HYDROLOGY H17/D6 TWO PLANS

H17/D6

2 Plans

DRAINAGE STUDY

MOUND TERRACE DEVELOPMENT

ALBUQUERQUE, NEW MEXICO

approved

1st PHAG Only 11-16-73

LIMBAUGH ENGINEERS, INC.

CONSULTING ENGINEERS
and PHOTOGRAMMETRISTS

November, 1974

Albuquerque, New Mexico

DRAINAGE STUDY

MOUND TERRACE DEVELOPMENT

ALBUQUERQUE, NEW MEXICO

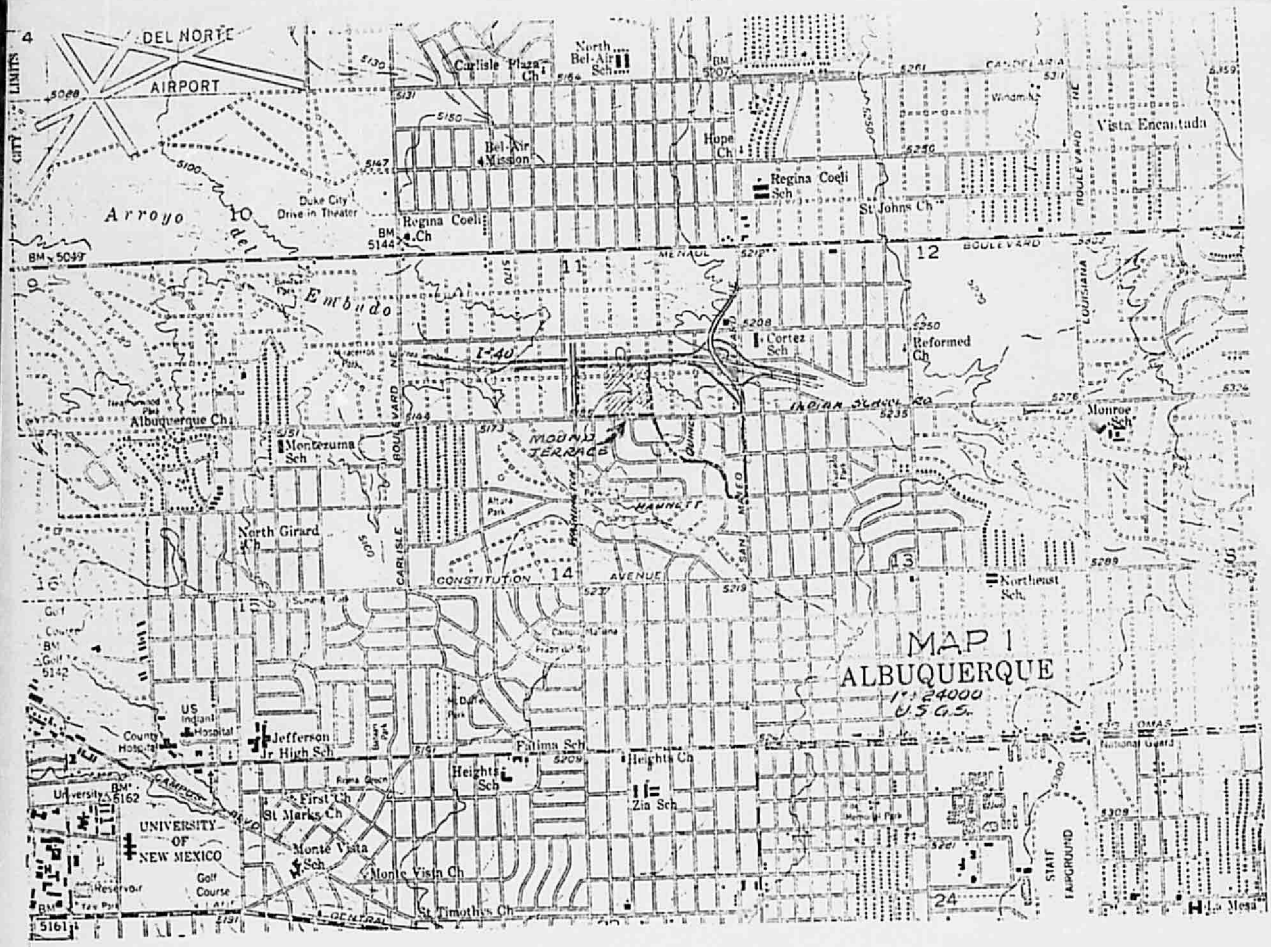
This is a study of the probable effect of the Mound Terrace Condominium Development east of Washington Street on Indian School Road in Albuquerque, New Mexico, on the existing drainage pattern in the vicinity.

The subject area is shown on Map 1 of this Report together with the adjoining areas which might have influence on, or be influenced by, the Mound Terrace Development. It will be seen from Map 1 that in addition to the Mound Terrace tract, the area between Interstate Highway 40 to Hannett Street, and San Mateo Blvd. to Quincy Street constitutes one such area. Another adjoining area is the Synagogue site at Indian School Road and Washington Street. Finally, the area of drainage for the Freeway along its south shoulder must be considered.

The consequence of the Mound Terrace Development will be to increase the amount of run-off from the Condominium site. The effect of the increased run-off volume and rate can be moderated by retention of the excess water in parking lot ponds and by release from this storage through orifices in a way that the rate of discharge is not greater than that which would have been experienced prior to the development. The following study developes the ponding volume and orifice dimensions such that the foregoing is achieved for a 6 hour, 100-year storm.

All of the drainage from the three above noted areas flows northward to the south ditch and culverts of I-40.

The first listed area, from Hannett to the Freeway, has been enlarged on the map on Computation Sheet 1 in the Appendix to facilitate computation by the "Rational Method" for routing of the storm run-off from the intersection of Hannett Street and Truman Street via Truman and Indian School Road to a point of confluence at the intersection of Indian School Road and Monroe Street. The "time of concentration" calculated from the water shed crest in Hannett Street to the confluence point is 36.1 minutes. Based on this time, an assigned run-off


MAP 1
ALBUQUERQUE
1" = 24000
U.S.G.S.
DEL NORTE
AIRPORT
Arroyo del Embudo
Carlisle Plaza Ch
North Bel-Air Sch
Bel-Air Mission
Hope Ch
Regina Coeli Sch
St Johns Ch
Vista Encantada
Duke City Drive-in Theater
Regina Coeli Ch
MENAUL
BOULEVARD
I-40
Cortez Sch
Reformed Ch
INDIAN SCHOOL RD
Monroe Sch
Albuquerque Ch
Montezuma Sch
MOON'S TERRACE
HANNETT
North Girard Ch
CONSTITUTION
AVENUE
Northeast Sch
US Indian Hospital
County Hospital
Jefferson Jr High Sch
Fatima Sch
Heights Ch
Heights Sch
Zia Sch
First Ch
St Marks Ch
UNIVERSITY OF NEW MEXICO
Monte Vista Sch
Monte Vista Ch
St Timothy's Ch
Golf Course
Reservoir
STATE FAIRGROUND
Hills Mesa

factor, "C" of 0.44, and a planimetered area of 24.8 Acres, the Q_{100} at the confluence point is found to be 34.9 cfs.

Referring to the map at scale 1"=100' titled "Drainage Study", the influent Q_{100} of 34.9 cfs when summed with the 16 cfs developed on the 14.8 acres between Indian School Road and the Freeway give an estimated 51 cfs at inlet into culvert 2A at a cumulative time of concentration of about 36 minutes.

The mentioned culvert 2A is a 36" diameter Reinforced Concrete Pipe (RCP) so located as to permit a hydraulic head to 5.5 ft. above its invert before spilling westward. By "Inlet Control" calculation, see nomograph 2A in the Appendix, the pipe has capacity under the cited conditions to pass 74 cfs. Accordingly, this pipe is competent to dispose of all waters from the first sub-area of study and this run-off may be dismissed from consideration with respect to the Mound Terrace Development.

Highway Culvert 2B, an 18" diameter RCP (see Drainage Study Map), has capacity to carry off 18 cfs. This amount is more than will run off from the I-40 pavement to the 2B drop inlet. There is no access for run-off from the south to get into this drop inlet.

In the Appendix, the estimated natural run-off from the Mound Terrace site is calculated on Computation Sheets 2 and 3 together with estimated run-off from the site when fully developed. The "natural" run-off in the amount of 12.7 cfs is the quantity which is adopted to rule the discharge rate from the developed site.

The site development will create an additional 56,480 cf of run-off to be managed and released by the device of ponding and orifice discharge.

The 12.7 cfs100 discharging from Mound Terrace tract will in part join the run-off discharged from the adjacent Synagogue tract in the public drainage ditch and culverts of the Interstate Highway. The arithmetic of the discharge scheme is posted on the "Drainage Study" Map.

It will be seen that the existing natural flow northward from the Freeway to the Embudo Diversion Channel as presently experienced is 19.5 cfs for the 100-year storm here considered. After development of the Mound tract, this flow will be 17.2 cfs. These quantities are the sum of the discharges from the 30" diameter RCP "2C" and the 30" diameter RCP "2D".

The original discharge from Culvert "2C" is calculated at 13.5 cfs. being the sum of the "natural" discharge from the Mound tract of 12.7 cfs and 0.8 cfs run-off from the highway.

The "Inlet Control" calculation to determine the capacity of RCP "2C" as given in the Appendix shows this culvert to be capable of handling 45 cfs. We project only 9.5 cfs from Mound plus 0.8 cfs from the Highway, totaling 10.5 cfs, to be delivered to this culvert after development.

The original discharge from Culvert "2D" was 6.0 cfs, being the sum of 5.3 cfs from the un-developed Synagogue tract plus 0.7 cfs of run-off from the Highway.

On the developed Synagogue tract, a ponding area and a 12" diameter corrugated metal discharge pipe with an estimated capacity of 3.0 cfs. maximum, have been installed. To this 3.0 cfs amount, we will add 3.0 cfs from the Mound Terrace site and 0.7 cfs from the highway, as before, to aggregate to 6.7 cfs <u>after</u> development. It is determined by calculation 2 D (Appendix) that pipe "2D" has capability of carrying 29 cfs.

Accordingly, the projected ponding and constricted release of storm waters based on the "natural" rate of run-off will allow ample latitude for variance, about 400%, before the capacity of the public channels is exceeded.

