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File 1989-0060 Name: State Farm Office – 2779 Crossroads Blvd. – Special Use

	Scanned	
		A few items are denoted with an asterisk (*), which means they are to be scanned for permanent record on the in some instances, not all entries designated to be scanned by the department are present in the file. There are also documents specific to certain files, not found on the standard list. For this reason, a checklist has been provided. Remaining items, (not selected for scanning), will be marked present on the checklist. This index can serve as a quick guide for the contents of each file. Files denoted with (**) are to be located using the ISYS Query System. Planning Clearance will need to be typed in full, as well as other entries such as Ordinances, Resolutions, Board of Appeals, and etc.
X	X	**Table of Contents**
X	X	**Review Sheet Summary**
X		Application Form
X		Review Sheets
X		Receipts for fees paid for anything
		***Submittal checklist**
X	X	***General project report**
		Reduced copy of final plans or drawings
		Reduction of assessor's map.
		Evidence of title, deeds, easements
		***Mailing list to adjacent property owners**
		Public notice cards
		Record of certified mail
X		Legal description
		Appraisal of raw land
		Reduction of any maps – final copy
		***Final reports for drainage and soils (geotechnical reports)**
		Other bound or non-bound reports
		Traffic studies
X	X	***Petitioner's response to comments**
		***Staff Reports**
		***Planning Commission staff report and exhibits**
		***City Council staff report and exhibits**
		***Summary sheet of final conditions**
		***Letters and correspondence dated after the date of final approval (pertaining to change in conditions or expiration date)**

DOCUMENTS SPECIFIC TO THIS DEVELOPMENT FILE:

X	X	Action Sheet	X	X	Letter from Linda Weitzel to Paul Klumb re: notice of approval – 1/22/89
X	X	Review Sheet Summary	X		Letter from Targie Hall, Director of Real Estate, Kettle Restaurants, Inc. re: asking for written approval for an extension of time to ge project completed – 1/25/90
X		Review Sheets	X	X	Letter from Don Newton to Katherine Crain, Kettle Restaurant re: approval of Storm Drain Facility – 8/6/90
X	X	Development Summary	X	X	Letter from Don Newton to Don Pettigrove, Banner Assoc., Inc. re: City will not accept 6" drain pipe – 10/24/90
X		Development Application – 12/7/89	X		Real Estate Appraisal – Robert O. Stevens, MAI
X	X	Planning Clearance - ** - 3/6/90	X	X	Drainage Report – State Farm Mutual Automobile Ins. Co. – 12/89
X		Request for Treasurer's Certificate of Taxes Due – 11/6/89	X	X	Exhibit "A" – Drainage PLan
X		Commitment for Title Ins., Chicago Title Ins. Co. – 11/15/89			
X	X	Perpetual Easement for Utilities and Storm Drainage purposes – Kettle Restaurants, Inc. and City of Grand Junction - ** - 8/30/90			
X		Notice of Special Use Application – 1/2/90			
X	X	Sign Diagram			
X	X	Landscaping Plan			
X	X	Site Plan			
X	X	Elevation Map			

Drainage Report

STATE FARM MUTUAL AUTOMOBILE INSURANCE CO.
LOT 3, BLOCK 2, CROSSROADS COLORADO WEST

for
Castillo Company

BANNER

BANNER ASSOCIATES, INC. — CONSULTING ENGINEERS
2777 CROSSROADS BOULEVARD — GRAND JUNCTION, COLORADO 81506
(303) 243-2242

DECEMBER 1989

DRAINAGE REPORT

for the

STATE FARM MUTUAL AUTOMOBILE INSURANCE COMPANY SITE
2779 Crossroads Boulevard
Grand Junction, Colorado

Prepared For

CASTILLO COMPANY
Post Office Box 21087
Phoenix, Arizona 85036-1087

Prepared By

BANNER ASSOCIATES, INC.
2777 Crossroads Boulevard
Grand Junction, Colorado 81506

COLORADO REGISTERED
GERALD R. WILLIAMS
26481
12/22/89
PROFESSIONAL ENGINEER

BAI Job No. 8200

December 20, 1989

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INTRODUCTION

Development is proposed on Lot 3 in Block 2 of the Replat of Lots 1 Through 5, Crossroads Colorado West, Mesa County, Colorado. (See Vicinity Map, Page 2.) The site is presently vacant, sloping towards the southwest. Development will significantly reduce permeable surface area, which, without mitigation, would result in an increase of runoff during storm activity.

The purpose of this Report is to estimate predevelopment peak flow from the site, design adequate detention facilities which result in post development peak flow which does not exceed predevelopment peak flow in the design storm, and design outlet and discharge facilities.

DESIGN CRITERIA

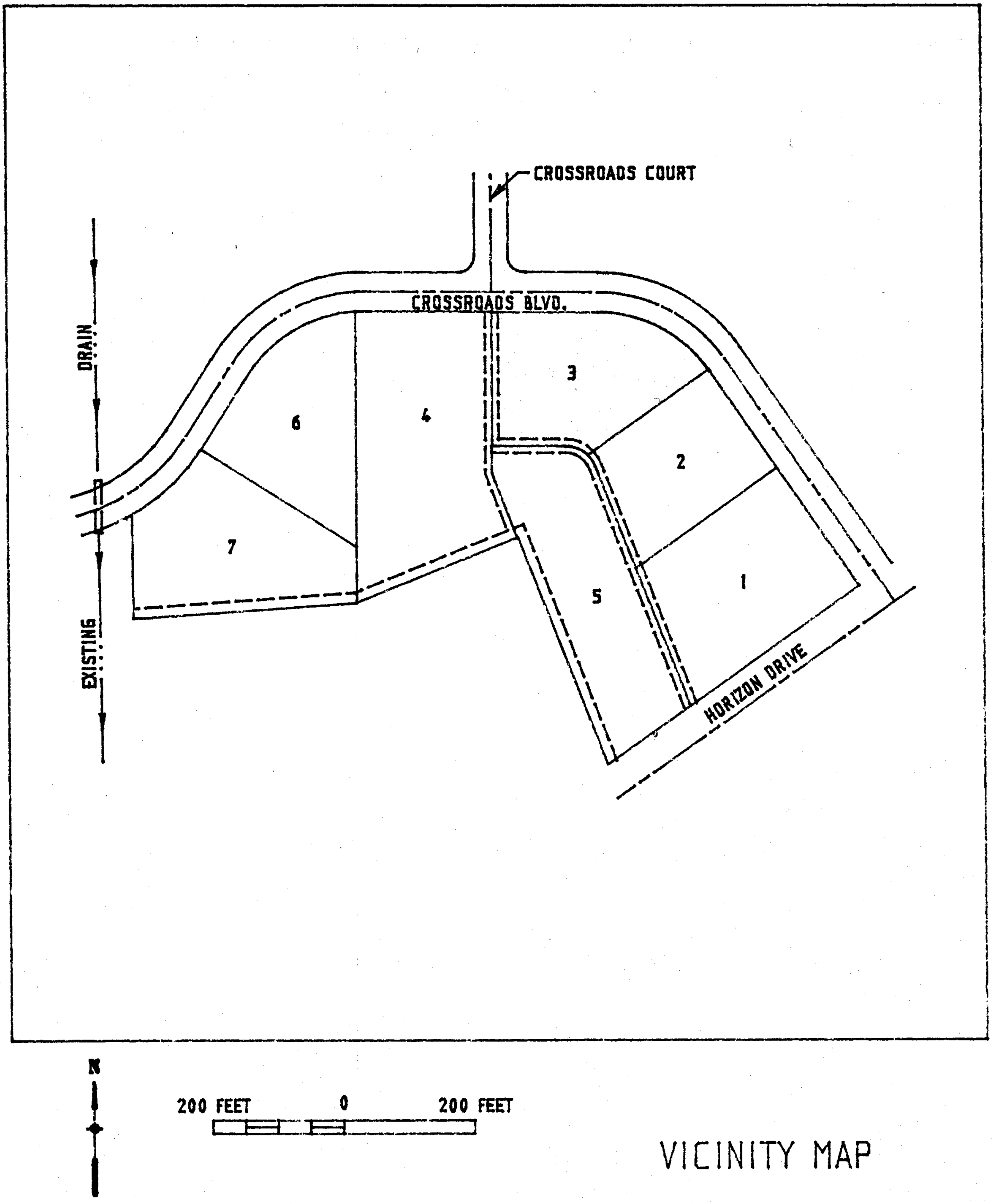
The design storm used in this Report is the 10-year storm. The rational method will be used in hydrology calculations, which is

$$Q = CIA.$$

where Q = Runoff in cubic feet per second (cfs) from a given area.
C = Permeability coefficient representing the ratio of runoff to rainfall (See Appendix "A").
I = Intensity of precipitation in inches per hour (See Appendix "A").
A = Area in acres.

