embudo soils (see Figure 2), which are classified as type "B" soils by the Soil Conservation Service. Embudo soils are generally deep, well drained soils formed in alluvium derived from decomposed, coarse-grained granite rocks. They exhibit medium runoff characteristics with moderate erosion.

EXISTING DRAINAGE CONDITIONS

The project site is presently undeveloped. On-site flows drain westerly to improved downstream drainage facilities. The eastern one-third of the site drains to de Vargas Loop which outfalls at the improved North Arroyo del Pino. The remainder of the site drains westerly into Ventura Street which also conveys flows to the North Arroyo del Pino. A small off-site basin impacts the site on the east at the Holbrook interface. Approximately 3.8 acres of undeveloped land drains into the site from an area east of Holbrook. This off-site flow merges with on-site flows and discharges into de Vargas Loop as discussed above. Off-site flows from the south drain within the 150' PNM easement along with flows from Wimbledon West and Fairways North. These flows are collected in a sedimentation basin adjacent to Ventura Street and are then piped in a 42-inch storm drain to the North Arroyo del Pino. No flows enter the site from the north, south, or west. Figure 4 is provided to illustrate how the off-site drainage basins affect this project site.

DEVELOPED DRAINAGE CONDITIONS

As shown by the Drainage and Grading Plan for **VENTURA ESTATES** (see Sheets 1 & 2 in back pocket) all on-site runoff will be conveyed by residential streets to the improved outfall points. The site will utilize two improved outfall points. Basins "F" and "G" will discharge into de Vargas Loop at analysis point 7. As outlined in the "Final Drainage Report, Heritage East, Unit III, Near San Antonio Avenue and Holbrook Street, Albuquerque, New Mexico", prepared by Scanlon & Associates, Inc., dated June 18, 1984, de Vargas Loop was designed to carry off-site flows from a portion of **VENTURA ESTATES**. As shown by the street depth calculations (see Appendix, page 6), sufficient capacity does exist within de Vargas Loop to accommodate these developed basins. De Vargas Loop discharges developed flows into the North Arroyo del Pino at an existing crossing structure.

The existing crossing structure at De Vargas contains 4 - 18 inch sidewalk culverts with no overflow weir. These culverts were apparently intended to drain the anticipated 100-year design flow. However, this is not the case. By our estimation, existing street flows are not being drained adequately and damage to private property by flooding is likely to occur. Therefore, since Ventura Estates drains a small basin to the bridge, we recommend construction of a drainage rundown to the North Arroyo del Pino just east of the existing bridge. Due to a lack of space the rundown will not drain the total 100 year design storm. Approximately 23 cfs will remain to weir flow over the existing bridge headwall. although this situation is not ideal, **VENTURA ESTATES** contributes only a small portion to the developed peak flowrate, and contruction of this channel significantly improves the drainage conveyance at the bridge. The sidewalk culverts will remain to drain nuisance flow and a small basin from the west. (see Appendix, page 9 and Detail Sheet 4 for analysis).

Basins "A" thru "F" will drain west and be collected by a storm drain system designed to pipe flows to the existing 42-inch storm drain located in Ventura Street. The 42-inch storm drain conveys off-site flows from Wimbledon West, Fairways North and the old San Antonio right-of-way to the North Arroyo del Pino. In order to control the Hydraulic Grade Line within Ventura Street it will be necessary to upsize a portion of the existing 42-inch line. The storm drain analysis (see Appendix, page 11 and the Storm Drain Plan and Profile Sheet 3) shows that upsizing a 60-foot section of the line downstream from the proposed connection point will improve the flow characteristics sufficient enough to allow for extension of the system into the project site.

Several small basins (B,D & F) will drain flows via private valley gutters within rear yards. These valley gutters will discharge into the public street system through sidewalk culverts. Private drainage easements are to be provided for placement of these facilities. Calculations for the capacity of the valley gutters and culverts are provided on page 4 of the Appendix.

Off-site flows from the east will be intercepted by improvement of Holbrook Street along the project frontage. Holbrook will convey the off-site flows north to the existing improved section of Holbrook which drains to the North Arroyo del Pino.