Management of the waters incident upon the Mound Terrace development to conform to the described discharge directions and rates as projected can be achieved by ponding and controlled discharge of the 100-year precipitation.

In the ultimate development of the site, two ponds, having storage volumes totaling 33.000 cubic feet and having sides 1.33 feet high, have been projected.

The rate of discharge from these ponds will be controlled by three square openings 0.33 feet high and of a length proportionate to the desired volumes as indicated on the "Drainage Study" Map.

The discharge through these openings is calculated by the formula:

$$Q_{cfs} = CdA_{sf} \sqrt{2gh}_{ft}.$$

The sub-scripts denote the units of measure. The factor Cd, the coefficient of discharge, adopted for these square openings, has been taken as 0.55. The head, h, is measured from the center of the opening. A graph of the discharge rate for varying head conditions is given on Computation Sheet 4 in the Appendix.

The hydrographic distribution of storm intensity assumed for this study is adopted from the run-off intensity curves which would indicate a peak load of over 50% of the total calculated run-off (77,272 cf) occuring within the first 45 minutes of the 6 hour storm. The ponding volume and orifice discharge rate here proposed can cope with this shock loading as is demonstrated by the In-Put/Out-Flow Tabulation on Computation Sheet 5 in the Appendix.

Referring to Computation Sheet 5, it is seen that the stored volume accumulates to 27,219 cf, maximum, being 11% less than overflow capacity. Also, the maximum rate of discharge of 12.0 cfs is about 6% less than the budgeted 12.7 cfs.

Consequent to the above, we are safe to adopt the following three orifices to be located as shown on the included "Site Plan."

Outlet	CFS, max.	Height, In.	Length, Ft.
# 1	3.0	4	1.75
# 2	4.0	4	2.75
# 3	5.7	4	3.50

In this study, all assumptions adopted have been taken so as to assure results to be on the safe side. The resulting computations have always yielded some additional margin of safety. Accordingly, it is felt that the ponding and release openings here described will provide the measures necessary for the developed Mound Terrace tract to be in substantial compliance with Section 3, Paragraph B. of Resolution No. 2, Series of 1972, of the Albuquerque Metropolitan Arroyo Flood Control Authority.

LIMBAUGH ENGINEERS, INC.

Rodney J. Dalton

Rodney J. Dalton, P.E.

P.E. & L.S. No. 2142
New Mexico

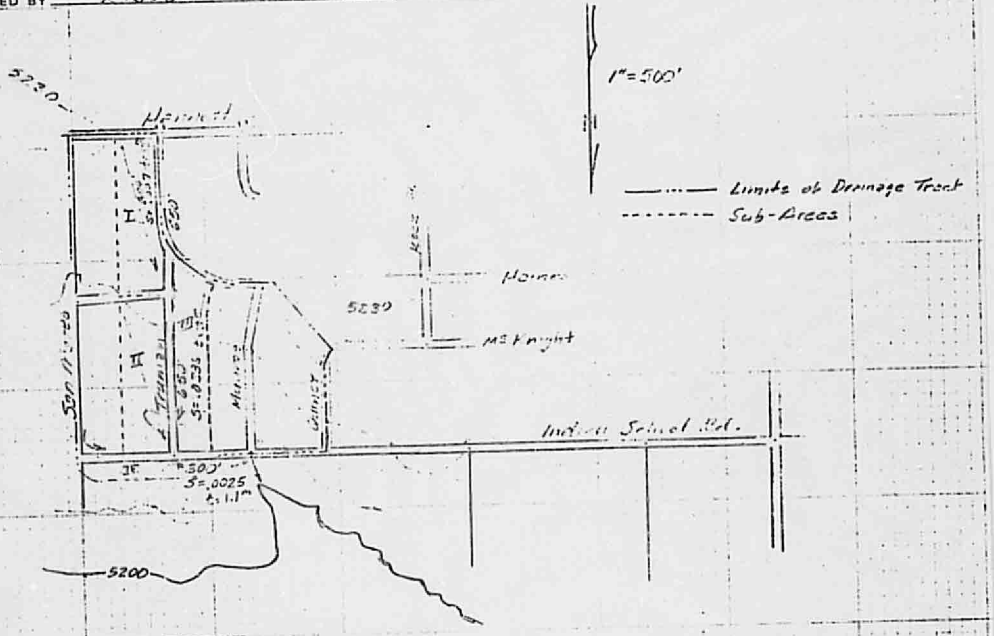
LIMBAUGH ENGINEERS, INC.
P.O. BOX 30068
ALBUQUERQUE NEW MEXICO 87110

PAGE 1 OF 5

SUBJECT Storm [illegible]

COMPUTATION Q=RCiA from [illegible] JOB NO.

COMPUTED BY R.J.B. DATE 7 Nov '72 CHECKED BY _____ DATE _____



Total Area = 24.8 Ac.

A_I = 175 × 650' = 113750 = 2.6 Ac. ; t = 52 min i = 3.6 in/hr Q = 2.6 × 0.7 × 3.6 = 6.57 cfs

A_{II} = 175' × 650' = 113750 = 2.6 Ac

A_{III} = 100' × 650' = 65000 = 1.5 Ac

A_{IV} = 60' × 350' = 21000 = 0.48 Ac.

Trumbull Drainage: Homes to Indian School [illegible] 500' S=0.0225 t=1.1 min ; Q = 1.37 cfs V = 3.5 fps t = 2.3 min

Q at Trumbull & Indian School: 2.6 × 5 [illegible] 350' [illegible] = 15.6 Ac ; t_c = 35 min ; Q = 15.5 × 0.4 × 3.3 = 20.5 cfs

Ind. School Rd. [illegible] = [illegible] 0.0025 t = 1.1 ; Q = 20.5 cfs ; V = 2.25 fps.

t_c = 32 + 3 + 1.1 = 36.1 min To Intersection Indian School & Trumbull ; ΣA = 24.8 Ac.

Q100 at Indian School & Trumbull = 24.8 × 0.4 × 3.2 = 31.7 cfs.

(Using C = 0.44, Q100 = 34.9 cfs)

Natural Runoff from the Property

A = 12.0 acres
C = .2
I = 5.3
Q = CIA = 12.7 cfs

Developed Runoff

Roof Area = 3.8 acres
C = .9
I = 7.2
Q = CIA = 24.6 cfs

Paved Area = 5.70 acres
C = .9
I = 7.2
Q = CIA = 37.0 cfs

Grass Area = 2.5 acres
C = .15
I = 4.00
Q = CIA = 1.5 cfs

Total Development Runoff = 63 cfs

COMPUTATION SHEET 3 of 5

Volume of Natural Runoff

522,000ft² X 2.4* in. X 1 ft./12 in. X .2 = 20,800 ft³ in 6 hrs.

Volume of Developed Runoff

Roof and Paved Area:

411,000ft² X 2.4* in. X 1 ft./12 in. X .9 = 73,980 ft³

Grass Area:

111,000ft² X 2.4* in. X 1 ft./12 in. X .15 = 3,300 ft³

Total Volume of Developed Runoff = 77,280 ft³ in 6 hrs.

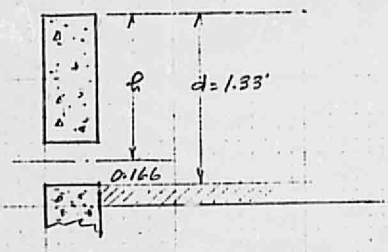
Difference in Developed & Natural Runoff = 56,480 ft³ in 6 hrs. or approximately 10,000 ft³/hr. (Considering the 100 yr. 6 hr. rain even distributed over the 6 hrs.)

* 100 yr. 6 hr. precipitation

LIMBAUGH ENGINEERS, INC.
P.O. BOX 30068
ALBUQUERQUE, NEW MEXICO 87110

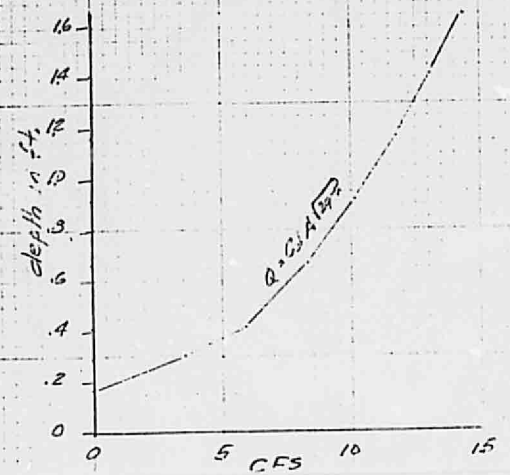
SUBJECT Discharge Through Rectangular Opening – Mound Terrace Study
COMPUTATION Incremental Orifice Discharge $Q = C_d A \sqrt{2gh}$ JOB NO.
COMPUTED BY RJO DATE 7 Nov '73 CHECKED BY [illegible] DATE 11 Nov. 73

RECTANGULAR OPENING 4" HIGH × 8.0' LONG



DISCHARGE CURVE FROM ABOVE OPENING 4" × 96"

h, ft	d, ft (h+.166)	Q, cfs
0.125	0.291	3.6
0.250	0.416	5.9
0.500	0.666	8.3
0.750	0.916	10.3
1.000	1.166	11.8
1.250	1.416	13.1
1.500	1.666	14.4



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ALBUQUERQUE, NEW MEXICO 87110

PAGE 5 OF 5

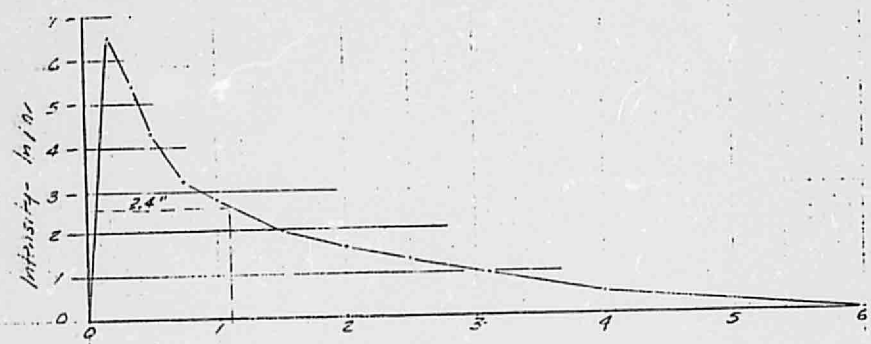
SUBJECT Developed Area Run-off vs Ponding & Discharge

COMPUTATION Q_{peak} - Storage ≤ 12.7 CFS JOB NO.