The time of concentration (Tc) is the time required for peak flow to be reached at a given location. For this site, this is assumed to be the time required for water to travel from the most remote point on the site to the point of discharge from the site. The Tc time is based on the SCS TR-55[1] and the NEH-4[2], with a minimum value of five (5) minutes.

Many methods exist for estimating the detention volume required to prevent an increase in runoff from a site due to development. Most are based on a triangular simplification of a runoff hydrograph, which is a graphical representation of runoff rate over time, starting at zero (0), increasing to a peak rate at the time of concentration, and decreasing back to zero (0). Generally, 37.5% of the runoff volume is assumed to occur between runoff start and the peak runoff (or at the time of concentration), and the



VICINITY MAP

balance of runoff volume occurring after the time of concentration[2]. Assuming a linear increase and decrease in runoff, the total storm runoff volume can be calculated. Detention for the difference in pre and post development volumes is provided. But this does not ensure that the predevelopment peak runoff rate is not exceeded under post development conditions.

A more accurate way to size detention basins is to perform a time/flow analysis. Flows are calculated at intermittent times, with corresponding accumulated volumes calculated. Once flow rates have decreased to predevelopment peak flow or less, then no more detention volume is required. This will result in post development peak flow not exceeding predevelopment peak flow for the design storm.

PREDEVELOPMENT CONDITION

Runoff from the site flows to the southwest. The site area is 1.22 acres. The Tc value is 8.1 minutes (Appendix "B"), and the corresponding intensity is 2.85. A "C" value of 0.30 is assumed. The resultant estimated peak 10-year storm runoff from the site is

$$Q_p = (0.30)(2.85)(1.22) = 1.0 \text{ cfs}$$

Post development flow cannot exceed this rate in the 10-year storm.

POST DEVELOPMENT CONDITION

The site remains the same at 1.22 acres. The Tc value is 8.5 minutes (Appendix "B"), and the corresponding intensity is 2.80. A weighted or composite "C" value is determined as follows, assuming future expansion is in place:

0.38 Acres @ 0.35 = 0.13
0.84 Acres @ 0.95 = 0.80
Composite "C" = 0.93/1.22 = 0.76

The peak runoff without detention would be

$$Q_p = (0.76)(2.80)(1.22) = 2.6 \text{ cfs}$$

As anticipated, detention is required because the predevelopment peak 10-year runoff rate of 1.0 cfs would be exceeded.

DETENTION AND OUTLET FACILITY

Detention is not the same as retention. Stored volume is only temporary, denoting a means of bleedoff. Avoiding a bleedoff facility that requires human intervention, such as opening a gate, means that bleedoff will be occurring simultaneously with the detention buildup. The outlet design then becomes an integral part of the detention design.

There are two (2) simple means of limiting outflow. The outlet opening can limit outflow, such as a curb opening or a grate opening sized to limit flow. Another means of limiting flow is to size and design the outlet pipe so that capacity limits outflow.

Outlet openings as a means of metering outflow has two (2) drawbacks. Capacity starts out low and increases slowly with depth. Outflow over much of the ponding period is minimal, resulting in more required detention area to hold water before runoff has decreased to predevelopment flows. Outlet openings are also subject to clogging with debris. When this occurs, outflow is reduced or even stopped. This is particularly true for horizontal inlet grates, but would also be true for vertical openings such as in a curb for flows as low as 1 cfs.

Less maintenance problems can be achieved by over sizing inlets and letting pipe capacity limit flow. Greater bleedoff rates can be obtained over the detention build-up period, which will allow use of less detention volume.

Pipe hydraulics are based on the Manning equation[3], or

$$Q = \frac{0.463\, d^{2.67}\, s^{.5}}{n}$$

where
- Q = flow in cfs.
- d = pipe diameter in feet.
- s = pipe slope if pipe is not flowing full or just full.
- s = hydraulic gradient if pipe is flowing full, surcharged, or above normal depth.
- n = Manning friction coefficient.

Since the pipe is to be designed at capacity, the latter condition of "s" will apply. For PVC pipe, an "n" value of 0.011 is used[4].

An eight (8) inch pipe will allow too much outflow under conditions which exist and will be designed into the site. However, a six (6) inch pipe is small enough that it would be too restrictive to be used for the full outlet length. A distance must be chosen which is long enough that outlet capacity does not vary too significantly as detention ponding head increases, and yet not so long that more hydraulic head is required to force flow through the pipe than is available.

It was determined that a six (6) inch pipe could probably be used from the detention outlet to the manhole, or about 170 feet. At this distance, a hydraulic gradient of 2.27% is required to push 1 cfs through a six (6) inch PVC pipe, or surface water 3.86 feet above the top of pipe at the downstream end.

Table 1 provides outlet capacity data, assuming that the outlet is horizontal, 2.86 feet above the hydraulic grade line in the manhole, that the outlet opening is 12" x 12", with 0.34 $feet^2$ open area, that the clogging factor reduces capacity by 50%, but that two (2) outlets are provided.

TABLE 1

OUTLET CAPACITY

Depth of Water above inlet (ft)	Hydraulic Gradient Available (%)	Pipe Capacity (cfs)	Weir[1] Inlet Capacity (cfs)	Orifice[2] Inlet Capacity (cfs)	Design Rate (cfs)
0.2	1.72	**0.87**	1.07	—	**0.87**
0.4	1.86	**0.90**	3.04	—	**0.90**
0.6	1.99	**0.94**	5.58	1.42	**0.94**
0.8	2.13	**0.97**	8.59	1.64	**0.97**
1.0	2.27	**1.00**	12.00	1.83	**1.00**

1. Weir flow based on the following equation applicable from 0.0 to 0.4 foot ponding depth, partially applicable from 0.4 foot to 1.4 foot ponding depth:

   $Q = 3pd^{1.5}$

   Where Q = Inlet capacity in cfs.
   p = Perimeter of inlet without subtraction for bars (feet).
   d = depth of ponding over grate (feet).

2. Orifice flow based on the following equation, applicable above 1.4 depth, partially applicable between 0.4 foot and 1.4 foot ponding depth.

   $Q = 0.67\ A(2gd)^{0.5}$

   Where Q = Inlet capacity in cfs.
   A = Open area of grate, feet^2.
   g = 32.2
   d = Depth of ponding over grate, feet.

Results of above equations (taken from HEC-12[5]) indicate that pipe capacity governs, not inlet capacity.

The five (5) flow depths in Table 1 above will be assumed to apply to the five (5) incremental time steps in the Time/Flow Analysis provided in Table 2.

