E21/D8

6 PLANS

ACADEMY HILLS
UNITS 4 - 6
DRAINAGE MANAGEMENT PLAN

PREPARED FOR:

[illegible]MHONE

by

COMMUNITY SCIENCES CORPORATION

OCTOBER 1977

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TABLE OF CONTENTS

		Page
A)	PURPOSE AND SCOPE	1
B)	SITE LOCATION AND TOPOGRAPHY	1
C)	DESIGN CRITERIA	
	1) Engineering Parameters	1
	2) Flood Control Regulations	2
D)	COMPUTATIONAL PROCEDURES	2
E)	OFF-SITE DRAINAGE	2
	1) Ultimate Plan	2
	2) Intermediate Plan	3
F)	ON-SITE DRAINAGE	3&4
	TABLES	
	Table I - Flow Rates	5
	Table II - Street & Channel Capacities	6
	PLATE 1 - Location Map	
	PLATE 2 - Offsite Drainage Areas	
	PLATE 3 - Upland Drainage Temporary Management Plan	
	PLATE 4 - Drainage Plan	
	PLATE 5 - Typical Plot Plan - R1	
	PLATE 6 - Typical Plot Plan - Fourplex	
	PLATE 7 - Typical Plot Plan - Townhouse	
	Appendix A - Calculations	7,8,&9

A) Purpose and Scope

Amhome, a joint venture of Home Planning Development Company and Affiliated Mortgage & Development Company is currently planning development of units 4 thru 6 of their Academy Hills Properties. The purpose of this report is to present a drainage management plan for units 4 thru 6, which is acceptable to the City of Albuquerque and to the Albuquerque Metropolitan Arroyo Flood Control Authority.

Since the lands under study are in various stages of subdivision and re-planning, the scope of work for this study varies for different sections of that parcel. For those areas where preliminary interior plats have been devised, detailed drainage patterns are defined. For those areas where nothing more than a specific type of land use has been chosen, drainage is treated in very general terms.

B) Site Location and Topography

Academy Hills Units 4 thru 6 are located in northeast Albuquerque immediately northwest of the proposed intersection of the extensions of Eubank and Juan Tabo Boulevards (see Plate 1). The land slopes from east to west at an average rate of 4%. Soils consist largely of alluvial sands and silts, and decomposed granite is the major soil constituent.

(1) Unit designation refers to those given in Academy Hills land division Masterplan as originally formulated by CCIC in 1976.

C) Design Criteria

(1) Engineering Parameters

For calculation of runoff rates & required storage volumes a C of 0.78 has been used for developed areas, and a C of 0.4 was used for undeveloped areas.

All runoff calculations have been based on a 100 year-6 hour rainfall of 2.6" (0.22') per AMAFCA requirements.

Rate of runoff calculations have been based on the frequency - intensity - duration relationship for a 100 year storm as presented by Gordon Herkenhoff and Associates in their 1963 Master Plan of Drainage for the City of Albuquerque. This relationship is expressed by the following equation: $I = 189/(Tc+25)$

1. Unit designation refers to those given in Academy Hills land division Masterplan as originally formulated by CCIC in 1976.

(2) Flood Control Regulations

The drainage plan presented in this report has been designed to comply with the 1972 AMAFCA Resolution in regard to rate and volume of runoff leaving the site. That Resolution has been interpreted to say that the rate and volume of runoff allowed to leave the site after development shall be no greater than the rate and volume running off prior to development.

D) Computational Procedures

Appendix A contains samples of the various types of hydraulic calculations performed. Proposed easement channels were sized based on the Manning equation for uniform flow. Street carrying capacities were calculated by means of the same equation using City of Albuquerque Standard Street Sections.

Hydrological flow rate calculations are based on the rational runoff method. Times of concentration were calculated using the Kirpich nomagraph for overland flow and the Manning equation for estimating street flow velocities. For flow convergence points having both developed and undeveloped areas contributing, composite runoff coefficients were calculated. Volume calculations for sizing individual lot ponds and for determination of overall ponding requirements for areas 1, 2, 5, and were performed by multiplication of the applicable C factor times the 100 year - 6 hour rainfall (0.22) times the appropriate area to obtain cubic feet of water.

E) Off-Site Drainage

(1) Ultimate Plan

Areas A, B, and C drain toward and onto the site from the north & east (See Plate 2). Area C will drain naturally into Academy Road when that arterial is constructed, and runoff will thus be channelized westward to the inlet structure located opposit the previously platted Unit 2. Runoff from Area A tends to collect in a well defined arroyo located just south of the Eubank-Academy intersection Since Eubank will drain southward from said intersection into the Bear Canyon Tributory, much of the flows generated by Area A can no doubt ultimately be conveyed along Eubank and into the Bear Tributory. However, the currently undefined alignment of the easterly extension of Academy Road will have a significant effect on the future flow patterns of Area A runoff. In Addition, development in Area A could have further

effects on said drainage patterns. For these reasons, the exact method of ultimately managing Area A runoff will be left to future studies. However, it can be stated with assurance that Eubank will have the capacity to handle at least 50% of these flows.

Area B, once developed, can discharge onto Eubank Boulevard since that conveyance can easily carry the resultant flows into the Bear Canyon Tributory.

(2) Intermediate Plan

Until such time when development plans are conceived for Areas A & B, temporary measures to divert, control, and manage flows and silt must be adopted. Construction of a berm across the Area A arroyo system in the location of Eubank extension will create a retention pond in that arroyo of adequate size to contain all the estimated runoff occuring at that point. In addition, when Eubank is to be paved this berm if constructed properly can serve as the road embankment itself since the roadbed will need to be elevated several feet above the arroyo bottom.

Runoff from Area B will be diverted southward into the Bear Canyon Tributory by a small graded earthen channel to be located just east of the Eubank ROW. This channel will prevent silt from washing onto Eubank Blvd, and it will be abandoned when development of lands east of Eubank takes place.

F) On-Site Drainage

Plate 4 illustrates the proposed drainage patterns for on-site runoff. Also shown, are the estimated ponding requirements for those areas not having a site plan. Areas marked "non Contributary" are those portions of the lots which will drain directly into back yard ponds. Plates 5 - 7 depict the typical plot plans for R1 single family, R2 fourplex and SU-1 townhouse type units respectively indicating required retention pond dimensions. The townhouse area will be sold to builders in parcel sizes to suit their individual needs. Although it is anticipated that virtually the entire townhouse area will be developed similarly to what's shown on Plate 7, the possibility exists that someone will wish to devise an alternate site plan. The owners therefore agree to put the requirement into

their sales contract that all townhouse developers must submit their plot plans to City Engineering for approval. Should said plan deviate from what's shown on Plate 7 significantly an additional individual drainage report will be required for the area.

Table 1 itemizes flows for several key points of flow confluence while Table 2 describes the proposed channel and/or street section geometries & flow capacities.

T A B L E I

FLOW RATES

Area/Point Designation	Area (Acres)	Areas Contributing	Gross Area (Acres)	Composite C	TC (min)	Inches /Hour	Q (cfs)
A	82.2	A	82.2	0.4	19	4.30	141
B	30.3	B	30.3	0.4	16	4.61	56
C	11.5	C	11.5	0.4	<10	5.4	25
1	10.15	1	10.5	0.4	<10	5.4	22
2	2.69	2	2.69	0.4	<10	5.4	6
3	1.57	2,3	4.26	0.54	<10	5.4	12
4	4.24	2,3,4	8.5	0.66	<10	5.4	30
5	1.07						
6	2.20	6	2.2	0.78	<10	5.4	9
7	3.65	6,7	5.85	0.78	<10	5.4	25
8	2.21	6,7,8	8.06	0.78	<10	5.4	34
9	6.38	1,9	16.53	0.55	<10	5.4	49
10	9.04	C,1,6-10	45.13	0.60	11	5.25	142
11	5.5*	C,1,6-10 7-1,7-3	50.63	0.62	12	5.1	160

* consists of areas 7-1 and 7-3 of Academy Hills Master Plan of Drainage dated July, 1976.

T A B L E 2

STREET & CHANNEL CAPACITIES*

Point Designation	Flow Rate(cfs)	Street (ft.)	Slope (ft/ft)	Bottom Width(ft.)	Depth (ft.)	Capacity (c.f.s.)
B	56	48	.035	NA	0.67	187
3	12	NA	.03	9	0.5	52
4	30	32	.03	NA	0.5	77
4	30	NA	.06	9	0.5	74
7	25	32	.03	NA	0.67	151
8	34	40	.049	NA	0.67	194
9	49	32	.02	NA	0.67	123
10	142	48	0.0073	NA	1.0	**216

Notes:* All flow calculations based on Manning formula for uniform flow. n taken as 0.015 asphalt and 0.013 concrete. Flow capacities at easements based on standard City of Albuquerque easement channel.

** Flow Capacity Based on City ROW grading standards of 0.33 rise from top of curb to ROW line. Flow Capacity assumes flow depth to ROW elevation.

A P P E N D I X A
Calculations

(1) Ponding - R-1 Lot

Typical lot = 70 X 110

Distance to ℄ street = 25'

∴ Typical design drainage area = (110+25)70
= 9450 SF

Area draining to street = 65 X 70 = 4550 SF

(0.78 X 4550 = 3549) < (0.4 X9450 = 3780)

Required pond volume = 0.22 X 9450 X (0.78-0.4)
= 788 cu.ft.

(2) Ponding - Fourplex Lot

Typical lot = 90 X 113

Distance to ℄ street = 25'

∴ Typical design drainage area = (113 + 25)90
= 12420 SF

Area draining to street = 75 X 90 = 6750 SF

(0.78 X 6750 = 5265) ≈ (0.4 X 12420 = 4968)

Required pond volume = 0.22 X 12420 X(0.78-0.4)
= 1040 cu.ft.

(3) Ponding - Townhouse Lot

Typical lot = 40 X 105

Distance to ℄ street = 25'

∴ Typical design drainage area = (105+25) 40
= 5200 SF

Area draining to street = 40 X 45 = 1800 SF

(0.78 X 1800 = 1404) < (0.4X5200 = 2080)
Required pond volume = 0.22 X 5200 X(0.78-0.4)
= 435 cu. ft.

(4) Area A - Temporary Pond

Area = 82.2 acres

Volume required = 0.22 X 0.4 X 82.2 X 43560
= 315,100 cu.ft.

Volume - increment table

CONTOUR	AREA	VOL	CUM VOL
4	3600		
6	14800	18400	18400
8	30400	45200	63600
10	51600	82000	145600
12	64400	116000	261600
14	76800	141200	402800

Estimated volume at contour 13 =

$$261600 + \frac{(64400 + 76800)/2 + 64400}{2}$$

= 329,100 cu.ft.

(5) Academy Drop Inlet

a) Capacity
Type C inlet

Co = 0.65
A (net grate)= 0.65 X 6.37 = 4.14 SF
h (grate) = 1'
Clog factor =0.67
Q (grate) = CoA (net) $\sqrt{2gh}$ X CF = 14.5 cfs.

A gutter inlet = 0.5 X 3 = 1.5 SF
h gutter inlet = 1.2 - 0.25 = 0.95

Q gutter inlet = Co A $\sqrt{2gh}$ = 7.6 cfs.
Q total = 22.1 cfs.

b) # type C inlets required = 160/22.1
= 8 inlets

c) Capacity 36" RCP@ 1% = 66 cfs
(from nomograph)

# of 36" RCP pipes required = 160/66
= 3 pipes

(6) Flow capacity Area B Temporary Channel

Bottom width = 8'
Sideslopes = 3/1
depth = 2'
n = .03 (earth)
slope = .025

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

= 269 cfs > >56

(7) Specific Energy of Flow @ Point 7

Flow depth in gutter = 0.32'
Flow velocity in gutter = 5 fps.

Specific energy = $0.32 + 5^2/64.4$
= 0.7'

Since curb height is 8" = 0.67' and conjugal depth of flow must be less than the specific energy of flow, there is no danger of the flows jumping the curb opposite point 7 into lots 28 & 29.

