PHOENIX-CASA GRANDE HIGHWAY
48TH STREET & BROADWAY ROAD T.I.'S
MARICOPA COUNTY
PROJECT ACI-10-3 (198)

FINAL DRAINAGE REPORT

JUNE, 1988

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ARIZONA
DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION

SECTION Bridge Drainage Section

☐ **PROJECT NO.** ACI-10-3(198)

☐ **PERMIT NO:**
- ☐ ENCROACHMENT-USE
- ☐ OUTDOOR ADVERTISING

☐ **PARCEL NO:**

☐ **OTHER** 48th St & Broadway T.I.'s

t• 62-4304 R3/86

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I. INTRODUCTION

Project ACI-10-3 (198) of the Phoenix-Casa Grande Highway, Interstate 10, is located in the southeastern portion of the City of Phoenix, Maricopa County, Arizona, as shown on the Vicinity Map, Figure 1. Project ACI-10-3 (198) will be the first project in a series of projects which encompasses the reconstruction of the 48th Street and Broadway Road traffic Interchanges and replacement of the super-elevated section of I-10 under 48th Street and Broadway Road in order to conform to the Federal Highway Administration Design Criteria.

The proposed construction in Project ACI-10-3 (198) includes the following: the widening of Ramp 'C' from Sta. 10+00 eastward to 48th Street; the reconstruction of the Ramp 'C'/48th Street Interchange to provide separate access onto 48th Street for traffic moving in the north and south directions; the widening of the I-10 eastbound lanes from Ramp 'D' eastward to Ramp 'K' for subsequent projects; tie-back of the south side of the Broadway structure, bridge deck, and retaining wall; realignment of Ramp 'J' through the tie-back, realignment of Ramp 'K' and construction of Ramp 'K-1'; and the removal of the pavement structural material at Ramps 'E' and 'H'.

With the exception of the construction associated with the Broadway structure, the proposed construction in Project ACI-10-3 (198) is temporary and will either be removed or replaced in subsequent projects.

The existing drainage system within the boundaries of Project ACI-10-3 (198) consists of a closed conduit drainage system. The existing closed conduit drainage system enters the project limits as a 48 inch conduit crossing I-10 east of the Broadway Road T.I. in the southwesterly direction. The conduit then turns in the westerly direction and runs along the I-10 south right-of-way, increasing in size to a 54 inch conduit. At the 48th Street T.I., the 54 inch conduit turns northward, runs parallel to 48th Street until it outfalls into the Tempe Drain. Runoff is collected in catch basins located along the shoulders of I-10 and the ramps and conveyed by laterals to the closed conduit drainage system.