TABLE 2

TIME/FLOW ANALYSIS

Time (min)	Intensity (in/hr)	Q (cfs)	Avg Q (cfs)	Bleedoff (cfs)	Q in Excess of Bleedoff (cfs)	Time Period (sec)	Volume to Be Detained (ft^3)
0.0		0.0					
			1.30	0.87	0.43	510	219
8.5	2.80	2.6					
			2.15	0.90	1.25	690	863
20.0	1.80	1.7					
			1.55	0.94	0.61	600	366
30.0	1.50	1.4					
			1.30	0.97	0.33	600	198
40.0	1.25	1.2					
			1.10	1.00	0.10	600	60
50.0	1.05	1.0					

Total Volume To Be Detained 1,706 ft^3

During storms of greater magnitude than the 10-year storm, flow will overtop the berming and follow its historic flow pattern.

DISCHARGE FACILITY

The City of Grand Junction has provided an outfall facility to receive runoff from the site. The primary facility is an existing drain which is approximately 600 feet west of the site. Another facility is a pipe and catch basin system which collects sheet runoff. (See Figure 1.)

The drain is accessible to the site through 10' or 15' wide utility easements which are provided along lot lines. Access to the street drain system is through the street.

Discharge of onsite runoff can be handled several ways, four (4) of which are analyzed in this Report. They are:

1. Bleedoff from the southwest corner of the site through a pipe sized to handle only on site bleedoff.
2. Same as above, but pipe enlarged as required at applicable locations to accommodate inflow from surrounding lots;
3. Pump water from the southwest corner of the site to Crossroads Boulevard and discharge into the street; and
4. Import fill to the site and provide for gravity drainage to Crossroads Boulevard.

Each of the four (4) options are discussed in more detail below.

The Grading and Drainage Plan (Exhibit "A") is applicable to Options 1, 2, and 3. Option 4, if chosen, will require a separate plan.

Option 1

This option proposes the installation of a 6" and 8" PVC pipeline within the lot line easements between Lots 4 and 5, Lot 4 and Holiday Inn, and Lot 7 and Holiday Inn. The pipe would only serve Lot 3. (Refer to Figure 2.)

This option provides for the needs of Lot 3, but ignores the potential to resolve drainages problems of sites along the route of the drain pipe.

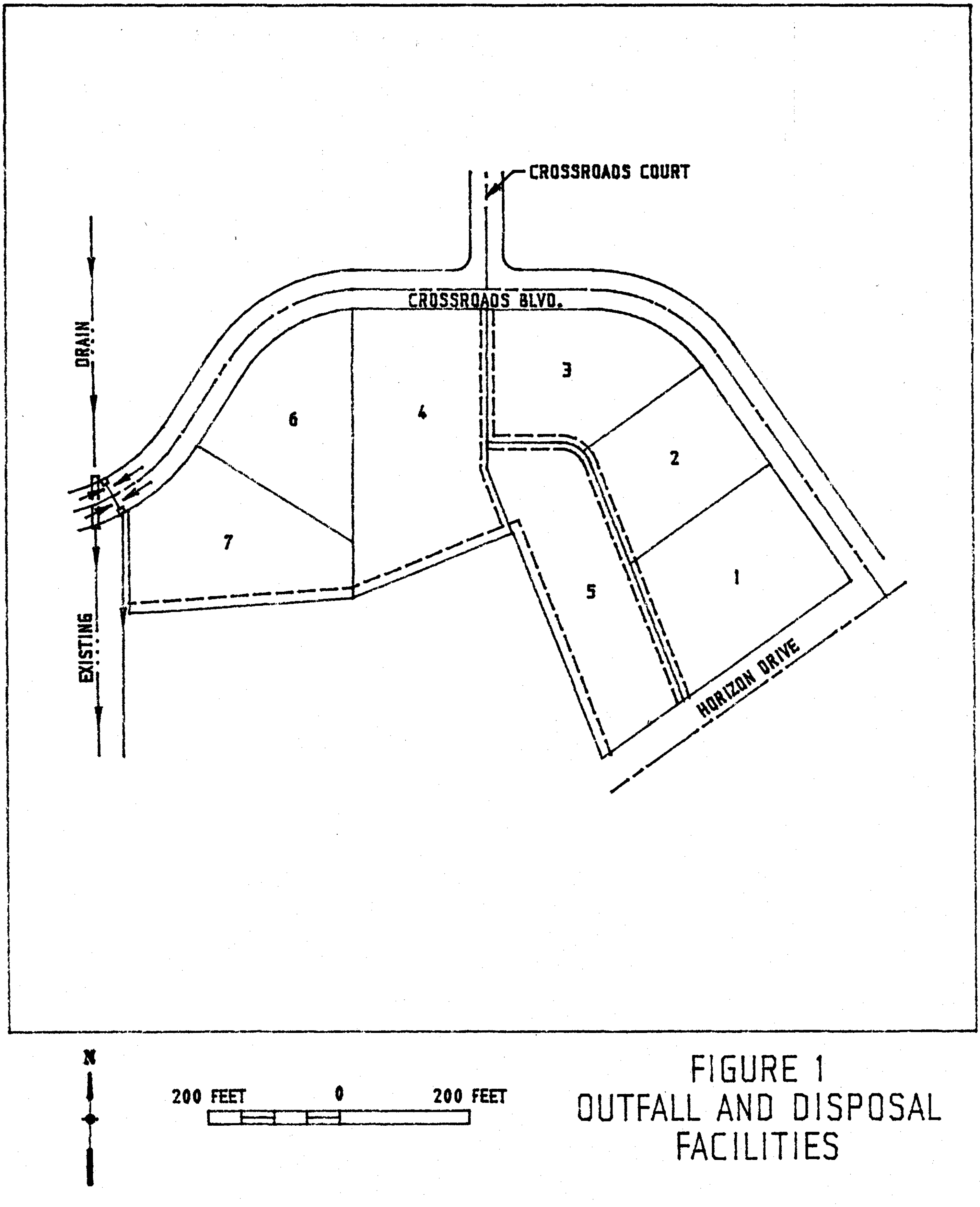
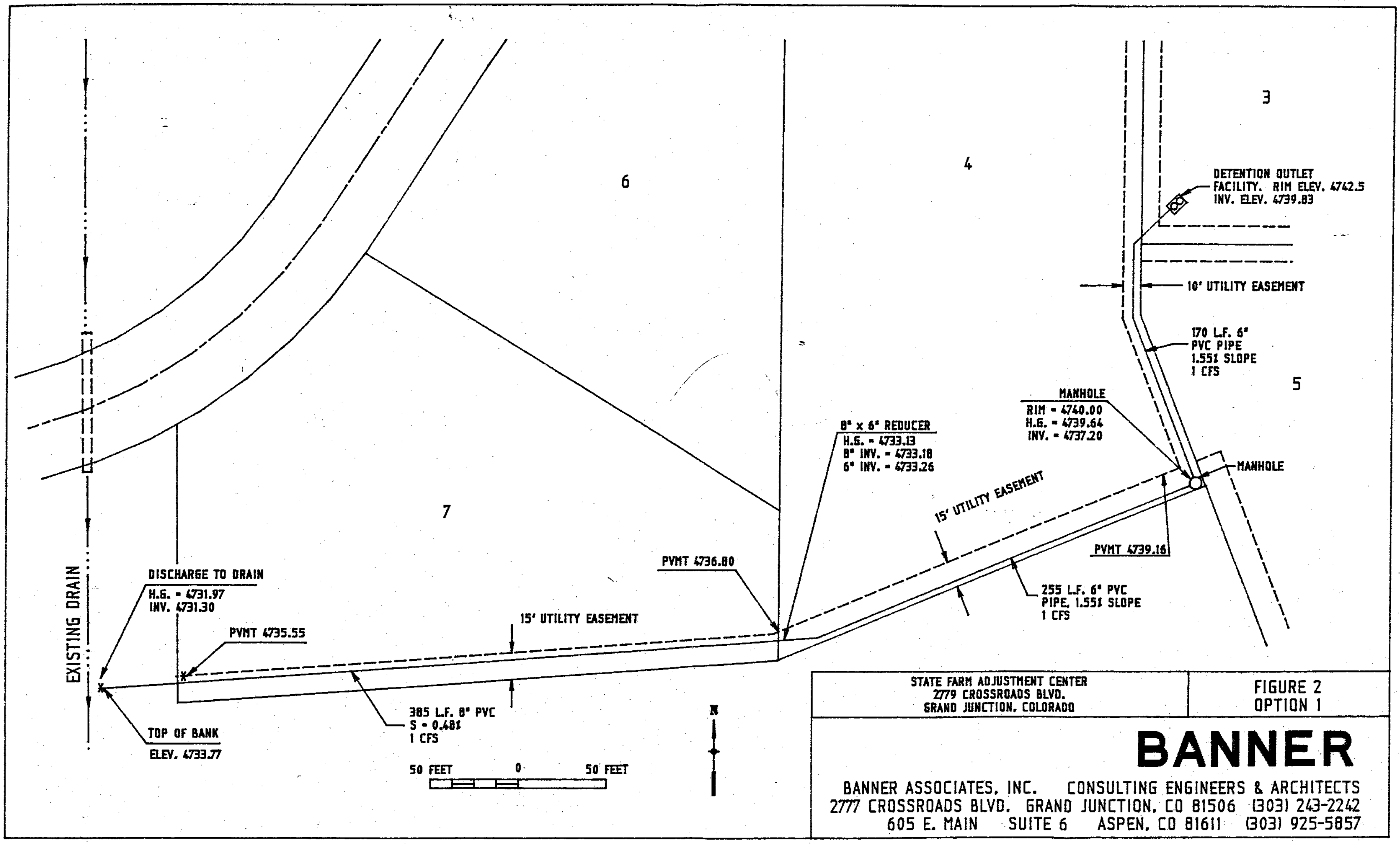