PLATE 1
LOCATION MAP

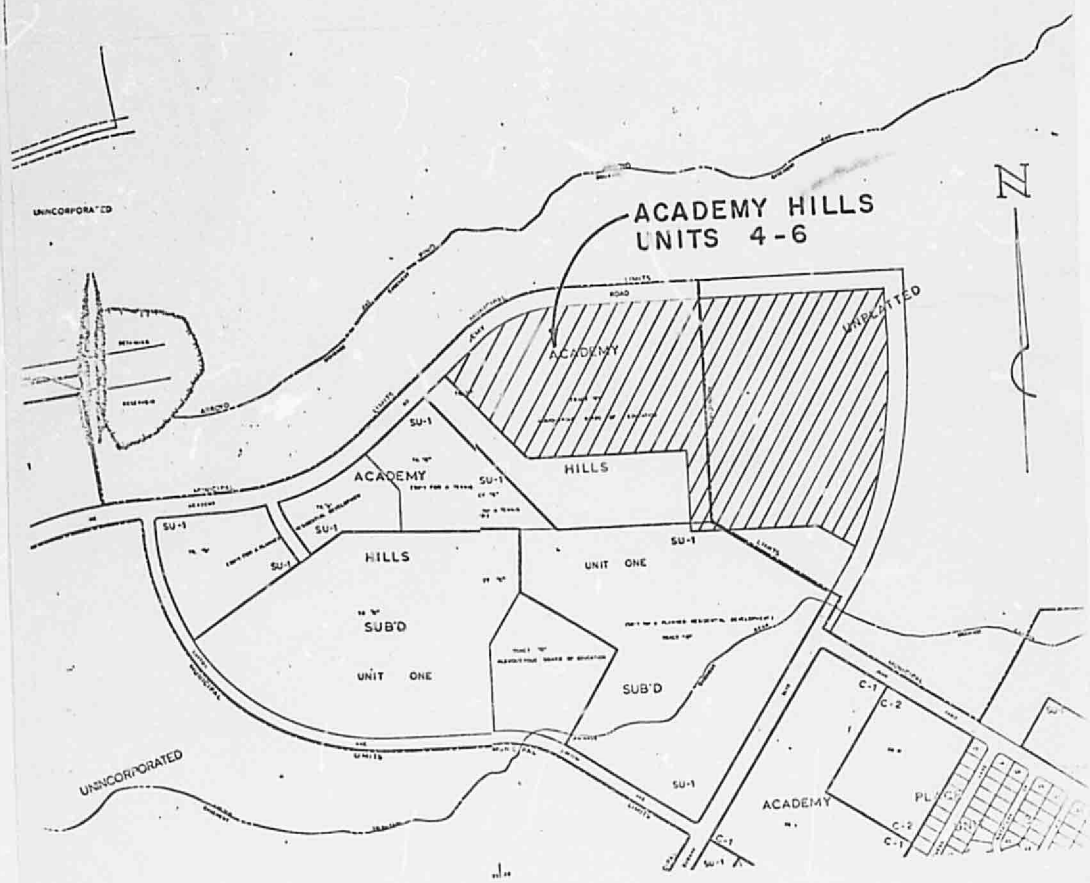


PLATE 2

OFFSITE DRAINAGE AREAS

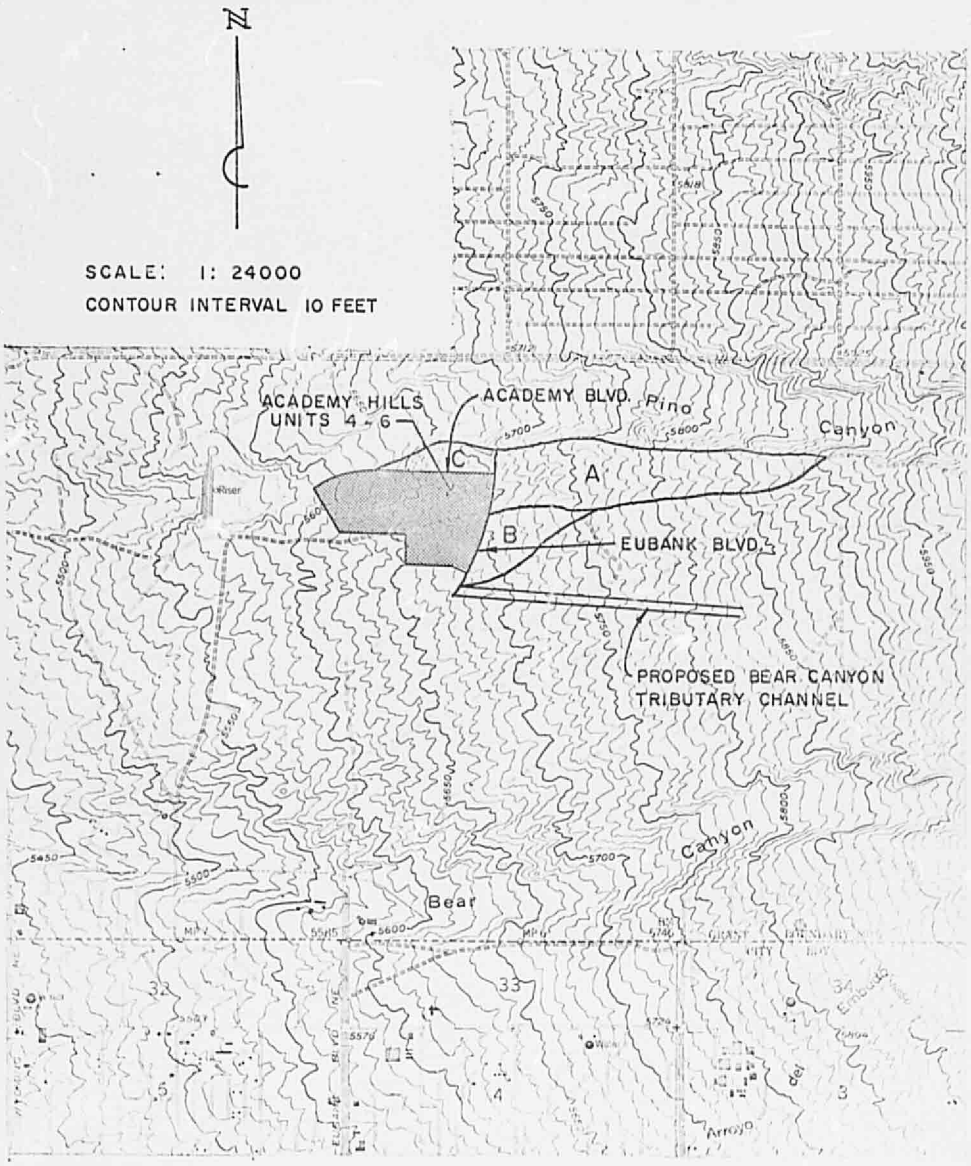


PLATE 5

TYP. PLOT PLAN
R-1 RESIDENTIAL

NTS

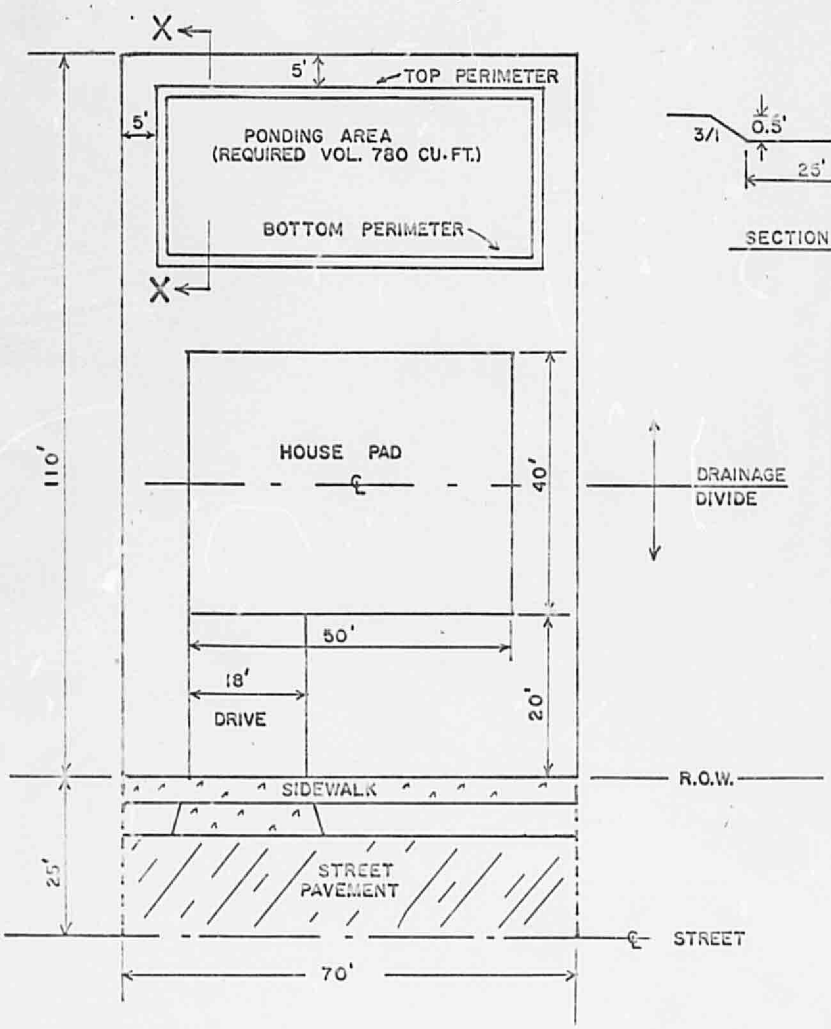
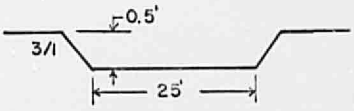


PLATE 6
TYPICAL PLOT PLAN
FOURPLEX UNIT

N.T.S.



SECTION X

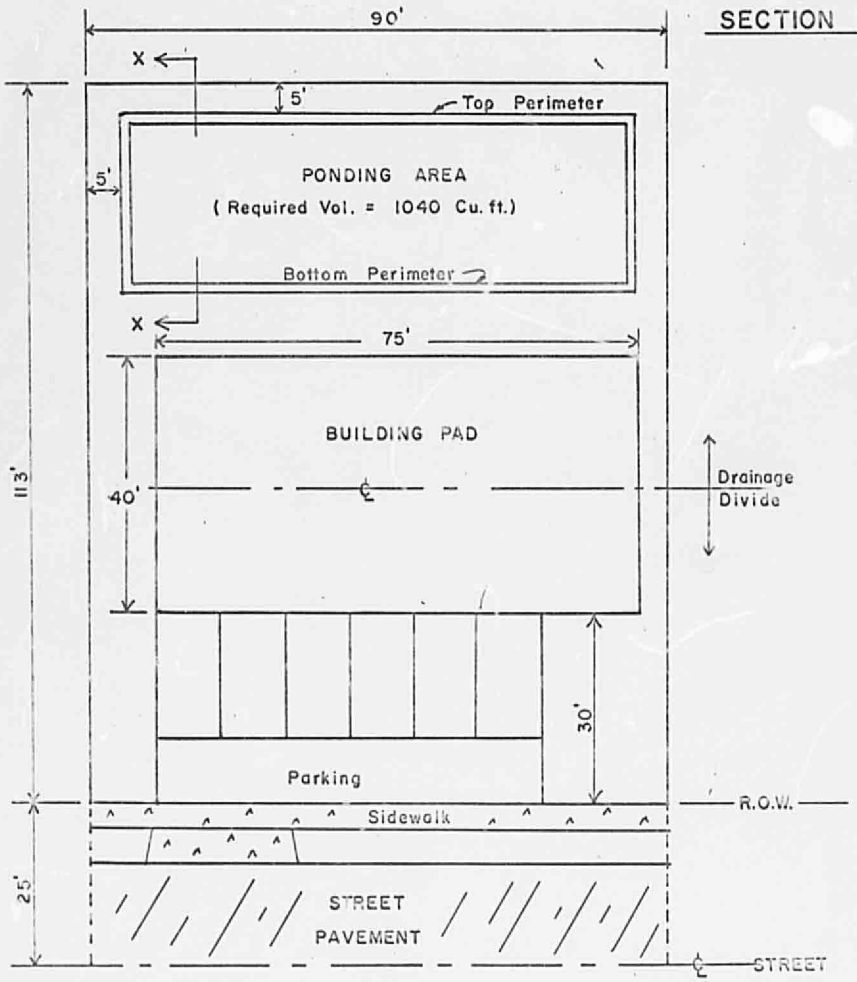
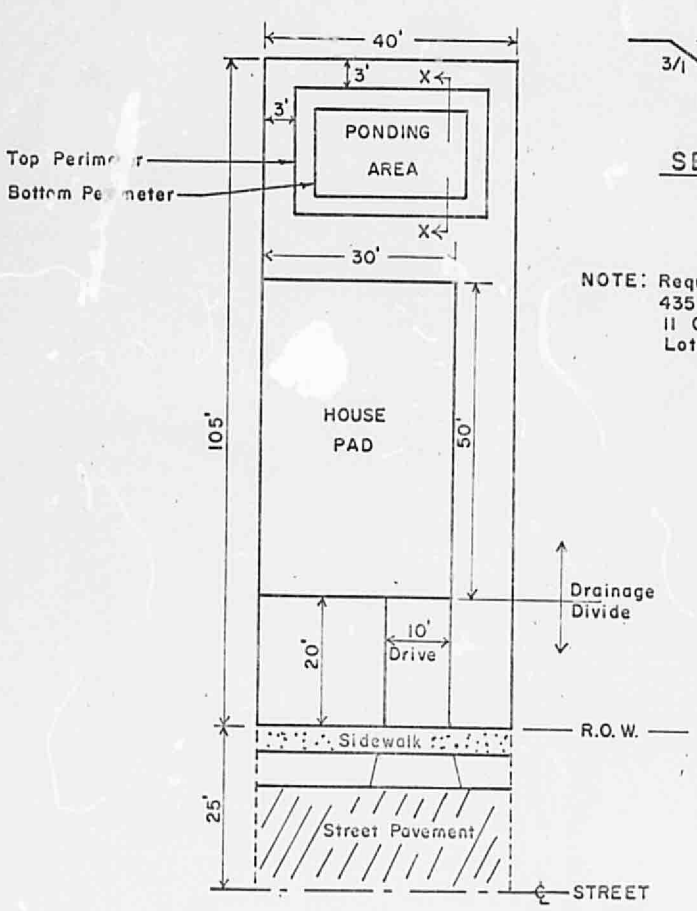
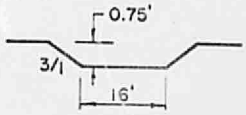


PLATE 7
TYPICAL PLOT PLAN
TOWNHOUSE UNIT

N.T.S.





SECTION

NOTE: Required Ponding Volume= 435 Cu.ft. for 40' Lot or 11 Cu.ft. per front ft. for Lots of varying width.

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DRAINAGE REPORT
FOR
EL PASEO DEVELOPMENT
(FORMERLY ACADEMY SQUARE)
ACADEMY HILLS SUBDIVISION
ALBUQUERQUE, NEW MEXICO

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(REVISED JUNE 20, 1981)

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TABLE OF CONTENTS

	DESCRIPTION	PAGE NO.
1.	PROJECT DESCRIPTION	1
2.	EXISTING CONDITIONS	1-2
3.	HYDROLOGY	3
4.	PROJECT ANALYSIS - DEVELOPED SITE RUN-OFF	3-6
5.	1977 MASTER PLAN DRAINAGE REPORT	7
6.	SUMMARY	7
7.	APPENDIX	
	A. LETTER FROM AMDEC REGARDING DIVERSION OF OFF-SITE DRAINAGE	
	B. CALCULATIONS (SHEETS 1-18)	
	C. CALCULATIONS (SHEETS A-1 THROUGH A-9)	
	D. ATTACHED DRAWINGS (L-4, L-5, L-6)	

DRAINAGE REPORT FOR
EL PASEO
(FORMERLY ACADEMY SQUARE)
ACADEMY HILLS SUBDIVISION
ALBUQUERQUE, NEW MEXICO

1. PROJECT DESCRIPTION

El Paseo is a development of 118 townhouse units of which 106 units will be built now and 8 units will be built when final area off-site drainage problems are resolved. The development is on Tract A of Academy Hills Subdivision on the southwest corner of the intersection of Eubank and Academy Boulevard, N.E. The basic site plan was approved by the Environmental Planning Commission at its September 18, 1980 meeting subject to submission of a drainage report and the working out of several minor traffic and fire department comments.

2. EXISTING CONDITIONS

The present tract is undeveloped with a large on-site holding pond which in part receives off-site water from the Academy-Eubank intersection through 2-24" CMP's from a temporary holding pond east of Eubank. The west lane of Eubank and the south lane of Academy as well as one half of the intersection have been built to final grades with curb and gutter and a concrete intersection. This work was completed in 1979 and is on record as a City of Albuquerque approved project No. P-28-18. Several related projects to this construction include Zone 6E water line (09-001-79) and Academy Hills Unit 4 (CT-1-78). An October, 1977 Drainage Study prepared for AMHOME by Community Sciences and a subsequent Academy-Tramway-Eubank Sector Development Area Master Conveyance Plan approved by the EPC and City Engineering, and the 1979 road construction define the existing and intended future off-site drainage plans. Long term drainage plans call for Eubank to be the north-south interceptor for all drainage east of the subject site with all water being diverted north on the Eubank extension to the Pino channel in the Tanoan Golf Course. It is our understanding that this improvement may be forthcoming in 1981 through an area wide assessment district being planned by AMDEC (see letter from AMDEC in the Appendix attached at the end of this report).

For purposes of this report, it is assumed that it will not be necessary to pond any of the off-site run-off on the EL PASEO site since the diversion will be constructed or AMDEC will adequately handle the upstream run-off before it reaches the subject project.

On-site existing conditions were developed in the 1977 Community Sciences Drainage Report, modified in 1979 and are as follows:

Q_{100} to Admiral Rickover = 5.81 cfs (1977)

Q_{100} to Denali = 21.92 cfs (1977)

With 1979 modifications:

Q_{100} to Admiral Rickover = 5.81 cfs (1979)

Q_{100} to Denali = 9.13 cfs (1979)

Q_{100} to Pond = 12.78 cfs (1979)

3. HYDROLOGY

The Rationale Formula was used in calculating peak flow rates based on the 100 year storm and for determining intensity duration curves for short duration storms of very high intensities (1-hour storm) which are common in the Northeast Heights. The Manning Formula was used to size underground culverts. The roughness coefficient used for corrugated metal pipe is .022. Entry, friction and velocity head losses were checked for all CMP and entry control governed in all cases.

Design aids used throughout the analysis include the following:

A. Intensity Duration Frequency Curves for the City of Albuquerque as developed by Gordon Herkenhoff & Associates.

B. 100 Year-6 Hour precipitation map for Albuquerque (1967).

C. Precipitation-Frequency Atlas of the Western United States (NOAA Atlas 2) Volume IV-New Mexico (1973).

D. Charts, graphs, and nomographs as illustrated in the Data Book for Civil Engineers, Volume 1, 3rd Edition by Seegle (1960).

E. Urban Storm Drainage Criteria Manual, Denver Regional Council of Governments (1969).

4. PROJECT ANALYSIS

The site was divided into 11 areas in order to clearly illustrate how on-site drainage is to be handled. Calculations for run-off, pond sizes, conveyance devices (pipes), etc. are contained in the Appendix bound to this report. The following descriptions of the various areas is a summary of those calculations.

Run-off rate quantities for the 100 year storm are shown on the drainage plan. However, in most cases, the short duration-high intensity storm governed the control of the design. This is reflected in the attached calculations found in the Appendix. The 1-hour storm was broken into five parts and the most severe arrangement of the storm for each separate area was analyzed. This is somewhat conservative but insures that ponds will be adequately sized for any high intensity/short duration storm.

A. Area 1
Size: 1.46 Acre
Run-off from the site is handled as sheet flow at the drive entrance from Admiral Rickover and as swale flow through two drains which pass below the sidewalk and discharge through the curb. Total discharges are as follows:

Q_{100} under sidewalk (east side Area 1) = 1.36 cfs

Q_{100} through drive entrance = 3.28 cfs

Q_{100} under sidewalk (south side Area 1) = 0.49 cfs

Total Q_{100} Developed = 5.13 cfs

Q_{100} undeveloped = 5.81 cfs $>$ Q_{100} developed, therefore no ponding is required. Total volume leaving this area would be 7742 cu. ft.

B. Area 2
Size = 0.28 Acre
One field catch basin will be utilized to intercept the run-off from this small area and convey same to a swale in Area 1. Eventually, this discharge (0.71 cfs) reaches Admiral Rickover and is included in the 1.35 cfs shown under Item A. Total volume discharged from Area 2 is 1244 cu. ft.

C. Area 3
Size = 0.25 Acre
Due to the constrictions of the confined site, a dry well will be utilized to intercept a portion of the run-off from Area 3 (126 cu. ft.). The balance of the run-off (due mainly to a short storm of high intensity) will be conveyed to the holding pond in Area 4. A maximum discharge rate of 0.57 cfs will begin entering Area 4 approximately 17.6 minutes after the storm starts and will reach the pond in Area 4 in 16 minutes thereafter. The total volume discharged to the pond in Area 4 will be about 718 cu. ft. for a short term storm and 783 cu. ft. for the 100 year storm.

D. Area 4

Size = 0.5 Acre

The majority of this area is hard surfaced and will discharge most of the rainfall falling on it. A detention pond will be utilized to slowly release the run-off from Area 4 to Area 5 and eventually through a second and third pond to Denali. Pond size was based upon the high intensity 1-hour storm. Based upon inflow rates of 0.57 cfs from Area 3 and a maximum rate of 1.63 cfs from Area 4, a pond size of 600 sq. ft. x 3 feet deep was calculated using an outflow rate of 0.4 cfs. Maximum storage capacity required = 1797 cu. ft. The capacity supplied = 1800 cu. ft. A 3.5' high chain link fence will be installed at the perimeter of the pond as it exceeds the 1.5' pond depth allowable limit. The pond in Area 4 will begin discharging to the pond in Area 5 approximately 10 minutes after the start of the short duration storm. The pond will empty in about 2.5 hours thereafter. Total volume discharged from the Area 4 pond (including the overflow from Area 3) is 3580 cu. ft. for a short duration storm (V100 = 4369).

E. Area 5

Size = 0.58 Acre

Run-off from this area will be detained in a 1020 sq. ft. x 1.5 foot deep pond. This pond will also receive discharge from the pond in Area 4. The size of the pond was based upon the high intensity short duration storm producing maximum run-off rates. Due to the topography of the area, a 3.0' high retaining/garden wall will surround the pond even though the maximum water depth will only reach 1.5 feet. No fence will be required. Maximum discharge to the pond from Area 5 will be 2.7 cfs after 16.5 minutes. Discharge from the pond will be at Q = 0.65 cfs and will flow to the retainage pond in Area 6. At this rate, the maximum pond capacity must be 1494 cu. ft. The pond is designed for 1500 cu. ft. of storage. The total discharge from this pond will be 7468 cu. ft. which also includes the discharges from Areas 3 and 4. The pond in Area 5 will completely drain after 3.5 hours. (See Sheet 11 of the calculations contained in the Appendix).

F. Area 6

Size = 3.83 Acres

The run-off from Area 6 will eventually drain to Denali, an existing street located to the west of the subject site. In order to reduce discharge rates, two Type C drop inlets will be installed to pick up about 61% of the street run-off and convey it to a detention pond. About 6 cfs of the total 9.8 cfs (100 year rate) will be diverted from the street to the detention pond. (See Sheets 13 and 14 of the calculations contained in the Appendix).

The detention pond was sized based upon a 1-hour high intensity storm. Total detention required due to run-off from Area 6 and inflow from Area 5 is 11,023 cu. ft. Total pond capacity is 9690 cu. ft. The excess (1333 cu. ft.) will discharge through a 4' wide by 1' deep notch in the top of the pond wall at a varying rate of 2 to 4 cfs. The pond will have an area of 1615 sq. ft. and a depth 6'. A 6 foot high chain link fence will be installed on the perimeter of the pond. It should be noted that the 4 cfs rate noted above will last for less than 1.7 minutes and will fall off rapidly to an average rate of about 2.8 cfs.

The detention pond will discharge to an internal street and eventually to Denali via a 12" CMP at a rate of 2.5 cfs. The pond will empty itself in approximately 3.7 hours. Total discharge from the pond will be 28,317 cu. ft. Additional direct discharge from Area 6, i.e. that run-off not intercepted by the drop inlets, will be 5103 cu. ft. at a maximum rate of 3.67 cfs. This rate will occur for a very brief period, namely 3 to 5 minutes. (The average rate Q_{100} will be approximately 3.8 cfs). The total (short storm) discharged volume will be 32,357 cu. ft. V100 total discharge = 39,719 cu. ft.