COMPUTED BY RJD DATE 9 Nov. 73 CHECKED BY PHO DATE 11 Nov 73

Period (Min)	Run-off Period Vol-CF.	Run-off Cum. Vol. CF	Pond Depth Ft	h Ft.	Discharge CFS	Period Discharge	Cum. Discharge	Retained Volume
0-12(1)	3 609	3 609	0.145	0.145	1.1	792	792	2 817
12-30	36 057	39 676	0.84	0.674	10.8	11 665	12 457	27 219
30-45	5 116	44 792	1.20	1.034	12.0	10 800	23 257	21 535
45-60	2 096	46 888	0.95	0.784	10.4	9 360	32 617	16 271
60-90(1)	6 516	53 404	0.79	0.624	7.3	16 740	49 357	6 047
90-120	4 915	58 319						
120-180	7 635	67 954						
180-240	4 915	72 869						
240-300	2 720	75 589						
300-360	1 683	77 272						

(1) Weir formula, $Q = CLh^{3/2}$, applies



Assumed Hydrograph
100 Yr - 6 hour Storm

FIG A. - ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 2 YEARS.

FIG B. - ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 10 YEARS.

FIG C. - ONE-HOUR RAINFALL, IN I
TO BE EXPECTED ONCE IN 50 Y

FIG D. - ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 5 YEARS.

FIG E. - ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 25 YEARS.

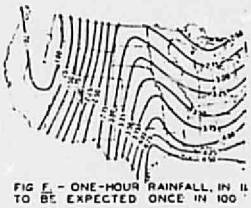


FIG F. - ONE-HOUR RAINFALL, IN I
TO BE EXPECTED ONCE IN 100

COMPUTATION OF i IN RATIONAL FORMULA.

EXAMPLE: Assume expectancy period = 5 years. See Fig. D, assume locality, find 1 hour intensity = 1.75 in. p

FIG.G-INTENSITY EXPECTATION FOR ONE-HOUR RAINFAL

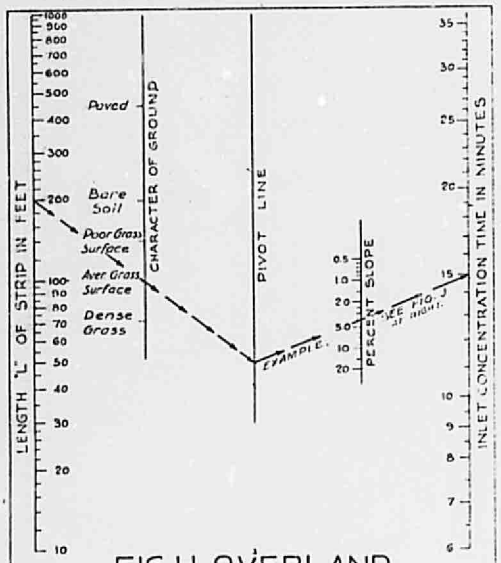


FIG. H-OVERLAND FLOW TIME.

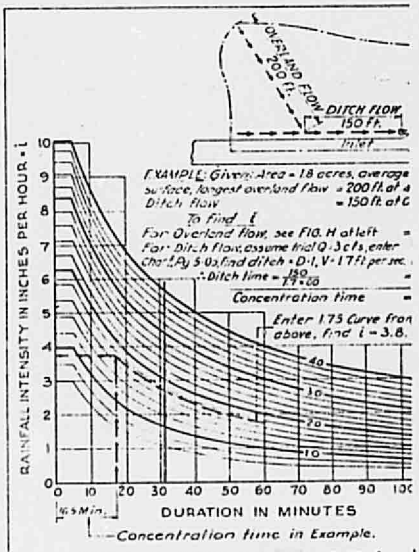


FIG. J-VALUES OF i
RAINFALL INTENSITY-DURAT

*Reproduced from Miscellaneous Publication No. [illegible] U.S. Dept. of Agriculture, by David L. Yarnell. Adapted from Engineering Manual of the War Dept. [illegible], Fort Sill, Okla., Dec. 45

PINETREE OFFICE PARK
4501 Indian School Road, N.E. — Suite D
Albuquerque, New Mexico 87110
Phone 505-255-8685

June 10, 1981

City of Albuquerque
Mr. Fred J. Aguirre
Assistant Hydrology Engineer
Public Works Department

Dear Mr. Aguirre,

Please accept this letter as our intent to go ahead and modify the drainage curb to comply with plans and specifications required.

It is not our intent to ever circumvent any part of our responsibility to make our buildings and property comply with all regulations.

Very Truly Yours,

Mel Davis
Property Manager

MD:saz

NEW MEXICO
STATE HIGHWAY DEPARTMENT
DISTRICT NO. 3

ADDRESS ALL CORRESPONDENCE
TO
DISTRICT ENGINEER
P. O. BOX 3768, STA. D

ALBUQUERQUE, NEW MEXICO 87190

September 28, 1978

Banco Mortgage Company
4665 Indian School Road, N. E.
Albuquerque, New Mexico 87106

Re: Pine Tree Office Complex
South Side - I-40 Between
Washington and San Mateo

Dear Sir:

Approval is granted for the construction of a concrete drainage rundown within highway right of way at approximately Sta. 102+50 in accordance with location drawings submitted July 10, 1978, and rundown detail submitted September 12, 1978.

Approval is granted subject to the following conditions:

1. All equipment and personnel must enter from the Pine Tree side of the freeway. Parking is not permitted on the highway right of way.

2. No equipment will be permitted within twenty-five (25) feet of the driving lane.

3. The R/W fence must be replaced in a condition satisfactory to the Highway Department.

4. The roadway slope and work area must be left in a neat and smooth condition upon completion of the work.

Harry L. Apodaca, Jr.
District Highway Engineer

By:
Dan Sabo
Traffic Engineer

DS:cd
cc: Chambers, Campbell, Isaacson, Chaplin, Inc.

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

June 23, 1978

Chris Weiss
C.C.I.C., Inc.
3500 Indian School Rd., NE
Albuquerque, New Mexico 87106

RE: Pine Tree Office Complex Phase IV

Dear Mr. Weiss:

I have reviewed the drainage report for Phase IV Pine Tree Office Complex and I am in agreement with the concept. Final acceptance of the drainage report is contingent upon approval by the N.M. Highway Dept. of the construction of the proposed drainage rundown.

Incidentally 2 signed copies of the drainage report need to be available to this office.

Very truly yours,

Bruno Conegliano
Asst. City Engineer-Hydrology

BC/gm

cc; V.M. Kimmick
Richard Leonard
Drainage file

A DRAINAGE ANALYSIS

FOR

PINE TREE OFFICE COMPLEX

PHASE IV

JUNE 1978

Chambers, Campbell, Isaacson, Chaplin, Inc.
architects, engineers, planners, 3500 indian school road ne albuquerque, new mexico 87106, 505/266-5521

A DRAINAGE ANALYSIS

FOR

PINE TREE OFFICE COMPLEX

PHASE IV

INTRODUCTION

The purpose of this study is to analyze the 100-year storm drainage characteristics of the remaining portion of an office complex called Pine Tree. The overall Pine Tree site is located south of Interstate 40, north of Indian School Road, N.E., between Washington Street and San Mateo Boulevard (Zone Atlas #H-17-Z). This report is concerned mainly with Phase IV of the development, which is located in the northeast quarter of the overall site.

General background information concerning the Pine Tree site is as follows: The drainage characteristics of the overall site were originally reviewed and approved in November, 1974.[1] The actual building construction took place in phases, with Phase I, IA, and II completing the initial construction. This firm became involved with the project at the beginning of Phase III. At that time, the City reviewed the project to determine the degree of compliance with the original drainage study. The general findings were that no ponding existed where called for by the report. This firm then made recommendations for correcting the problem areas.[2]

Phase IV of the Pine Tree Office Complex involves the final portion of the site, the northeast quarter. The drainage characteristics of the site remain the same as covered by the Phase III report -- that is, the west pond will require a total volume of 20,050 ft.^{3} and the east pond will require a total volume of 12,950 ft.^{3} These two ponds equal the total storage volume of 33,000 ft.^{3} required by the original report. Since that time, however, the owners of the site acquired an additional 45-foot wide strip of land, for access purposes, along the east side of the property. This additional area will be paved and will drain directly into the east pond. The total additional area will be 45 ft. x 555 ft., for a runoff volume increase of (24,975 ft.^{2} x 2.4" x 0.9) ÷ 12 = 4496.ft.^{3}

---

[1]Drainage Study - Mound Terrace Development by Limbaugh Engineers, Inc. (A copy is attached to this narrative.)

[2]Revised Drainage Study - Pine Tree Phase III, by Chambers, Campbell, Isaacson, & Chaplin, Inc. (Sheet C-3 - a copy is attached to this narrative.)

The total volume requirement of the east pond will then become 12,950 ft.^{3} plus 4496 ft.^{3} = 17,446 ft.^{3}. It should be recognized that this volume accounts for overflow capacity. The actual stored volume will accumulate to a figure approximately 12% less than the overflow capacity, because of the simultaneous release of flows through a controlled orifice.

The following is a tabulation of the volume of the east pond, by proposed finished contours.

Contours	Area	Volume	Acc. Volume
88 - 89	5,250 ft.^{2}	3,030 ft.^{3}	3,030 ft.^{3}
89 - 90	9,800 ft.^{2}	7,525 ft.^{3}	10,555 ft.^{3}
90 - 90.6	13,856 ft.^{2}	7,097 ft.^{3}	17,652 ft.^{3}

Therefore, at a high water elevation of 90.6, the pond will have an overflow capacity of 17,652 ft.^{3}, which is 1% greater than the required amount of 17,446 ft.^{3}.

From the original drainage study, the east pond was to discharge into a drainage swale along the south side of the freeway, which in turn empties into an existing drop inlet to the west. The discharge from the east pond will occur through an opening in the curb and cannot exceed 5.7 cfs, as per the approved report.

Then,

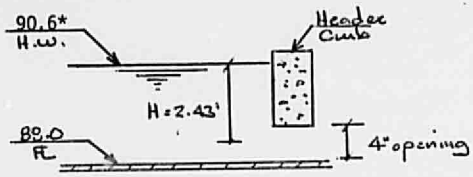
$Q = C_d \times A \sqrt{2gh}$, where $C_d = 0.55$

for a Q of 5.7 cfs,

$5.7 = (0.55)\sqrt{(64.4)\ (2.43)}\ A$

$A = 0.83$ ft.^{2}

$L = 2.5$ ft.