FIGURE 1
OUTFALL AND DISPOSAL
FACILITIES



STATE FARM ADJUSTMENT CENTER
2779 CROSSROADS BLVD.
GRAND JUNCTION, COLORADO

FIGURE 2
OPTION 1

BANNER

BANNER ASSOCIATES, INC. CONSULTING ENGINEERS & ARCHITECTS
2777 CROSSROADS BLVD. GRAND JUNCTION, CO 81506 (303) 243-2242
605 E. MAIN SUITE 6 ASPEN, CO 81611 (303) 925-5857

Option 2

Owners of Lot 4 and 5 also have access to the existing drain for disposal of storm runoff. However, under current conditions, Lot 5 disposes of its water onto the surface of the parking area of Lot 4 and into a swale which is not entirely within the 15' utility easement. This has resulted in flooding on Lot 4 that has damaged pavement and resulted in legal action. Lot 5 runoff combined with Lot 4 runoff surface drains through Lot 7 (which also spreads out beyond the limits of the 15' utility easement) and toward the existing drain. Some of the runoff overflows the swale between Lots 4 and 7 and flows through a wall opening into the Holiday Inn parking lot.

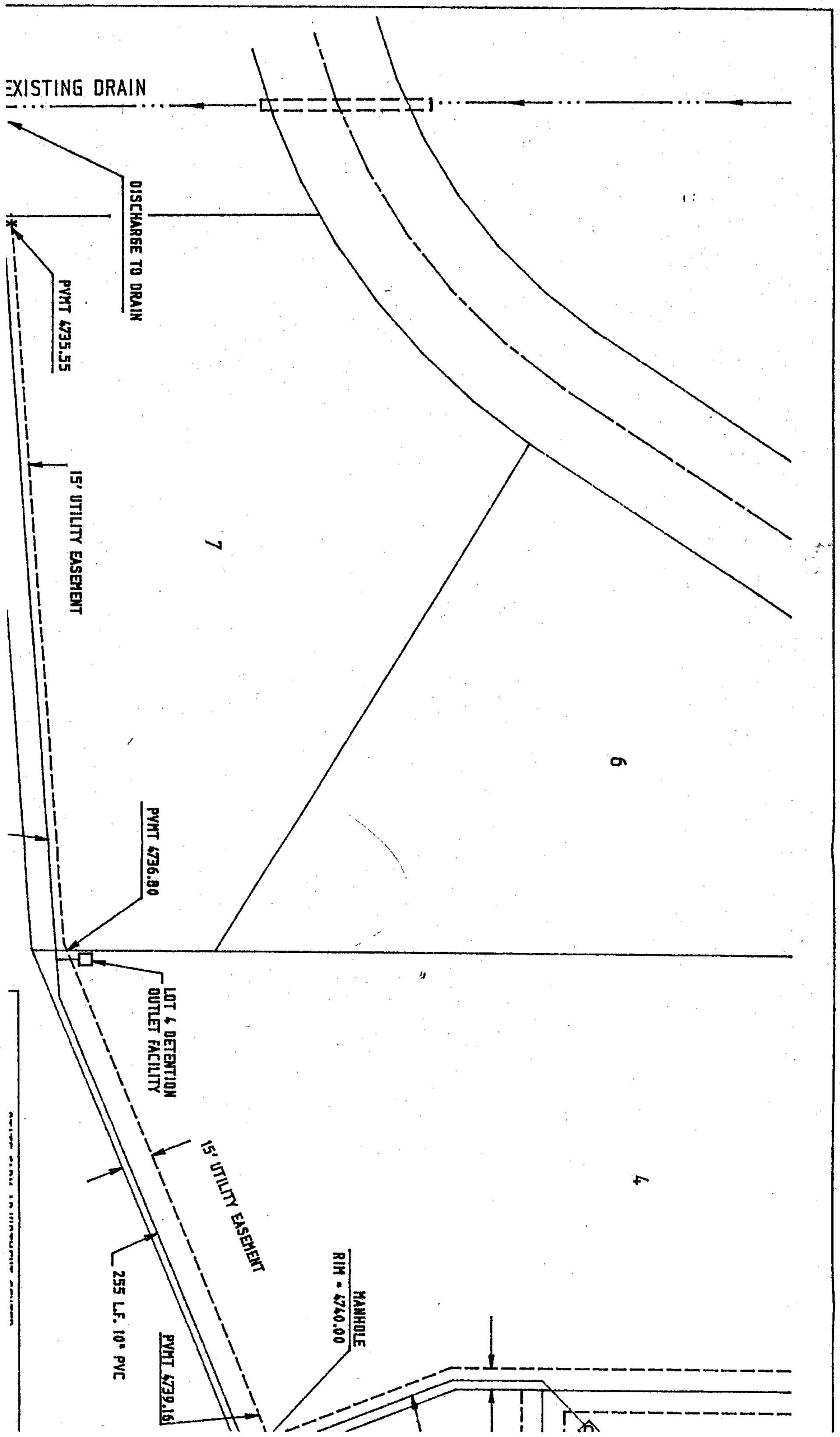
It would seem advantageous to owners of Lots 4 and 5 to participate in a drain pipe project. Lot 5 drainage could be metered into the drain pipe (with a corresponding pipe size increase) at the southeast corner of Lot 4. Runoff from Lot 4 could be added to the drain pipe at the west side of the lot with a corresponding pipe increase. The pipeline would then serve and provide proper means of drainage for Lots 3, 4, and 5, and also mitigate current flooding problems.

