HYDROLOGY E21/D20 4 PLANS

E21/D20

4 Plans

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

April 6, 1982

Mr. Rich Allen
Design Professionals, Inc.
4301 Carlisle Blvd. N.E.
Albuquerque, N.M. 87107

RE: EL PASEO DRAINAGE REPORT PHASE II

Dear Rich:

The revised plan for the El Paseo II Development is approved.

Very truly yours,

Brian G. Burnett
Civil Engineer/Hydrology

BGB/tsl

HALL SURVEYING CO.

PHONE 884-6444 / 884-6200 ALBUQUERQUE, NEW MEXICO 87110 2625 PENNSYLVANIA, N.E. SUITE 350

TO: Brian Burnett
City Engineer
Hydrology
Albuquerque, N.M.

FROM: Richard Hall
Engineer
2625 Pennsylvania Ave. #350
Albuquerque, N.M. 87110

RECEIVED
JUL 20 1982
ENGINEERING

Reference: El Paseo Townhomes

Dear Brian,

We at Hall Surveying and Engineering Co. have been involved in the project El Paseo Townhomes for over a year. Our Responcibilities have been to stake and lay-out the entire project and construction grading and finish floor elevations. I know from personal on-site inspection that the grading plan has been followed. Every finished floor elevation has been tied to city datum and is accurate to 2" plus or minus. Enclosed is a copy of field survey elevations marked on a grading plan. If I can be of further service, let me know.

RECEIVED
JUL 20 19
ENGINEERING

Sincerely,

Richard Hall

Richard Hall
PE # 7918

RICHARD V. HALL
NEW MEXICO
No. 7918
REGISTERED PROFESSIONAL ENGINEER

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

March 11, 1982

E21

RECEIVED
MAR 12 1982
CITY ENGINEER

Mr. Chuck Easterling, P.E.
Hydrology Department
City of Albuquerque
Post Office Box 1293
Albuquerque, New Mexico 87103

SUBJECT: Drainage Report for EL PASEO - PHASE II (8212.)

Dear Mr. Easterling:

Attached are two copies of the Drainage Report for the referenced project. The report contains calculations and information which indicates that the intent of Resolution 72-2 has been met. Also, the calculations show that the minimum detention pond areas which were agreed upon by your office and Grupe Construction has provided. (See copy of Grupe's 7/7/81) letter in Appendix).

A preliminary meeting regarding the subject site was held with Brian Burnett on February 17, 1982. Basic concepts, flows and volumes which were presented at that time were acceptable to Brian and have formed the basis for this report.

Also included herewith is a copy of the PHASE I drainage report drawings for your files. The calculations and written report were transmitted to you under separate cover.

Your earliest attention to the review of this report would be appreciated. Please advise if there are any questions.

Sincerely,

DESIGN PROFESSIONALS, INC.

Richard H. Allen

RICHARD H. ALLEN, P.E.
Vice President

cc: Dave Kochert

RECEIVED
MAR 15 1982
ENGINEERING

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

DRAINAGE REPORT FOR
EL PASEO DEVELOPMENT
PHASE II
ACADEMY HILLS SUBDIVISION
ALBUQUERQUE, NEW MEXICO

PREPARED FOR
GRUPE CONSTRUCTION WEST, INC.
POST OFFICE BOX 7576
STOCKTON, CALIFORNIA 95207

BY
DESIGN PROFESSIONALS, INC.
4301 CARLISLE BOULEVARD, N.E.
ALBUQUERQUE, NEW MEXICO 87107

MARCH 11, 1982

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

ARCHITECTS/ENGINEERS
GEORGE H. BOLLING
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JOHN PETER VARSA
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RICHARD G. VAUGHAN

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

DRAINAGE REPORT INFORMATION SHEET

PROJECT TITLE EL PASEO DEVELOPMENT (PHASE II)

ZONE ATLAS PAGE NO. E-21-Z CITY ADDRESS 6401 EUBANK, N.E. (PHASE I AND II)

LEGAL ADDRESS TRACT A, OF SUMMARY PLAT, ACADEMY HILLS SUBDIVISION, UNIT 5

ENGINEERING FIRM DESIGN PROFESSIONALS, INC. CONTACT RICHARD H. ALLEN, P.E.

ADDRESS 4301 CARLISLE BOULEVARD, N.E. PHONE 881-6336

OWNER GRUPE CONSTRUCTION WEST, INC. CONTACT DAVE KOCHERT

ADDRESS 5023 JUSTIN, N.W. PHONE 821-0111

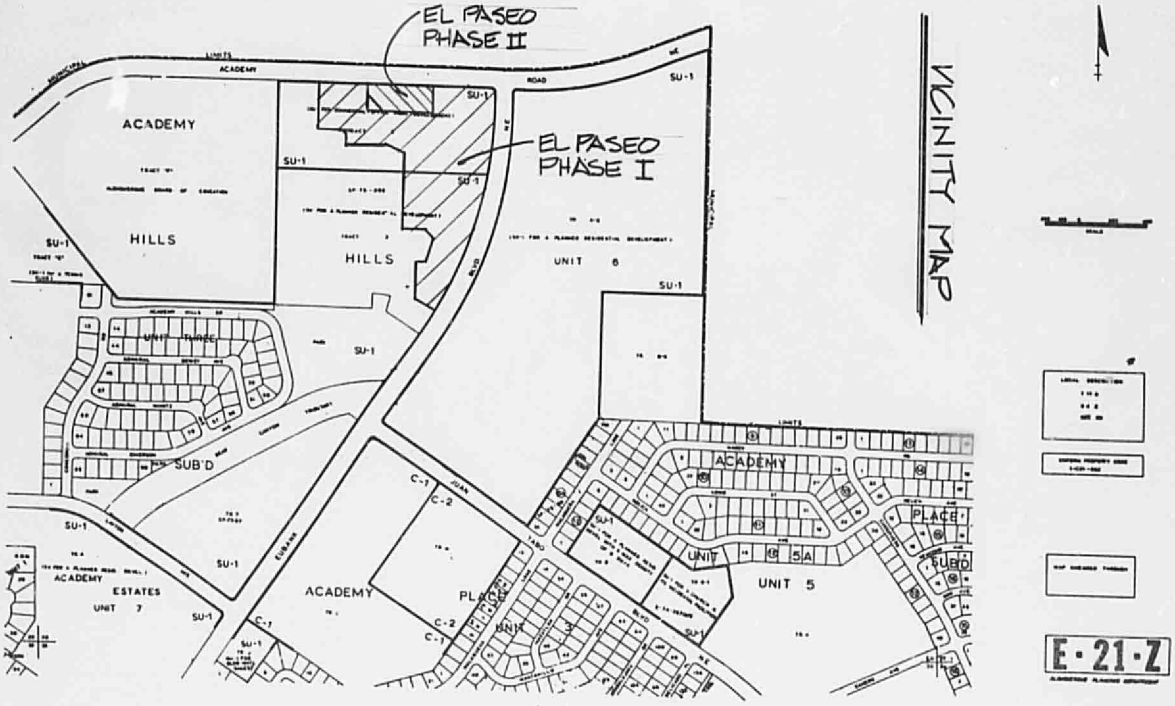
~~ARCHITECT~~/SURVEYOR HALL SURVEYING CO. CONTACT BOB SEXTON

ADDRESS 2625 PENNSYLVANIA, N.E. PHONE 884-6200

DATE SUBMITTED MARCH 10, 1982

BY Richard H. Allen
RICHARD H. ALLEN, P.E.