G. Area 7

Size = 2.28 Acres

Discharge from Area 7 will flow down the internal street system to Denali. Due to the long time of concentration (14.8 minutes), the short storm maximum discharge to Denali is about the same as Q_{100}, namely 6.43 cfs. (See Sheet 8 of the calculations in the Appendix). The Q_{100} rate was therefore used to determine the total discharge rate to Denali from both Areas 6 and 7. This is summarized below:

Q from Area 6 (pond)	=	2.5 cfs	
Q from Area 6	=	3.8 cfs	
Q from Area 7	=	5.91 cfs	
Q from Area 9	=	1.87 cfs	(See pages 5 & 6)
Q from Area 11	=	0.66 cfs	(See page 6)

Maximum Discharge to Denali = 14.74 cfs

This rate (14.74 cfs) is less than the Q_{100} of 21.92 cfs proposed in the 1977 Community Science's Drainage Report referred to earlier. Total volume discharged to Denali over a 3.7 hour period (based upon a high intensity short storm) would be 46,335 cu. ft. V100 discharged to Denali will be 55,082 cu. ft.

H. Area 8

Size = 0.39 Acre

Undersidewalk drains will convey approximately 1900 cu. ft. of run-off at a combined discharge rate of 1.10 cfs directly to Academy along the north side of the property. The maximum discharge rate for a high intensity storm is 1.5 cfs and is still well within reason for direct discharge to Academy. No detention ponding is deemed necessary.

I. Area 9

Size = 0.55 Acre

Based upon a short term high intensity storm, the maximum discharge rate (for a period of 5 minutes or less) will be 2.66 cfs and the total volume discharged to Denali will be 2743 cu. ft. over the length of the storm. Both the maximum Q and volume discharged are such that detention ponds are not required. V100 discharged to Denali will be 3242 cu. ft. The 100 year rate (Q) 1.87 cfs was used as the reasonable runoff rate expected to reach Denali and is reflected in the table under letter G.

J. Area 10

Size = 2.75 Acres

Area 10 was subdivided into three smaller sections to better define the run-off. Area 10A flows directly to Academy through drains under the sidewalk. Maximum Q is about 0.91 cfs and the volume discharged is 2770 cu. ft.

Area 10B flows directly to Eubank at a maximum discharge rate of 0.68 cfs. The volume discharged is 1182 cu. ft. These are 100 year rates and volumes.

The remainder of Area 10 drains to an on-site detention pond. The pond's size was based upon 100% retention of a 1-hour high intensity storm with a slow release to Academy at Q = 2.2 cfs. The pond volume is 9896 cu. ft. at a depth of 0.99. Total volume of water discharged to Academy will be 9896 cu. ft.

It should be noted that the pond area noted above will be developed at a later date. Detention ponds will be sized to handle the run-off generated by this future expansion.

K. Area 11

Size = 0.27 Acre

This area will drain to the internal street system and eventually to Denali. The maximum Q will be 0.66 cfs and the volume discharged will be 1147 cu. ft. (This rate and volume were included in the calculations for the run-off to Denali under Item G for Area 7). No detention was provided due to the low flows and volumes involved.

5. 1977 MASTER PLAN DRAINAGE REPORT

It should be noted that the Drainage Report prepared for the subject area by Community Sciences Corporation for AMHOME in 1977 was considered in the preparation of this report. Many of the statements contained in the Community Sciences' report are very general in nature and therefore could not be strictly followed. A composite C of 0.78 presented in the CS report for developed areas is for example too high for many of the sections of the proposed development. Separate C factors were developed for each drainage area and are contained on Sheet A-1 of the Appendix. The requirement for detention/retention ponding suggested by the CS report has been closely met. A total of 0.99 Acre feet of water will be temporarily detained on the site in lieu of the 1.073 suggested by the earlier report.

6. SUMMARY

Detention ponding with controlled release has been provided for those areas of the project which could overtax the capacities of surrounding streets. Direct discharge to Academy and Eubank has been utilized where such discharge was feasible and warranted. It is felt that the proposed drainage plan outlined in this report and shown on the attacned drawings labeled L-4, L-5 and L-6, complies with the intent of Resolution 72-2 as adopted by the City of Albuquerque.

A summary of volumes by Area, runoff rates and total detention volume for the site is shown at the end of the Appendix, Sheets A-7, A-8, and A-9.

affiliated mortgage & development company amdec

a wholly owned subsidiary of Albuquerque Federal Savings and Loan Association

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AM - 742

February 18, 1981

Mr. Richard S. Heller, P.E.
City Engineer
City of Albuquerque
P. O. Box 1293
Albuquerque, New Mexico 87103

Subject: Drainage Swale North of the Intersection of Academy Road N.E. and Eubank Blvd., N.E.

Dear Dick:

The intent of this letter is to confirm our conversation in your office this morning during our meeting with Dick Vaughn and Dave Kochert.

As we discussed, plans are currently being prepared for the construction of a 10 year storm sewer and a 100 year earth channel section from subject intersection north to a desilting pond. These inprovements are scheduled for construction this summer (approximate start date is July 1, 1981) in a City of Albuquerque Special Assessment District.

It is my understanding that you will require at least the earth section north from subject intersection to be excavated before the City will issue occupancy permits for any of the Grupe development southwest of this intersection.

AMDeC shall excavate subject earth channel section at the time it is required for occupancy permits if this work has not already been performed as part of the above noted Special Assessment District.

I hope this is what you needed. If not please contact me immediately at 883-3501.

Thank you for your help in this situation.

Sincerely,

AFFILIATED MORTGAGE & DEVELOPMENT COMPANY

Allan L. Whitesel, P.E.
Chief Engineer

c.c. Richard Vaughn & David R. Kochert

SUPPLEMENT TO RGV. DRAINAGE REPORT

Hardsurface = 0.66 Acre (from RGV report, page 1)

AREA 1

①	1/2(65)(35) =	1138
②	1/2(8)(112+66) =	712
③	1/2(72+85)(20) =	1570.
③'	1/2(72+54)(18) =	1134.
④	120(190) =	22800.
⑤	190(150) =	28500.
⑥	1/2(171)(80) =	6840.
⑦	1/2(80+44)(18) =	1116.
	TOTAL AREA 1	63810 ϕ = 1.46 Acre

Landscaped: 1.46 − 0.66 = 0.80 Acre.

$Q = CiA$ $\quad C_{ave} = 0.95\left(\frac{.66}{1.46}\right) + .24\left(\frac{.80}{1.46}\right) = 0.56$

From RGV report, page 1, T_c = 10 Min.; i_{100} = 5.4 in/hr. $\quad i_{10}$ = 2.7 in/hr

$\therefore Q_{100} = .56(5.4)(1.46) = 4.42$ cfs $\quad Q_{10} = .56(2.7)(1.46) = 2.21$ cfs

$V_{100} = 7742 \text{ ft}^3$

$Q_{100\ \text{undevel.}} = 5.81$ cfs (RGV report, page 2)

For Area 1, No ponding required since $Q_{100\ \text{devel}} < Q_{100\ \text{undevel.}}$

Q to Rickover under Sidewalk:

$2480\phi + 720 + 1200 = 4400\phi$ hard surf.

$30(20) + 160(15) + 30(15) + (35)^2 = 4675\phi$ lawn $\quad \Big\rangle 9075\phi$

Ave C: $C_{ave} = \frac{4400}{9075}(.95) + \frac{4675}{9075}(.24) = .58$

$Q = .58(5.4)(.21) = 0.65$ cfs

$Q_{100} = .58(2.7)(.21) = 0.33$ cfs.

From Catch Basin (Sheet 2)

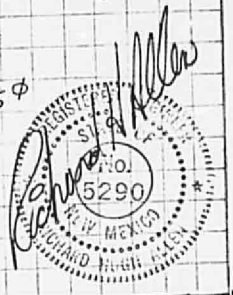
$Q_{100} = 0.71$ cfs

$Q_{10} = 0.36$ cfs

TOTAL To Rickover under S.W.:

$Q_{100} = .65 + .71 = 1.36$ cfs

$Q_{10} = .33 + .36 = 0.69$ cfs



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JOB ACADEMY SQUARE JOB # 8042

BY RH ALLEN DATE 5/7/81

CHK Revised DATE 6/19/81

SCALE SHEET NO. 1 OF 18

AREA 2 (Localized drainage to Catch Basin)

Total Area

① 12(48) = 3456
② ½(60)(22) = 660
③ ½(50+32)(60) = 2460
④ ½(85+120)(55) = 5637
TOTAL = 12213 ϕ = 0.28 Acre

Hard Surf. = 3710 ϕ
Lawn = 8503 ϕ
TOTAL = 12213 ϕ

Ave. C (composite) = 0.47 (See Sheet A-1 in Appendix)

Q_{100} = (5.4)(.47)(0.28) = 0.71 cfs
Q_{10} = (2.7)(.47)(0.28) = 0.36 cfs
V_{100} = 2.6/12 (12213)(.46) = 1244 ft³

SINCE AREA 1 & AREA 2 go to Rockover:

Total Q_{100} = 4.42 + 0.71 = 5.13 cfs < 5.31 OK
Total Q_{10} = 2.21 + 0.36 = 2.57 cfs

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SURFACE under S.W. To the South: (Part of Area 1)

Total Area = 9120 ϕ
Hard = 2465

Ave c = 2465/9120 (.95) + 6655/9120 (.24) = .43

Q_{100} = .43(5.4)(.21) = .49 cfs
Q_{10} = .43(2.7)(.21) = .25 cfs

Q_{100} Out driveway = 4.42 − .49 − 0.65 = 3.28 cfs
Q_{10} Out driveway = 2.21 − .33 − .25 = 1.63 cfs

AREA 3 (TO DRYWELL)

Hard Area = 3580 ϕ
Landscaped = [(50)(47) + ½(60)(105+127) + ½(33)(74)] − 3580 = 6951 / 10531 ϕ
Calc (composite) = 0.49 (See Sheet A-1 in Appendix)

*Herkenhoff maps

Use Tc = 10 min i_{100} = 5.4 i_{10} = 2.7

Q_{100} = .49(5.4)(.24) = .64 cfs
Q_{10} = .49(2.7)(.24) = .32 cfs

V_{100} = (25*/12)(10531)(.49) = 1118 ft³
V_{10} = (1.25*/12)(10531)(.49) = 537 ft³

Drywell Volume = (π)(12²)(10) = 1256 ft³
Vol/area to contour 96 : [5(35) + 20(4) + ½(25)(13)](.5) = 209 ft³
Overflow to Area 4 = 1118 − 335 = 783 ft³

100

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SCALE SHEET NO. 2 OF 8

Overflow to Area 4 = 526 - 335 = 191 ft³
10

For 1 hr. storm, V = .85 (2.6) = 2.2 "
5 MIN 7.68"/hr. 5 MIN 3.12"/hr, 30 MIN = 0.92"/hr.
5 MIN 4.20"/hr 15 MIN 1.96"/hr.

8" pipe @ .013%

$Q = \frac{1.49}{.019} (.913)^{1/2} (.1625)(.342)^{2/3} = .91$ cfs

$R = \frac{.1963}{1.57} = .125$

Available for discharge

NOTE: For 1 hour storm discharge to AREA 4 will start as follows:

				To ④
	After 15 MIN	Q = 0.23 ft³/sec	Vstore = 207 ft³	
0.91	Next 5 MIN	Q = 0.91 ft³/sec	Vstore = 128 ft³	145
	Next 5 "	Q = 0.50		150
	Next 5 "	Q = 0.37		111
	" 30 "	Q = 0.11		198
			TOTAL 17.6 MIN	604

Discharge to AREA 4 starts after 17.6 MIN.
Time to reach pond = 17.6 + 16 = 33.6 MIN.
(Area 4)

AREA 4

TOTAL AREA = 40(30) + 120(44) + 90(170) = 21780 ft² = .5 Acre
Hard Surface: 3700 + 1056 + 3(528) + 780 + 26(195) + 25(25)(3) + 3200 = 15965 ft² = .37 Acre

Ave C: $.95\left(\frac{15965}{21780}\right) + .24\left(\frac{5815}{21780}\right) = 0.76$

$Q_{100} = (.76)(5.4)(.5) = 2.05$ cfs
$Q_{10} = 1.03$ cfs.

$V_{T_{100}} = 2.6(21780)/12(.76) = 3586$ ft³

1 Hr Storm:

		Vol
5 MIN	Q = 2.91 cfs	873 ft³
" "	= 1.60 "	480
" "	= 1.18 "	354
15 "	= 0.74 "	666
30 "	= 0.35"	630
		3003 ft³

w/ 8" CMP from D.I. capacity = 1.63 cfs, ponding will occur during about 5 minutes of storm. Pond available

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JOB ACADEMY SQUARE JOB # 8042
BY R.H. ALLEN DATE 5/8/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO. 3 OF 18

To contour 97, will hold 600 ft³
In 5 Minutes (worst) Vol = 873,
Runoff by 8" pipe = 490 ft³
Storedge req'd = 384 ft³ ≅ 600 ft³ OK

Note: Overflow will occur to pond at curb when street pond elev = 696.8

Q_{100} @ street:
(21780 − 6300) = 15480 ϕ
Q_{100} = .76(5.4)(.36) = 1.48 cfs.

AREA 5
Gross Area: 145(51) + ½(85)(147+110) + 136(38) + 17² + 73(18) = 25088 ϕ
Hard Surface:
1038 + 1110 + 828 + 2484 + 3700 + 750 = 9910 ϕ
Landscaped: 25088 − 9910 = 15178

C_{ave} (composite) = 0.53 (See Sheet A-1 IN Appendix)

Slope = 9/235 = .038
∴ Tc = 16.5 Min
S_{100} = 4.5

Q_{100} = .53(4.5)(.58) = 1.38 cfs
Q_{10} = .53(2.25)(.58) = 0.69 cfs
$Volume_{100}$ = (25088)(2.6/12)(.53) = 2881 ft³

PONDS for Area 4: (See also Sheet A-4) (0.4 cfs 6" Pipe Outlet)

Inflow/outflow for 1 Hr Storm:		VT		Stor. Req'd
15 Min	Q = 0.74	666	360	306
5 Min	Q = 2.91	873	120	753
5 "	= 1.6	480	120	360
5 "	= 1.18	354	120	234
30 "	= 0.35 (30 Min)	864	720	144
	+ .15 (26 Min) overflow		Min Pond =	1797 Ft³

Pond area = 570 ϕ
Depth = 1797/600 = 2.99' Use 3'
B.P. Elev = 91.0
T. Wall = 94.0

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JOB Academy Square JOB # 8042
BY RHALLEN DATE 5/8/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO. 4 OF 18

POND FOR Area 5: (1 Hr Storm, Rainfall = .85(2.6) = 2.2

				Cum. Run:
15 Min	i = 1.96 "/hr.	Q = 0.60 cfs		5 0.64"
5 Min	i = 2.68 "/hr.	= 2.36		10 0.99"
5	4.20	= 1.29		15 1.25"
5	3.12	= 0.96		30 1.74"
30	0.92	= 0.28		60 2.20
			.65 cfs	

T	Q in (Total)	Vol (G)	Vout	Vstorage
15	0.6 + .4 = 1.00	900	585	315
5	2.36 + .4 = 2.76	828	195	633
5	1.29 + .4 = 1.69	507	195	312
5	0.96 + .4 = 1.36	408	195	213
30	0.28 + .4 = 0.68	1224	1170	54
		3867	TOTAL REQ'D =	1527 ft³

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Area @ bottom = 1020 ϕ

Depth = 1.497' Say 1.5' (No fence req'd.)

Due to topo, Make top of wall @ 93.0

Bot. Pond = 90.0

AREA 6

GROSS AREA

①	58 X 60	= 2280
②	140 X 35	= 4900
③	½ (197 + 260)(190)	= 43415
④	(170)(145)	= 24650
⑤	½ (240 + 188)(95)	= 20330
⑥	230 (50)	= 11500
⑦	½ (230 + 285)(50)	= 12875
⑧	½ (34)(20)	= 340
⑨	½ (250)(100)	= 12500
⑩	90 (250)	= 22500
⑪	58 (195)	= 11310
	TOTAL	= 166600 ϕ = 3.825 Acre

Hard Surface.

ABC	2 (3700) =	7400
AED	3 (3580) =	10740
A&C&E	5 (780) + 3336 =	7236
our.	36 (240) =	8640
	Roads (26')(1045) =	27170
	Pads, Drives =	15864
	Sidewalks = 1325 (4) =	5300
	Misc.	3000
	Club House/Patio	6500
	TOTAL	91850 ϕ = 2.109 Ac

Can be developed = 166600 − 91850 = 74750 ϕ = 1.716 Acre.

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JOB ACADEMY SQUARE JOB # 8042
BY R.H. ALLEN DATE 5/8/81
CHK REVISED DATE 6/19/81
SCALE SHEET NO. 5 OF 18

Composite Runoff factor "C":
C_{ave} (composite) = 0.61 (See Sheet A-1 IN Appendix)

Misc. Data to be used in calcs that follow:
100 Year 6hr precip = 2.6" (Herkenhoff maps)
1 Hr. precip = 2.2"
Tc Calculation:
$\frac{9.9}{275} = 3.6\%$ Tc = 17.5 Min (Overland Flow)
$\frac{(7.00-6.87)}{400} = 3.25\%$ Tc = 2.1 Min (See Sheet A-5 IN Appendix for calcs
Total Tc = 19.6 Min
From intensity/frequency curves I = 4.2"/hr.

$\therefore Q_{100} = (.61)(4.2)(3.825) = 9.8$ cfs
$Q_{10} = 4.7$ cfs

Vol. = 2.6 (c)(A)/12 = 2202 ft³
$Vol._{100} = 11011$ ft³

Analyze 1 hour storm:
.85(2.6) = 2.2"

T	S	Q = CiA = .61(i)(3.825)	V = 5376
5	7.68	17.92 cfs	
5	4.20	9.80	2940
5	3.12	7.28	2184
15	1.96	4.5	4113
30	0.92	2.5	3870

Vol_1 = 18483 ft³

	(Cum")
5	.64
10	.99
15	1.25
30	1.74
60	2.2"

The high intensity one hour storm will be used to calculate in flow/out flow rates for 17'x95'x6' detention pond.