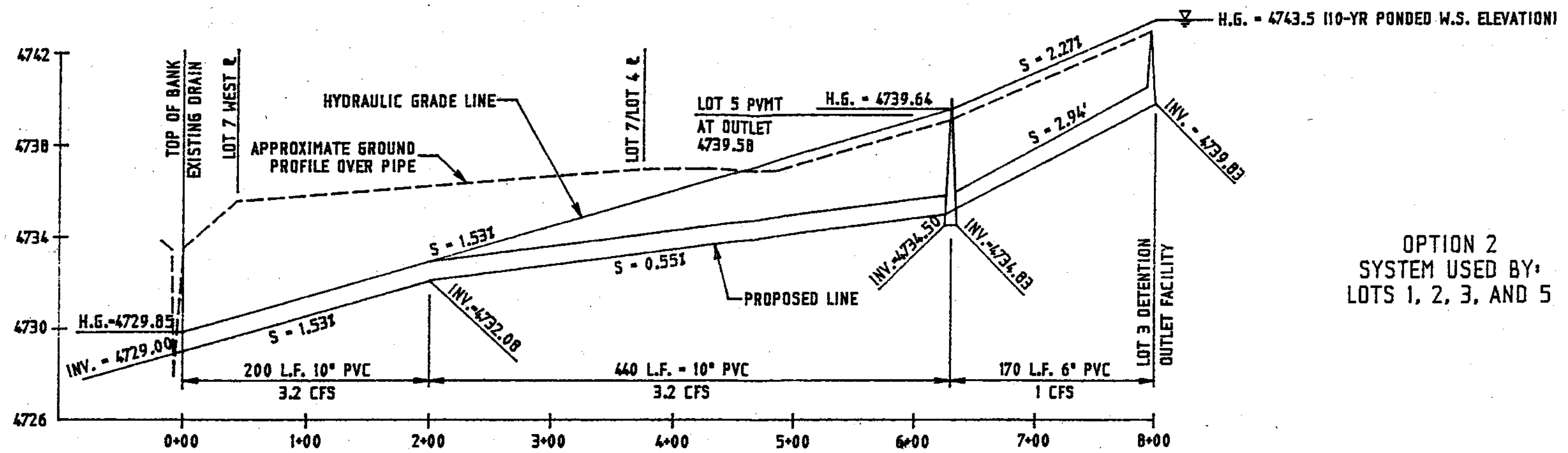
Lot 5 comprises 1.61 acres. About half of the runoff from Lots 1 and 2, which were developed prior to drainage ordinances, also flows onto Lot 5, adding another 1.23 acres of contributing area. Assuming a 10 minute time of concentration, the precipitation intensity would be 2.6 inches per hour for the 10-year storm. With a "C" value of 0.30 for the predeveloped condition, the 10-year predeveloped peak flow is estimated to be

$$Q = CIA = (0.30)(2.6)(2.84) = 2.2 \text{ cfs}$$

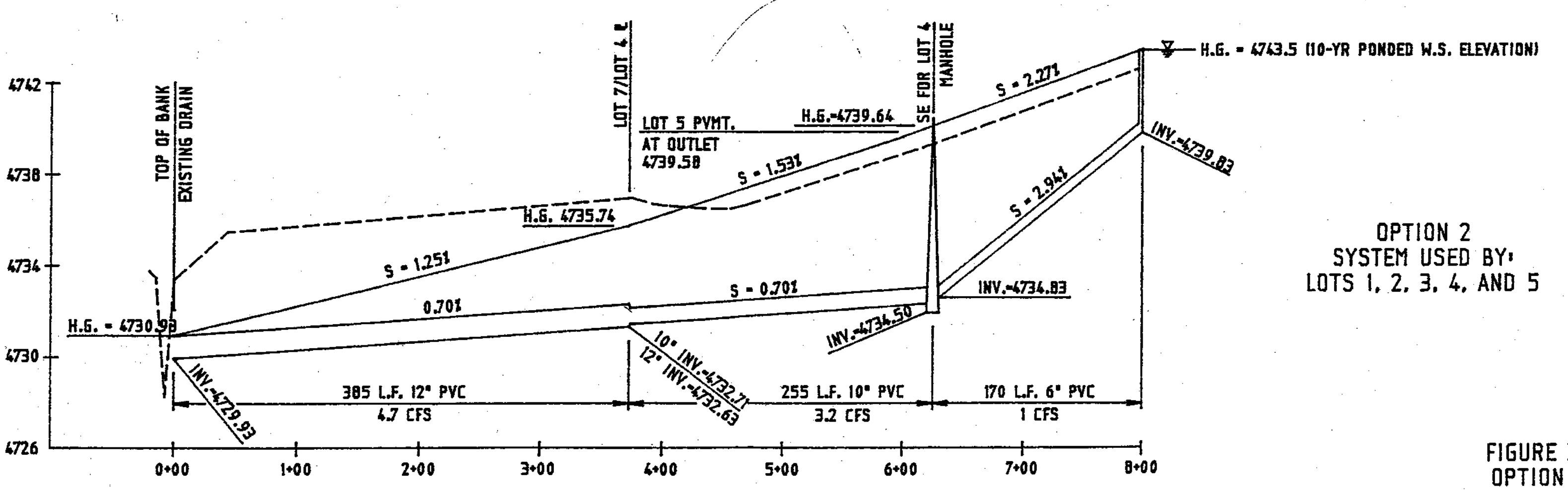
If the owner of Lot 5 chooses to use the drain pipe, the inlet device allowing inflow to it must be designed so that for the 10-year storm, a maximum of 2.2 cfs may enter the pipe.

Lot 4 comprises 1.93 acres. Assuming a 10 minute time of concentration, the precipitation intensity would be 2.6 inches per hour for the 10-year storm. With a "C" value of 0.30 for the


EXISTING DRAIN
DISCHARGE TO DRAIN
PVMT 4735.55
15' UTILITY EASEMENT
7
6
PVMT 4736.80
LOT 4 DETENTION OUTLET FACILITY
15' UTILITY EASEMENT
4
255 L.F. 10" PVC
PVMT 4739.16
MANHOLE
RIM = 4740.00



OPTION 2
SYSTEM USED BY:
LOTS 1, 2, 3, AND 5



OPTION 2
SYSTEM USED BY:
LOTS 1, 2, 3, 4, AND 5

FIGURE 3b
OPTION 2

STATE FARM ADJUSTMENT CENTER
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predeveloped condition, the 10-year predeveloped peak flow is estimated to be

$$Q=CIA=(0.30(2.6)1.93) = 1.5 \text{ cfs}$$

If Lot 4 chooses to use the drain pipe, the inlet(s) to it must be designed so that for the 10-year storm, a maximum of 1.5 cfs may enter the pipe.

This option meets the drainage needs of Lot 3 and allows for a single solution of a multiple problem, providing a low-maintenance gravity outlet for several lots currently experiencing drainage problems.

Figure 3a and 3b show a schematic of the possible joint facility drain pipe.

Option 3

Rather than drain runoff south and west, water could be pumped north to Crossroads Boulevard. Surface water would drain into a sump (a manhole with a grated lid) and be pumped through a four (4) inch pressure line and outlet into the face of the curb. The sump would be designed with a dissipation pit underneath it (a hole backfilled with 3/4" washed rock) so that water left in the lines and pit could drain off into the ground.

Observing bleedoff rates in Table 2, it is found that if a bleedoff rate of 1.0 cfs is provided by pumps, then the detention volume calculated previously would still be sufficient.

This option meets the drainage needs of Lot 3, but results in a system that could be problematic, requires more maintenance than gravity systems, is subject to power outages, and is nonfunctionable when electrical services are off. Also, solutions to current drainage problems on surrounding lots cannot be incorporated into this option.

Figure 4 is a sketch of facilities proposed with this option.

Option 4

The Grading and Drainage Plan (Exhibit "A") does not apply to this option. This option requires the importation of fill and raising the site such than runoff could be bled off to Crossroads Boulevard.