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


EL PASEO PHASE II
EL PASEO PHASE I
VICINITY MAP
ACADEMY
HILLS
HILLS
UNIT 6
UNIT THREE
SUB'D
ACADEMY
ESTATES
UNIT 7
ACADEMY
PLACE
UNIT 3
ACADEMY
PLACE
UNIT 5A
UNIT 5
SUBD
SU-1
C-1
C-2
E-21-Z

DRAINAGE REPORT FOR
EL PASEO - PHASE II

ACADEMY HILLS SUBDIVISION
ALBUQUERQUE, NEW MEXICO

1. PROJECT DESCRIPTION:

PHASE II of the EL PASEO DEVELOPMENT consists of eight townhouse units to be added to 106 units currently under construction. In addition, due to EPC concerns, several of the 106 units from PHASE I have been reoriented and drainage has been reworked for the units to the west of the new eight (8) units. The development is on Tract A of Academy Hills Subdivision on the southwest corner of the intersection of Eubank and Academy Boulevard, N.E. (This area is shown on Zone Atlas page E-21). The PHASE II development is on the northern side of the site and abuts Academy. The basic site plan was approved by the Environmental Planning Commission at its September 18, 1980 meeting subject to the submission of a drainage report and the resolution of several minor traffic and Fire Department comments. The Drainage Report for PHASE I was approved on July 2, 1981 and is on file with the City Engineer's office. The traffic/fire comments were resolved and incorporated into the final approved drawings.

The PHASE II conceptual site plan was reviewed by the EPC at their January 21, 1982 meeting. Access to the new units has been changed as recommended by the EPC based on comments received from the Traffic Engineer's Office. The new access has required the reorientation of several of the PHASE I units. Effects on the overall site drainage due to this reorientation are reflected in this Drainage Report.

2. EXISTING CONDITIONS:

PHASE I development required that a large holding/detention pond be constructed on the PHASE II site. This pond was designed to intercept runoff from other parts of the site, detain it and slowly release it at a rate of 2.2 cfs to Academy. By a commitment letter from Grupe Construction, dated July 7, 1981, the Owner has agreed to maintain ponding in this area to detain a minimum of 9,896 cu. ft. after development is completed. (See Item a. in the Appendix). New permanent detention areas will be provided in the PHASE II development to fulfill this requirement and the existing PHASE I pond will be filled and graded to receive the new

units. Thus, the temporary drainage easement requested by the City in their letter of July 2, 1981 will not be required once the final grading/development is completed in PHASE II. The plat for this site has been revised to reflect the elimination of this easement.

Runoff released from the detention ponds of PHASE II will be conveyed underground to a field drop inlet and discharged to Academy through a culvert drain which will pass below the sidewalk and curb on Academy.

An erosion control plan should not be required as the total site will be built out at one time. Should delays in construction necessitate some kind of dust/erosion control plan, however, steps will be taken to work out an acceptable plan with the City.

Additional information regarding the total site is contained in DPI's Drainage Report for EL PASEO PHASE I, revised on June 20, 1981 and accepted by the City on July 2, 1981. Pages 1 and 2 of said report also contain existing on-site flow calculations for the site prior to development.

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 flowing less than full. The roughness coefficient used for corrugated metal pipe was .022, that for RCCP was .013, and that for transite pipe was .010. Entry, friction and velocity head losses were checked for underground pipes 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 and Associates.

B. 100 Year-6 Hour Precipitation Map for Albuquerque (1967).

dpi

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 Seeyle (1960).

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

4. PROJECT ANALYSIS:

The site was divided into ten (10) major areas for analysis. Areas 2, 3, 5 and 7 drain to the internal street system and eventually reach Denali. Area 6 drains to an internal drywell. Areas, 1, 4, and 9 drain directly to Academy. Areas 8 and 10 drain to the upper and lower detention ponds and ultimately empty into Academy. Runoff from PHASE I development enters the upper holding pond and eventually drains to Academy. Based on a 1-hour high intensity storm, all ponds will empty via gravity flow in approximately 2 hours and 10 minutes.

There is no offsite runoff to contend with on this project due to the construction by AMDEC of a diversion channel east of Eubank.

Calculations for runoff, pond sizes, pipe sizes, etc. are contained in the Appendix bound in this report. Flow rates, pipe sizes, etc. are shown on the attached Drainage Plan.

5. SUMMARY:

AREA NO.	COMPOSITE 'C'	AREA (SF)	Q100(CFS)	V100(CF)
1	0.64	2,452	0.19	340
2A	0.66	3,630	0.30	519
2B	0.60	4,466	0.33	581
2C	0.81	1,146	0.12	201
3	0.67	2,262	0.19	328
4	0.63	6,586	0.51	500
5	0.71	21,204	1.56	3,262
6	0.52	2,760	0.15	273
7	0.74	12,510	1.08	2,006
8	0.70	17,774	1.35	2,520
9	0.50	12,142	0.61	1,315
10	0.54	18,318	0.94	2,143

UPPER POND:

Bottom Area: 1,670 SF (Bottom Elev. = 5697.5)
Max. Available Capacity: 3,120 C.F. @ 1.5 feet deep
Outlet: 12" diameter R.C.C.P. @ S = .008
Q Max. = 3.19 cfs @ 4.06 fps
Volume detained (max.) for 1-hour high intensity storm = 2,400 C.F.
Emergency Overflow Spillway: Q Max. = 6.73 cfs
Down Stream Channel Velocity = 2.2 fps

LOWER POND:

Bottom Area: 3,130 SF (Bottom Elev. = 5694.0)
Max. Available Capacity: 7,825 C.F. @ 2.5 feet deep
Outlet to Drop Inlet: 12" diameter R.C.C.P. @ S = .005
Q Max. = 2.21 cfs @ 2.81 fps to field drop inlet.
Outlet to Academy: 12" diameter R.C.C.P. @ S = .0237
Q Max. = 3.28 cfs @ 6.99 fps
Emergency Spillway (Notch) Q Max. = 6.06 cfs
Volume detained (max.) for 1-hour high intensity storm = 7,469 C.F.