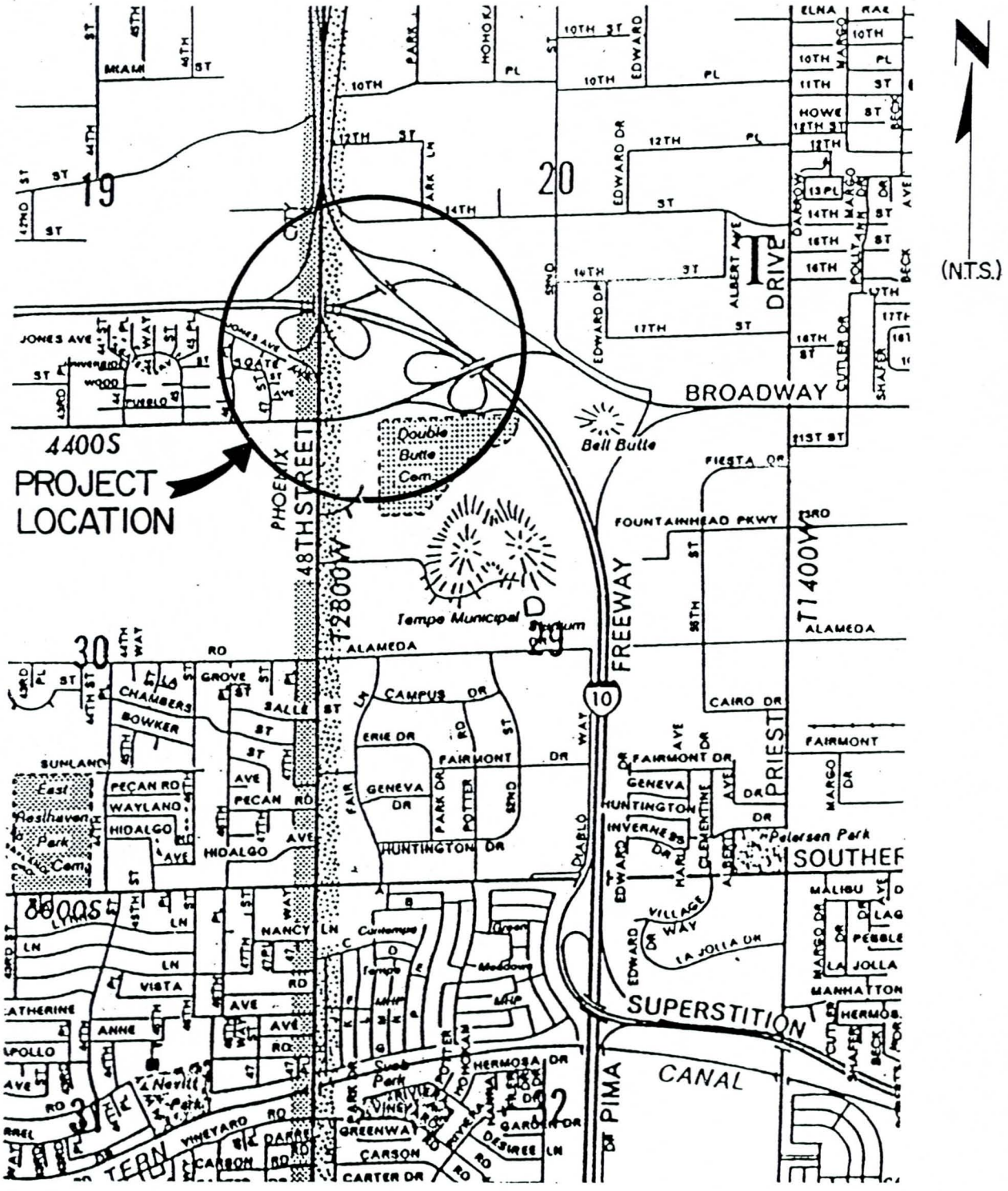

FIGURE 1: VICINITY MAP

II. SCOPE OF WORK

Project ACI-10-3 (198) Drainage Report is the first of several drainage reports addressing the phase construction of the 48th Street and Broadway Road traffic interchanges.

The Drainage Report will address the drainage requirements associated with the temporary roadway construction proposed in Project ACI-10-3 (198).

The scope of work for this project shall include the design of detention and retention facilities, drainage structures, connector pipes, and drainage swales as required.

III. DESIGN PROCEDURES AND CRITERIA

The design of the storm drainage collection and conveyance systems proposed in Project ACI-10-3 (198) shall conform to the requirements of the Arizona Department of Transportation. Including all design criteria and methods as outlined in the Project Design Guidelines, Arizona Statewide Management Program and the supporting Technical Memorandums.

Because the roadway construction proposed in Project ACI-10-3 (198) will be removed in subsequent projects, the design life is estimated at between one and two years. Therefore, in an effort to minimize the construction costs of the drainage systems, the following modifications to the present design criteria, as agreed to by the Management Consultant, will be applicable to this project. A copy of the documentation, summarizing the modification to the design criteria, has been included in Appendix D.

- On-site retention is applicable, using the 50-year 1-hour storm event to determine storage requirements of the retention facilities.

- The designer is allowed to waive the 0.5 foot of freeboard between the pavement structural section and the maximum ponding level in the detention and retention facilities, since the existing roadways within the project limits will be removed in subsequent projects.

- The new maximum allowable ponding elevations at the detention and retention facilities is established at one foot below the shoulder elevation.