The site could be designed so that a peak of 1.0 cfs drains directly to the road in the 10-year storm. With a 5 minute Tc time, precipitation intensity is 3.3 inches per hour, and "C" = 0.76 (previously determined). The area that could drain toward the road can then be determined as follows:

$$Q = CIA$$
$$1 = (0.76)(3.3)A$$
$$0.4 \text{ acres} = A$$

The balance of the site would need to be designed so that runoff flowed through detention and metering facilities.

Although this option meets the needs of Lot 3, solutions to current drainage problems on surrounding lots cannot be incorporated into this option. Figure 5 depicts the concepts involved in Option 4.

A maximum of 1.0 cfs in the 10-year storm may be allowed to exit the site. However, in the predevelopment condition, little of the flow goes towards Crossroads Boulevard. Permission from the City of Grand Junction would be required to allow runoff to be drained towards the road per Options 3 and 4.

Rough estimates of Lot 3 share of costs for the various options are provided in Table 3.

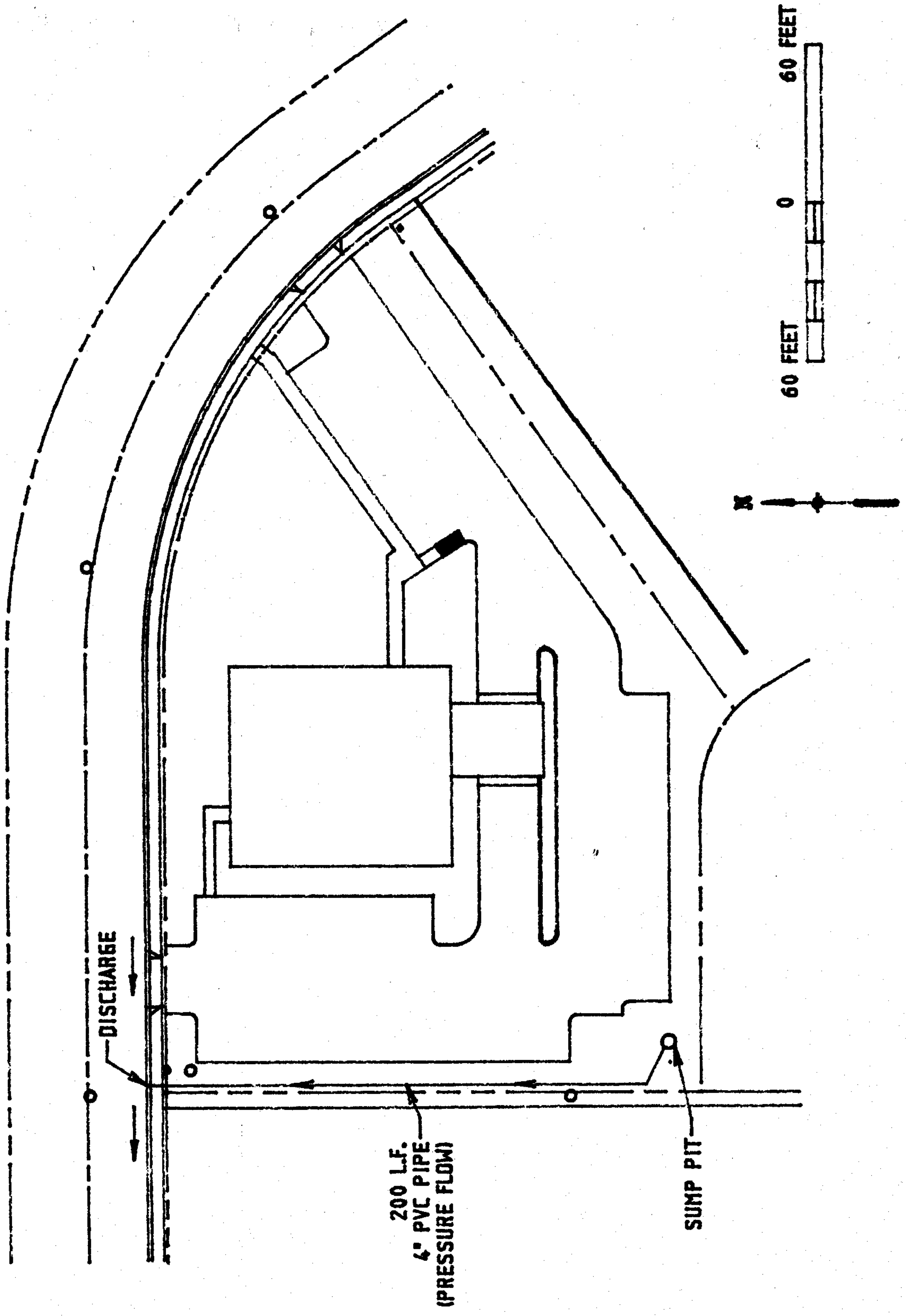

FIGURE 4
OPTION 3

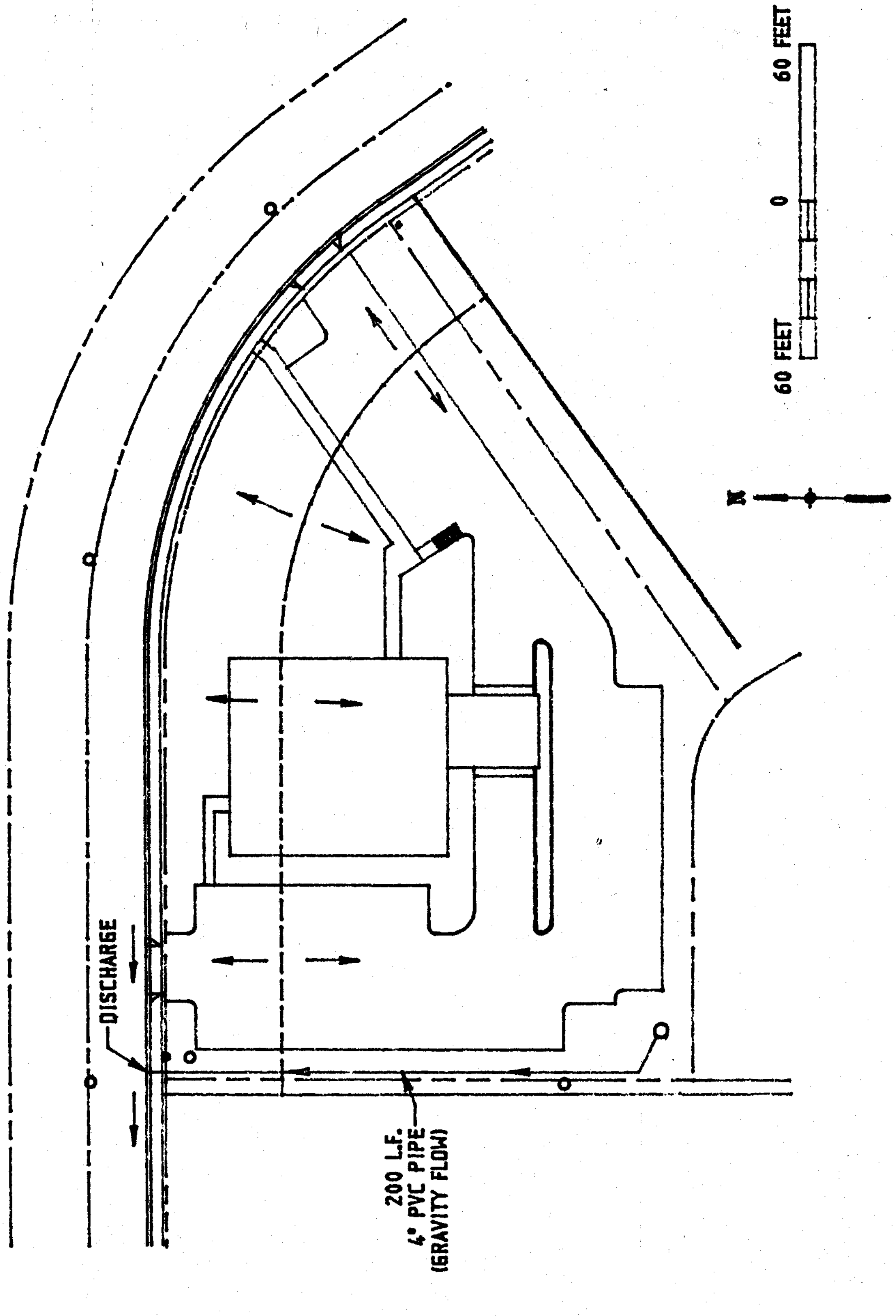

FIGURE 5
OPTION 4

TABLE 3

OPTION COST ESTIMATE: LOT 3 SHARE

Option 1

425 L.F. 6" Pipe @ $7/L.F.	$ 2,975.00
385 L.F. 8" Pipe @ $9/L.F.	3,465.00
350 L.F. Pavement Replacement @ $3/L.F.	1,050.00
1 EA Manhole	1,200.00
Detention Outlet Facility	500.00
TOTAL =	$ 9,190.00

Option 2 (Lots 3 and 5 & 1 & 2)

640 L.F. 10" Pipe @ $10.50/L.F. x (1/2 share)	$ 3,360.00
170 L.F. 6" Pipe @ $7 /L.F.	1,190.00
350 L.F. Pavement Replacement @ $3/L.F. x (1/2 share)	525.00
1 EA Manhole	1,200.00
Detention Outlet Facility	500.00
TOTAL =	$ 6,775.00

Option 2 (Lots 3, 5 & 1 & 2, and 4)

170 L.F. 6" Pipe @ $7/L.F.	$ 1,190.00
255 L.F. 10" Pipe @ $10.50/L.F. x (1/2 share)	1,339.00
385 L.F. 12" Pipe @ $12/L.F. x (1/3 share)	1,540.00
350 L.F. Pavement Replacement @ $3/L.F. x (1/3 share)	350.00
1 EA Manhole	1,200.00
Detention Outlet Facility	500.00
TOTAL =	$ 6,119.00

Option 3

200 L.F. 4" Pipe @ $4/L.F.	$ 800.00
Sump (Manhole) with Dissipation Pit	2,500.00
Lead and Lag pumps and meters, hardware, and electrical supply	3,500.00
Discharge through Curb	500.00
TOTAL =	$ 7,300.00

Option 4

2,500 CY (rough figure) @ $4/C.Y.	$ 10,000.00
200 L.F. 4" Pipe @ $4/L.F.	800.00