EROSION CONTROL

Temporary erosion control facilities will be required throughout the construction phase of the project. The contractor is required to obtain a topsoil disturbance permit prior to beginning any earthwork operations. The contractor will be required to adhere to all local dust control and air pollution ordinances during the construction phase of the project. In order to limit the discharge of sediment downstream, a ditch-dike system shall be placed in appropriate locations, as indicated and detailed on the Final Grading Plan. These interim erosion control measures shall remain in place until all paving and downstream drainage facilities are in place.

CALCULATIONS

I. CRITERIA

Criteria for hydrologic calculations is per the Rational and SCS methods of estimating storm runoff as outlined in the City of Albuquerque "Development Process Manual", Volume II, Chapter 22.

Rainfall: P100/6 hour = 2.60 in.
P10/6 hour = 1.71 in.

Rainfall Intensity: $I = 6.84P(Tc^{-.51})$ (in/hr)

where P = rainfall (in)
Tc = time of conc.(min)

Time of Concentration: $Tc = 0.0078(L^{0.77})/S^{0.385}$ (min.)

where L = length (ft.)
S = slope (ft./ft.)

Soil: Embudo (EmB) & (EtC), Group 'B'

SCS curve number: CN per DPM, Vol. II, plates 22.2 C2 & C3

Rational "C" factor: "C" factor by Notice of Emergency Rule

Runoff: Q = CIA (c.f.s.) - Rational Method

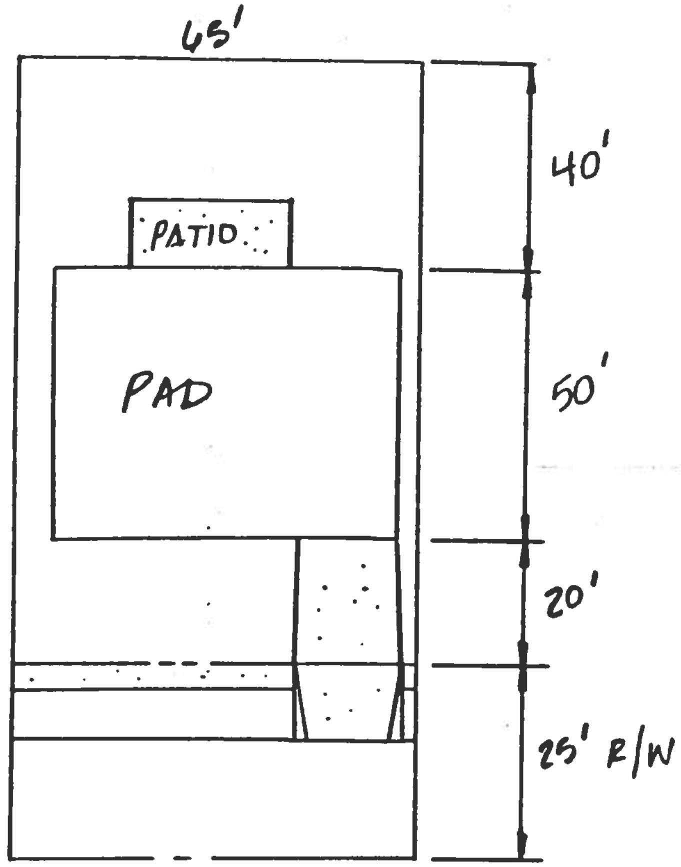
where A = basin area (ac.)
I = intensity (in/hr)
C = "C" factor

Volume: 3630AR (c.f.) - SCS Method

where A = basin area (ac.)
R = direct runoff (in.)

PROJECT NAME ____ JOB NO. ____
SUBJECT ____
BY DL CK BY ____ APPROVED BY ____ DATE ____ PAGE 2 OF ____

RATIONAL 'C' FACTOR DETERMINATION (ON-SITE)



AVE LOT SIZE · 65×110' + PUBLIC R/W = 10,400 SF

- PAD = 2750 SF @ C = 0.90
- DRIVE/PATIO = 600 SF @ C = 0.95
- LANDS/SOD = 3800 SF @ C = 0.25
- PUBLIC R/W = 3250 SF @ C = 0.95

'C' FACTOR = 0.68

PROJECT NAME ______ JOB NO. ______
SUBJECT ______
BY DL CK BY ______ APPROVED BY ______ DATE ______ PAGE 4 OF ______

PRIVATE VALLEY GUTTER CAPACITIES

AP#	Q_{100}	V.G. SLOPE	* CAPACITY	# S/W CULVERTS **
2	1.6 cfs	0.015'/1	1.8 CFS	2- 2'
5	1.6	0.019	2.0	2- 2'
6	1.1	0.010	1.4	2- 2'

* BY MANNINGS EQN, WHERE: $n = 0.013$
$A = 0.5$ SF
$P = 4$ FT

** BY WEIR EQN, WHERE: $Q = 3.33 L h^{1.5}$
$h = 0.33$ FT
$L = 2.0$ FT
$Q_{MAX} = 1.3$ CFS

- FOR PRIVATE VALLEY GUTTER DETAIL, SEE PAGE 5.