The 50-year 1-hour storm event will be used to size the detention and retention facilities. The precipitation value for the 50-year 1-hour storm event in addition to the drainage areas weighted runoff coefficients based on 'C' coefficients of 0.90 for pavement and 0.5 for right-of-way and the contributing areas, are used to calculate the volume of runoff at the detention and retention facilities. The procedures presented in HEC-12 will be used to size catch basins and slotted drains. The outlet structures at the detention facilities are checked using the weir and orifice equations and the projected discharges are based on the maximum allowable ponding elevations.

IV. DRAINAGE DESIGN AND PROPOSED CONCEPTS

The drainage concept proposed for Project ACI-10-3(198) is to redirect existing runoff, that is presently being collected in catch basins along I-10 eastbound lanes shoulder and along the ramps, into either detention facilities or retention facilities, as shown on Figures 1E and 2E in Appendix E. Although the detention facilities have been provided with outlet structures, they are designed to retain the 50-year 1-hour storm event with the one foot of freeboard below the adjacent roadway shoulder, should their outlet structures become plugged. Discharge from the detention facilities will be into an existing closed conduit drainage system located parallel to I-10 along the south right-of-way. Because of the storage capacities provided at the detention facilities, undersized outlet pipes (18" diameter) will be utilized to further attentuate peak flows into the existing drainage system.

An Intergovernmental Agreement (C-3891) and a Supplemental Agreement between the City of Tempe, Maricopa County Flood Control, and Arizona Department of Transportation (ADOT) limits ADOT's discharge into the Tempe Drain to 93 cfs. The I-10 Corridor Study, Final Drainage Report Volume II, 40th Street to Baseline Road, by DMJM, dated March 1988, estimates the existing discharge into the Tempe Drain at 45 cfs and 71 cfs for the 10-year and 50-year storm events, respectively. Analysis was performed using the SCS methodology and the hydraulic program EXTRAN. Although the methodology differ from the methodology presented in this report, the analysis shows that additional available capacity exists in the present drainage system beyond its present requirements.

Since most of the runoff is existing runoff being redirected from existing catch basins to detention and retention facilities, and considering the peak flow attenuation at the detention facilities, the change in peak flow in the existing closed conduit drainage system is expected to be minimal.

The supporting calculations are included in Appendix C.

V. CONCLUSIONS

The drainage requirements for Project ACI-10-3(198) includes eight detention facilities designated Detention Facilities 'A' through 'H' and two retention facilities designated Retention Facilities 'A' and 'B'. Each of the eight detention facilities are provided with outlet structures (catch basins with aprons) connected to the existing closed conduit drainage system. Because the majority of runoff is existing runoff redirected to the detention facilities, and considering the peak flow attentuation provided by detention, the projected flows into the existing drainage system can be expected to be equivalent or less than the existing flows. Although each detention facility has been provided with an outlet structure, the facility has been sized to store the runoff from a 50-year 1-hour storm event should their outlet structures become plugged.

Retention Facilities 'A' and 'B' at the intersection of Ramp 'C' and 48th Street are implemented because of adequate available storage, the small drainage areas, and the monitary savings in not having to provide drainage structures.

Because of the short design life of the proposed drainage facilities, it was decided that discharge protection for the conduits discharging into the detention facilities did not warrant the additional cost, and therefore, discharge from the outlet pipe would be allowed to flow overland to the detention facilities with the minor erosion occurring.

In summary, runoff within the limits of Project ACI-10-3 (198) is redirected to either detention or retention facilities where temporary on-site storage is provided until discharged into the existing drainage system or permanent on-site storage is provided. The changes to the drainage within the project limits will not adversely affect the existing drainage system nor exceed the outfall discharge limitation at the Tempe Drain.