Detention Outlet Facility	500.00
TOTAL =	$ 11,300.00

CONCLUSION

The maximum runoff allowed from the site in the 10-year storm is 1.0 cfs. For Options 1 or 2, this can be accomplished by flow capacity on a 6" pipe as previously explained. For Option 3, sump pump capacity at 10.0 foot of head cannot exceed 1.0 cfs. For Options 1, 2, or 3, the Grading and Drainage Plan (Exhibit "A") applies, providing a minimum detention storage of 1,706 feet^3. Option 4 would require a separate grading and drainage plan and report.

Pipe sizes for the various options should be as contained in this report.

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1. TR-55 Urban Hydrology for Small Watersheds, 2nd Edition, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 20250, June 1986.

2. SCS National Engineering Handbook, Section 4, (NEH-4), Hydrology, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C., 20250, August 1972.

3. Handbook of Hydraulics, Brater and King, 6th Edition, McGraw Hill Book Company, 1986.

4. Handbook of PVC Pipe Design & Construction, Uni-Bell Plastic Pipe Association, Dallas, Texas.

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6. Design Guidelines for Stormwater Management, Mesa County, Colorado.

APPENDIX "A"

HYDROLOGIC PARAMETERS

1. Runoff coefficient

The values for the coefficient of runoff for use in the Rational Method within Mesa County are as shown in Table 2-2, RECOMMENDED RUNOFF COEFFICIENTS (C). The design engineers judgment must be used to select the runoff coefficient that will best represent the end result of the development.

TABLE 2-2
RECOMMENDED RUNOFF COEFFICIENTS
(C)

Description of Area or Surface Areas		Runoff Coefficients
Business		
	Downtown	0.70 to 0.95
	Neighborhood	0.50 to 0.70
Residential		
	Single-family	0.30 to 0.50
	Multi-units, detached	0.40 to 0.60
	Multi-units, attached	0.60 to 0.75
Residential (suburban)		0.25 to 0.40
Apartment		0.50 to 0.70
Industrial		
	Light	0.50 to 0.80
	Heavy	0.60 to 0.90
Parks, cemeteries		0.10 to 0.25
Playgrounds		0.20 to 0.35
Railroad yard		0.20 to 0.35
Unimproved		0.10 to 0.30
Surfaces		
Pavement		
	Asphalt and Concrete	0.70 to 0.95
	Brick	0.70 to 0.85
Roofs		0.75 to 0.95
Lawns, sandy soil		
	Flat, 2 percent	0.13 to 0.17
	Average 2 to 7 percent	0.18 to 0.22
	Steep, 7 percent	0.25 to 0.35