*It is recognized that the high water elevation of 90.6 will not be achieved with simultaneous release of flows but was used here as a maximum head for the 5.7 cfs release rate. In actuality, the flow rate will always be less than 5.7 cfs for smaller heads.

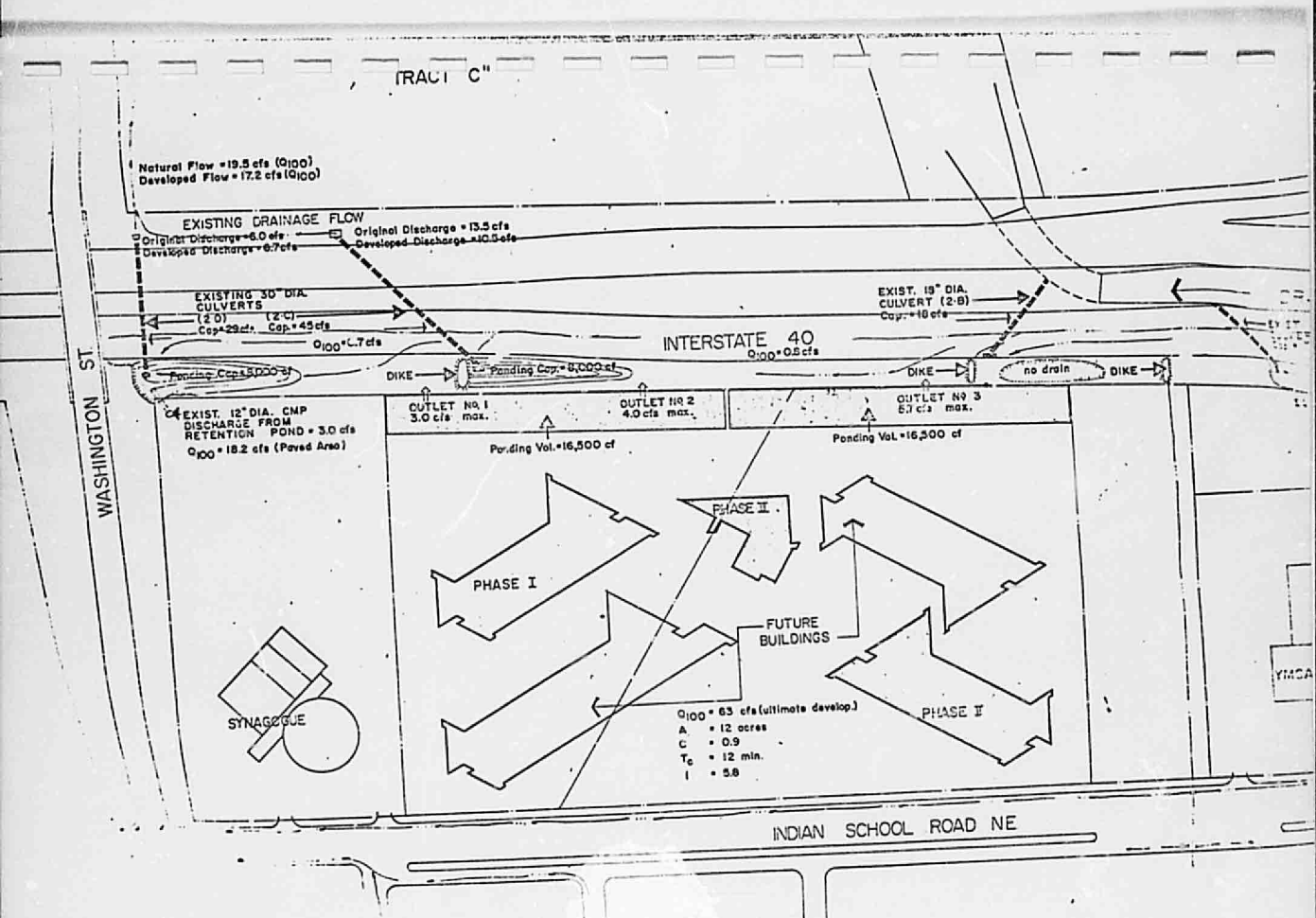
RECOMMENDATIONS

1. Build a header curb, top at 90.6, along north boundary of parking-pond area, and taper curb into regular curb at the east and west returns at the 90.6 elevation.

2. Construct an opening in the curb, 4" high x 2.5' long at the location shown on the plan.

3. Construct a concrete rundown down the slope into the freeway drainage swale at the location shown on the plan.

CHRISTOPHER L. REISS
NEW MEXICO
6853
REGISTERED PROFESSIONAL ENGINEER



REDUCED COPY OF "DRAINAGE STUDY PLAT"
FROM ORIGINAL APPROVED REPORT

DRAINAGE STUDY

MOUND TERRACE DEVELOPMENT

ALBUQUERQUE, NEW MEXICO

November, 1974

DRAINAGE STUDY

MOUND TERRACE DEVELOPMENT

ALBUQUERQUE, NEW MEXICO

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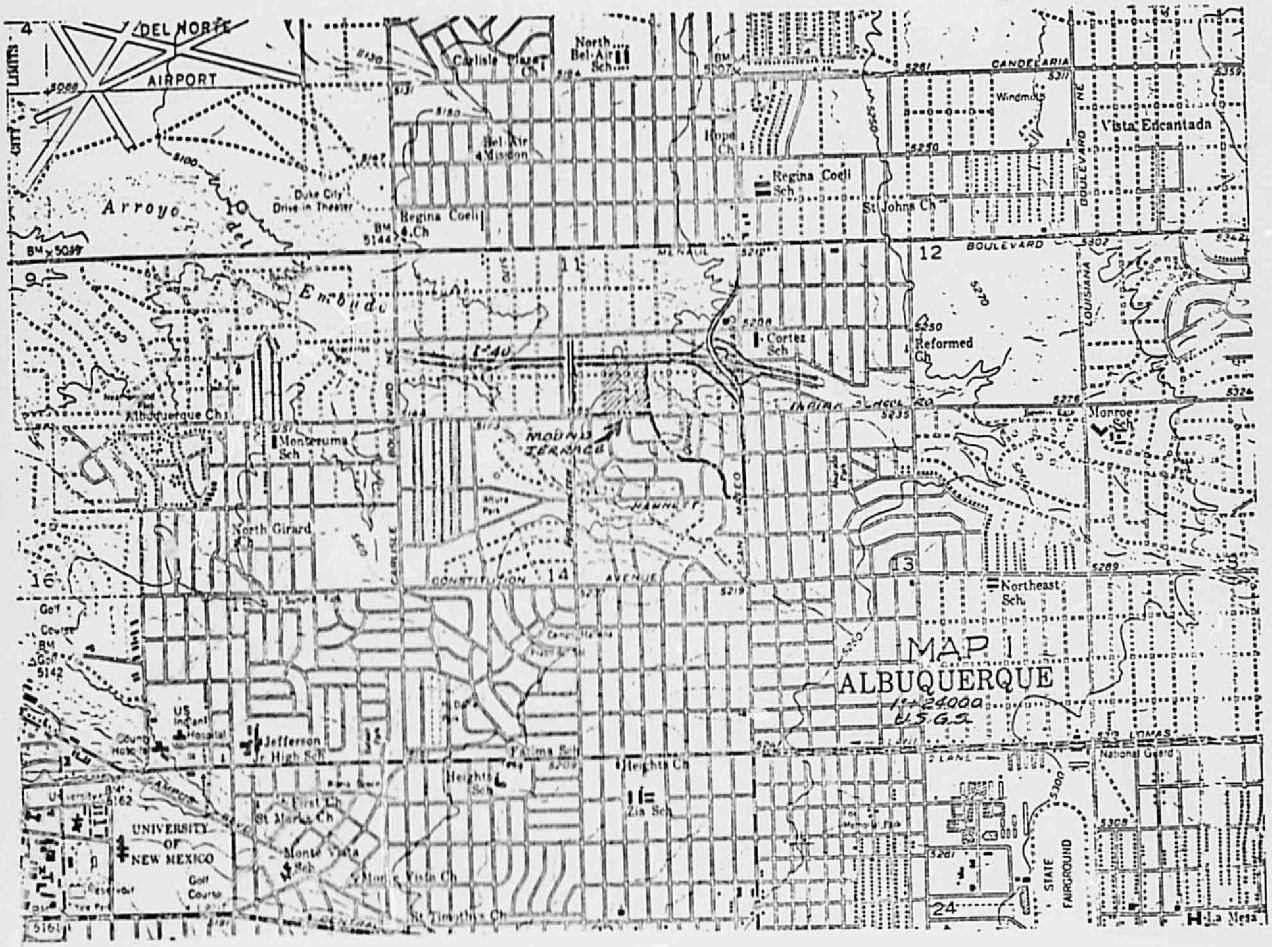
All of the drainage from the three above noted areas flows northward to the south ditch and culverts of I-40.

The first listed area, from Hannett to the Freeway, has been enlarged on the map on Computation Sheet 1 in the Appendix to facilitate computation by the "Rational Method" for routing of the storm run-off from the intersection of Hannett Street and Truman Street via Truman and Indian School Road to a point of confluence at the intersection of Indian School Road and Monroe Street. The "time of concentration" calculated from the water shed crest in Hannett Street to the confluence point is 36.1 minutes. Based on this time, an assigned run-off


MAP 1
ALBUQUERQUE
1:24000
U.S.G.S.
DEL NORTE
AIRPORT
CITY LIMITS
Arroyo del Embudo
Carlisle Chapel Ch
North Bel-Air Sch
Bel Air Mission
Hope Ch
Regina Coeli Sch
St Johns Ch
Regina Coeli Ch
Duke City Drive in Theater
CANDELARIA
Windmills
Vista Encantada
LOUISIANA BOULEVARD NE
BOULEVARD
Cortez Sch
Reformed Ch
Albuquerque Ch
Montezuma Sch
MADRID TERRACE
Monroe Sch
North Girard
CARLISLE
CONSTITUTION AVENUE
Northeast Sch
Golf Course
US Indian Hospital
Jefferson Jr High Sch
Palms Sch
Heights Ch
Heights Sch
Zia Sch
National Guard
LANE
UNIVERSITY OF NEW MEXICO
Golf Course
First Ch
St Marks Ch
Monte Vista Sch
Monte Vista Ch
St Timothy's Ch
STATE FAIRGROUND
La Mesa

factor, "C" of 0.44, and a planimetered area of 24.8 Acres, the Q_{100} at the confluence point is found to be 34.9 cfs.