APPENDICES

APPENDIX A

- DESIGN CRITERIA

DRAINAGE DESIGN CRITERIA/GUIDELINE

102.1 DESIGN REFERENCES

Arizona Highway Department Bridge Division, "Hydrologic Design for Highway Drainage in Arizona." (ADOT Manual I)

Arizona Highway Department Structures Section Hydraulics Division, "Hydrologic and Hydraulic Training Session," Revised December 1973. (ADOT Manual II)

State of Arizona Department of Transportation, "Construction Division of Highways Standard Drawings," June 1986. (ADOT Standards C-__.__)

U.S. Department of Transportation, Federal Highway Administration, "Drainage of Highway Pavements," Hydraulic Engineering Circular No. 12, March 1988. (HEC No. 12)

U.S. Department of Transportation, Federal Highway Administration, "Hydraulic Charts for the Selection of Highway Culverts," Hydraulic Engineering Circular No. 5, latest edition. (HEC No. 5)

102.2 HYDROLOGIC DESIGN

1. Peak Rates of Runoff

Reference ADOT Manuals and data sheets included in manuals.

a) For Urban areas

(1) Use Rational Method for urban area under 0.1 square miles (64 acres) and pavement and median drainage.

(2) Use TR-20 or TR-55 method as developed by the Soil Conservation Service or HEC-1 Flood Hydrograph Package as developed by the U.S. Army Corps of Engineers for urban areas greater than 0.1 square miles.

b) Non-Urban Areas

(1) Use Soil Conservation Service Method, Part I, for areas up to 10 square miles. Reference ADOT Manuals.

(2) Use Soil Conservation Service Method, Part II for areas greater than 10 square miles. Reference ADOT Manuals.

c) Design should consider land development expected in the area for 20 years in the future.

2. Frequencies

a) The following frequencies are applicable:

Type of Structure	Type of Project		
	Interstate	Primary	Secondary
Bridge	50 yr.**	50 yr.**	50 yr.*
Culvert	50 yr.**	50 yr.**	25 yr.
Storm Sewer	10 yr.	10 yr.	10 yr.
Storm Sewer for Depressed Section	50 yr.	25 yr.***	10 yr.***
Pavement Drainage	10 yr.	10 yr.	10 yr.

* A lesser value may be used when conditions warrant. Justification shall be included in the report.

** The flood of record should be used if greater than the 50 year event.

*** A higher value may be used where conditions warrant. Justification shall be included in the report.

b) In cases where the highway encroaches into a regulatory floodway, the 100 year design frequency shall be used.

c) The design frequencies shall be reviewed to assure compliance with Federal Highway Program Manual, Volume 6, Chapter 7, Section 3, Subsection 2.

102.3 PAVEMENT DESIGN

This section is limited to the drainage of pavements where an underground sewer system is not required and where pavement runoff must be concentrated at the roadway shoulder to prevent erosion of the embankment face.

1. Storm frequency shall be in accordance with criteria in Section 1.2, Design Frequency, ADOT Manual I, pages 1 and 2.

2. Storm runoff shall be calculated by the Rational Method.

3. Allowable ponding widths shall be as follows:

Roadway Type	Maximum Spread	Frequency
Rural 4-lane divided	Lt. shoulder width Rt. shoulder width plus 1/2 adjacent traffic lane width	10 years
Urban 4 or 6 lane divided by raised median	Lt. gutter width plus any shoulder width	10 years
	Rt. shoulder, parking or distress lane width plus 1/2 adjacent traffic lane width	
4 or 6-lane undivided	Shoulder, parking or distress lane width plus 1/2 adjacent traffic lane width	10 years
2-lane undivided	Gutter and/or shoulder parking or distress lane width	10 years
22′ or 24′ ramps (Incl. accel. & decel. lane)	Left 2′ Right 8′	10 years

4. Embankment curb shall be used at the edge of paved roadways on fills where storm water cannot be permitted to run down and erode the embankment face.

5. Hydraulic Capacity Charts have been developed for standard ADOT embankment inlets and shall be used in the design. Reference ADOT Standards C-04.10 (Formerly C-4.01) and C-04.20 (Formerly C-4.02).

The hydraulic capacity charts for these standards have the clogging factors incorporated into them and no additional reduction should be made.

6. Inlets on grade

   Portion of flow permitted to by-pass inlet and be picked up at succeeding inlets downstream.

7. Inlets at Sag

   Inlets must be sized to intercept all of the flow. No flow will be permitted to overtop the curb. If the inlet capacity in the sag is less than the approach flow, then additional inlets shall be placed upstream to reduce the flow at the sag.