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JOB ACADEMY SQUARE JOB # 8047
BY R.H. Allen DATE 5/8/81
CHK. Revised DATE 6/19/81
SCALE SHEET NO. 6 OF 18

AREA 7 AND AREA 8
TOTAL Area

① 255(288) = 73440.
② ½(87)(255+340) = 25882.
99322 = 2.28 Acres.

AREA 8
340(35)+124(90) = 16860 = .39 Acre (Sidewalk)
Hard Surface: 770(4)+35(18)+65(21)+(272)(4') = 6163 ϕ
Landscaped = 16860 − 6163 = 10697
Ave C (Composite) = 0.52 (See Sheet A-1 in Appendix) (Use Tc = 10) ∴ (I = 5.4)
Q100 = (.52)(5.4)(.39) = 1.10 cfs
Q10 = 0.55 cfs.
V100 = 16860 (.52)(2.6/12) = 1900 ft³

Since Q & V are small, (even high intensity short storm gives max Q for entire area of 1.5 cfs) adequate under sidewalk drains will be used to convey runoff directly to Academy through curb.

AREA 7 − Gross = 2.28 − 0.39 = 1.89 Acres.
Hard Surface:

3 C-C Units	= 3(2480)	= 7440
4 D-C "	= 4(2340) (In Area 8)	= 9360
4 F-C "	= 4(2770) − 3080	= 8000.
Roads	(410')(26)+30(150)2	= 15160
Garages	22(240)	= 5280
Parking Spaces	20(180)	= 3600
Side walk	(1073')(4')	= 4292
Dumpster & Misc.	300+48(41)	= 1968
	TOTAL	= 55100 ϕ

Hard = 1.26 Acre
Landscaped = 1.89 − 1.26 = 0.63 Acre

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JOB ACADEMY SQUARE JOB # 8042
BY RH ALLEN DATE 8/9/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO. 7 OF 18

AREA 7 (Cont'd)

Ave C (Composite) = 0.68 (See Sheet A-1 IN Appendix)

Assume Total runoff down streets to Denali:

Tc: Grass to Street (115 @ 3%) = 12 Min 12

Street to R @ Denali (480' @ 3.2%) = 2.8

Tc = 14.8 MIN

∴ i = 4.6"/hr.

Q_{100} = .68(4. X 1.89) = 5.91 cfs. V_{100} = 2.5/12 (82328)(.68) = 12130 ft³

Q_{10} = 2.95 cfs

1 HOUR STORM ANALYSIS:

Time	i	Q	Vol.
5	7.68	9.87 cfs	2961 ft³
5	4.20	5.40	1620
5	3.12	4.01	1203
15	1.96	2.52	2268
30	0.92	1.18	2124
		TOTAL	10,176 ft³

Tc for Area 7 = 14.8 Min. Therefore for worst 3-5 MIN periods of storm (Total T=15 MIN) Ave. Q reaching Denali would be as follows:

$$\frac{(9.87 + 5.40 + 4.01)}{3} = 6.43 \text{ cfs} > Q_{100}$$

Although localized runoff rates may be as high as 6.43 cfs for very short periods, the overall Q to Denali of a short duration high intensity storm will not exceed the Q_{100} rate of 5.91 cfs for only a very short period.

∴ The Q_{100} rate (5.91 cfs) will be used as the maximum rate in calculating the most reasonable run-off rate to Denali.

REGISTERED ENGINEER STATE OF NEW MEXICO 6190 RICHARD HUGH ALLEN

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JOB ACADEMY SQUARE JOB # 8042
BY RHALLEN DATE 5/4/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO. 8 OF 18

AREA 9 (DISCHARGE DIRECTLY TO DEVAUL)

AREA = 1248 + 180(125) = 23748 ϕ

Hard: 385(4) + 6(240) + 6(180) + 11(48) + 2340 + 30(105) + 2(2980) − 35(18) − 65(21) = 13043 ϕ = 0.30

To Area 8

Landscape = 23748 − 13043 = 10705 = 0.25 Acre

Tc < 10 MIN ∴ Use i = 5.4"/hr.

$C_{ave} = \frac{.25(.24) + .30(.95)}{.55} = \frac{.35}{.55} = 0.63$

$Q_{100} = .63(5.4)(.55) = 1.87$ cfs

$V_{100} = \frac{2.6}{12}(23748)(.63) = 3242$ ft³

$Q_{10} = 0.99$ cfs

1-Hour Storm: Rainfall$_T$ = .85(2.6) = 2.2"

5 MIN	i = 7.68"/hr	Q = .63(7.68)(.55) =	2.66 cfs.
" "	= 4.20	=	1.46
" "	= 3.12		1.08
15 "	= 1.96		0.68
30 "	= 0.92		0.32

Overall, rates & volumes discharged directly to Devaul appear reasonable. No detainage or retainage will be required.

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JOB ACADEMY SQUARE JOB # 8042
BY RH ALLEN DATE 5/1/91
CHK. DATE
SCALE SHEET NO. 9 OF 18

DETENTION POND ANALYSIS:

1. Discharge from AREAS 3, 4 & 5 Reaches Pond in $16.5 + (140 / 3.25 \text{ ft/sec}) \div 60 = 16.5 + .7 = 17.2 \text{ MIN}$
   Rate = 0.65 cfs
2. Install two Type C drop inlets in main street to pick up portion of AREA 6 runoff.

⇒ See Sheets 11, 12, 13 & 14 for calculations

Pond inflow:
0.65 cfs @ T = 17.2 Min } Based on 100 year
6.0 cfs @ T = 19.6 Min } storm.

For 1 Hour high intensity storm:
Area 6: Ave $Q_1 = \frac{17.92 + 9.8 + 7.28}{3} = 11.67$ cfs (From Sheet 6)

$Q_{3,4,5} = 0.65$ cfs

Total inflow to pond will still be 6.65 cfs @ T = 19.6 Min. However flow in Denali will be 11.67 − 6 = 5.67 cfs for high intensity storm. This is theoretically the highest rate of flow for Denali beyond holding pond.

Assume discharge from pond @ 1.5 cfs.

A. AREA 3 4 5 (High Intensity Short Storm)
- Discharge 3 to pond in 4 after 33.6 Min Q = 5.7 cfs, V = 577 ft³
- Discharge 4 to pond in 5 after 10 Min
  Q = 0.4 cfs
  $V_T = 577 + 3003 = 3580$ ft³
  Total time to discharge = 149 Minutes
  Time after start of storm = 149 + 10 = 159 Min.

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JOB ACADEMY SQUARE JOB # 8042
BY EHAllen DATE 5/11/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO. 10 OF 19

• Discharge 5 to AREA 6 Pond:
- Start @ T = 17.2 MIN (Sheet 10)
(See calc sheets A-4 and A-5)
Q = 0.65 cfs (reserve IN 5)
V_T = 3834 + 2140 + 1494 = 7468 FT³
(485 IN 1 hr) (from res. IN 4 & 5)

TOTAL TIME to Discharge:
17.2 + 7468 / .65 = 208.7 MIN. from start of storm.

Areas 3, 4 & 5 will be empty at T = 208.7 MIN.

B. AREA 6 Discharge to Pond
AT T = 19.6 MIN, Q = 6 cfs into pond
AT T = 19.6 following would have occured based on a 1-hour storm:

	T	Q			Deficit	Storage
20	15 MIN	4.57 cfs		4113	—	4113
	5 "	17.92 cfs	V_T = 5376	9489	3576	1800
	5 "	3.80 cfs			1140	1800
	5 "	7.28 cfs			384	1800
	30 "	2.15 cfs			—	3870
					TOTAL	13383

SUMMARY

Time	Q_5	Q_6	V_5 (from #5)	V_6	Q_{out}	V_{out}	V_{stored}
0	0	0	0	0	0	0	0
17.2	0.65	6.07	671	5040	1.5	1548	4163
23.5	0.65	11.12	245	6131	1.5	1548	4828
25	0.65	11.12	58	1126	1.5	135	1049
30	0.65	7.52	95	2256	1.5	1548	903
60	0.65	2.22	1170	3996	1.5	2700	2466
90	0.65	0.8	1170	1440	1.5	2700	-90
150	0.65	—	2340	—	1.5	5400	-3060
208.7	0.46	—	1619	—	1.5	5283	-3664

N.G. See Sheet 12

STORAGE REQ'D = 13409
AVAILABLE = 9700.

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JOB EL PASO JOB # 8042
BY EH ALLEN DATE 5/10/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO. 11 OF 18

SINCE POND CANNOT BE INCREASED IN SIZE, outflow rate will be increased to 2.5 cfs w/ overflow notch in top of wall for excess.

Time	Q5	Q6	V5	V6	Qout(pipe)	Qout(over)	Vout	Vstore
0								
17.2	0.65	6.28	671	6478	2.5	0	2580	4569
19.6	0.65	11.67	94	1680	"	0	375	1399
25	0.65	10.76	210	3486	"	0	810	2886
30	"	7.28	195	2184	"	793	750	836
60	"	2.15	1170	3870	"	540	4500	-630
90	"	1.75	1170	3150	"	0	4500	-180
150	"	—	2340	—	2.5	0	9000	-6660
208.7	"	—	1619	—	2.5	0	4469	-2850
		TOTAL	7469	20848		1333	26984	0

28317 Ft³ 28317 Ft³

All ponds empty after 209 MIN = 3.48 hours ✓

Outflow pipe for 2.5 cfs 108' $S = 2/108 = 1.85\%$

Try 8": $Q = \frac{1.49}{.019}(.0185)^{1/2}(.342)(.1625)^{2/3} = 1.086$ cfs

Try 12" $Q = \frac{1.49}{.019}(.0185)^{1/2}(.7854)(.2501)^{2/3} = 3.32$ cfs

Try 10" $Q = \frac{1.49}{.019}(.0185)^{1/2}(.554)(.2125)^{2/3} = 2.1$ cfs

Use 12" CMP @ $S = .011$ (1.1%) $Q = 2.49$ cfs OK

Invert @ Pond = 82.0 $V = 3.79$ fps

Req'd $h = \frac{1}{.62^2}\frac{(2.0)^2}{64.4(.62)} = 0.41$ Water depth = 0.91'

Losses: $\frac{3.72^2}{64.4}(1.5+1) + \frac{(.022)^2(3.79)^2(108)}{2.22(.2501)^{4/3}} = 2.15'$

TOTAL $H_L = 2.48'$ OK

Inlet governs control of flow.

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SCALE SHEET NO. 12 OF 19

DROP INLET CALCS.

Given: Type C inlet

$Q = (CA\sqrt{2gh}) \times (2/3)$

C = coeff. for opening = 0.6

A = Net area in ft² =

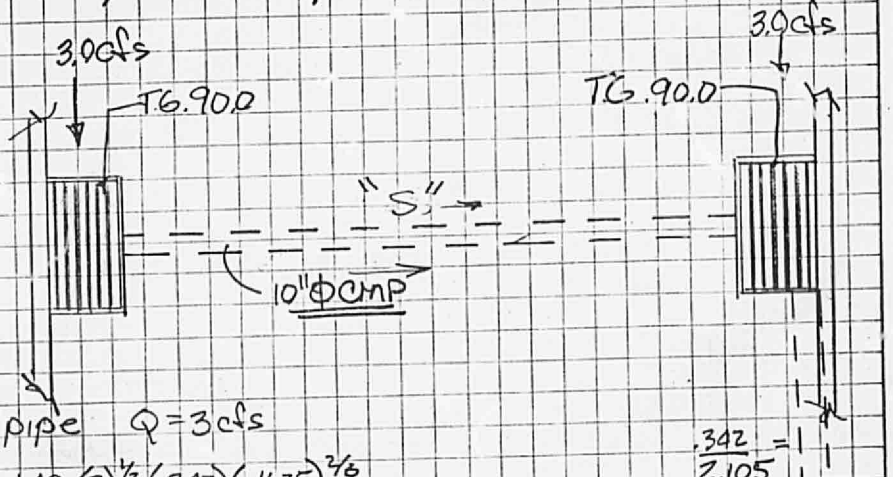
= (.06)(3.33) 21ea. = 4.2 ft

h = allowable head = 0.42'

$Q = .6(4.2)(\sqrt{2}(32.2)(.42)) \times 2/3$

$Q = 8.71$ cfs.

1- Type C on each side of street will adequately handle the quantity of water to be picked up. Total of 3 cfs needed at each inlet. Control will be provided by outlet pipes.



Try 8" pipe Q = 3 cfs

$3 = \frac{1.49}{.019}(S)^{1/2}(.342)(.1625)^{2/3}$

$.1119 = S^{1/2}(.1625)^{2/3}$

$S^{1/2} = .3758$

$S = .14$ (14% too steep)

Try 10" pipe

$3 = \frac{1.49}{.019} S^{1/2}(.554)(.2125)^{2/3}$

$S^{1/2} = .1939$ $S = .0376$ (Say .038)

$\frac{.342}{2.105} = .554$

See attached sheet for continuation

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SCALE SHEET NO. 13 OF 18

Outlet from 2nd Catch Basin to Holding Pond

Total Q = 6 cfs

.7854 / 3.14

Try 12" pipe

$6 = \frac{1.49}{0.019}(.7854)(.2501)^{2/3} S^{1/2}$

$S^{1/2} = .2454$

$S = .06$ $\quad V = Q/A = 6/.7854 = 7.64$ fps Too High

Try 16" pipe

$6 = \frac{1.49}{.019}(1.4103)(.3375)^{2/3} S^{1/2}$

1.4103 / 4.1783

$S^{1/2} = .1119$

$S = .0125$ Use 1.5%

1.5% for 75' = 1.12'

87 - 1.12 = 85.88 (Use 85.9 as Outlet at Pond.

$V = 6/1.4103 = 4.25$ fps. Ok ✓

Vel = 5.56 fps

Losses in 10" Pipe:

Bend: $\frac{1.5(5.56)^2}{64.4} = 0.72'$

Friction & Velocity: $\frac{(.022)^2(5.56)^2(22)}{2.22(.2125)^{4/3}} + \frac{5.56^2}{64.4} = 0.87$

Total $h_L = 1.59'$

h for 3.0 cfs discharge $= \frac{1}{.62^2}\frac{(3^2)(}{64.4(.31}$ $= 1.171$ (D.I has 2.2' of p) OK ✓

Losses in 16" Pipe: $Q = 6$ cfs $V = 4.25$ fps

Losses: $\frac{1.5(4.25^2)}{64.4} + \frac{4.25^2}{64.4} + \frac{.022^2(4.25)^2(75)}{2.22(.3375)^{4/3}} = 1.96'$

Req'd head $= \frac{6^2}{.62^2(64.4)(1.4103)^2} = 0.73'$

Inlet control governs. D.I. will fill partially to arrive at 0.73' head & design discharge (Q = 6 cfs) will be achieved.

REGISTERED LAND SURVEYOR 7896 NEW MEXICO RICHARD HUGH ALLEN

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JOB ACADEMY SQUARE JOB # 8042
BY R.H.Allen DATE 5/1/81
CHK. REVISED DATE 6/19/81
SCALE SHEET NO 14 OF 18

AREA 10

GROSS AREA:

192 × 427	81984
½(12)(30)	180.
½(90)(134+78)	4095.
½(21)(85+47)	1386
½(60)(35)	1050
½ 173(137+149)	24739.
½ 46(137)	3151.
45(33)+67(17)	2624
½(15)(37+32)	517
TOTAL	119726 = 2.75 Acres

Hardsurface

A3C 2½	2.5(3700)	= 9250
PCA	3(3580)	= 10740.
Roads	40(35)+26"(255)	= 8030
Garages	15(240)	= 3600
Stalls	15(180)	= 2700
Walks	4'(705)	= 2820
Patio, Misc.	17(50)+1670	= 2570
		39660 ϕ = 0.91 Acre

REGISTERED LAND SURVEYOR 5290 NEW MEXICO RICHARD H. ALLEN

Landscape/Open = 2.75 − .91 = 1.84

Ave C: $\frac{1.84(.24)}{2.75} + \frac{.91(.95)}{2.75} = 0.47$ (See also Use i = 5.4 sheet A-1)

$Q_{100} = .47(2.75)(5.4) = 6.98$ cfs $V_{100} = 12192$ Ft³

$Q_{10} = 3.49$ cfs

AREA 10A ⇒ DRAINS TO ACADEMY UNDER SIDEWALK

TOTAL AREA = ½(37(65)+66(127)+200+92(6)) + 38(8) + ½(65)(45) + ½(18)(76) = 12787 ϕ

Hard surface: ½(3580)+2467+48 = 6095

= 0.14 Acre

Landscaped = 0.29 − .14 = 0.15 Acre

Composite C for AREA 10A:

$C_{ave} = \frac{.14(.95)}{.29} + \frac{.15(.24)}{.29} = 0.58$ (See also Sheet A-1)

$V_{100} = \frac{26}{12}(.58)(12787) = 2770$ Ft³

$Q_{100} = .58(.29)(5.4) = 0.91$ cfs ⇒ Flow is negligible since it is discharged @ 2 points to ACADEMY OK

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SCALE SHEET NO 15 OF 18

AREA 10B ⇒ DRAINS TO CURBING

GROSS AREA = 38(30)+58(70)+64(98)+½(27)(64+58) = 9919 ☐'

Hard = 40(45)+2386+50 = 4236 = .10 Acre

Landscape = .23 − .10 = .13 Acre

Cover = .13(.24)/.23 + .10(.95)/.23 = .55

Q_{100} = .55(.23)(5.4) = .68 cfs OK (Negligible)

V_{100} = 2.6/12(9919)(.55) = 1182 FT³

BALANCE OF AREA 10 ⇒ DRAINS TO INTERIOR HOLDING PONDS.

GR. AREA = 2.75 − (.23+.29) = 2.33

Hard " = 0.91 − (.14+.10) = 0.67

Landscaped = 1.84 − (.15+.13) = 1.56

Cover (composite) = 0.45 (See Sheet A-1 in Appendix)

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Q'_{100} = .45(5.4)(2.33) = 5.66 cfs V'_{100} = 9896 FT³

Q'_{10} = 2.83 cfs

Pond Size: 9896/1.0 = 9896 ☐' Bot. Pond Elev: 96.0

Use 100 X 100 = 10,000 ☐'

Depth of water = 0.99' OK

Outlet: Rate: Use 2 cfs.

$Q = \frac{1.49}{0.19}(S^{1/2})(A)R^{2/3}$ Try 10" Pipe

S = 2% Q = 2.2 cfs

Run to MH in landscaped area.