Referring to the map at scale 1"=100' titled "Drainage Study", the influent Q_{100} of 34.9 cfs when summed with the 16 cfs developed on the 14.8 acres between Indian School Road and the Freeway give an estimated 51 cfs at inlet into culvert 2A at a cumulative time of concentration of about 36 minutes.

The mentioned culvert 2A is a 36" diameter Reinforced Concrete Pipe (RCP) so located as to permit a hydraulic head to 5.5 ft. above its invert before spilling westward. By "Inlet Control" calculation, see nomograph 2A in the Appendix, the pipe has capacity under the cited conditions to pass 74 cfs. Accordingly, this pipe is competent to dispose of all waters from the first sub-area of study and this run-off may be dismissed from consideration with respect to the Mound Terrace Development.

Highway Culvert 2B, an 18" diameter RCP (see Drainage Study Map), has capacity to carry off 18 cfs. This amount is more than will run off from the I-40 pavement to the 2B drop inlet. There is no access for run-off from the south to get into this drop inlet.

In the Appendix, the estimated natural run-off from the Mound Terrace site is calculated on Computation Sheets 2 and 3 together with estimated run-off from the site when fully developed. The "natural" run-off in the amount of 12.7 cfs is the quantity which is adopted to rule the discharge rate from the developed site.

The site development will create an additional 56,480 cf of run-off to be managed and released by the device of ponding and orifice discharge.

The 12.7 cfs_{100} discharging from Mound Terrace tract will in part join the run-off discharged from the adjacent Synagogue tract in the public drainage ditch and culverts of the Interstate Highway. The arithmetic of the discharge scheme is posted on the "Drainage Study" Map.

It will be seen that the existing natural flow northward from the Freeway to the Embudo Diversion Channel as presently experienced is 19.5 cfs for the 100-year storm here considered. After development of the Mound tract, this flow will be 17.2 cfs. These quantities are the sum of the discharges from the 30" diameter RCP "2C" and the 30" diameter RCP "2D".

The original discharge from Culvert "2C" is calculated at 13.5 cfs, being the sum of the "natural" discharge from the Mound tract of 12.7 cfs and 0.8 cfs run-off from the highway.

The "Inlet Control" calculation to determine the capacity of RCP "2C" as given in the Appendix shows this culvert to be capable of handling 45 cfs. We project only 9.5 cfs from Mound plus 0.8 cfs from the Highway, totaling 10.5 cfs, to be delivered to this culvert after development.

The original discharge from Culvert "2D" was 6.0 cfs, being the sum of 5.3 cfs from the un-developed Synagogue tract plus 0.7 cfs of run-off from the Highway.

On the developed Synagogue tract, a ponding area and a 12" diameter corrugated metal discharge pipe with an estimated capacity of 3.0 cfs, maximum, have been installed. To this 3.0 cfs amount, we will add 3.0 cfs from the Mound Terrace site and 0.7 cfs from the highway, as before, to aggregate to 6.7 cfs after development. It is determined by calculation 2 D (Appendix) that pipe "2D" has capability of carrying 29 cfs.

Accordingly, the projected ponding and constricted release of storm waters based on the "natural" rate of run-off will allow ample latitude for variance, about 400%, before the capacity of the public channels is exceeded.

Management of the waters incident upon the Mound Terrace development to conform to the described discharge directions and rates as projected can be achieved by ponding and controlled discharge of the 100-year precipitation.

In the ultimate development of the site, two ponds, having storage volumes totaling 33,000 cubic feet and having sides 1.33 feet high, have been projected.

The rate of discharge from these ponds will be controlled by three square openings 0.33 feet high and of a length proportionate to the desired volumes as indicated on the "Drainage Study" Map.

The discharge through these openings is calculated by the formula:

$$Q_{cfs} = CdA_{sf} \sqrt{2gh}_{ft.}$$

The sub-scripts denote the units of measure. The factor Cd, the coefficient of discharge, adopted for these square openings, has been taken as 0.55. The head, h, is measured from the center of the opening. A graph of the discharge rate for varying head conditions is given on Computation Sheet 4 in the Appendix.

The hydrographic distribution of storm intensity assumed for this study is adopted from the run-off intensity curves which would indicate a peak load of over 50% of the total calculated run-off (77,272 cf) occuring within the first 45 minutes of the 6 hour storm. The ponding volume and orifice discharge rate here proposed can cope with this shock loading as is demonstrated by the In-Put/Out-Flow Tabulation on Computation Sheet 5 in the Appendix.

Referring to Computation Sheet 5, it is seen that the stored volume accumulates to 27,219 cf, maximum, being 11% less than overflow capacity. Also, the maximum rate of discharge of 12.0 cfs is about 6% less than the budgeted 12.7 cfs.

Consequent to the above, we are safe to adopt the following three orifices to be located as shown on the included "Site Plan."

Outlet	CFS, max.	Height, In.	Length, Ft.
# 1	3.0	4	1.75
# 2	4.0	4	2.75
# 3	5.7	4	3.50

In this study, all assumptions adopted have been taken so as to assure results to be on the safe side. The resulting computations have always yielded some additional margin of safety. Accordingly, it is felt that the ponding and release openings here described will provide the measures necessary for the developed Mound Terrace tract to be in substantial compliance with Section 3, Paragraph B. of Resolution No. 2, Series of 1972, of the Albuquerque Metropolitan Arroyo Flood Control Authority.

LIMBAUGH ENGINEERS, INC.

Rodney J. Dalton

Rodney J. Dalton, P.E.

P.E. & L.S. No. 2142
New Mexico

REFERENCES

1. Date Book For Civil Engineers, Design Vol. 1; SEELYE, Elwyn E. 1945

2. Hydraulic Engineering Circular No. 5, U.S. Department of Commerce, Bureau of Public Roads

3. Handbook of Hydraulics; King and Brater, 1963.

4. Maps of the U.S. Geological Survey

5. Maps of the N.M.S.H.D.

APPENDIX

LIMBAUGH ENGINEERS, INC.
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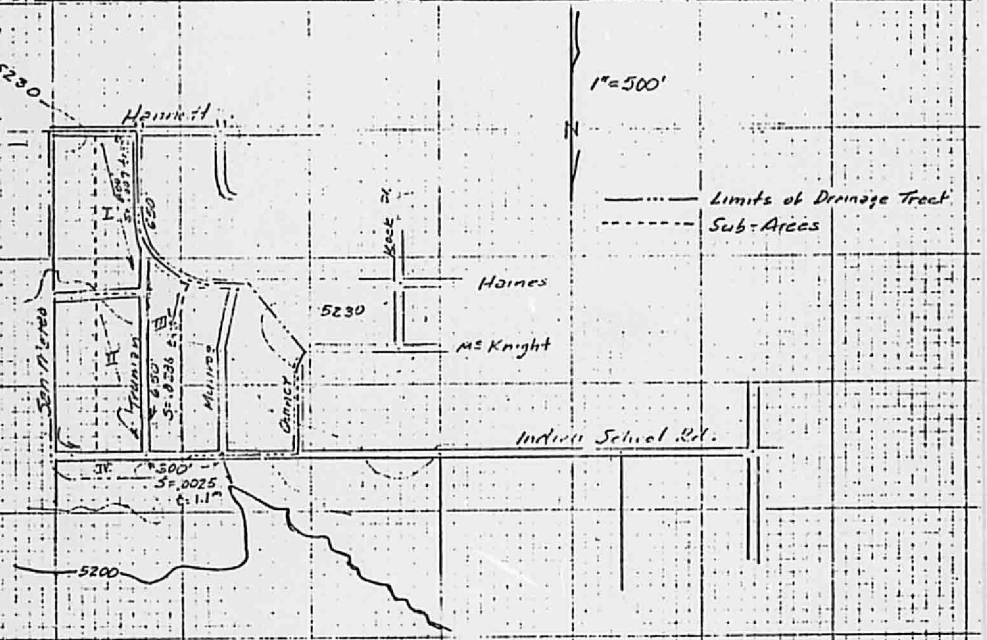
ALBUQUERQUE, NEW MEXICO 87110

SUBJECT Storm Runoff to Indian Hills, Phase II Drainage Study Intersection

COMPUTATION Q=ACi from Hannett x Truman to Hancock x Indian School JOB NO.

COMPUTED BY R.J.D. DATE 7 Nov. '73 CHECKED BY DATE



Total Area = 24.8 Ac.

A_I = 175 x 650' = 113750 = 2.6 Ac. ; t_c = 32m ; i = 3.6 in/hr ; Q = 2.6 x 0.2 x 3.6 = 1.87 cfs

A_{II} = 175 x 650' = 113750 = 2.6 Ac

A_{III} = 175 x 650' = 113500 = 2.6 Ac

A_{IV} = 60 x 350 = 20000 = 0.7 Ac.

Truman between Haines & Indian School = 650' @ 0.0256% ; t @ Q = 1.87 cfs ; V = 3.5 fps ; t = .03m

Q at Truman & Ind. Sch. = 2.6 x 5 (2.6 + 350 x .6 / 43560) = 15.5 Ac ; t_c 35m ; Q = 15.5 x 0.4 x 3.3 = 20.5 cfs

Ind. School between Truman & Quincy = 500' @ .0025 @ Q = 20.5 cfs ; V = 2.25 fps ; t = 1.11

t_c = 32 + 3 + 1.1 = 36.1m To Intersection Indian School x Truman ; ΣA = 24.8 Ac.

Q_{100} @ Ind. School x Truman = 24.8 x 0.4 x 3.2 = 31.7 cfs.