TOTAL DETENTION:

2400 + 7469 = 9,869 C.F.
(Storage/detention required is within 0.3% of that required by earlier commitments dated July 7, 1981).

Detention ponding with controlled release has been provided for the entire area under consideration. The proposed drainage plan outlined herein and shown on the attached drawings complies with the intent of Resolution 72-2 as adopted by the City of Albuquerque.

July 7, 1981

Mr. Charles M. Easterling, P.E.
Prin. Asst. City Engr./Hydrology
City of Albuquerque
P. O. Box 1293
Albuquerque, New Mexico 87103

RE: EL PASEO DEVELOPMENT

Dear Mr. Easterling:

Grupe Construction West, Inc. plans to construct an additional eight dwelling units within the subject project on the area currently occupied by a temporary retention pond. The specific site is noted as "Area 10" in the approved drainage report for the referenced project.

The purpose of this letter is to state that the storage volume for Area 10, namely 9896 cubic feet, will be preserved after the eight additional units are constructed. This is in compliance with your letter dated July 2, 1981.

Thank you for your assistance with our project, and let us know if you have any questions.

Very truly yours,

GRUPE CONSTRUCTION WEST, INC.

CHARLES A. WALEK
Director of Construction

CAW/hb

cc: Dave Kochert - Superintendent

GRUPE CONSTRUCTION WEST, INC.
2291 W. March Lane, P.O. Box 7576, Stockton, California 95207, 209/951-6660
5023 Justin N.W., Albuquerque, New Mexico 87114, 505/897-1125
A GRUPE CO. California Lic 306470 New Mexico Lic 18150

EL PASEO – PHASE II RE-DO

1. AREA ANALYSIS (Using Planimeter)

AREA No.	Asphalt (SF)	House/Garage (SF)	Sidewalk (SF)	Landscape (SF)	Total (SF)
1		1300	124	1028	2452
2A		1560	791	279	3630
2B		1616	380	470	4466
2C		1062		84	1146
3		1104	376	782	2262
4		3318	400	2868	6586
5	6513	5658	2796	6237	21204
6		936		1824	2760
7	7116	1100	948	3346	12510
8	4884	5000	922	6968	17774
9		3142	436	8564	12142
10		5552	1712	11054	18318
Subtotals	18513 ϕ	31348 ϕ	8885 ϕ	46504 ϕ	105250 ϕ

TOTAL PHASE II DRAINAGE AREA = 2.42 Acre

2. RUNOFF COEFFICIENTS ('C') FOR RATIONALE FORMULA

Character of surface	Range of 'C'	'C' Used
Asphalt (Streets)	0.7 – 0.95	0.90
Concrete Sidewalks	0.75 – 0.85	0.80
Roofs (B.U.R.)	0.75 – 0.95	0.85
Lawns/Landscape	0.25 – 0.35	0.35

A composite 'C' factor for each AREA was calculated based on weighted averages for each type of surface.

Sample Calculation using AREA 8:

$(4884/17774)(.90) + (5000/17774)(0.85) + (922/17774)(.85) + (6968/17774)(.35) = 0.67$ USE 0.70

[Composites for all areas are shown in TABLE on sheet 2.]

REGISTERED ENGINEER STATE OF NEW MEXICO No. 5290 RICHARD HUGH ALLEN

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

JOB EL PASEO PHASE II JOB # 8212
BY RH ALLEN DATE 3/1/82
CHK DATE
SCALE SHEET NO 1 OF 18

TABLE 1 – COMPOSITE 'C' FACTORS

AREA NO.	'C' comp.	AREA NO.	'C' comp.
1	0.64	5	0.71
2A	0.66	6 (Deletions)	(.55) 0.52
2B	0.60	7	0.74
2C	0.81	8	0.70
3	0.67	9	0.50
4	0.63	10	0.54
TOTAL AREA (2.42Ac)	0.63		

NOTE:
Since PHASE II overlaps a portion of PHASE I, the units involved will be re-analyzed. Total runoff, (V & Q) will be maintained so as not to exceed street flows calculated & approved in PHASE I Report.

3. ANALYZE AREA WEST OF PHASE II
A. Flow to Parking Area to West of Exist'g Units

$C_{comp} \cong 0.75$

$Q = C i A$ — Use $T_c = 10$ MIN. $i = 5.4''/HR$

$Q^{100} = 0.75(5.4)(.07)$ — $A = 15(25) + 45(25) + 35(20) + 45(20) = 3100\,\phi$

$Q^{100} = 0.28$ CFS — NEGLIGIBLE

$V_T = 504\ FT^3$ — "

B. Flow To New Drywell

$A = 20(17) + 20(12) = 580\,\phi = 0.013$ Ac.

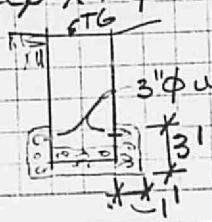
$Q = .75(5.4)(.013) = .05$ cfs

Vol. of runoff $= .75(2.6''/12'')(580) \cong 94\ FT^3$ (6 hrs)

- Percolation rate for soil at Drywell $\cong 2 \times 10^{-2}$ FT/MIN
- Horiz. percolation rate $\cong 10 \times 10^{-2}$ FT/MIN

5' Deep x 4' ϕ Drywell

VOL $= 12.6\,\phi(5) + .33(15.7)(3)$
$\cong 80\ FT^3$



REGISTERED ENGINEER STATE OF NEW MEXICO No. 5290

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

JOB EL PASEO – PHASE II — JOB # 8212
BY RH Allen — DATE 3/1/82
CHK — DATE
SCALE — SHEET NO 2 OF 18

In 6 Hrs., Vert. Percolation will have proceeded
$(6\ \text{Hrs})(60\ \text{Min/Hr})(.02\ \text{FT/Min}) = 7.2'$

Additional Vol. stored $\cong 7.2(\pi)(6^2)(.15)$
$\cong 31\ FT^3$

$80+31 = 111\ FT^3 > 94\ FT^3$
Percolation will also occur horizontally & add to dissipation of water.

USE 5' DEEP WELL.