NOTE: Pond & 10" Pipe to M.H. are temporary. Long term system will be designed w/ B additional W.S. in the area of temp. pond. M.H. will remain to carry runoff from newly developed area.

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SCALE SHEET NO. 16 OF 18

AREA 10 (Cont'd)

Max. discharge rate to pond will be due to short-term high intensity storm. (1 hour)

Q max based on a Tc = 10 min

5 min i = 7.68"/hr $Q = .45(7.68)(2.33) = 8.05$ cfs

3 min i = 4.20"/hr $Q = .45(4.2)(2.33) = 4.4$ cfs

weighted ave $Q = 6.23$ cfs

Pond sized for Total retention. Swales, streets etc... will carry this short period high flow.

Inverts for D.I. & M.H.

D.I. inv of 10" pipe = 95.5 − 1.5 = 94.0

Manhole invert: 94.0 − 1.6 = 92.4

Slope = 2.0%

AREA 11 – Drains to inner street system & then to Pond.

Area = 11700 ϕ (.27 Acre) Landscaped = 8955 ϕ = (0.21 Acre)

Hard area = 2745 ϕ = .06 Acre

Cave (Composite) = 0.45 (See Sheet A-1 in Appendix)

$Q_{100} = .45(.27)(5.4) = 0.66$ cfs $V_{100} = 1147 ft^3$

Inflow to street & thus to Pond is negligible and should not present a problem for Pond drainage

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JOB ACADEMY SQUARE JOB # 8042
BY RH ALLEN DATE 5/12/81
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SCALE SHEET NO 17 OF 18

BUILDING SIZES USED IN RUN-OFF CALCULATIONS

Drainage Study - Grupe Tract

Developed hard surfaces

Unit sizes in various combination

A, B, C	= 75'x30' + 37'x39'	=	3700 s.f.
C, D	= 36'x65'	=	2340
A, E, D	= 32x45 + 69x31'	=	3580
C, C	= 67'x37'	=	2480
F, C	= 36x77	=	2770
Garage	= 11x22	=	240
Parking Space	= 9x20	=	180
Roadway	= L.f x26		

BUILDING SIZES USED IN RUN-OFF CALCULATIONS

Drainage Study - Group Tract

Developed hard surfaces

Unit sizes in various combination

A, B, C	= 75'x30' + 37'x39'	=	3700 s.f.
C, D	= 36'x65'	=	2340
A, E, D	= 32x45 + 69x 31'	=	3580
C, C	= 67'x 37'	=	3480
E C	= 36x77	=	2770
Garage	= 11x22	=	240
Parking Space	= 9x20	=	180
Roadway	= L.f x26	=	

SUPPLEMENT TO EL PASEO DRAINAGE REPORT
ORGINAL DATED 5/14/81

1. Analysis of Composite C factors
The following table was obtained from the Denver Regional Council of Governments Drainage Criteria Manual (1969) and was used to calculate composite runoff coefficients for the Rational Formula.

Character of Surface	Range of 'C'	'C' used
Streets (asphalt)	0.7 to 0.95	0.90
Walks (Concrete)	0.75 to 0.85	0.80
Roofs	0.75 to 0.95	0.85
Lawns (Heavy Soil, 7% slope±)	0.25 to 0.35	0.30

Area 1 Approximately 90% of hard surfaced area is comprised of streets and roofs. A weighted average will be used to calculate a composite 'C' value.

$\left(\frac{0.9+0.85}{2}\right)90\% + 0.8(10\%) = 0.87$ (Hard surf.)

$\therefore$ Composite 'C' $= 0.87\left(\frac{.66\ Acre}{1.46\ Acre}\right) + 0.3\left(\frac{0.80\ Acre}{1.46\ Acre}\right)$

$C_{comp.} = 0.56$

Similar calculations can be performed for the remaining areas resulting in composite run-off coefficients as follows:

	Area	Comp.	Area	
Rickover	1 Swale under walk	0.58	7	0.68
	2 (localized at catch basin & swale)	0.47	8	0.52
	3	0.49	9	0.63
	4	0.76	10A	0.58
	5	0.53	10B	0.55
	6	0.61	10 (Balance)	0.45
			11	0.45

Composite 'C' for overall Site = 0.56 The above calculations were used for calcs in body of report.

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SCALE SHEET NO A-1 OF

2. Analysis of street f/ ws, velocities & Times of concentration Tc.

A. Area 1 ⇒ Tc = 10 Min. (See Sheets 1 & 2)

B. Area 2 ⇒ No street flows.

Drop inlet and pipe analysis in Area 2:

Q_{100} reaching inlet = 0.71 cfs

$Q = VA \quad \therefore \quad V = Q/A = .71/A$ = design discharge

For Drop Inlet, assume square bend @ bottom of inlet & ∴ $K_b = 1.5$ $\quad n = .022$ Corrugated Mtl. pipe. $\quad L = 65'$

$$h_L = \left(1 + \frac{29n^2L}{R^{4/3}} + K_b\right)\frac{V^2}{2g}$$

$R = A/WP$

$R = \frac{\pi r^2}{\pi r} = \frac{r}{2}$

Check that Inlet is not submerged:

h_w elev at Inlet (Assume weir action)

$Q = C_w L h_w^{3/2}$ $\quad C_w = 2.7$ (Since Drop Inlet is assumed not to fail)

$0.71 = (2.7)(8')(h_w^{3/2})$

$h_w = 0.10'$ ⇒ This is reasonable, & therefore the assumption of weir action and a non-submerged inlet is OK.

$h_L = 702.3 - 697.2 = 5.1'$ (Total loss from T.G. to outlet)

- Entrance loss ≅ 0 as drop inlet works as weir with $h_w = 0.10'$ ∴ Inlet is not submerged.

Velocity at grate:

$$V_g = \frac{1.49}{.025}\left(\frac{.5}{200}\right)^{1/2}(.095)^{2/3} = 2.23 \text{ fps}$$ Assume 2' channel

$R = A/WP \quad (.095)/2.1$

- Loss @ bend: $1.5(V_g^2/2g) = 0.12'$
- Friction: $\frac{(.022)^2(65)(3.74)^2}{2.22(.17)^{4/3}} = \frac{.44}{.209} = 2.1$
- Velocity head loss: $\frac{V^2}{2g} = .22'$

$H_L = .12 + 2.1 + .22 = 2.44' < 5.1'$ actual head loss ∴ OK

Note: Outlet losses are negligible as it is a free end discharge.

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Max. Capacity of 8" pipe flowing full @ 3.23% is

$Q = \frac{1.49}{.022}(.0323)^{1/2}(.17)^{2/3}(.342) = 1.28$ cfs

A smaller pipe could accomodate the .71 cfs flow. However, 8" CMP will be utilized.

Area 3

Note: 8" Pipe is merely an overflow device between pond areas. No hydraulic calcs are needed. Q discharge rate = max. rate to be seen by pipe under 1 hour storm condition.

Total Volume to Area 4 due to 100 year 6 hr. precip. is:

$V_{61}{}_{100} = \left(\frac{2.6}{12}\right)(10531)(.49) - 335 = 783$ ft^3

Area 4

Analysis of 8" pipe in drop inlet.

Max. capacity of 8" pipe flowing full = 1.63 cfs

$Q = \frac{1.49}{.022}(.053)^{1/2}(.17)^{2/3}(.34)$

$Q_1 = 1.63$ cfs $\quad V = 4.79$ fps

Losses @ entrance

$h_L = 1.5\frac{V^2}{2g} = 1.5\frac{(4.79)^2}{64.4} = 0.07'$

$\text{Friction} = \frac{(.022)^2(30)(4.79)^2}{2.22(.17)^{4/3}} = 1.59'$

Velocity hd loss $= V^2/2g = 0.36'$

Total $h_L = 2.02'$

$h_L = 3.63 > 2.02$ ∴ Ok.

depth @ curb = .13'

$V_1 = \frac{1.48}{A}$ $\quad A = \frac{1}{2}(6.5)(.13)2 = .84$

$V_1 = 1.76$ fps

Check weir effect: Perim = 8'

$1.46 = (2.7)(8)(h_w)^{3/2}$

$h_w = 0.17'$ $\quad 0.14$

Grate not submerged ∴ operates at free surface.

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E. AREAS
No street flows; Tc values checked - OK.

F. POND @ AREA 4
(INflow out flow on sheet 4)
Check outflow pipe ⇒ (6" ϕ) CMP
Max capacity = 0.4 cfs. flowing full @ 1% slope

Check head required for a given flow in 6" pipe:

$$h = \frac{1}{C_d^2}\frac{Q^2}{2gA^2} \qquad C_d = .62$$

$$h = \frac{1}{(.62)^2}\left(\frac{.4^2}{64.4(.039)}\right) = 2.6(.064) = .166' \approx (.17')$$

∴ Min head req'd to obtain 0.4 cfs discharge is 0.17' above the ℄ of the pipe. As head increases, discharge will increase to a total of 1.55 cfs with a 2.5' head. (Qave = .98 cfs)
6" CMP IS OKAY FOR ANTICIPATED FLOWS BETWEEN AREA 4 AND AREA 5.

G. Pond at AREA 5
Max capacity of 6" pipe @ 2.67% = 0.65 cfs
Ck. head req'd.

$$h = \frac{1}{(.62)^2}\left(\frac{.65^2}{64.4(.039)}\right) = .44'$$

Min. head to obtain .65 cfs discharge is .44' above ℄ orifice.

Max Q @ h = 1.25' is 1.1 cfs
V = .85/.196 = 3.3 fps
head losses: (entrance) $\frac{.5(3.3^2)}{64.4} = .08'$
(Velocity) $\frac{3.3^2}{64.4} = .17'$
(friction) $\frac{(.022)^2(3.3)^2(137)}{2.22(.125)^{4/3}} = 5.2'$
5.45'
Vertical drop in grade = 5.0'
∴ Design discharge will be attained when water depth in pond is 0.25 + 0.44 = 0.69'.

A = .196 ft²
R = .125'

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SCALE SHEET NO. 4.4 OF

A. POND 2 AREA 6 AND STREET FLOWS

- Calculate Tc for Area 6:
  Overland flow Tc = 17.5 min (i = 4.5"/hr) (See Sheet 6)
  Tc in street:
  Ave. slope = 13'/400' = 0.0325
  Street crown = .02 (3"/12.5')
  n = 0.015 (asphalt)
  Q from overland flow to street ≈ 1.6 cfs.
  $Q = 0.63(0.54)(4.5)$

$Q = 0.56\left(\frac{d}{n}\right)S^{1/2}y$ y = depth of flow.

$1.6 = 0.56\left(\frac{50}{.015}\right)(.0325)^{1/2}(y)$ $y_{\frac{1}{2}} = 1'/.02 = 50$

$\therefore y = 0.14'$

$A = \frac{1}{2}(.14)(7') = 0.5\,\phi$

$V = Q/A = 1.6/.5 = \underline{3.2\ fps}$

Tc for street $= \frac{400'}{3.2\ fps} = 125\ sec. = 2.1\ min.$

$\therefore$ Total Tc (Area 6) = 17.5 + 2.1 = 19.6 min.

$i = 4.2"/hr$ (Intensity duration curve)

$Q_{100} = 4.2(66)(3.825) = \underline{9.80\ cfs}$

$Q_{10} = \underline{4.9\ cfs}$

(See sheets 12, 13 & 14 for Drop Inlet analysis)

- Weir on top of pond wall to discharge excess volume entering retainage basin:
  Weir formula: $Q = C_w L h^{3/2}$ → (Water Resources Engineering by Linsley, 1964)
  Hd = weir ht. = 1' L = 2'
  h = head on weir (assume .5')
  Hd/h = 1'/.5' = 2.0 $\therefore C_w = 3.485$
  $Q = 3.485(2)(.5)^{3/2} = 4.39\ cfs < 7.93$
  $7.93 = 3.485(L)(.5)^{3/2}$
  $L = 3.61'$ Use L = 4'
  V = 1.98 fps
  Disch. will meet max. requirement of 7.93 cfs for 1.7 min. After 1.7 minutes discharge will fall off rapidly to only 2.3 cfs. (See Table on sheet 12)

REGISTERED STATE ENGINEER NO. 5020 NEW MEXICO RICHARD HUGH ALLEN

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JOB EL PASO JOB # 8042.
BY RH Allen DATE 6/15/81
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I. AREA 10

Losses in 10" pipe to MH & to outlet @ Academy.

• Head req'd.

$h = \frac{1}{.62^2} \frac{(2.2)^2}{64.4(0.31)} = 0.63'$ Vert drop @ OI = 1.5'

$V = \frac{2.2}{.56} = 3.93$ f/s.

$R = .2125$

Head losses:

At drop inlet (Bend loss): $h_L = 1.5\left(\frac{V^2}{2g}\right) = 0.36'$

Friction: $h_L = \frac{(.022)^2(3.93)^2(77)}{2.22(.2125)^{4/3}} = 2.04'$

Velocity: $h_L = \frac{3.93^2}{64.4} = .0.24'$

Total h_L to M.H. = 0.36 + 2.04 + 0.24 = 2.64'

Total Vert. drop = 3.11'

Note:

Inlet will control flow. With 1.5' vert. drop @ O.I., sufficient head will be available to discharge design Q of 2.2 cfs. Pond will hold 100% of run-off assigned to it even if design flow (Q) is not achieved. (See sheet 16 of main report)

Head loss from M.H. to final outlet.

Inlet: $.5\frac{(3.93)^2}{64.4} = 0.12$

Velocity: $\frac{3.93^2}{64.4} = 0.24$

Friction: $\frac{(.022)^2(3.93)^2(181')}{2.22(.2125)^{4/3}} = 4.81'$

TOTAL H_L = 5.17'

Total Vert. drop = 92.2 − 86 = 6.2'

Additional losses at M.H. due to turbulence at high flows and bending of pipe will result in lower flows (Q < 2.2 cfs) to street. Since pond size was based on 100% retention these lower flows will not jeopardize the area around the pond.

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JOB EL PASEO JOB # 8042.
BY RH Allen DATE 6/16/81
CHK. DATE
SCALE SHEET NO. 16 OF

SUMMARY

1. AREAS 1 and 2 ⇒ Flow to Admiral Rickover:
   From Area 1 Q = 3.28 + .49 + 1.65 = 4.42 cfs.
   From Area 2 Q = 0.71 cfs .71
   TOTAL Q_{100} = 5.13 cfs
   This is less than Q_{100} undeveloped = 5.81 cfs ∴ OK
   V_{100} = 7742 + 1244 = 8986 ft³

2. AREAS 3 and 4 ⇒ Flow to pond in AREA 4

Retained in dry well (V_{100})		126 ft³
Retained in ponded area (AREA 3)		209 ft³
Overflow to AREA 4 Pond (V_{100})	783 ft³	
Retained at AREA 4 Drop Inlet (V_{100})	384 ft³	
AREA 4 Pond (V_{100})	1800 ft³	
TOTAL RETAINED/DETAINED	2967	335
	3302 ft³	

(See sheets 3 & 4 for Q and discharge times, sheets 10 & 11)

3. AREA 5
   Retained in Pond 1527 ft³
   (See sheets 4 & 5 for Q and discharge times, shts 10 & 11.)

4. AREA 6
   Pond Volume = 9490 ft³
   Total Volume run through pond = 28317 ft³
   Note: All 28317 ft³ will be released over a period of 3.48 hours

5. AREA 7
   V_{100} to Denalp = 12130 ft³
   Runoff rate to Denalp = 5.91 cfs

6. AREA 8
   V_{100} to Academy = 1900 ft³
   Q_{100} to Academy = 1.10 cfs @ 4 locations.

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JOB EL PASO JOB # 8047
BY ZH ALLEN DATE 6/19/81
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SCALE SHEET NO A-7 OF

7. AREA 9
$Q_{100} = 1.87$ cfs
$V_{100} = 3242$ ft^3
> No detainage required
Discharge to Arvalle

8. AREA 10A, 10B & 10
10A Q_{100} to Academy = 0.91 cfs $V_{100} = 2770$ ft^3
10B Q_{100} to Eubank = 0.68 cfs $V_{100} = 1182$ ft^3
10 Q_{100} to Pond = 5.7 cfs $V_{100} = 10000$ ft^3

9. AREA 11
Q_{100} to Inner Streets, then to Arvalle = 0.66 cfs
$V_{100} = 1147$ ft^3 (No detention)

SUMMARY OF FLOWS

1. To Rickover
$Q_{100} = 5.13$ cfs < 5.81 cfs
$V_{100} = 10136$ ft^3

2. To Arvalle
Q from Area 6 Pond 2.5 cfs
Q from Area 6 3.8 cfs
Q from Area 7 5.91 cfs
Q from Area 11 0.66 cfs
Q from Area 9 1.97 cfs
14.74 cfs < 21.92 cfs

Total Volume discharged to Arvalle over a 6 hour period would be 783 + 3586 + 2881 + 22021 + 12130 + 3242 + 1147 = 45,790 ft^3

TOTAL VOLUMES RETAINED / DETAINED
AREAS 3, 4, 5 4329 ft^3
AREA 6 9490
AREA 10 10000
24319 ft^3

An additional (28317 − 9490) = 18827 ft^3 is channeled through holding pond 6 & is detained about 2.5 hours.

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Therefore, in effect the total volume being detained/retained on the site is 24319 + 18827 = 43146 ft³

This is (0.99 Acre Feet)

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SUPPLEMENT TO RGV. DRAINAGE REPORT

AREA 1

Hardsurface = 0.66 acre (from RGV report page 1)

① ½(65)(35) = 1138
② ½(8)(112+66) = 712
③ ½(72+85)(20) = 1570.
③' ½(72+54)(18) = 1134.
④ 120(190) = 22800.
⑤ 190(150) = 28500.
⑥ ½(171)(80) = 6840.
⑦ ½(80+44)(18) = 1116.
TOTAL AREA 1 63810 ϕ = 1.46 acre

Landscaped: 1.46 − 0.66 = 0.80 Acre.

$Q = CiA$ $C_{ave} = \frac{0.95(.66)}{1.46} + \frac{.24(.80)}{1.46} = 0.56$

From RGV report, page 1, T_c = 10 min. i_{100} = 5.4 in/hr. i_{10} = 2.7 in/hr

$Q_{100} = .56(5.4)(1.46) = 4.42$ cfs $Q_{10} = .56(2.7)(1.46) = 2.21$ cfs

$V_{100} = 7752$ cf

$Q_{100\ undevel.} = 5.81$ cfs (RGV report, page 22)

For Area 1, No ponding required since $Q_{100\ devel} < Q_{100\ undevel.}$

Q to Rickover under Sidewalk:

2480 ϕ + 720 + 1200 = 4400 ϕ hard surf.
30(20) + 160(15) ÷ 30(15) + (35)² = 4675 ϕ lawn
> 9075 ϕ

Ave C: $C_{ave} = \frac{4400}{9075}(.95) + \frac{4675}{9075}(.24) = .58$

$Q = .58(5.4)(.21) = 0.65$ cfs
$Q_{10} = .58(2.7)(.21) = 0.33$ cfs.