Source: Mesa County Stormwater Mgmt

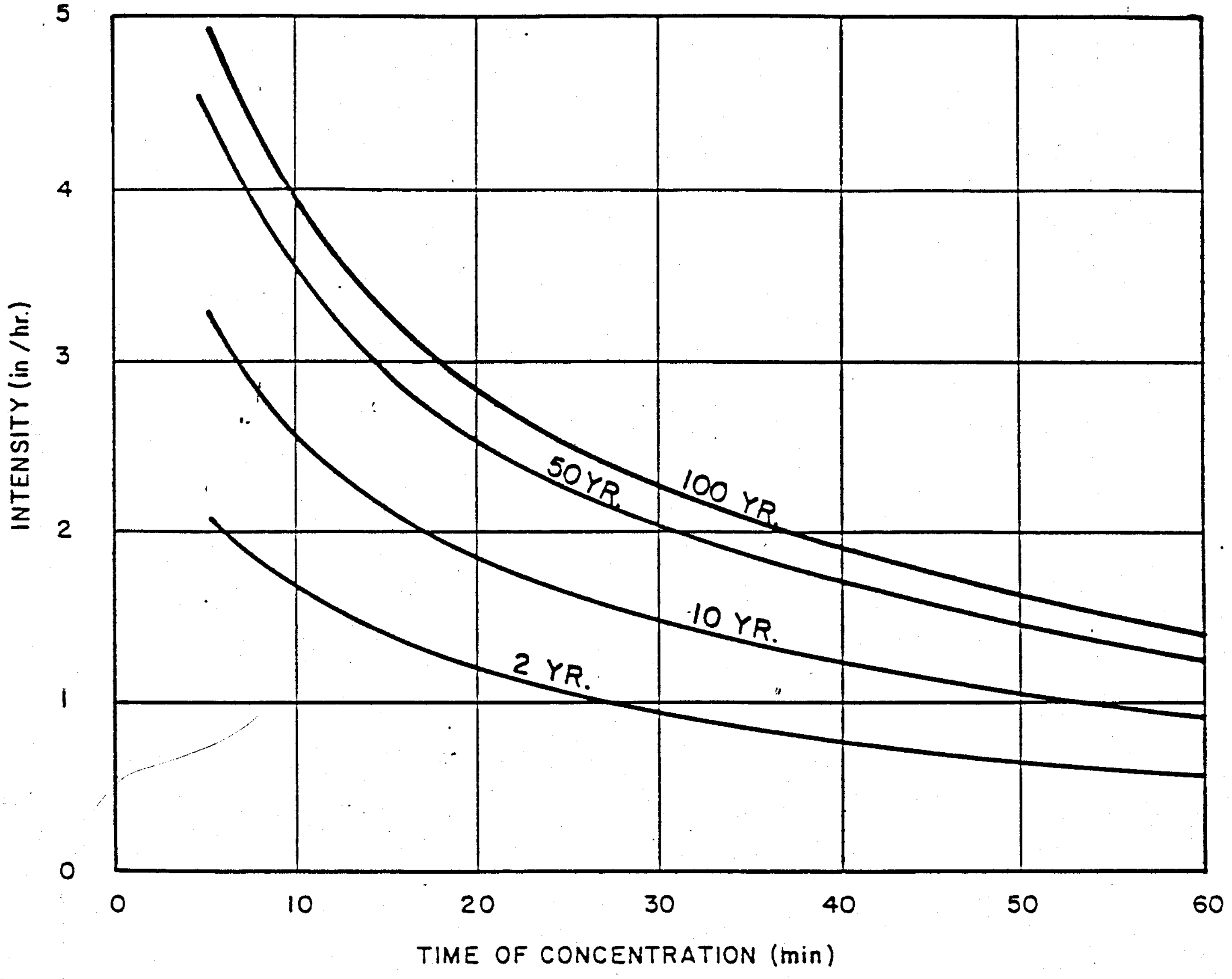

Figure 2-1 INTENSITY DURATION CURVES

APPENDIX "B"

TIME OF CONCENTRATION CALCULATIONS

Worksheet 3: Time of concentration (T_c) or travel time (T_t)

Project ______________________ By ______ Date ________

Location ______________________ Checked ______ Date ________

Circle one: Present Developed ______________________

Circle one: T_c T_t through subarea ______________________

NOTES: Space for as many as two segments per flow type can be used for each worksheet.

Include a map, schematic, or description of flow segments.

Sheet flow (Applicable to T_c only)

	Unit	Segment ID: Present	Segment ID: Developed	
1. Surface description (table 3-1)		Bare Ground	Gravel	
2. Manning's roughness coeff., n (table 3-1) ..		0.02	0.03	
3. Flow length, L (total L $\le$ 300 ft)	ft	100	80	
4. Two-yr 24-hr rainfall, P_2	in	1.0*	1.0*	
5. Land slope, s	ft/ft	.012	.010	
6. $T_t = \frac{0.007\,(nL)^{0.8}}{P_2^{0.5}\, s^{0.4}}$ Compute T_t	hr	0.071	+ .089	=

Shallow concentrated flow

	Unit	Segment ID	Segment ID	
7. Surface description (paved or unpaved)		Bare Ground	Paved	
8. Flow length, L	ft	220	390	
9. Watercourse slope, s	ft/ft	0.009	.01	
10. Average velocity, V (figure 3-1)	ft/s	0.95	2.0	
11. $T_t = \frac{L}{3600\,V}$ Compute T_t	hr	0.064	+ .053	=

Channel flow

	Unit	Segment ID	Segment ID	
12. Cross sectional flow area, a	ft^2			
13. Wetted perimeter, p_w	ft			
14. Hydraulic radius, $r = \frac{a}{p_w}$ Compute r	ft			
15. Channel slope, s	ft/ft			
16. Manning's roughness coeff., n				
17. $V = \frac{1.49\, r^{2/3}\, s^{1/2}}{n}$ Compute V	ft/s			
18. Flow length, L	ft			
19. $T_t = \frac{L}{3600\,V}$ Compute T_t	hr	X	+	= .135 (Pre)
20. Watershed or subarea T_c or T_t (add T_t in steps 6, 11, and 19)	hr			.142 (Post)

T_c = 0.135 hr = 8.1 min (Pre)

T_c = 0.142 hr = 8.5 min (Post)

* Based on NOAA ATLASS II

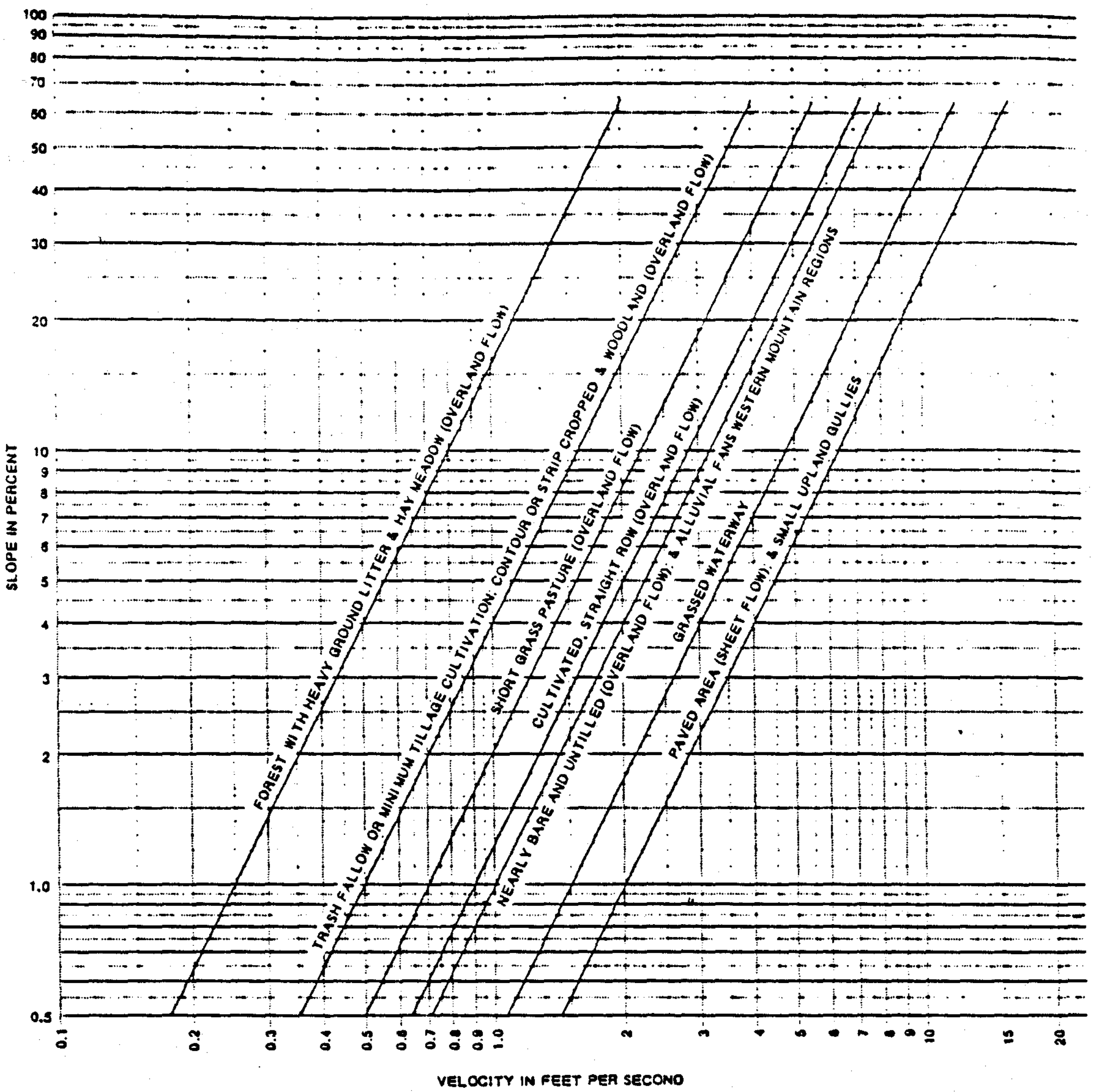

Figure 15.2.—Velocities for upland method of estimating T_c

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