Original Flow to Parking Area West of Existing PHASE I BUILDINGS (AREA 7 IN PHASE I DRAINAGE REPORT) $\cong 850\ FT^3$ @ 0.4 cfs
∴ New plan does not exceed Vol. or Q. OK.

NOTE: 1 Hr/100yr. precip = .85 6 Hr/100yr precip.

4 PHASE II AREA ANALYSIS

A. AREA 1 (Drains to Academy via culvert)
Use Tc = 10 Min C = 0.64 A = 2452 ☐ = .056 Acre
$Q_{100} = (.056)(5.4)(.64) = 0.19$ cfs $Q_{1HR} \cong 28$ cfs (max in 5 min)
$V_{100} = (.64)(2.6/12)(2452) = 340\ FT^3$

B. AREA 2 (Divided to size pipes internally)
(2A) Use Tc = 10 Min, i = 5.4"/hr A = 3630 ☐ = .083 Acre
$Q_{100} = 0.30$ cfs C = .66 $Q_{1HR} \cong .42$ cfs
$V_{100} = 519\ FT^3$

Since flows are quite small, use standard pedestrian grate in sidewalk to pick up flows. (Similar to NEENAH's R-4940-A Floor drain (2-2' long sections)
Use NEENAH's R-4941-A Grate frame in yard above to intercept flows & discharge through wall to S.W. grate.
4" Transite pipe n ≅ .010
(Johns Manville Literature.

REGISTERED PROFESSIONAL ENGINEER STATE OF NEW MEXICO No. 5290 RICHARD HUCH ALLEN

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

JOB EL PASEO - PHASE II JOB # 8212
BY R.H. Allen DATE 3/3/82
CHK DATE
SCALE SHEET NO 3 OF 18

$Q = \frac{1.49}{0.010} (S^{1/2})(R^{2/3})(A)$

$Q = 149 (.1643)(.302)(.086)$

$Q = .636 \text{ ft}^3/\text{sec} > .30$ OK
7.42

$A = .086\phi$
$S = 2.7\%$
$R = \frac{.086}{.518} = 0.166'$ (Assume 1/2 full)

Use 4"ϕ TRANSITE FOR DRAINS.

(2B) (AREA DRAINS to S.W. INlet)

Let $T_c = 10$ MIN $i = 5.4''/hr$ $C = 0.60$ $A = 4466\phi = .102 Ac$

$Q = 0.33$ cfs, $Q_{100} = 0.47$ cfs

$V_{100} = 581$ cf

Using analysis similar to AREA 2A, pedestrian type S.W. drain will be adequate w/ 4"ϕ transite outlet pipe. Slope to C.O. @ $S = .01'/ft.$

AT Clean out, Δ to 6"ϕ Transite.

$Q_{cap.} = \frac{1.49}{.010} (.018)^{1/2} (.25)^{2/3} (.196)$

$= 149 (.134)(.397)(.196)$

$= 1.55$ cfs $> .30 + .33 = .63$ cfs
$> .42 + .47 = .89$ cfs

1/2 full
$R = .196/.785 = .25$

$A = .196\phi$

∴ **Use 6"ϕ Transite to daylight at ELEV. 805**

(2C) (AREA DRAINS roofs to transite line)

$Q_{100} = .12$ cfs $Q_{100} = .16$ cfs

$V_{100} = 201 \text{ ft}^3$

Dumping directly into 4" line

$Q_T = .12 + .30 = 0.42 < .64$
$.16 + .42 = .58 < .64$ OK

Check 6":

$Q_{T_{100}} = .63 + .12 = .75 < 1.55$
$Q_{T_{100}} = .89 + .16 = 1.05 < 1.55$ OK

REGISTERED ENGINEER STATE OF NEW MEXICO No. 5290 RICHARD H. ALLEN

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

JOB EL PASEO - PHASE II JOB # 8212
BY R.H. Allen DATE 3/3/82
CHK DATE
SCALE SHEET NO 4 OF 18

C. AREA (3) (Drains directly to LAS CASITAS)

$Q_{100} = 0.19$ cfs $Q_{1HR} = 0.27$ $A = 2262 = .052$

$V_{100} = 328$ ft^3

D. AREA (4) (Drains to Academy under sidewalk)

$Q_{100} = 0.51$ cfs $Q_{1HR} = .73$ cfs $A = .151$ Acre

$V_{100} \cong 900$ cf. $i = 5.4$"/hr $C = 0.63$

Use A & culvert (City Std) Width = 12"

$Q = \frac{1.49}{.013}(.06)^{1/2}(d)(d/1+2d)$ Assume S = 6% $A = (1)d$ $R = \frac{d}{1+2d}$

$.73 = 114.62\,(d^2/1+2d)^{2/3}\,(2449)$

$.0026 = \left(\frac{d^2}{1+2d}\right)^{2/3}$

$.00419 = \frac{d^2}{1+2d}$ $d \cong .07$ (3/4")

Use 12" wide culvert (Min. Width) @ 5.4% ±

E. AREA (5) (Drains to Las Casitas under sidewalk)

Tc Computation:

25'	Roof:	1.5 Min.
45'	Lawn/S.W. to Alg:	7.5 Min
137'	Asphalt	2.5 Min
62'	Cobbles to Street	4.5 Min
	TOTAL Tc =	16.0 Min

∴ i = 4.5"/hr. A = 21204 ϕ

$Q_{100} = (.487)(4.5)(0.71) = 1.56$ cfs

$V_{100} = (.71)(2.6/12)(21204) = 3262$ ft^3

$Q_{1HR} \cong (.71)(7.68)(.487) = 2.66$ cfs.

NOTE: In figuring Q_{1HR}, NOAA Atlas values for durations of 1 Hr. storm were used:

5 Min	29% of 1 Hr. precip will fall
10"	45% " " " " "
15"	57% " " " " "
30"	79% " " " " "

∴ i for worst 5 Min of storm = i_5

$i_5 = *(.85)(2.6)(.29)(60/5) = 7.68$"/Hr.