From Catch Basin #1 (Sheet 2)

$Q_{100} = 0.7$ cfs
$Q_{10} = 0.35$ cfs

TOTAL To Rickover under S.W.:
$Q_{100} = .65 + .7 = 1.35$ cfs
$Q_{10} = .33 + .35 = 0.68$ cfs

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JOB FALCON Y SQUARE JOB # 8042
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CHK DATE
SCALE SHEET NO 1 OF 18

AREA 2 (Localized drainage to catch Basin)

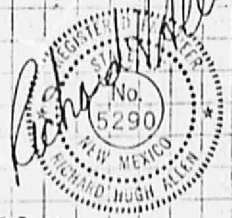
Total Area

① 72(48) = 3456
② ½(60)(22) = 660
③ ½(50+32)(60) = 2460
④ ½(85+120)(55) = 5637
TOTAL = 12213 ϕ = 0.28 Acre

Hard Surf. = 3710 ϕ
Lawn = 8503 ϕ
TOTAL = 12213 ϕ

Ave. C = 3710/12213 (.95) + 8503/12213 (.24) = 0.46

Q_{100} = (5.4)(.46)(0.28) = 0.70 cfs
Q_{10} = (2.7)(.46)(0.28) = 0.35 cfs
V_{100} = 2.4/12 (12213)(.46) = 1217 ft³



Since Area 1 + Area 2 go to Ackover.:

Total Q_{100} = 4.42 + 0.7 = 5.12 cfs < 5.81 OK
Total Q_{10} = 2.21 + .035 = 2.56 cfs

SWALE Under S.W. To the South: (Part of Area 1)

Total Area = 9120 ϕ
Hard = 2465

Ave c = 2465/9120 (.95) + 6655/9120 (.24) = .43

Q_{100} = .43(5.4)(.21) = .49 cfs
Q_{10} = .43(2.7)(.21) = .25 cfs.

Q_{100} Out driveway = 4.42 − .49 − 0.65 = 3.28 cfs
Q_{10} Out driveway = 2.21 − .33 − .25 = 1.63 cfs

AREA 3 (TO DRY WELL)

Hard Area = 3550 ϕ
Landscaped = [(50)(47) + ½(60)(105+127) + ½(22)(74)] − 3580 = 6959 / 10531

Conc = 3580/10531 (.95) + 6951/10531 (.24) = 0.48

*Herkenhoff maps

Use Tc = 10 min i_{100} = 5.4 i_{10} = 2.7

Q_{100} = .48(5.4)(.24) = .62 cfs
Q_{10} = .48(2.7)(.24) = .31 cfs

V_{100} = (2.5/12)(10531)(.48) = 1053 ft³
V_{10} = (1.2/12)(10531)(.48) = 526 ft³

Dry well Volume = (π)(2²)(10) = 126 ft³
Vol/area to contour 96 : [5(35) + 20(4) + ½(20)(13)](.5) = 209 ft³
Overflow to Area 4 = 1053 − 335 = 718 ft³
100

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SCALE SHEET NO. 2 OF 18

Overflow to Area 4 = 526 - 335 = 191 ft³

For 1 hr. storm, V = .55 (2.6) = 2.2" ← might fertain

5 min 7.68"/hr. 5 min 3.12"/hr. 30 min = 0.92"/hr.
5 min 4.20"/hr 15 min 1.96"/hr.

8" pipe @ .013%

$Q = \frac{1.49}{.019} (.013)^{1/2} (.1625)(.342)^{2/3} = .91$ cfs

R = .1963/1.57 = .125

Available for discharge

Saiz who?

NOTE: For 1 hour storm discharge to AREA 4 will start as follows:

				To ④
	After 15 min	Q = .22 ft³/sec	Vstore = 198 ft³	
0.91	Next 5 min	Q = .88 ft³/sec	Vstore = 137 ft³	127
	Next 5"	Q = .48		144
	Next 5"	Q = .30		108
	" 30"	Q = .11		98

190 @ 1.0 for 3.0 min!

Discharge to AREA 4 starts after 17.6 min

Time to reach pond = 17.6 + 16 = 33.6 min.
(Area 4)

AREA 4

TOTAL AREA = 40(50) + 120(44) + 90(170) = 21780 ft² = .5 Acre

Hard Surface: 3700 + 1056 + 3(528) + 780 + 26(145) + 25(25)(3) + 3200 = 15965 ft² = .37 Acre

Ave C: $.95\left(\frac{15965}{21780}\right) + .27\left(\frac{5815}{21780}\right) = 0.76$ .843

$Q_{100} = (.76)(5.4)(.5) = 2.05$ cfs

$Q_{10} = 1.03$ cfs.

$V_{T_{100}} = 2.6(21780)/12(.76) = 3586$ ft³

1 Hr Storm:			Vol
	5 min	Q = 2.91 cfs	873 ft³
	" "	= 1.60"	480
	" "	= 1.8 "	354
	15 "	= 0.14 "	660
	30 "	= 0.35"	630

3003 ft³

W/8" CMP from D.I. capacity = 1.8 cfs ponding will occur during about 5 minutes of storm. Pond available

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BY R.H. ALLEN DATE 5/8/81
CHK. DATE
SCALE SHEET NO. 3 OF 18

To contour 97 will hold 600 ft³
In 15 minutes (worst) Vol = 873 ft³
Outlet by 8" pipe = 540 ft³
Storage req'd = 333 ft³ ≅ 600 ft³

Note: Overflow will occur to pond at curb when street pool elev = 696.8.

Q_{100} @ street:
(21780 − 6300) = 15480 ϕ
Q_{100} = .76(5.4)(.36) = 1.48 cfs.

AREA 5

Gross Area: 145(51) + ½(85)(147+110) + 136(38) + 17² + 73(18) = 25088

Hard Surface:
1038 + 1110 + 878 + 2484 + 3700 + 750 = 9910 ϕ

Landscaped: 25088 − 9910 = 15178

$C_{ave} = \frac{15178}{25088}(.24) + \frac{9910}{25088}(.95) = .52$

Slope = 9/235 = .038
∴ Tc = 16.5 min
S_{100} = 4.5

Q_{100} = .52(4.5)(.58) = 1.36 cfs
Q_{10} = .52(2.25)(.58) = 0.68 cfs
Volume$_{100}$ = (25088)(2.6/12)(.52) = 2827 ft³

PONDS for Area 4:
Inflow/outflow for 1 hr storm:

15 min	Q = 0.74	(M) 666	360	306
5 min	Q = 2.91	873	120	753
5"	= 1.6	480	120	360
5"	= 1.18	354	120	234
30"	= 0.35 (30 min)	} 864	720	144
	+ .15 (21 min) overflow			

(0.4 cfs) 6" Pipe Outlet

Min Pond = 179.7 Ft³

Pond area = 570 ϕ
Depth = 1797/600 = 2.99' Use 3'
T.P. Elev = 91.0
T. Wall = 94.0

REGISTERED PROFESSIONAL ENGINEER
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JOB ACADEMY SQUARE JOB # 8042
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SCALE SHEET NO. 4 OF 18

POND FOR Area 5: (1 Hr Storm, Rainfall = .85(2.6) = 2.2

				Cum Run:
15 Min	i = 1.96 "/hr.	Q = 0.59 cfs	5	0.64"
5 Min	i = 7.68 "/hr.	= 2.32	10	0.99"
5	4.20	= 1.27	15	1.25"
5	3.12	= 0.94	30	1.74"
30	0.92	= 0.28	60	2.20
		.65 cfs		

T	Q_{IN} (Total)	(Vol(G)	V_{out}	$V_{storage}$
15	.59+.4 = .99	891	585	306
5	2.32+.4 = 2.72	816	195	621
5	1.27+.4 = 1.67	501	195	306
5	0.94+.4 = 1.34	402	195	207
30	0.28+.4 = 0.68	1224	1170	54
		3834	TOTAL REQ'D =	1494 ft³

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Area @ bottom = 10000 ft²
Depth = 1.494 Say 1.5' (No fence req'd.)
Due to topo, Make top of wall @ 93.0

Bot. Pond = 90.0

AREA 6

GROSS AREA

①	8 X 20	= 2280	
②	140 X 35	= 4900	
③	½(197+260)(190)	= 43415	
④	(170)(145)	= 24650	
⑤	½(240+188)(175)	= 20330	
⑥	230(50)	= 11500	
⑦	½(230+285)(50)	= 12875	
⑧	½(34)(20)	= 340	
⑨	½(250)(100)	= 12500	
⑩	90(250)	= 22500	
⑪	58(195)	= 11310	
	TOTAL	= 166600 ft²	= 3.825 Acre
Col. Landscaped		= 166600 − 91850	= 74750 ft² = 1.716 Acre

Hard Surface.

Apt.	2(3700)	=	7400
Apt.	3(3580)	=	10740
Acres	5(780) ÷ 3336	=	7236.
Gar.	36(240)	=	8640
	Roads (26')(1045)	=	27170
	Pads, Drives	=	15864
	Sidewalks = 1375(4)	=	5500
	Misc.		3000
	Club House/Patio		6500
	TOTAL	91850 ft²	= 2.109

.70%

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SCALE SHEET NO 5 OF 18

703

Composite Runoff factor "C":

Calc = .24(1.716)/3.825 + .95(2.109)/3.825 = 0.63

Misc. Data to be used in calcs that follow:

100 Year 6hr precip = 2.6" (Herkenhoff maps)

1 Hr. precip = 2.2"

Tc Calculations:

9.9/275 = 3.6% Tc = 17.5 Min Overland flow

(700-687)/400 = 3.25% Tc = 6.6 Min Channel

Total = 24.1 Min

From intensity/frequency curves } = 3.9"/hr.

(14.5 Min)

∴ Q_{100} = (.63)(3.9)(3.825) = 9.40 cfs

Q_{10} = 4.72 cfs

V = 0.467 ft/sec²

V = 25,399 cu ft

Vol. = 2.6(c)(A)/12 = 22?

$Vol._{100}$ = 11370 ft³

$Vol._{10}$ =

Street crown = 1/8"/ft.

Depth @ curb line = .42' (5")

Depth @ ℄ of street = .30' (3 5/8")

Analyze 1 hour storm:

.85(2.6) = 2.2"

T	i	Q = CiA = .63(i)(3.825)	V = 6553
5	7.68	18.5 cfs	
5	4.20	10.12	3036
5	3.12	7.52	2256
15	1.96	4.72	4248
30	0.92	2.22	~~4896~~

Vol. = 19989 ft³

(Cum")	
5	.64
10	.99
15	1.25
30	1.74
60	2.2"

The high intensity one hour storm will be used to calculate in flow/out flow rates for 17X95'X6' detention pond.

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BY RH Allen DATE 5/8/81

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SCALE SHEET NO. 6 OF 18

AREA 7 AND AREA 8
TOTAL Area

① 255(280) = 71400
② ½(87)(255+340) = 25882
199322 = 2.28 Acres.

AREA 8
340(35)+124(40) = 16860 = .39 Acre (sidewalk)
Hard Surface: 770(4)+35(18)+65(21)+(272)(4') = 6163 ft²
Landscaped = 16860 − 6163 = 10697
Ave C = 10697(.24)/16860 + .95(6163)/16860 = .50 (Use Tc=10)
∴ (i = 5.4)
Q100 = (.50)(5.4)(.39) = 1.05 cfs
Q10 = 0.53 cfs.
V100 = 16860(.5)(2.6/12) = 1827 ft³

Since Q & V are small (even high intensity short storm gives max Q for entire area of 1.5 cfs) adequate under sidewalk drains will be used to carry runoff directly to Academy through curb.

AREA 7 − GROSS = 2.28 − 0.39 = 1.89 Acres
Hard Surface:

3 C-C Units	= 3(2480)	=	7440
4 D-C "	= 4(2340) (In Area 8)	=	9360
4 F-C "	= 4(2770) − 3080	=	8000.
Roads	(410')(26)+30(150)±	=	15160
Garages	22(240)	=	5280
Parking Spaces	20(180)	=	3600
Sidewalk	(1073')(4')	=	4292
Dumpster & Misc.	300+48(41)	=	1968
	TOTAL	=	55100 ft²

Hard = 1.26 Acre
Landscaped = 1.89 − 1.26 = 0.63 Acre

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SCALE SHEET NO 7 OF 18

AREA 7 (Cont'd)

Ave C = $\frac{1.26(.95) + .24(.63)}{1.89} = \frac{1.39}{1.89} = 0.71$

Assume Total runoff down streets to Denali:

Tc: Gross to Street (115' @ 3%) = 12 min 12 · 1.4 f.s.

Street to R.P. @ Denali (480' @ 3.2%) = 6.5

= 18.5 min

$\frac{Tc}{80}$ i = 4.3"/hr.

$Q_{100} = .71(4.3)(1.89) = 5.77$ cfs $V_{100} = \frac{2.5}{12}(82323)(.71) = 12665$

$Q_{10} = 2.89$ cfs

if V = 3 f.p.s. L = 4320'

$T_c < 5'$

1 HOUR STORM ANALYSIS:

Time	i	Q	Vol.
5	7.68	10.31	3093 ft³
5	4.20	5.64	1692
5	3.12	4.19	1257
15	1.96	2.63	2367
30	0.92	1.23	2214
		TOTAL	10623 ft³

use 7.68"/hr

$Q_{100} = 7.68 \cdot 1.26 \cdot .95$

= 9.19 cfs !

Tc for Area 7 = 18.5 min. Therefore for worst 3-5 min periods of storm (Total T = 15 min) Ave. Q reaching Denali would be as follows:

$\frac{(10.31 + 5.64 + 4.19)}{4} + 2.63 = 5.17$ cfs < Q100

Although localized runoff rates may be as high as 6 or 7 cfs for very short periods, the overall Q to Denali of a short duration high intensity storm will not exceed the Q100 rate of 5.77 cfs.

* The Q100 rate (5.77 cfs) will be used as the maximum rate to be expected to reach Denali.

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JOB ACADEMY SQUARE JOB # 8042
BY RH ALLEN DATE 5/11/81
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SCALE SHEET NO 8 OF 19

AREA-9 (DISCHARGE DIRECTLY TO DEVARGAS)

AREA = 1248 + 180(175) = 23748 ft²

Hard: 385(4) + 6(240) + 6(180) + 11(48) + 2340 + 30(105 + 2(2480) − 35(18) − 45(21) = 13043 ft² = 0.30 Acre (To Area 8)

Landscape = 23748 − 13043 = 10705 = 0.25 Acre

Tc < 10 MIN ∴ Use i = 5.4"/hr.

$C_{ave} = \frac{.25(.24) + .30(.95)}{.55} = .63$

$Q_{100} = .63(5.4)(.55) = 1.87$ cfs $\quad V_{100} = \frac{2.6}{12}(23748)(.63) = 3242$

$Q_{10} = 0.99$ cfs

1-Hour Storm: Rainfall = .85(2.6) = 2.2"

5 MIN	i = 7.68"/hr	Q = .63(7.68)(.55) =	2.66 cfs.
" "	= 4.20	=	1.46
" "	= 3.12		1.08
15 "	= 1.96		0.68
30 "	= 0.92		0.32

Overall, rates & volumes discharged directly to Devargas appear reasonable. No detainage or retainage will be required.

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JOB ACADEMY SQUARE JOB • 8042
BY RH Allen DATE 5/11/81
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SCALE SHEET NO 9 OF 18

DETENTION POND ANALYSIS:

1. Discharge from AREAS 3, 4 & 5 reaches pond in 16.5 + (140 / 3.25 ft/sec) ÷ 60 = 16.5 + .7 = 17.2 min
   Rate = 0.65 cfs.
2. Install two Type C drop inlets in main street to pick up portion of AREA 6 runoff.
   ⇒ See Sheets 4 & 5 for calculations

Pond inflows:
0.65 cfs @ T = 17.2 min } Based on 100 year
6.0 cfs @ T = 23.5 min } storm.

For 1 Hour high intensity storm:
Area 6: Ave $Q_1 = \frac{18.51 + 10.12 + 4.72}{3} = 11.12$ cfs

$Q_{345} = 0.65$ cfs.
Total inflow to pond will still be 6.65 cfs @ T = 23.5 min. However flow in Arvada will be 11.12 − 6. = 5.12 cfs for high intensity storm. This is theoretically the highest rate of flow for Arvada beyond holding pond.

Assume discharge from pond @ 1.5 cfs.

A. AREA 345 (High Intensity Short Storm)
- Discharge 3 to pond in 4 after 33.6 min Q = .57 cfs, V = 577 ft³
- Discharge 4 to pond in 5 after 10 min
  Q = 0.4 cfs
  V_T = 577 + 3003 = 3580 ft³
  Total time to discharge = 149 minutes
  Time after start of storm = 149 + 10 = 159 min.

REGISTERED ENGINEER STATE OF NEW MEXICO 5281 Richard Hugh Allen

DESIGN PROFESSIONALS, INC.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-6338 87107

JOB AGRADENT/SQUIRE JOB # 8042
BY RHAllen DATE 5/11/21
CHK. DATE
SCALE SHEET NO 10 OF 18

- Discharge 5 to AREA 6 Pond
  - Start @ T = 17.2 MIN (Sheet 10)
    Q = 0.65 cfs
    V_T = 3834 + 2140 + 1494 = 7468 ft³
    (485 in 1 hr) (reserve in 5) (Sranres. in 4)

TOTAL TIME to Discharge:
17.2 + 7468/.65 = 208.7 MIN. from start of storm

Areas 3, 4 & 5 will be empty at T = 208.7 MIN.