(Using C = 0.44, Q_{100} = 34.9 cfs)

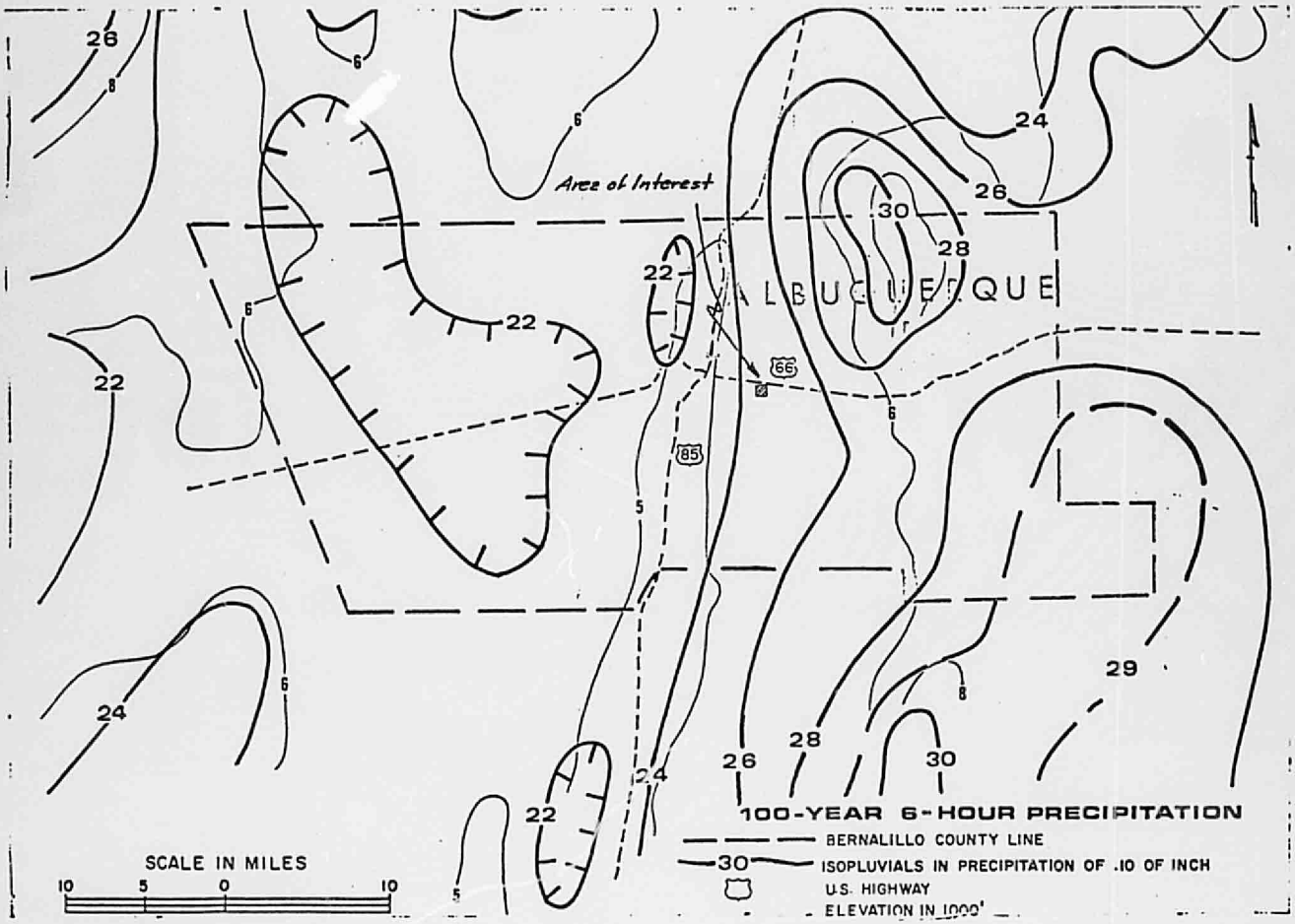

Area of Interest
ALBUQUERQUE
66
85
26
8
6
22
24
30
28
29
5
100-YEAR 6-HOUR PRECIPITATION
BERNALILLO COUNTY LINE
30 ISOPLUVIALS IN PRECIPITATION OF .10 OF INCH
U.S. HIGHWAY
ELEVATION IN 1000'
SCALE IN MILES
10 5 0 10

FIG. A. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 2 YEARS.

FIG. B. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 10 YEARS.

FIG. C. – ONE-HOUR RAINFALL, IN I
TO BE EXPECTED ONCE IN 50 Y

FIG. D. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 5 YEARS.

FIG. E. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 25 YEARS.

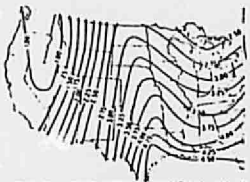


FIG. F. – ONE-HOUR RAINFALL, IN I
TO BE EXPECTED ONCE IN 100

COMPUTATION OF *i* IN RATIONAL FORMULA.

EXAMPLE: Assume expectancy period = 5 years, See Fig. D, assume locality, find 1 hour intensity = 1.75 in. p

FIG. G-INTENSITY EXPECTATION FOR ONE-HOUR RAINFAL

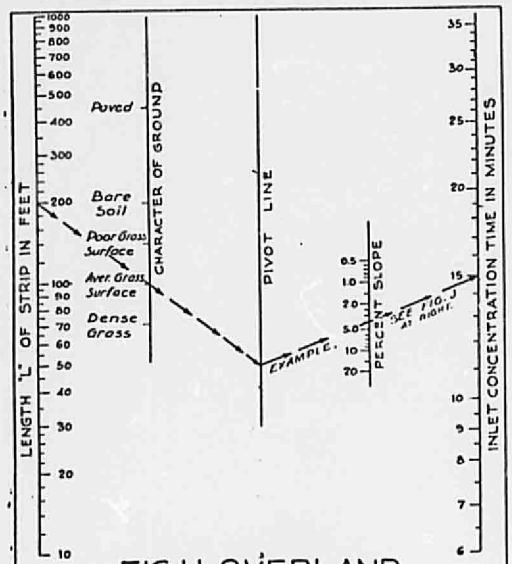


FIG. H-OVERLAND FLOW TIME.

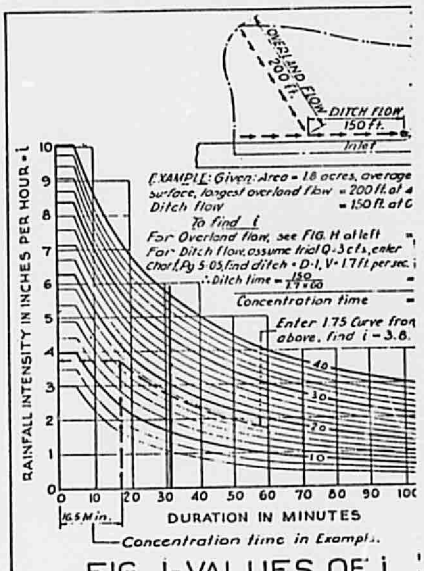


FIG. J-VALUES OF i
RAINFALL INTENSITY-DURAT

*Reproduced from Miscellaneous Publication No 204, U.S. Dept. of Agriculture, by David L. Yarnell.
+ Adapted from Engineering Manual of the War Department, Part XIII Chap 1, Dec 45

Natural Runoff from the Property

A = 12.0 acres
C = .2
I = 5.3
Q = CIA = 12.7 cfs

Developed Runoff

Roof Area = 3.8 acres
C = .9
I = 7.2
Q = CIA = 24.6 cfs

Paved Area = 5.70 acres
C = .9
I = 7.2
Q = CIA = 37.0 cfs

Grass Area = 2.5 acres
C = .15
I = 4.00
Q = CIA = 1.5 cfs

Total Development Runoff = 63 cfs

COMPUTATION SHEET 3 of 5

Volume of Natural Runoff

522,000ft^2 X 2.4* in. X 1 ft./12 in. X .2 = 20,800 ft^3 in 6 hrs.

Volume of Developed Runoff

Roof and Paved Area:

411,000ft^2 X 2.4* in. X 1 ft./12 in. X .9 = 73,980 ft^3

Grass Area:

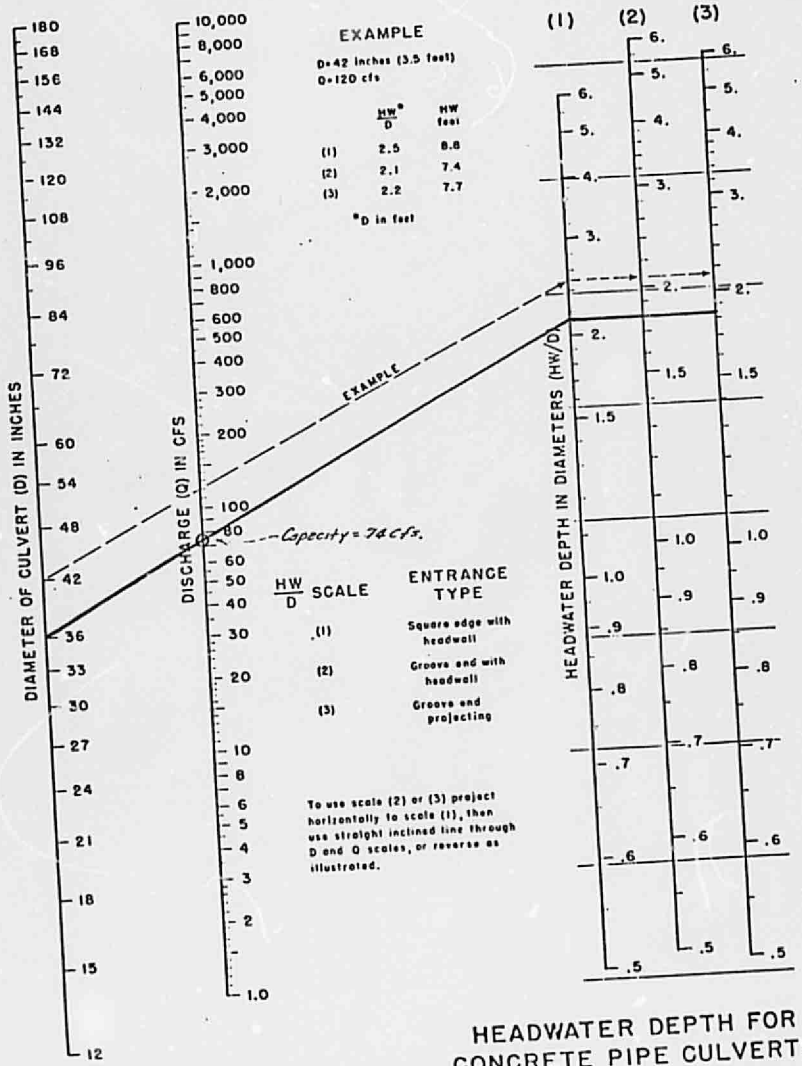
111,000ft^2 X 2.4* in. X 1 ft./12 in. X .15 = 3,300 ft^3

Total Volume of Developed Runoff = 77,280 ft^3 in 6 hrs.