*1Hr/100 yr precip = .85 (6Hr/100 yr. precip)

REGISTERED ENGINEER STATE OF NEW MEXICO No. 5290 RICHARD HUGH ALLEN

Check ft capacity (AT Pk'g) S = 5% n = .013

$Q_{max} = (.71)(.377)(5.2)$ $T_c = 11.5$; $i = 5.2''/hr$

$= 1.45$ cfs $A = 21204 - 35(55) - 37(68) - 17(19) = 16440 = .377$ Ac

$Q_{100} = 2.06$ cfs

$2.06 = \frac{1.49}{.013}(.05)^{1/2}\left(\frac{2d}{2d+2}\right)^{2/3}(2d)$ $A = 2d$, $R = \frac{2d}{2d+2}$

$.0804 = \left(\frac{2d}{2d+2}\right)^{2/3}(2d)$

$d \simeq 2''$ (0.17')

Vel = 2.06 / 2(.17) = 6.1'/sec. OK

Check ft @ Street $Q_{max} = 2.66$ cfs S = 1/27 = .037

$2.66 = \frac{1.49}{.017}(.037)^{1/2}\left(\frac{2d}{2d+2}\right)^{2/3}(2d)$ n = .017

$.158 = \left(\frac{2d}{2d+2}\right)^{2/3}(2d)$

$d \simeq 3''$ (.25')

V = 2.66 / .5 = 5.32 ft/sec OK

Q_{100} & Vel_{100} will be less than those calculated for Q_{100} (short duration / high intensity storm)

F. AREA (6) Drains to Interior Drywell

Note: Fill in tiered walls is 6" below top of wall & therefore storage will take place in this area. No water will be contributed to the runoff going to the drywell from this area.

Area (scheduled) = 2760 ϕ − tiers (472) = 2288 ϕ

New $C_a = 0.55$

$Q_{100} = (56)(.54)(.05) = .15$ cfs $Q_{100} = 21$ cfs

$V_{100} = 273$ Ft³

Use 6' deep × 4' ϕ dry well (See Std. Dtl.)

Vol. stored = $(\pi(4)^2/4)(6) = 75$ Ft³ > 90 Ft³

Rock at bottom: 3'(1')(15.7)(.33) = 15 Ft³

Vol. ponded = 273 − 90 = 183 Ft³

T.G. = 85.3 : Store to Contour 86

Vol = ½(.7)[(3.4)(11) + ½(10)² + ½(33)(6)]

= 183 Ft³ OK

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JOB ELPASEO - PHASE II
BY RH Allen DATE 3/5/82
CHK DATE
SCALE SHEET NO 6 OF 18

BASED on percolation rates, pond will dissipate through drywell as follows:

$.02'/MIN\ (520\ \Phi)(X\ MIN) = 183\ ft^3$
(Area of ponding)

$X = 18\ MIN.$

After 20 MIN ± Area of pond should be drained down.

G. AREA (7) (Flows down street to Las Casitas)

$A = 12510\ \Phi = .287\ Ac$
$C = 0.74$

Tc: 10" Roof : 0.6 MIN
45' Lawn : 7.5 MIN
160' Pav't asphalt : 3.0 MIN
Tc = 11.1 MIN i = 5.1"/hr.

$Q_{100} = (5.1)(.287)(.74) = 1.08\ cfs$ $Q_{1HR} = 1.63\ cfs.$
$V_{100} = 2006\ ft^3$

Runoff will flow 1/2 on each side of street to Las Casitas

H. AREA (8) (DRAINS TO Lower Pond & Double D Inlet)

A. To Inlet:
Compute separate Cc for 8A:

$C_c = \frac{4884}{9938}(.90) + \frac{2462}{9938}(.85) + \frac{922}{9938}(.80) + \frac{1670}{9938}(.35) = 0.78$

$Q_{100} = .78(5.4)(.228) = 0.96\ cfs$ $Q_{1HR} = 1.36\ cfs$ (max)
$V_{100} = 1680\ cf$
Velocity = Q/A = 136/.12 = 11.3'/sec (Assumes gutter is full & all flow is in gutter)

If depth of flow is 1" Vel = 5.7'/sec.
This appears more reasonable due to the slope & short "run" of the water.

*Double 'D' Inlet will handle flow of 1.36 cfs

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NO. 5129

Determine pipe size from inlet to pond
(Use RCCP)
Try 8" ϕ $Q_{max} = 1.36$ cfs $S = .6/24 = .025$
$n = .013$

$1.36 = \frac{1.49}{.013}(.025)^{1/2}(\pi)(.67)^2/4\,(R)^{2/3}$

$.2129 = (R)^{2/3}$ circ. of 8" pipe = 2.1' N.G.

$.0982 = R = A/WP$ $R = .176/1.052 = .167$

For 8" pipe flowing 1/2 full
By trial & error, depth of flow in pipe $\cong$ 3"

Velocity $= \frac{1.36}{0.145\phi} = 9.4$ ft/sec OK ✓

*USE 8" ϕ RCCP @ 2.5% SLOPE TO POND
INV. (INLET) 94.6
INV. (OUTLET) 94.0
*USE CLASS III Pipe Due to minimal depth of cover over pipe

B. TO HOLDING POND (DIRECTLY)
Compute separate Tc & C
$C_c = (2538/7836)(.85) + (4998/7836)(.35) = 0.50$
Tc (Time to Field Inlet) =
25' Roof = 1.5 Min
127' Grass 6 min + 11.5 = 17.5
Tc = 19.0 Min ∴ i = 4.3"/hr.

$Q_{100} = (.5)(4.3)(.18) = .39$ cfs
$V_{100} = 850$ CF
$Q_{1HR} = 0.69$ cfs

I. AREA (9) DRAINS TO FIELD INLET A = 12142 = .279
Tc = Roof (35') 2.1 Min C = 0.50
S = 3.6% Grass (240') 16.0
18.1 Min ∴ i = 4.35"/Hr.

$Q_{100} = (.5)(4.35)(.279) = 0.61$ cfs $Q_{1HR} = 1.07$ cfs
$V_{100} = 1315$ FT^3

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*SINCE FLOWS ARE SMALL, USE 8" Φ CMP below sidewalk ok

J. AREA (10) DRAINS TO UPPER POND

A = 18318 Φ = .42 Acre
C = 0.54

Tc to Field Inlet:
(20') Roof 1.2 MIN
(600') Grass (8%) 14.0 MIN
15.2 MIN ∴ i = 4.15"/HR.

Q_{100} = (.54)(4.15)(.42) = 0.94 cfs
Q_{10} = 1.74 cfs
V_{100} = 2143 FT³

5. ANALYSIS OF UPPER DETENTION POND

An analysis was made in the Phase I Drainage Report for the flows & volumes entering the area partially occupied by the upper holding pond. These flows were used in the original PHASE II Report approved on October 5, 1981. This same information will be used for this PHASE II Analysis since the area contributing to this flow has not changed.