B. AREA 6 Discharge to Pond
AT T = 23.5 MIN, Q = 6 cfs into pond
At T = 23.5 following would have occured based on a 1-hour storm:

	T	Q				Dvault	
23.5 MIN	15 MIN	4.72 cfs		4248		—	4248
	5 "	10.12 cfs	V_T =	3036	11171	1236	1800
	3.5 "	18.51 cfs		3887		2627	1260
	1.5 "	18.51 cfs				1126	540
	5 "	7.52 cfs				456	1800
	30 "	2.22 cfs					3996
						-	13644
						13409	

REGISTERED ENGINEER STATE OF NEW MEXICO RICHARD HUGH ALLEN

SUMMARY

Time	Q_5	Q_6	V_5 (Sranr. 5)	V_6	Q_{out}	V_{out}	V_{stored}
0	0	0	0	0	0	0	0
17.2	0.65	6.07	671	5040	1.5	1548	4163
23.5	0.65	11.12	245	6131	1.5	1548	4828
25	0.65	11.12	58	1126	1.5	135	1049
30	0.65	7.52	95	2256	1.5	1548	903
60	0.65	2.22	1170	3996	1.5	2700	2406
90	0.65	0.8	1170	1440	1.5	2700	-90
150	0.65	—	2340	—	1.5	5400	-3060
208.7	0.46	—	1619	—	1.5	5283	-3664

N.G. See Sheet 12

STORAGE REQ'D = 13409
AVAILABLE = 9700.

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4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-6336 87107

JOB ____ JOB # ____
BY ____ DATE ____
CHK. ____ DATE ____
SCALE ____ SHEET NO. 11 OF 18

SINCE POND CANNOT BE INCREASED IN SIZE, OUTFLOW RATE WILL BE INCREASED TO 2.5 cfs W/ OVERFLOW NOTCH IN TOP OF WALL FOR EXCESS.

Time	Q5	Q6	V5	V6	Qout(pipe)	Qout(over)	Vout	V5+6
0	—	—	—	—	—	—	—	—
17.2	0.65	6.07	671	5040	2.5	0	2580	313
23.5	0.65	11.12	245	4203	"	0	945	350
25	0.65	11.12	58	1001	"	0	225	83
30	"	7.52	195	2256	"	0	750	170
60	"	2.27	1170	3996	"	145	4500	521
90	"	1.94	1170	3493	"	163	4500	0
150	"	—	2340	—	2.5	0	9000	−64
208.7	"	—	1619	—	2.5	0	4649	−30
	TOTAL		7468	19989		308	27149	0

27457 Ft³ 27457 Ft³

All ponds empty after 209 MIN = 3.48 hours ✓

Outflow pipe for 2.5 cfs 108' S = 2/108 = 1.85%

Try 8": $Q = \frac{1.49}{.019}(.0185)^{1/2}(.342)(.1625)^{2/3} = 1.086$ cfs

Try 12": $Q = \frac{1.49}{.019}(.0185)^{1/2}(.7854)(.2501)^{2/3} = 3.32$ cfs

Try 10": $Q = \frac{1.49}{.019}(.0185)^{1/2}(.554)(.2125)^{2/3} = 2.1$ cfs

Use 12" CMP @ S = .011 (1.1%) Q = 2.49 cfs OK

Invert @ Pond = 82.0

REGISTERED PROFESSIONAL ENGINEER STATE OF NEW MEXICO RICHARD HUGH ALLEN

DESIGN PROFESSIONALS, INC.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-6338 87107

JOB ACADEMY SQUARE JOB # 8047
BY RHA (LEA) DATE 5/11/81
CHK. DATE
SCALE SHEET NO 12 OF 18

DROP INLET CALCS.

Given: Type C inlet

$Q = (CA\sqrt{2gh}) \times (2/3)$

C = coeff. for open q = 0.6

A = net area in ft² =

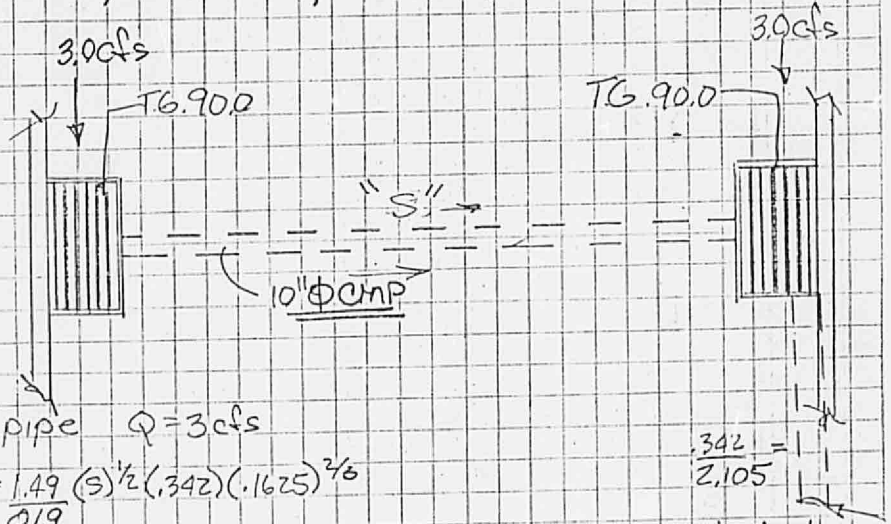
= (.06)(3.33) 21ea. = 4.2 ft

h = allowable head = 0.42'

$Q = .6(4.2)(\sqrt{2}(32.2)(.42)) \times 2/3$

$Q = 8.74$ cfs.

1- Type C on each side of street will adequately handle the quantity of water to be picked up. Total of 3 cfs needed at each inlet. Control will be provided by outlet pipes.



Try 8" pipe Q = 3cfs

$3 = \frac{1.49}{.019}(S)^{1/2}(.342)(.1625)^{2/3}$

$.1119 = S^{1/2}(.1625)^{2/3}$

$S^{1/2} = .3758$

$S = .14$ (14% too steep)

Try 10" pipe

$3 = \frac{1.49}{.019} S^{1/2}(.554)(.2125)^{2/3}$

$S^{1/2} = .1939$ $S = .0376$ (Say .038)

$\frac{.342}{2.105} =$

See attached sheet for continuation

.554

DESIGN PROFESSIONALS, INC.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-8338 87107

JOB ACADEMY SOUARE JOB #

BY RH ALLEN DATE 5/11/81

CHK. DATE

SCALE SHEET NO 13 OF 18

OUTLET FROM 2ND Catch Basin TO #6 Holding Pond

Total Q = 6 cfs

.7854 / 3.14

Try 12" pipe

$Q = \frac{1.49}{.019} (.7854)(.2501)^{2/3} S^{1/2}$

$S^{1/2} = .2454$

$S = .06$

$V = Q/A = 6/.7854 = 7.64$ fps Too High

Try 16" pipe

$Q = \frac{1.49}{.019} (1.4103)(.3375)^{2/3} S^{1/2}$

1.4103 / 4.1783

$S^{1/2} = .1119$

$S = .0125$

Use 1.5%

$V = 6/1.4103 = 4.25$ fps. Ok ✓

1.5% for 75' = 1.12'

87 - 1.12 = 85.88 (Use 85.9 as Outlet at Pond.

POND INFLOW:

0.65 cfs @ T =

REGISTERED SURVEYOR 9196 NEW MEXICO RICHARD HUGH ALLEN

DESIGN PROFESSIONALS, INC.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-6338 87107

JOB ACADEMY SQUARE JOB # 8042
BY R.H. Allen DATE 5/14/81
CHK. DATE
SCALE SHEET NO. 14 OF 18

AREA 10

GROSS AREA:

192 × 427	81984
½(12)(30)	180.
½(90)(137+78)	4095.
½(21)(85+47)	1386
½(60)(35)	1050
½ 173(137+149)	24739.
½ 46(137)	3151.
45(33)+67(47)	2624
½(15)(37+32)	517
TOTAL	119726 = 2.75 Acres

Hardsurface

ABC 2½	2.5(3700)	= 9250
PCA	3(3580)	= 10740.
Roads	80(35)+26'(255)	= 8030
Garages	15(240)	= 3600
Stalls	15(180)	= 2700
Walks	4'(705)	= 2820
Patio, Misc.	17(50)+1670	= 2570
		39660 ft² = 0.91 Acre

Landscape/Open = 2.75 − .91 = 1.84

Ave C: 1.84(.24)/2.75 + .91(.95)/2.75 = 0.47 Use I = 5.4

Q_{100} = .47(2.75)(5.4) = 6.98 cfs V_{100} = 12192 Ft³

Q_{10} = 3.49 cfs

AREA 10A ⇒ DRAINS TO ACADEMY UNDER SIDEWALK

TOTAL AREA = ½(37(65)+66(127)+200+92(6)) + 38(8) + ½(65)(45) + ½(18)(76) = 12787 ft²

Hard surface: ½(3580)+2467+48 = 6095 = 0.14 Acre

Landscaped = 0.29 − .14 = 0.15 Acre

Composite C for AREA 10A:

Ave = .14(.95)/.29 + .15(.24)/.29 = 0.58

V_{100} = 2.6/12(.58)(12787) = 2770 Ft³

Q_{100} = .53(.29)(5.4) = 0.91 cfs ⇒ Flow is negligible since it is discharged at 2 points to ACADEMY OK

DESIGN PROFESSIONALS, INC.
4701 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-6338 87107

JOB ACADEMY SQUARE JOB # 2077

BY RH ALLEN DATE 5/12/81

CHK. DATE

SCALE SHEET NO 15 OF 18

AREA 10B ⇒ DRAINS TO CULBANE

GROSS AREA = 38(30) + 58(70) + 64(98) + ½(27)(64+58) = 9919 ft²

Hard = 40(95) + 2386 + 50 = 4236 = .10 Acre

Landscape = .23 − .10 = .13 Acre

Cave = .13(.24) + .10(.95) = .55
.23 .23

Q_{100} = .55(.23)(5.4) = .68 cfs OK (Negligible)

V_{100} = 2.6/12 (9919)(.55) = 1182 Ft³

BALANCE OF AREA 10 ⇒ DRAINS TO INTERIOR HOLDING PONDS

Gr. AREA = 2.75 − (.23+.29) = 2.33

Hard " = 0.91 − (.14+.10) = 0.67

Landscaped = 1.84 − (.15+1.13) = 1.56

Cave = (.24)(1.56) + .95(.67) = 0.43
2.33 2.33

REGISTERED ENGINEER STATE NEW MEXICO 6290 RICHARD HUGH ALLEN

Q'_{100} = .43(5.4)(2.33) = 5.41 cfs V'_{100} = 9456 Ft³

Q'_{10} = 2.71 cfs

Pond Size: 9456 / 1.0 = 9456 ϕ Bot. Pnd. Elev: 96.0

Use 100 X 100 = 10,000 ϕ

Depth of water = 0.95' OK

Outlet: Rate: Use 2 cfs.

$Q = \frac{1.49}{0.19}(S^{1/2})(A)R^{2/3}$ Try 10" Pipe

S = 2.7% Q = 2.2 cfs

Run to M.H. in landscaped area.

NOTE: Pond & 10" Pipe to M.H. are temporary. Long term system will be designed w/ 8 additional units in the area of temp. pond. M.H. will remain to carry runoff from newly developed area.

DESIGN PROFESSIONALS, INC.
4301 Carlisle Boulevard N.E.
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(505) 881-6338 87107

JOB ACADEMY SQUARE JOB # 8042

BY RH ALLEN DATE 5/12/81

CHK. DATE

SCALE SHEET NO 6 OF 18

AREA 10 (Cont'd)

Max. discharge rate to pond will be due to short term high intensity storm. (1 hour)

Qmax based on a Tc = 8 min

5 min I = 7.68"/hr Q = .43(7.68)(2.33) = 7.7 cfs

3 min I = 4.20"/hr Q = .43(4.2)(2.33) = 4.2 cfs

weighted ave Q = 6.4 cfs

Pond sized for total retention. Swales, streets etc... will carry this short period high flow.

Inverts for D.I. & M.H.

D.I. inv of 10" pipe = 95.5 - 1.5 = 94.0

Manhole invert: 94.0 - 1.6 = 92.4

Slope = 2.0%

AREA 11 - Drains to river street system & thus to Dwali.

Area = 11,00 ± (.27 Acre) Landscaped = 8955 ± (0.21 Acre)

Hard area = 2745 ± = .06 Acre

$C_{ave} = \frac{.21}{.27}(.25) + \frac{.95}{.27}(.06) = 0.40$

$Q_{100} = .4(.27)(5.4) = 0.58$ cfs $V_{100} = 1014 ft^3$

Inflow to street & thus to Dwali is negligible and should not present a problem for Dwali drainage.

REGISTERED ENGINEER STATE No. 5286 NEW MEXICO RICHARD HUGH ALLEN

DESIGN PROFESSIONALS, INC.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico
(505) 881-6338 87107

JOB ACADEMY SQUARE JOB # 3042

BY RH ALLEN DATE 5/12/21

CHK. DATE

SCALE SHEET NO. 17 OF 18

BUILDING SIZES USED IN RUN-OFF CALCULATIONS

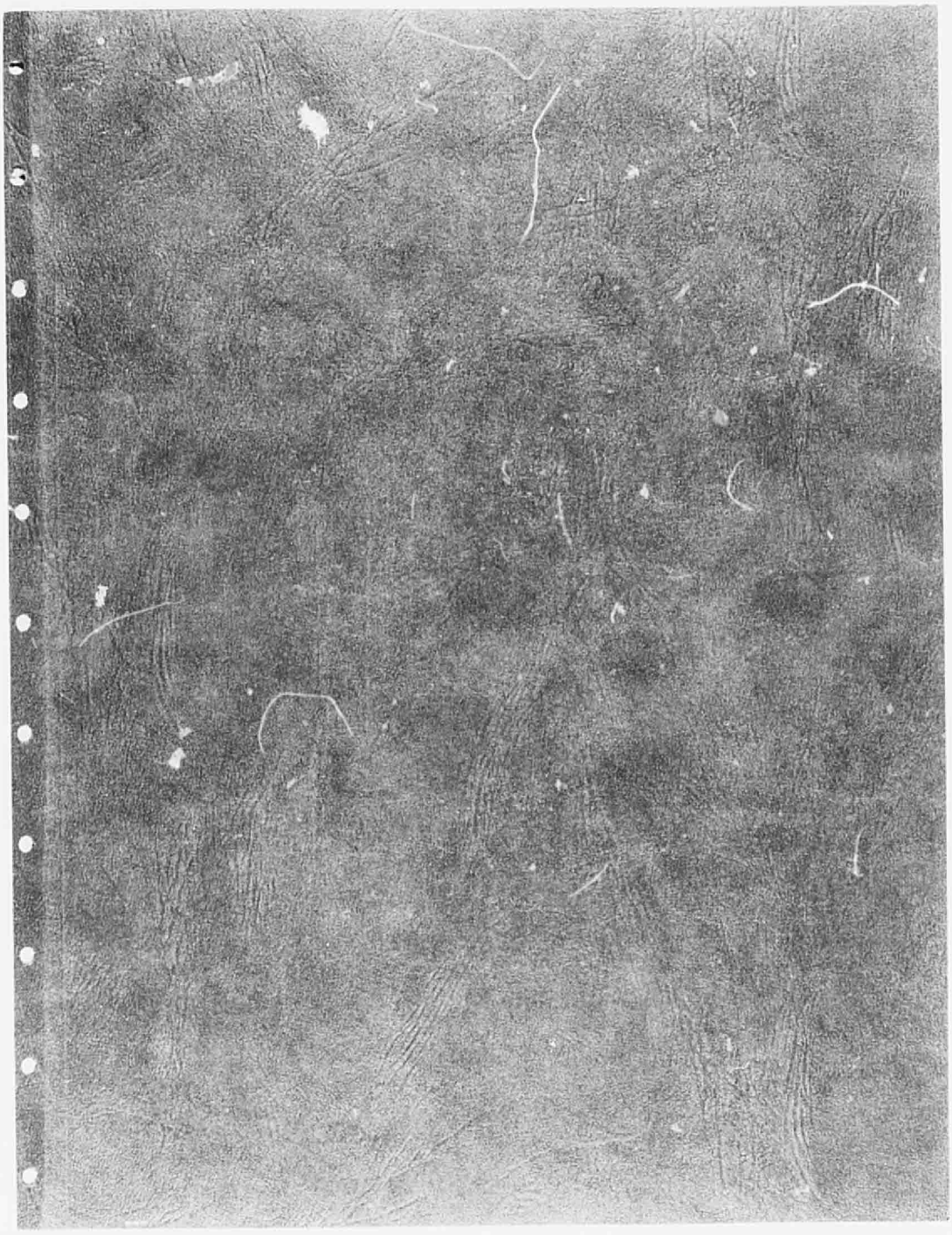
Drainage Study - Grupe Tract

Developed hard surfaces

Unit sizes in various combination

A, B, C	=	75'x30' + 37'x39' =	3700 s.f.
C, D	=	36'x65' =	2340
A, E, D	=	32x95 + 69x 31' =	3580
C, C	=	57'x37' =	2480
A, C	=	36'x77' =	2770
Garage	=	11'x22' =	240
Parking Space	=	9'x20' =	180
Roadway	=	L.f x26' =	

Sheet 18 of 18



E21-D8

DESIGN PROFESSIONALS, INC.
4301 CARLISLE BOULEVARD, NE/ALBUQUERQUE, NEW MEXICO 87107/(505) 881 6336

March 20, 1981

RECEIVED
MAR 20 1981
CITY ENGINEER

City of Albuquerque Drainage Division
Post Office Box 1293
Albuquerque, New Mexico 87103

SUBJECT: Academy Square for Grupe Development Company

Gentlemen:

Attached are three copies of the following data pertaining to the previously submitted drainage report for the subject project:

1. Landscape drawings at 1:20 scale indicating detailed internal site grading. Please note that the original drainage report concept for handling run-off has not changed with the exception of the off-site run-off ponding requirement. The off-site water will be handled per AMDEC's letter of 2/18/81, copies of which are enclosed for your files.

2. Overall site grading plan at 1:50 scale.

It should be noted that the area containing the pond which was originally designed to hold the off-site run-off will be developed at some future point in time. A revised grading/drainage plan will be submitted for this area after plans are finalized by the Owner. The current drainage plan will utilize this area for temporary ponding of on-site run-off as delineated in the original drainage report.

Please advise if there are any questions.

Sincerely,

DESIGN PROFESSIONALS, INC.