8. Spillways and Downdrains:

   a) Use between inlets and toe of slope and in accordance with ADOT Standards C-04.10 (formerly C-4.01), C-04.20 (Formerly C-4.02) and C-04.40.

   b) The curb opening inlets on grade and in sags shall be spaced such that curbs are not overtopped and the pavement spread remains within the specified limits.

   c) If possible, space the curb opening limits to eliminate water by-passing. This is not required, but is preferable.

   d) Downdrains will be used where embankment slopes are steeper than 4:1 and where spillways will be visible from the main roadway.

   e) Spillways will be used where embankment slopes are 4:1 or flatter.

9. All computations to be performed on ADOT Pavement Drainage Design Sheets contained in ADOT Manuals.

102.4 MEDIAN DRAINAGE

1. The storm runoff from medians will be removed by median inlets or standard culverts.

2. Storm frequency shall be in accordance with criteria in Section 1.2, Design Frequency, ADOT Manual I, pages 1 and 2.

3. Storm runoff shall be calculated by the Rational Method.

4. The maximum allowable ponding depth in the median shall not exceed the elevation of the top of the subgrade at the edge of pavement.

5. Median dykes will be required at the downstream side of all median drains except where the inlet is located in a sag. The top of the median dyke should be at least 0.5 feet higher than the design highwater elevation of the inlet.

6. Pave median when velocity is excessive and erosion is a problem.

7. Hydraulic capacity charts have been developed for standard ADOT median inlets and shall be used in the design. Reference ADOT Standards C-15.30 (Formerly C-15.08), C-15.80 (Formerly C-15.09) and C-15.90 (Formerly C-15.10).

   The hydraulic capacity charts for these standards have the clogging factors incorporated into them and no additional reduction should be made.

8. All computations are to be performed on ADOT Median Drainage Design Sheets contained in the ADOT Manuals.

102.5 STORM SEWER SYSTEM

1. The storm sewer system shall consist of a system of inlets, conduits and other appurtenances which are designed to collect and convey storm runoff from the roadways and adjacent contributory drainage areas to an outlet.

2. Storm frequency shall be in accordance with criteria in Section 1.2, Design Frequency, ADOT Manual I, pages 1 and 2, with the exception of underpasses or other depressed roadways where ponded water can be removed only through a storm sewer system. For this exception, the storm frequency shall be 50 years.

3. Storm runoff shall be calculated by the Rational Method.

4. The allowable ponding width shall be in accordance with criteria in Section 2.2, Pavement Design, Item 3 of part 102.3 of this section.

5. Hydraulic capacities of the standard ADOT inlets shall be determined utilizing the general equations.

6. The following factors shall be applied to grate, curb opening and combination inlets due to clogging:

   a) Grates

(1) Sump Conditions

(a) Orifice Flow - actual area = 2.0 x required area

(b) Weir Flow - actual Perimeter = 2.0 x required perimeter

(2) Continuous Grade Conditions - actual length of opening = 2.0 x required length or greater

b) Curb Opening Inlets

(1) Sump Conditions - actual length of opening = 1.25 x required length or greater

(2) Continuous Grade Conditions - actual length of opening = 1.25 x required length or greater

c) Combination Grate and Curb Opening - ADOT Standard C-15.10 Catch Basin, Type 1 and 2

(1) Sump Conditions

(a) Orifice Flow - actual area = 1.0 x required area of grate only

(b) Weir Flow - actual perimeter = 1.0 x required perimeter of grate only

(2) Continuous Grade Conditions - actual length of opening = 1.0 x required length of grate only

d) Standard C-15.40 Catch Basin, Type 5

(1) Sump Conditions

(a) Orifice Flow - actual area = 1.0 x required area of grate only

(b) Weir Flow - actual perimeter = 1.0 x required perimeter of grate only

(2) Continuous Grade Conditions - actual curb opening length upstream from grate = 1.25 x required length

e) Slotted Drains

(1) Sump Conditions - actual length of opening = 2.0 x required length

(2) Continuous Grade Conditions - actual length of opening = 1.25 x required length

7. Hydraulic profile of the storm sewer system shall be required to assure the system will perform as designed.

8. Catch Basins

   a) All catch basins and grates shall conform with ADOT Standards, C-15.10 thru C-15.90.

   b) Catch basins shall be spaced such that curbs are not overtopped and the pavement spread remains within the specified limits.