Q_{100} = 5.32 cfs V_{100} = 9313 FT³

Also entering pond will be run-off from Area 10 (See above)

Q_{100} = 0.94 cfs V_{100} = 2143 FT³

* Analysis of 1 Hour high intensity storm on Upper Pond:

Note: Rainfall IN 1hr = 2.6(.85) = 2.2"

For Upper Area	Q_{max}	Area 10
5 MIN	(.68)(7.68)(1.45) = 7.57 cfs	1.74 cfs
5"	4.14	0.95
5"	3.08	0.71
15"	1.93	0.44
30"	0.91	0.21

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CHK DATE
SCALE SHEET NO 9 OF 18

Examining Tc for both areas, examine Max 5 Min inflow occuring during 2nd 5 Min period & 3rd 5 Min period.

	Offsite	Area 10	Total Pond inflow
0-5 Min	4.14/2	0.95/3	2.39
5-10	(4.14+7.57)/2	(1.74+.95)2/3 = 1.79	7.65
10-15	(7.57+3.08)/2	(.71)1/3 + 1.74(2/3) .14	6.73
15-30	(3.08+1.93)/2	0.44 1/3 + .71(2/3)	3.13
30-60	(1.93+0.91)/2	(0.21)1/3 + 2/3(.44)	1.78

0-5	3.08/2	.71 (1/3)	1.78
5-10	(4.14+3.08)/2	(.71+.95)2/3	4.72
10-15	(7.57+4.14)/2	1.74(1/3)+2/3(.95)	7.07
15-30	[(7.57)1/3+1.93]/2	1.74(2/3)+.44(1/3)	3.53
30-60	(1.93.0.91)/2	.21(1/3)+.44(2/3)	1.78

USE 2ND 5 MIN w/Max rate rainfall rate as controlling for pond sizing & outflow pipe sizing.

INFLOW - OUTFLOW ANALYSIS OF UPPER POND

Pond Size = 1670 ⌀ w/1:3 side slopes & 1.5' deep

Volume = 3120 FT^3

Using 16" ⌀ Outlet pipe @ S = 0.008 (.9/92) RCCP

$Q_{max} = \frac{1.49}{.013}(.008)^{1/2}(1.41/4.18)^{2/3}(1.41) = 7.0$ cfs

Try 12" ⌀ Outlet

$Q_{max} = \frac{1.49}{.013}(.008)^{1/2}(.785/3.142)^{2/3}(.785) = 3.19$ cfs

V = 4.06 fps

Increase slope to 1%

Q max = 3.57 cfs

* USE 12" RCCP @ .008 slope & Drop Inlet (See Sheet 11 for ck on Inlet)

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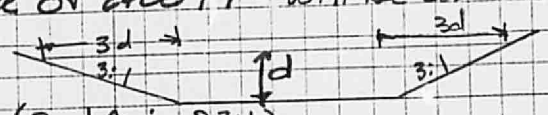
Determine Pond storage required

Time	Area 10	Offsite Qout	Vout	Vol @ Pond	VT
0-5	2.39	2.39	717	-0-	717
5-10	7.65	3.19	957	1338	2295
10-15	6.73	3.19	957	1062	2019
15-30	3.13	3.19	2871	-54	2817
30-60	1.78	2.5 Ave	4500	-1296	3204
60-73	0.80	2.5 *	1950	-1350	600
					11652**

*Pipe 1/2 full ±

**Based on Total Volume for 1 hr Storm (≅ 9900 FT³) this is Number (Storage & through flow) is conservative. Therefore pond is ok.

Actual pond depth based on MAX Stored volume of 2400 FT³ will be as follows:



(Pond Perim @ 3:1)

$$1670d + 180(1/2)(d)(3d) = 2400$$

$$270d^2 + 1670d - 2400 = 0$$

$$d = 1.2' < 1.5' \text{ ok}$$

Check Drop Inlet Capacity

Outlet from pond will be controlled by pipe size. As Pond fills, D.I. will cease to act as a weir & will become submerged. Velocity @ grate ≅ 0. Since water depth (above grate) will vary from 0 to 1.2', the inlet will function somewhere between an orifice & a weir.

Weir: Grate capacity $= Q = C_w L h_w^{3/2}$

$C_w = 3$, $h_w \cong .4'$, $L = 2+2+2+2 = 8'$

$\therefore Q = 6.0$ cfs > 3.19 Pipe governs.

Orifice: Grate capacity $= CA_o\sqrt{2g\Delta h}$

$C = 0.6$ $\Delta h = 1.2'$ $A_o = 2.08\,\phi$

$\therefore Q = 10.97$ cfs $\gg 3.19$ Pipe governs

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BY RH Allen DATE 3/6/82
CHK DATE
SCALE SHEET NO 11 OF 18

Head losses in 12" ϕ pipe

Velocity $V^2/2g = 4.06^2/2g = .26'$

Loss @ bend $1.5\,V^2/2g = .39'$

Friction $\dfrac{(.013)^2\,(92)(4.06^2)}{2.22\,(.585)^{4/3}} = .52'$

$R = .785/3.142 = .585$

$= \dfrac{n^2 L V^2}{2.22\,(R)^{4/3}}$ $\quad n = .013$

Total head loss $= .26 + .39 + .52 = 1.17'$

Head available = T.G. − outlet ℄ = 97.3 − 94.7 = 2.6' > 1.17 OK

Check head req'd on 12" pipe to obtain Q = 3.19 cfs

$h = 1/C_d^2\,(Q^2/2gA^2) = 1/.62^2\,(3.19^2/64A(.785)^2)$

$h = 0.52'$

This head should be attained prior to arrival of peak flows ∴ Pipe should flow full most of the time.

SUMMARY (LOWER POND)

BPE = 97.5

T.G. (Drop Inlet 2'x2') = 97.2

Outlet pipe: 12" RCCP @ .008 slope

Storeage available = 3120 Ft³

Actual storeage for 1 hr storm = 2400 FT³ (1.2')

Pond empties in 73 MIN. from start of storm

Overflow spillway: 2' wide conc. @ S = .2

Inv. @ outlet = 98.8

Vol. through pond (actual) = 9900 FT³ (1 Hr. Storm)

V_{100} = 11456 FT³ = (9313 + 2143)

Qmax @ emergency spillway = 6.73 cfs (Assumes D.I. 2d totally clogged)

$Q = \dfrac{1.49}{.013}(.18)^{1/2}\left(\dfrac{2d}{2d+2}\right)^{2/3}$

$d \cong .21'$

Vel = 16 fps

Provide stilling box at bottom

Vel. to street ≅ 2.2 fps (4' channel in cobbles @ S = .02)