Richard H. Allen

RICHARD H. ALLEN, P.E.
Vice President

cc: Dave Kochert

ARCHITECTS/ENGINEERS
GEORGE H. BOLLING
ROBERT L. ROCHELEAU
JOHN PETER VARSA
RICHARD H. ALLEN
HOWARD S. COTTRELL
RICHARD G. VAUGHAN

affiliated mortgage & development company amdec

a wholly owned subsidiary of Albuquerque Federal Savings and Loan Association

6400 uptown blvd., n.e.
albuquerque, new mexico

mailing address
p. o. box 23000
albuquerque, new mexico 87125

AM - 742

February 18, 1981

Mr. Richard S. Heller, P.E.
City Engineer
City of Albuquerque
P. O. Box 1293
Albuquerque, New Mexico 87103

Subject: Drainage Swale North of the Intersection of Academy Road N.E. and Eubank Blvd., N.E.

Dear Dick:

The intent of this letter is to confirm our conversation in your office this morning during our meeting with Dick Vaughn and Dave Kochert.

As we discussed, plans currently being prepared for the construction of a 10 year storm sewer and a 100 year earth channel section from subject intersection north to a desilting pond. These inprovements are scheduled for construction this summer (approximate start date is July 1, 1981) in a City of Albuquerque Special Assessment District.

It is my understanding that you will require at least the earth section north from subject intersection to be excavated before the City will issue occupancy permits for any of the Grupe development southwest of this intersection.

AMDeC shall excavate subject earth channel section at the time it is required for occupancy permits if this work has not already been performed as part of the above noted Special Assessment District.

I hope this is what you needed. If not please contact me immediately at 883-3501.

Thank you for your help in this situation.

Sincerely,

AFFILIATED MORTGAGE & DEVELOPMENT COMPANY

Allan L. Whitesel

Allan L. Whitesel, P.E.
Chief Engineer

c.c. Richard Vaughn & David R. Kochert

affiliated mortgage & development company amdec

a wholly owned subsidiary of Albuquerque Federal Savings and Loan Association

6400 uptown blvd., n.e.
albuquerque, new mexico

mailing address
p. o. box 25000
albuquerque, new mexico 87125

AM - 742

February 18, 1981

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City of Albuquerque
P. O. Box 1293
Albuquerque, New Mexico 87103

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c.c. Richard Vaughn & David R. Kochert

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mailing address
p. o. box 250 10
albuquerque, new mexico 87125

AM - 742

February 18, 1981

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City Engineer
City of Albuquerque
P. O. Box 1293
Albuquerque, New Mexico 87103

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Allan L. Whitesel

Allan L. Whitesel, P.E.
Chief Engineer

c.c. Richard Vaughn & David R. Kochert

E21-D0 pm

81 40476

307

DRAINAGE COVENANT

THIS COVENANT made this ____ day of July, 1981, by and between the CITY OF ALBUQUERQUE, a municipal corporation ("City"), and STATE SAVINGS AND LOAN ASSOCIATION, a California State Savings and Loan Association ("Owner" which term includes successors and assigns).

RECITAL

The Owner is owner of certain real property located at South-West Corner of Eubank and Academy, N.E., in Albuquerque, New Mexico (the "Property"), and more particularly described as follows:

> Tract Lettered "A" of the summary plat of Academy Hills Subdivision, Unit 5, an Addition to the City of Albuquerque, New Mexico, as the same is shown and designated on the Plat thereof, filed in the Office of the County Clerk of Bernalillo County, New Mexico, on May 30, 1978.

That pursuant to City ordinances, regulations, and other applicable laws, the Owner is required to install and maintain certain drainage facilities on the Property, and the parties wish to provide for an agreement as to the obligations and responsibilities for same.

DESCRIPTION OF FACILITIES

Those ponding facilities as more fully shown and described on Exhibit "A" attached hereto and incorporated herein by reference (the "Drainage Facilities") are to be constructed on the Property and maintained by Owner.

CONSTRUCTION OF DRAINAGE FACILITIES

The Owner shall construct the Drainage Facilities in accordance with those plans and specifications shown on Exhibit "A" which have been approved by the City.

MAINTENANCE OF FACILITIES

The Owner shall, at his cost in accordance with the standards, plans and specifications prescribed by the City, maintain the Drainage Facilities. Upon prior written notice to the Owner and during normal business hours, the City shall have the right to enter upon the Property to inspect the Drainage Facilities.

FAILURE TO COMPLY AND LIEN

In the event that the Owner shall fail to construct or adequately maintain the Drainage Facilities, the City shall give the Owner notice in writing to construct, correct, or maintain said facilities, and if the Owner fails to comply therewith within fifteen (15) days, the City may enter upon said Property to perform the necessary construction or maintenance. The cost of the City's performing such construction or maintenance shall be paid by the Owner. In the event the Owner fails to pay said cost within thirty (30) days after being billed for same, the City may file a lien against the Property.

LIABILITY

The City shall not be liable for any damages to the Owner resulting from its construction, modification, or maintenance of said facilities except as that which results from the gross negligence, willful or intentional acts of the City. However, the City shall be subject to no tort liability from which they are immune pursuant to state law.

NOTICE

The written notice provided for herein shall be accomplished by mailing same to:

State Savings and Loan Association
c/o Mr. John A. Myers
Johnson and Lanphere, P.C.
6400 Uptown Blvd., N.E.
Suite 200-West
Albuquerque, New Mexico 87110

The Owner may change said address by giving written notice, certified mail, return receipt requested, to the City Engineer, City Hall, Room 403, at 505 Marquette Street, Albuquerque, New Mexico, 87103.

INDEMNIFICATION AND HOLD HARMLESS

The Owner shall agree to defend, indemnify, and hold harmless, the City, its officials, agents and employees from and against any and all claims, actions, suits, or proceedings of any kind brought against said parties for or on account of any matter arising from the Drainage Facilities provided for herein or the Owner's failure to construct, maintain, or modify the Drainage Facilities under this Covenant.

COVENANT RUNNING WITH THE PROPERTY

The obligation of the Owner set forth herein shall be binding upon the Owner, his heirs, and assigns, and the Property and will run with the Property until released by the City.

OWNER:

STATE SAVINGS AND LOAN ASSOCIATION,
A California State Savings and
Loan Association

By _____
Its Vice President

REVIEWED BY THE LEGAL DEPARTMENT:

Malcolm W. deVesty
Assistant City Attorney

CITY OF ALBUQUERQUE

By _____
Chief Administrative Officer

STATE OF _____)
) ss.
COUNTY OF _____)

On _____, 1981, before me, the undersigned, a Notary Public in and for said State, personally appeared _____ John A. Larson _____, known to me to be the person whose name _____ subscribed to the within instrument and acknowledged that _____ executed the same.

WITNESS my hand and official seal.

Signature _____

OFFICIAL SEAL
SUSAN K. WILLIAMS
NOTARY PUBLIC - CALIFORNIA
SAN JOAQUIN COUNTY
My Commission Expires June 25, 1982

STATE OF NEW MEXICO)
) ss.
COUNTY OF BERNALILLO)

The foregoing instrument was acknowledged before me this 20th day of July, 1981, by [illegible], Chief Administrative Officer of the CITY OF ALBUQUERQUE, a municipal corporation, on behalf of said corporation.

[illegible]
Notary Public

My commission expires:

10/10/83

Drainage Report for El Paseo Development (formerly Academy Square), Academy Hills Subdivision, Albuquerque, New Mexico, prepared for Grupe Construction Company, P. O. Box 7576, Stockton, California, 95207, by Design Professionals, Inc., 4301 Carlisle Blvd., N.E., Albuquerque, New Mexico, 87107, on May 14, 1981 (Revised June 20, 1981), and supporting drawing, City of Albuquerque drawing Number 1130 Sheet 1 of 13 through 13 of 13 and Letter of Acceptance from the City of Albuquerque, dated July 2, 1981.

STATE OF NEW MEXICO
COUNTY OF BERNALILLO
FILED

JUL 29 1 25 PM '81
Misc 868 307-311
TENNY C. ULP
CO. CLERK & RECORDER
DEPUTY

EXHIBIT "A"

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

January 22, 1981

Design Professionals, Inc.
4201 Carlisle Blvd. N.E.
Albuquerque, New Mexico 87107

Attention: Mr. Richard Vaughn

RE: ACADEMY SQUARE DEVELOPMENT - EUBANK BLVD. AND ACADEMY RD.

Dear Mr. Vaughn:

We have reviewed your proposal for the subject subdivision and have found that site subject to substantial flood hazard.

The previous use of that land was for the detention of storm waters emanating from lands east of Eubank Blvd. and for the detaining of same so that they would not flow down Academy Rd. The development of this property without a suitable discharge point for the upland flows will not be granted by the City of Albuquerque, until such time as there is a solution for the waters emanating from the east.

Should you have any further questions regarding this property or its development, please call me.

Very truly yours,

Richard S. Heller
City Engineer

RSH/BC/tsl

MUNICIPAL DEVELOPMENT DEPARTMENT
Richard S. Heller, P.E., City Engineer ENGINEERING DIVISION Telephone (505) 766-7467

COMMENTS PERTAINING TO ACADEMY SQUARE-PLAN PREPARED BY

Dick Vaughn and Associates

1. The definition of the different watersheds within the development is very approximate when present, so that it is impossible for the writer to verify the Engineer's thoughts or intent. In some cases the ~~device~~ divide is shown to run parallel to the contours which is obviously not acceptable. I find that there is no indication on where the buildings will drain; in some cases this is important. In order to verify ~~lots~~ flow quantities and directions. As an example: ~~Building~~ The grading adjacent to Academy Blvd. indicates the potential for some drainage to occur over the sidewalks. If only a small strip of landscaped land is involved there is no concern, but uncontrolled drainage of the buildings over the sidewalk is not acceptable.

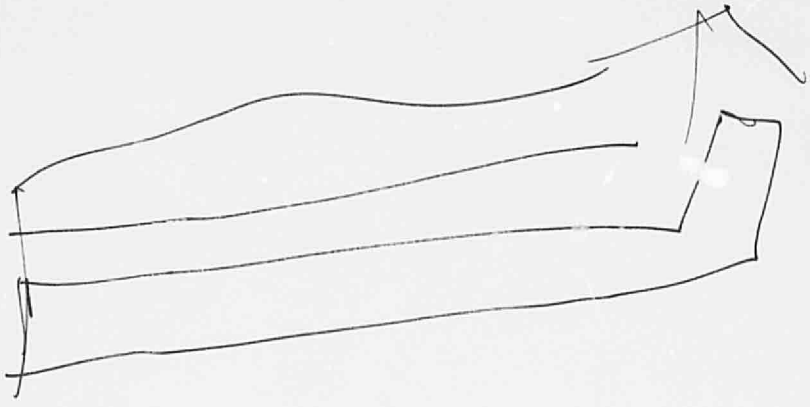
2. The drawing is of difficult interpretation due to the small scale; a great amount of information needs to be recorded ~~from~~ on the drawing and ~~they have~~ the writer has a harder time understanding the relationship ~~under~~ between the existing and the proposed ground elevations. This problem is particularly evident along the west and south property line(adjacent to the existing development). More specifically to the north and east of lots 116, 117, and 118 of Academy Hills 5 the grading plan shows the ground to be sloped towards the mentioned lots. The proposed contour line adjacent to the property line has an elevation of 5697 msl versus an existing elevation of 5694-5695 msl.

3. It is essential for the reviewer to understand the realtionship between the proposed grading of this parcel and the existing topography of the adjacent townhouse development. I want to remind the engineer that the AMAFCA Resolution 72-2 (Adopted and part of the City rules) require that the topographic information supplied by the engineer extend 100 feet beyond the limits of his client's property. I, therefore, expect some reasonable compliance to this requirement. Another problem pertaining to this draining plan is related to the overflow channel. The dimensions of this channel are such that the top width is 28 feet or slightly more than 1/2 inch on the scale shown. It is obvious that this channel cannot be fitted between the property line and the top of the curb. There is also the observation that regardless of the acceptability of the concept, the City could never concurr to the construction of such a channel outside of the property of the engineer's client and within the street right-of-way.

5. The pond adjacent to lots 103,104, and 105 Academy Hills Unit 5 is somewhat difficult to visualize in that the grading plan seems to convey the message that the pond is constructed on a slope. In general, regarding the ponds could be constructed within the development, if their depth exceeds 18". Said ponds must be properly fenced with a fence no less than 3 feet in height. If the depth of the pond exceeds 3 feet this office will require a cyclone fence no less than 6 feet in height. The drawing seems to suggest the construction of a fence along the property line which

is not sufficient indication for this office.

6. As a concluding remark on this draining plan, I note that much better details will have to be supplied with the construction plans in order to obtain approval from the department. For instance, the cutfall pipes between ponds and from the major pond at Academy Road must be snown n profile with a clear indication of the existing ground, the proposed ground in relationship of the pipe tc the above. It is unclear to the writer what is the proposed relationship of the outfall pipe from the pond collecting the cutflow from the two 21 inch pipes and coveying to Academy with the proposed ground elevation. Even more how this 10 inch pipe is going to drain onto to Academy Road. This information has to be supplied in much better detail than what is available and certainly not with notes.



R R F.

City of Albuquerque

P.O. BOX 1293 ALBUQUERQUE, NEW MEXICO 87103

May 4, 1981

E21-D8

Mr. Richard Allen
Design Professionals, Inc.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico 87107

Subject: Drainage Report for Academy Square

Dear Mr. Allen:

As verbally indicated I have some problems with the drainage report and plan submitted. I am sure that with your cooperation the objections and concerns of the City can be expeditiously resolved. Regretfully the grading plan, on a large scale, recently forwarded to my office by the architect bears but little resemblance to your drainage plan (i.e. uncontrolled discharge on the sidewalks onto unimproved public streets, existing topography absent and proposed topography impossible to execute correctly etc.) Please contact my office at your earliest convenience for a resolution of this matter.

Very truly yours,

Bruno Conegliano
Assistant Hydrology Engineer

BC/fs

cc - R. S. Heller
R. Leonard
Drainage File

Check if this is a fact why sign the site plan

RSH RSH ADM
HRO SUR
CDS COUN
DES SEC
INSP FILE
HYDRO RETURN

MUNICIPAL DEVELOPMENT DEPARTMENT

Telephone (505) 766-7467

RSH

Mr. Richard Allen
Design Professionals, Inc.
4301 Carlisle Boulevard N.E.
Albuquerque, New Mexico 87107

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Very truly yours,

Bruno Conegliano
Assistant Hydrology Engineer

BC/fs
cc - R. S. Heller
R. Leonard
Drainage File

Check if this is a fact why sign the site plan

RSH	RSH	ADM	
HRO		SUR	
CDS		COUN	
DES		SEC	
INSP		FILE	
HYDRO		RETURN	

MUNICIPAL DEVELOPMENT DEPARTMENT

Telephone (505) 766-7457

RSH
- SIGNED THE SITE PLAN
TO GET RICH'S COPY OF
HIS DRAIN REPORT. HE HAS
THE PROJECT COORDINATOR
AND I NEED TO CALL HIM
I NEED TO FIND OUT
WHAT HAVE BEEN DONE AND
I DO NOT NEED HIM ON THIS

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

May 28, 1981

Mr. Richard Allen
Design Professionals, Inc.
4201 Carlisle Blvd. N.E.
Albuquerque, N.M. 87107

RE: EL PASEO (ACADEMY SQUARE) DRAINAGE REPORT

Dear Mr. Allen:

I have reviewed the revised drainage report for the referenced development and I find that it represents a commendable and significant improvement over the previous submittal. Unfortunately, the needs of the City still have not been met in two significant ways:

First, in his drainage report for Academy Hills, Units 4-6, Mr. Whitman, under a set of perfectly reasonable assumptions, identified the need to supply a detention volume of 1.077 acre feet; this amount of detention was considered necessary in order to prevent adverse consequences downstream, or, at the very least, not to worsen the existing conditions.

That report was approved by the City, and I expected to receive from your office a demonstration of essential compliance with the findings of that report.

As I have indicated to you, I am concerned with the existing unimproved conditions of Academy Road and with the absence of adequate drainage facilities to the west of the existing development. Therefore, I consider it imperative, that all the runoff generated by proposed new development, finds an ultimate destination in the Pino Arroyo.

Secondly, it is essential that from the grading plan, both the City Engineers Office and the Zoning Officer be able to verify compliance with the existing codes. Specifically, the manner in which existing facilities are protected must be evident from the plan. Notwithstanding my prior request, the topography and conditions of the adjacent property is not defined. On sheet L-4, it is shown that the existing ground elevation will be lowered, by excavating adjacent to the existing property. This excavation which appears to approach or exceed 8

feet may place in serious jeopardy the existing improvements. Without a clear indication on how you propose to proceed, the City will not be able to approve the grading plan submitted.

If you desire to review these matters further, please feel free to contact Bruno Conegliano or me for an appointment.

Very truly yours,

Charles M. Easterling
PACE/Hydrology

BC/CE/tsl

cc: R. S. Heller, City Engineer
Rich Leonard, AMAPCA

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

R.F.
E 21-D8

July 2, 1981

Mr. Richard Allen
Design Professionals
4201 Carlisle Blvd. N.E.
Albuquerque, N.M. 87107

Re: EL PASEO DEVELOPMENT

Dear Mr. Allen:

Purpose of this letter is to indicate the concurrence of the City Engineer's Office with the drainage plan for the referenced development. While legitimate differences of opinion exist regarding the pertinence or applicability of certain assumptions of the report (i.e. C. values, temporal distribution of design storm) I feel the need to express to you my appreciation for the thoroughness and professionalism evident in your report.

It is my hope that efforts presently underway to develop check lists, procedures and design manuals will assist the consulting engineers community in serving better the private developers and the community. These documents should remove uncertainties by establishing standards; I hope that this office may be able to obtain your constructive criticism before final publication.

As I have indicated to you, difficulties encountered in obtaining adequate maintenance of the drainage facilities and of the detention areas have prompted the City Engineer to request the execution of maintenance covenants. Therefore, before the plat for this development is signed off by the City Engineer the attached covenant will have to be executed by the Developer.

Further, in view of the need to preserve the ponding volume presently available in Area 10, a temporary drainage easement will have to be supplied over the pond in question. Said temporary easement should contain a reversionary clause to allow development of the remainder of Area 10; at the same time the City must receive a commitment for the preservation of the storage volume discussed in the report.

MUNICIPAL DEVELOPMENT DEPARTMENT
Richard S. Heller, P.E., City Engineer ENGINEERING DIVISION Telephone (505) 766-7467

In case you need clarification on any of the above, please do not hesitate to contact my office.

Sincerely,

Charles M. Easterling
Princ. Asst. City Eng./Hydrology

CME/BC/tsl