PROJECT NAME ______ JOB NO. ______
SUBJECT ______
BY pL CK. BY ______ APPROVED BY ______ DATE ______ PAGE 6 OF ______

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STREET DEPTH ANALYSIS

AP #	Q100	STREET WIDTH	STREET SLOPE	d100	CURB TYPE
1	39.0 cfs	32 FT	0.0225 %	0.48 FT	STANDARD
3	14.9	32	0.0292	0.35	"
4	10.2	32	0.0292	0.32	"
7	14.7	32	0.0390	0.32	"
10	78.1	32	0.0260	0.59	"
8	14.6	60	0.004 min	0.59	"
11	1.2	32	0.0085	0.20	"
12	19.3	32	0.0292	0.46 *	STANDARD
13	13.0	32	0.0333	0.33	MOUNTABLE

d_{MAX} = 0.33' MOUNTABLE
0.67' STANDARD

* SEE SUPERELEVATION ANALYSIS PAGE 7.

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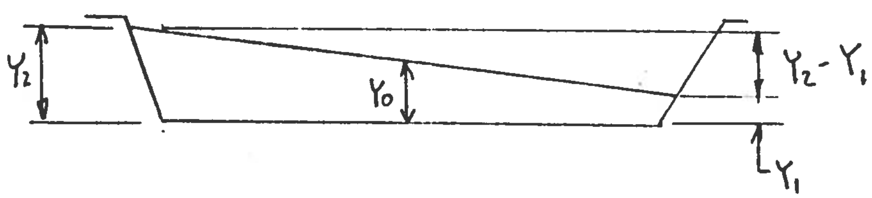
SUPER ELEVATION ANALYSIS

PER DPM VOL II CHP 22 PG 58

$$S = \frac{V^2 B}{gR}$$

WHERE: V = VELOCITY, FPS
B = TOP WIDTH, FT
g = 32.2 F/S^2
R = CURVE ℄ RADIUS, FT



$S_T = Y_2 - Y$ = TOTAL SUPER ELEVATION

$S = Y_2 - Y_0$ = SUPER ELEVATION EACH SIDE

$$S = \frac{Y_2 - Y_1}{2} = \frac{V^2 B}{2gR}$$

AP	AREA	Q_{100}	d_{100}	V	SLOPE	R	S	d_{TOTAL}
3	3.98	14.9	0.35'	4.0	0.0292	75'	0.10'	0.45'
11	0.32	1.2	0.20'	1.5	0.0085	75'	0.02'	0.22'
12	5.15	19.3	0.46'	5.4	0.0292	75'	0.20'	0.66'

STD CURB & GUTTER PROVIDED AT ALL AP's

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HYDRAULIC JUMP ANALYSIS:

CHECK FOR HYD. JUMP AT T-INTERSECTIONS (AP 7 AT DE VARGAS & GREENWOOD, AND AP 13 REDMONT & GREENWOOD)

PER DPM VOL II CHP 22 PG 87, USE PLATES 22.3 E-1 & 22.3 E-2 TO DETERMINE HEIGHT & LENGTH OF JUMP.