Difference in Developed & Natural Runoff = 56,480 ft^3 in 6 hrs. or approximately 10,000 ft^3/hr. (Considering the 100 yr. 6 hr. rain even distributed over the 6 hrs.)

* 100 yr. 6 hr. precipitation

CHART 2 A (36")



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2B3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN 1963

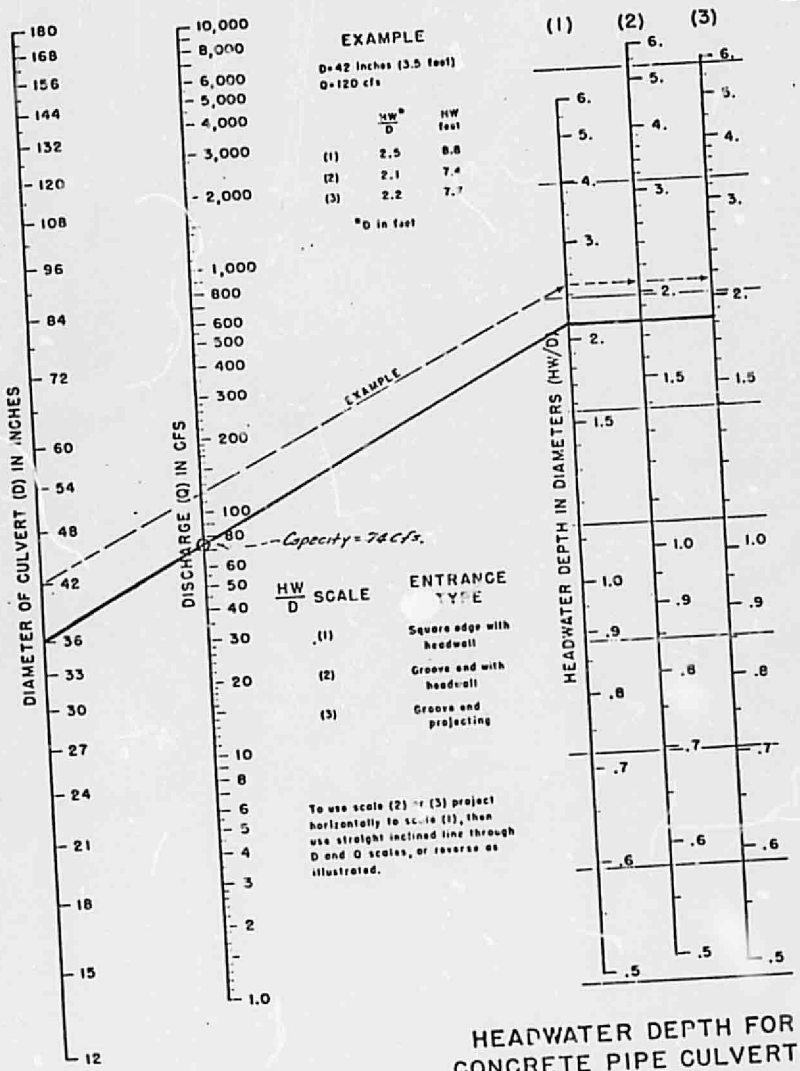
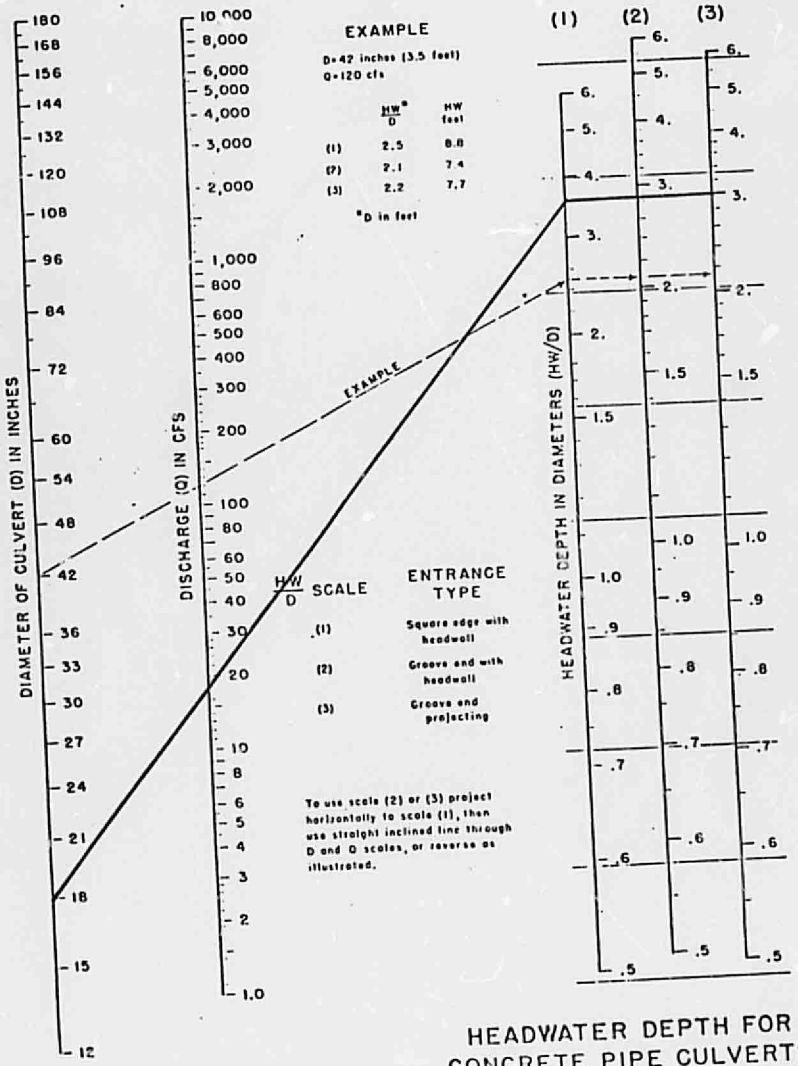


CHART 2 A (36")
EXAMPLE
D=42 inches (3.5 feet)
Q=120 cfs
HW*/D
HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HEADWATER DEPTH IN DIAMETERS (HW/D)
EXAMPLE
Capacity = 74 cfs.
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
HEADWATER SCALES 2B3
REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN 1963

CHART 2 B (18)