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SCALE SHEET NO 12 OF 18

6. ANALYSIS OF LOWER DETENTION POND

The lower pond will intercept runoff from the upper pond, from drainage AREA 8B and from the catch basin located in AREA 8A. (See sheet 8 for calculations)

Area 8A: $Q_{100} = 0.96$ cfs $V_{100} = 1680$ cf
$Q_{1HR} = 1.36$ cfs (max)

AREA 8B: $Q_{100} = 0.39$ cfs $V_{100} = 850$ cf
$Q_{1HR} = 0.69$ cfs (max)

Upper Pond: $Q_{max} = 3.19$ cfs $V_{100} = 11456$ Ft^3

The 1-HR. High Intesity storm will be analysed to check storeage requirements at the pond. (Worst rate @ 5-10 Min)

Time	8A $T_c=10$	8B $T_c=19$	Upper Pond	Total inflow (Q) (1)
0-5	½(.75)	⅓(.38)	2.39	2.89
5-10	.75+½(1.37)	⅔(.38)+⅓(.69)	3.19	5.11
10-15	1.37+½(.55)	.38+⅔(.69)+⅓(.28)	3.19	5.77
15-30	.55+½(.35)	.69+⅔(.28)+⅓(.18)	3.19	6.06
30-60	.35+½(.16)	.28+⅔(.18)+⅓(.08)	2.5	2.95
60-73	(.16)	.18+⅔(.08)	2.5	2.89
73-79	—	.08	—	0.08

Outlet to M.H. near Academy = 126 L.F. Flow to Academy must be restricted to 2.21 cfs or less.

Try 12" ϕ RCCP & determine slope. Assume it flows full.

$$2.21 = \frac{1.49}{.013}(S^{1/2})(.785/3.142)^{2/3}(.785)$$

$S^{1/2} = .0619$

$S = .0038$ [Use .005 to assure cleaning action.]

$V = 2.81$ ft/sec.

INV @ M.H. = 92.30 - .63 = 91.67 Set outlet pipe from M.H @ 91.50

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SCALE SHEET NO 13 OF 18

INFLOW/OUTFLOW FROM LOWER POND

Time	Q_{IN}	Q_{out}	Vout	Vol @ Pond	Total	
0-5	2.89	2.21	663	204	867	
5-10	5.11	2.21	663	870	1533	
10-15	5.77	2.21	663	1068	1731	
15-30	6.06	2.21	1989	3465	5454	
30-60	2.95	2.21	3978	1332	5310	
60-73	2.89	2.21	1724	530	2254	
73-79	0.08	2.21	796	-767	29.	
79-129	—	2.21	6702	-6702	—	
TOTALS			17178	-0-	17178	14924

Total Volume stored = 7469 FT^3

Pond capacity = (3130 ⌀) 2.5' = 7825 > 7469 ∴ Ok

*Analysis will show @ Drop Inlet, pipe size governs flow.

* Head loss check

Vel. $V^2/2g$ = .123'

Bend $1.5\,V^2/2g$ = .184'

Fric. .155'

Total loss = .462'

Head available = T.G − Outlet ℄ = 93.5 − 92.17 = 1.33' Ok

Head req'd on 12" pipe:

$h = 1/C_d^2 (Q^2/2gA^2) = (1/.62^2)(2.21^2/64.4(.785)^2$

= 0.37'

This head should be achieved prior to arrival of peak flows. ∴ Pipe should flow full most of time.

SUMMARY:

BPE = 94.0 T.G. = 93.5

Outlet = 12" RCP @ .005 slope

Storeage = 7469 FT^3 ∴ 7825 Capacity

Depth (max) = 2.39'

Pond empties in 129 Min from start of storm

Vol. through pond ≅ 17000 FT^3

V_{100} = 11456 + 1680 + 850 = 14000 FT^3

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SCALE SHEET NO 14 OF 18

NOTE: Volume emptied for 1 Hr. storm > V100. Analysis for 1 Hr storm ∴ is conservative.
For purpose of showing storage capacities, 100 year V will be used, namely 14000 CF.

An emergency overflow device located at the southwest corner of the pond will discharge to an interval street via a drop inlet & 2-6" ϕ P/C pipe. This flow will eventually reach Duvall.

Outflow from D.I. to Academy. (12" CMP)
Q = 2.21 cfs L = 253' to ℄ @ S.W.
Elev. @ outlet = 85.5 S = (91.5 − 85.5)/253 = .0237

Due to Δ in slope, pipe will not flow full but will act as open channel flow.
For 12" pipe flowing full: (12" CMP)
$Q = \frac{1.49}{.022}(.0237)^{1/2}(.785/3.142)^{2/3}(.785) = \underline{3.25 cfs.}$

$\frac{2.21}{3.25} = 68\%$ full .68(.785) = .534 ϕ

Depth in pipe (by trial & error) = 0.7' ±

Velocity = 2.21/.534 = 4.14 fps OK

NOTE: Area 9 will add to flow to Academy as it enters flow at D.I. $Q_{1HR\,max}$ = 1.07 cfs
Q_{100} = 0.61 cfs.

Q_T = 2.21 + 1.07 = 3.28 cfs ≅ 3.25 cfs capacity.
For short period, pipe will flow near full. OK
Velocity = 3.28/.785 = 4.2 fps OK.

USE 12" RCCP

If Conc. (RCCP) is used:
Q max, @ = 5.49 cfs @ V = 7 fps
2.21 + 1.07/5.49 = .597
d ≅ 0.6' V = 7 fps.

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SCALE SHEET NO 15 OF 18

SUMMARY

Per GRUPE's letter to City Hydrologist dated 7/7/81, a minimum detention storage (on site) was to be provided in Phase II after development. The agreed to volume was 9948 Ft³.

The foregoing analysis indicates the following storages:

Area	Available	Utilized
Upper Pond	3120 Ft³	2400 Ft³
Lower Pond	7825 Ft³	7469 Ft³
	10945 Ft³	9869 Ft³ ≅ 9896 ok

The storage provided approximately equals that req'd.

All ponds will be empty at T ≅ 130 min or 2 hrs & 10 min after the 1 hour storm starts.