① AP 7 Q_{100} = 14.7 CFS STREET S = 0.039
d_{100} = 0.32' V = 4.8 FPS

$F_R = \frac{V}{\sqrt{gD}} = 1.4$ $\frac{TW}{D} = 1.5$ $\frac{L}{D} = 4$

TW = JUMP HT = 0.56 FT OK STD C/G EXISTING

L_{jump} = 1.1'

② AP 13 Q_{100} = 13.0 S = 0.033
d_{100} = 0.33 V = 4.5 FPS

F_R = 1.4 TW/D = 1.4 $\frac{L}{D} = 3$

TW = JUMP HT = 0.50 FT OK STD C/G

L_{jump} = 1.1'

DRAINAGE RUNDOWN @ BRIDGE

1. EXISTING S/W CULVERT CAPACITY

EXISTING (3) 18" S/W CULVERTS

A. CHECK ENTRANCE BY WEIR:

$Q = 3.33 L H^{3/2}$

$L = 1.5'$

$H = 8'' = 0.67'$

$Q = 2.7$ CFS (EA)

$\Sigma_1^3 Q = 8.1$ CFS

B. CHECK CHANNEL FLOW BY MANNINGS'

$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$

$S = 1\%$

$n = 0.013$

$A = 1.0$ SF

$R = 0.35$

$Q = 5.7$ CFS (EA)

$\Rightarrow$ INLET CONTROL

$Q_{EXCESS} = 78.1 - 8.1 = 70$ CFS

2. PROVIDE RUNDOWN CHANNEL UPSTREAM FROM BRIDGE.

MAXIMUM AVAILABLE WIDTH = 18'

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VENTURA ESTATES

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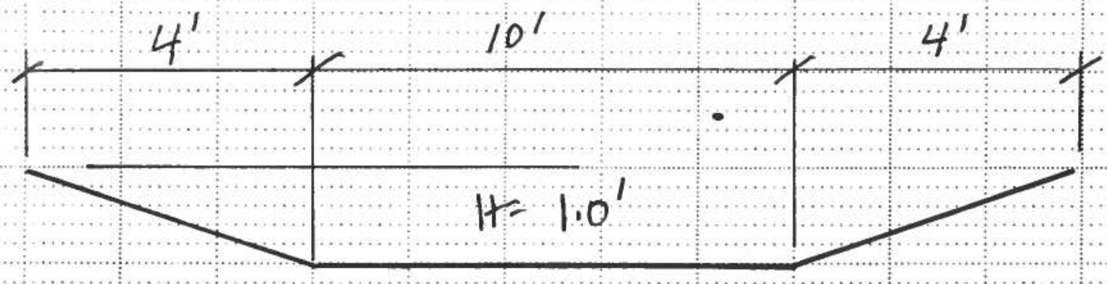
DRAINAGE RUNDOWN @ BRIDGE



SECTION

CHECK ENTRANCE CAPACITY BY WEIR

$Q = 3.33\,LH^{3/2}$ $L = 14'$ (AVE)
$H = 1'$

$Q = 46.6$ CFS

$Q_{EXCESS} = 70 - 46.6 = 23.4$ CFS.

EXCESS TO SPILL OVER BRIDGE HEADWALL

SEE SHEET 4 FOR COMPLETE DETAILS.

PROJECT NAME _______ JOB NO. _______
SUBJECT _______
BY DL CK BY _______ APPROVED BY _______ DATE _______ PAGE 11 OF _______

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STORM SEWER DESIGN - CONCEPTUAL

A STORM SEWER WILL BE EXTENDED FROM THE EXISTING 42" RCP IN VENTURA ST. TO DRAIN FLOWS FROM BASINS A-F. ALL PIPE SIZING IS BY MANNING'S EQN. DROP INLET REQUIREMENTS PER DPM PLATES 22.3 D-5-7

AP#	Q100	d100	INLET TYPE	NO. INLETS	INLET Q	PIPE DIA.	PIPE Q	RESIDUAL Q
1	39.0	0.48'	A	2	16.0	24	16.0	23.0
	23.0	0.41	CC	2	12.0	24	28.0	11.0
	9.0	0.32	CC	2	11.0	30	39.0	-0-

ON-SITE STORM SEWER WILL CONNECT TO EXISTING 42" RCP WHICH DRAINS TO N. PINO CHANNEL. DESIGN FLOW FOR 42" SD IS 104 CFS (PER AS BUILTS AND DRAINAGE REPORT FOR WIMBELDON WEST). ADDITION OF ON-SITE FLOW INCREASES Q TO 143 CFS. 42" SD WILL FLOW UNDER PRESSURE CONDITIONS. IN ORDER TO KEEP HGL FROM AFFECTING THE ON-SITE SYSTEM, APPROX 130 LF OF 42" SD WILL BE UPSIZED TO 60" SD IMMEDIATELY NORTH OF NEW MANHOLE. SEE SHEET 3 FOR PROFILE.