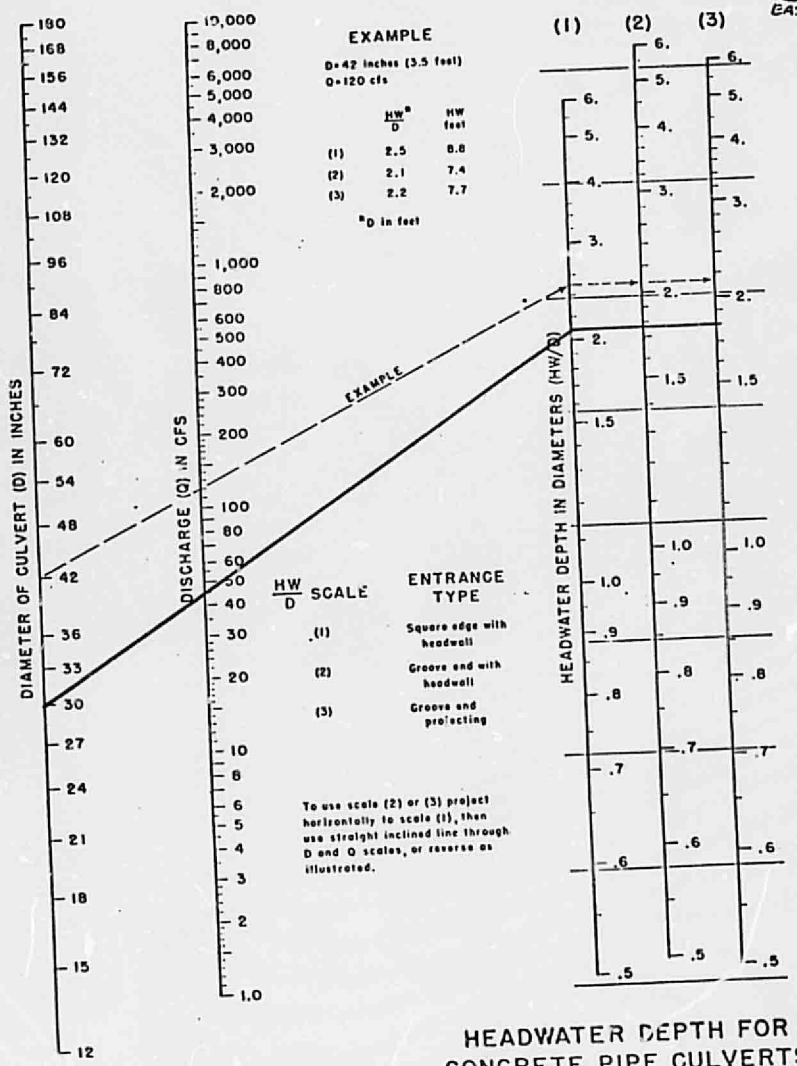


HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2B3
REVISED MAY 1964

BUREAU OF PUBLIC ROADS JAN 1963

CHART 2 C (30) EAST



HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL

HEADWATER SCALES 2B3
REVISED MAY 1964

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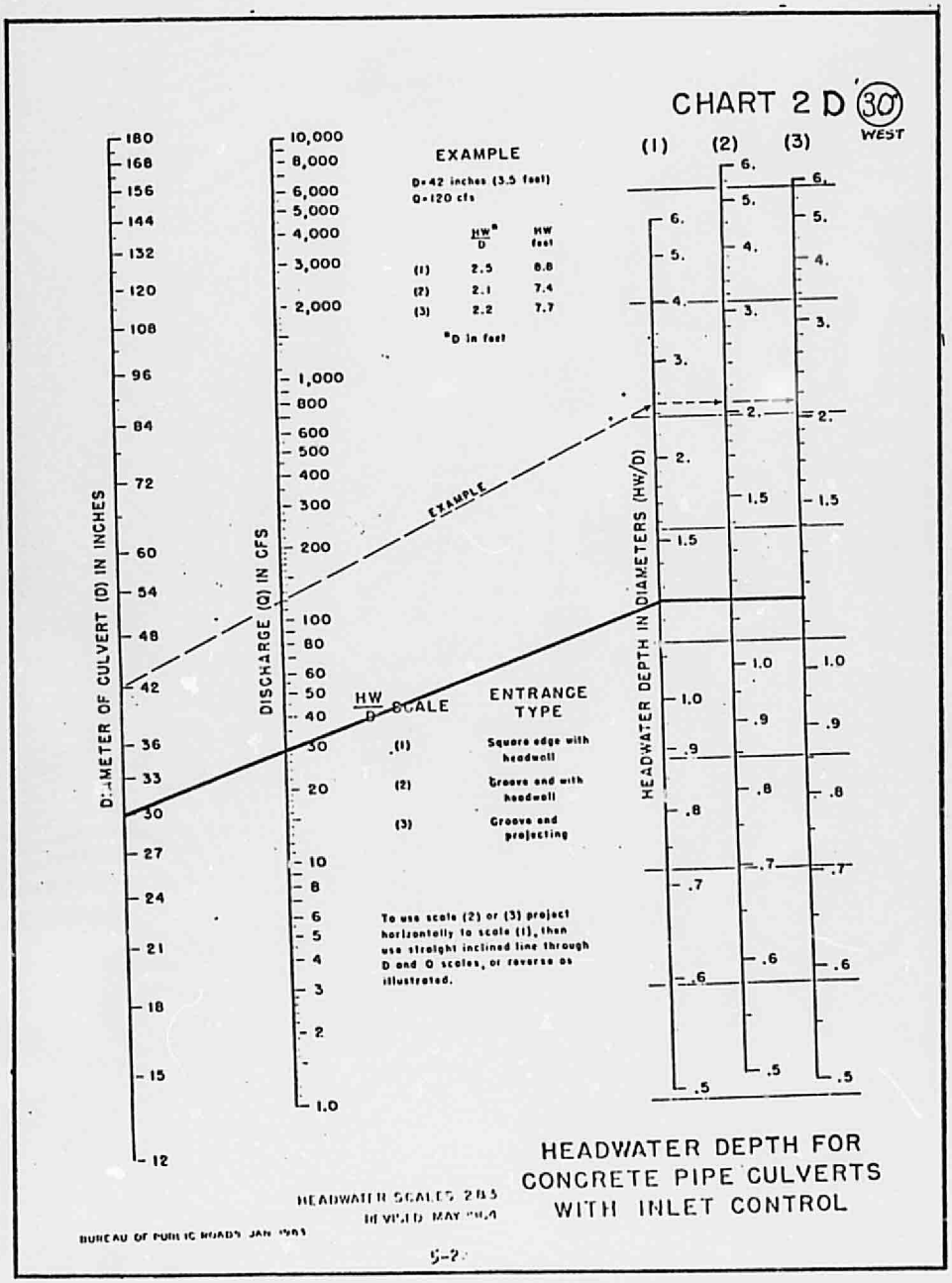
CHART 2 D
30
WEST
EXAMPLE
D=42 inches (3.5 feet)
Q=120 cfs
HW*/D
HW feet
(1) 2.5 8.8
(2) 2.1 7.4
(3) 2.2 7.7
*D in feet
EXAMPLE
DIAMETER OF CULVERT (D) IN INCHES
DISCHARGE (Q) IN CFS
HW/D SCALE
ENTRANCE TYPE
(1) Square edge with headwall
(2) Groove end with headwall
(3) Groove end projecting
To use scale (2) or (3) project horizontally to scale (1), then use straight inclined line through D and Q scales, or reverse as illustrated.
HEADWATER DEPTH IN DIAMETERS (HW/D)
(1)
(2)
(3)
HEADWATER DEPTH FOR CONCRETE PIPE CULVERTS WITH INLET CONTROL
HEADWATER SCALES 2&3
REVISED MAY 1964
BUREAU OF PUBLIC ROADS JAN 1963

LIMBAUGH ENGINEERS, INC.
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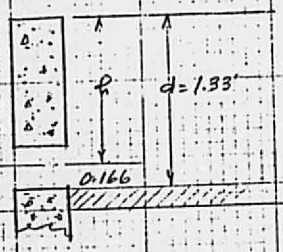
ALBUQUERQUE, NEW MEXICO 87110

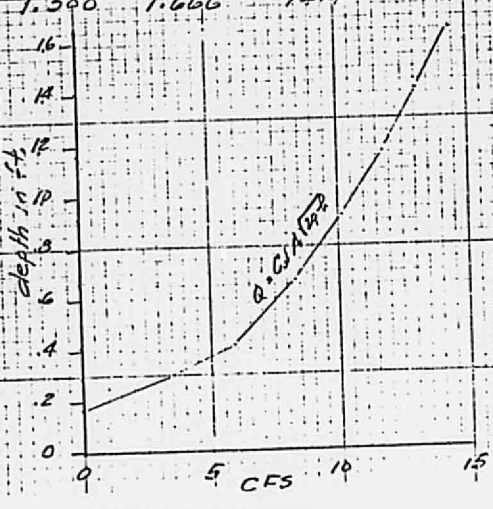
SUBJECT Discharge Thru' Rectangular Opening – Mound Terrace Study
COMPUTATION Incremental Orifice Discharge; $Q = CA\sqrt{2gh}$ JOB NO.
COMPUTED BY RJO DATE 7 Nov '73 CHECKED BY GH DATE 11 Nov. 73

RECTANGULAR OPENING 4" HIGH × 8.0' LONG



DISCHARGE CURVE FROM ABOVE OPENING 4" × 96"

h, ft	d, ft (h+.166)	Q, cfs
0.125	0.291	3.6
0.250	0.416	5.9
0.500	0.666	8.3
0.750	0.916	10.2
1.000	1.166	11.8
1.250	1.416	13.1
1.500	1.666	14.4



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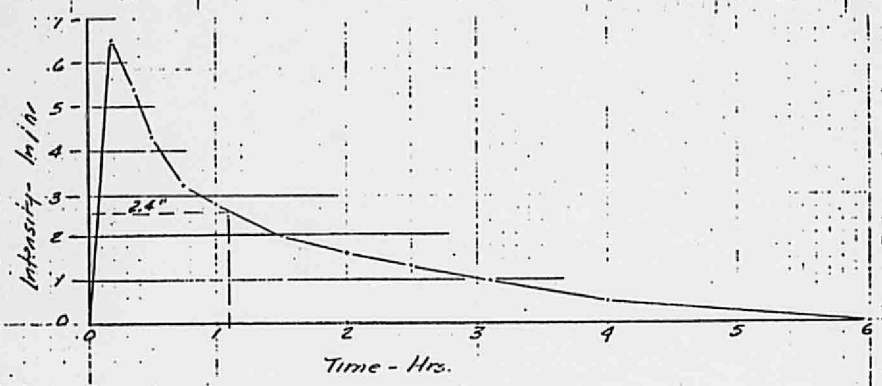
SUBJECT Developed Area Run-off vs Ponding & Discharge

COMPUTATION Q_{100} - Storage ≤ 12.7 CFS JOB NO.

COMPUTED BY RJD DATE 9 Nov. 73 CHECKED BY PHO DATE 11 Nov 73

Period (Min)	Run-off Period Vol.-CF.	Run-off Cum. Vol. CF.	Pond Depth Ft.	h Ft.	Discharge CFS	Period Discharge	Cum. Discharge	Retained Volume
0-12 (1)	3 609	3 609	0.145	0.145	1.1	792	792	2 817
12-30	36 067	39 676	0.84	0.674	10.8	11 665	12 457	27 219
30-45	5 116	44 792	1.20	1.034	12.0	10 800	23 257	21 535
45-60	4 096	48 888	0.95	0.784	10.4	9 360	32 617	16 271
60-90 (1)	6 516	55 404	0.79	0.624	7.3	16 740	49 357	6 047
90-120	4 915	60 319						
120-180	7 635	67 954						
180-240	4 915	72 869						
240-300	2 720	75 589						
300-360	1 683	77 272						

(1) Weir formula, $Q = CLh^{3/2}$, applies



Assumed Hydrograph
100 Yr - 6 hour Storm

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PAGE 4 OF 5

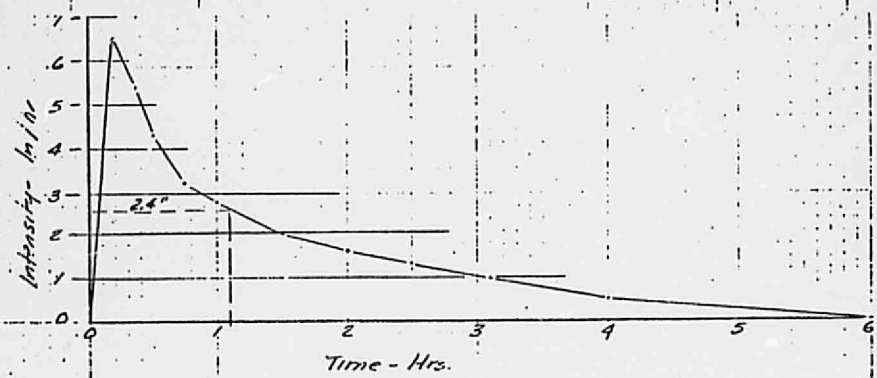
SUBJECT: Developed Area Run-off vs Ponding + Discharge

COMPUTATION: Q_{100} - Storage ≤ 12.7 CFS JOB NO.

COMPUTED BY: RJD DATE: 9 Nov. 73 CHECKED BY: PHO DATE: 11 Nov 73

Period (Min.)	Run-off Period Vol.-CF.	Run-off Cum. Vol. CF.	Pond Depth Ft.	h Ft.	Discharge cfs	Period Discharge	Cum. Discharge	Retained Volume
0-12 (1)	3.609	3609	0.145	0.145	1.1	792	792	2817
12-30	36 067	39 676	0.84	0.674	10.8	11 665	12.457	27 219
30-45	5116	44 792	1.20	1.034	12.0	10 800	23 257	21 535
45-60	4.096	48 888	0.95	0.784	10.4	9 360	32617	16 271
60-90 (1)	6516	55 404	0.79	0.624	7.3	16 740	49 357	6047
90-120	4 915	60 319						
120-180	7 635	67 954						
180-240	4.915	72 869						
240-300	2.720	75 589						
300-360	1 638	77272						

(1). Weir formula, $Q = CLh^{3/2}$, applies



Assumed Hydrograph
100 Yr - 6 hour Storm