RUNOFFS & VOLUMES FOR 100 YEAR STORM

AREA (West of PHASE II)	Q_{100}	V_{100}	
	0.28 cfs	504	Goes to Pkg/Denali
	.05	108	Goes to Drywell
1	.19	340	Goes to Academy
2	.75	1301	Goes to Internal St. then to Denali
3	.19	328	Goes to Las Casitas & then Denali
4	.51	900	Goes to Academy
5	1.56	3262	Las Casitas/Denali
6	.15	273	Drywell
7	1.08	2006	LAS CASITAS/DENALI
8	1.35	2530	LOWER POND
9	1.07	1315	Acad'y
10	.94	2143	UPPER POND
OFFSITE	5.32	9313	UPPER POND

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SCALE SHEET NO 16 OF 18

TOTALS ADDED:	Q	V
TO DENALI	3.86	7401
TO ACADEMY	1.77	2555
TO DRYWELLS	.20	381
TO PONDS & EVENTUALLY TO ACADEMY	7.61*	14000

*This is not an actual flow rate to Academy. Actual rate from Ponds = 2.21 cfs.

The following rates/volumes were accepted as part of Phase I Report

	ALLOWED	ACTUAL
TO DENALI	21.92 cfs	14.74 cfs
TO ACADEMY		
.91+1.1-.22 Internally	1.79 > 4.89	4.89 cfs
(2.2+.9) Original Pond (PART Pond + Areas 11 & 8)	3.1 >	

Academy New:

PHASE I AREA B	1.10 − 0.6 =	0.5
PHASE I AREA 10A	=	0.91
PHASE II AREA 9	=	1.07
PHASE II AREA 1	=	0.19
PHASE II AREA 4	=	0.51
PHASE II PONDS	=	2.21
TOTAL		5.39 cfs > original by .5

Denali (New)

PHASE II	3.86
PHASE I	
AREAS 3,4,5,6	5.90
AREAS 11	0
Area 7 (Partial) .6(5.77)	3.46
Area 9	1.62
TOTAL	14.84 cfs ≅ ORIGINAL

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SCALE SHEET NO 17 OF 18

Flows to Academy under PHASE II Development will exceed PHASE I Flows by 0.5 cfs. This is very small & therefore OK.

5.39 (new) vs. 4.89 (old)

Flows to Donald are essentially the same

14.84 (new) ≅ 14.74 (old) OK

Overflow notch @ lower pond: Assume D.I. totally clogged. Max Q = Inflow Q at time when pond fills to notch. Q = 6.06 cfs

Pond will fill to spillway notch at time T:

$T_s = 3.49/6.06 = 10.2$

$T = T_s + 15 = 25.2$ Min.

Notch will function as weir

$Q = C_w L h_w^{3/2}$

$6.06 = 3(L)(.2)^{3/2}$ $L = 22'$

Try $h_w = .3'$ $L = 12.3'$ Use 3 Notches 4' wide ea.

Outlet from field inlet (Try 6"φ PVC) $S = .13$

$Q = \frac{1.49}{.009}(.13)^{1/2}(.196/1.57)^{2/3} = 2.92$ cfs

2-6" PVC, $Q_T = 5.84$ cfs ≠ 6.06

To account for losses, use 3-6"φ PVC pipe

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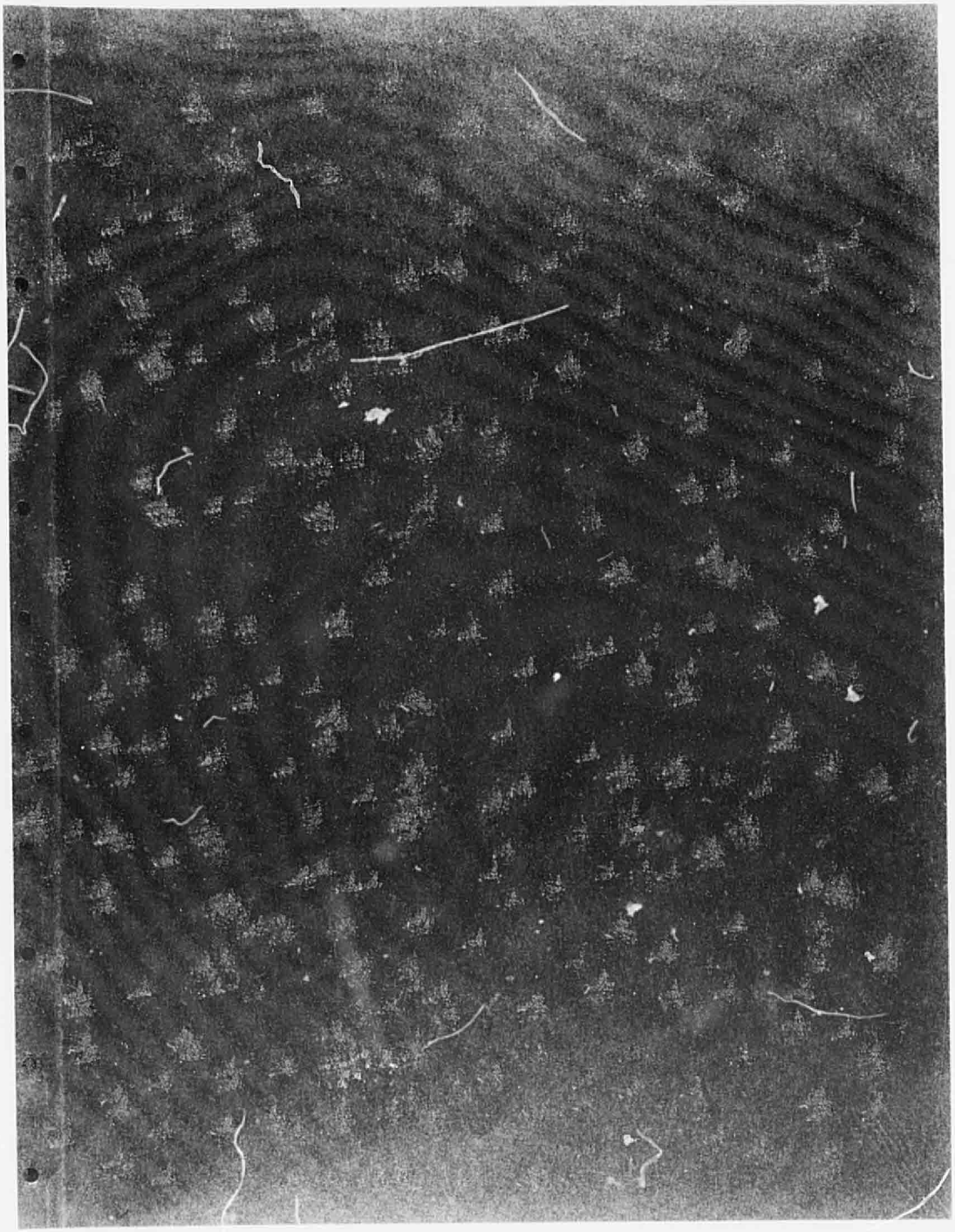

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