9. All computations shall be shown on ADOT standard design forms contained in ADOT Manuals.

   a) Runoff Calculation Sheet - all design data and hydrologic calculations

   b) Inlet Calculations Sheet - all inlet design data and calculations

   c) Storm Sewer Calculation Sheet - all storm sewer design data and calculations

10. Depressed Roadways

   a) Design the storm sewer for a 50-year frequency.

   b) Determine the elevation of the 50-year hydraulic grade line (HGL) and check to see that the HGL is below the low steel at all roadway inlets.

   c) If the 50-year HGL elevation is less than six inches below the elevation of the low steel of any roadway inlet grate, adjust the conduit sizes or flow line elevations as necessary to lower the HGL elevation at that point.

   d) If revisions have been made, recheck the 50-year HGL to determine the effect on the remainder of the system.

   e) If the storm sewer terminates at a pump station, the 50-year HGL must consider the water surface elevation in the pump station.

   f) The 50-year HGL shall be shown on the final profiles.

102.6 CROSSROAD CULVERTS

1. Storm frequency shall be in accordance with criteria in Section 1.2, Design Frequency, ADOT Manual I, pages 1 and 2.

2. Storm water runoff shall be calculated utilizing the correct method for the conditions as stipulated in Section 1.2, Design Frequency, ADOT Manual I, pages 1 and 2.

3. Calculate size and type of structure in accordance with HEC No. 5. The maximum allowable headwater shall not exceed the elevation of the top of subgrade at the edge of pavement.

4. The culvert inlet shall be as close as possible to the natural stream bed. Any culvert having an invert elevation more than six inches below the natural stream profile shall be assumed to have only the waterway opening above the stream bed profile for hydraulic capacity calculations.

5. Outlet Protection

   a) Structures in range and desert areas, with drainage areas less than 0.1 square miles, will not require outlet protection unless specifically requested at the field review.

   b) Structures in urban areas, with drainage areas less than 0.1 square miles, will be analyzed for potential scour and protection designed accordingly.

   c) All structures in areas with drainage areas greater than 0.1 square miles shall be designed with outlet protection. The following parameters are suggested for determining the type of outlet protection required. The Consultant shall be responsible for the design and type of material used.

Ratio of Outlet Velocity to Natural Stream Velocity	Outlet Protection
1.0 to 1.5	No protection required
1.5 to 2.0 with outlet velocity less than 10 fps	Dump rock riprap
1.5 to 2.5 with outlet velocity greater than 10 fps	Wired tied rock riprap
Greater than 2.5	Concrete energy dissipater or consider larger culvert

Use rock density of 140 lb/cu. ft. in the design of graded riprap. The dumped rock riprap shall be designed in accordance with HEC No. 15.

d) For outlet flow velocities greater than either 2.5 times the natural stream velocity or 16 feet per second, an energy dissipator should be considered. The design shall be in accordance with HEC No. 14.

6. The 100-year event shall be analyzed for potential damage to upstream property, roadway, and channel erosion for all crossroad culverts.

102.7 CHANNEL AND DITCH

1. Hydraulic design shall be in accordance with ADOT manuals, Hydraulic Engineering Circulars prepared by the Federal Highway Administration, and sound engineering practice.

2. The suggested minimum freeboard for improved channels shall be as follows:

   a) Supercritical Flow - 0.25 x water depth

   b) Subcritical Flow - 0.20 x (depth + $V^2/2g$)

   c) No freeboard less than 1.0 foot

3. Paving shall be used where necessary for erosion protection.

102.8 EROSION CONTROL

1. Temporary erosion control methods shall be employed during construction to control erosion and sediment until the permanent protection is provided. The types of protection are generally as follows:

   a) Direct Protection of Ground Surface - ground cover, soil stabilization or riprap

   b) Control of Runoff Pattern - diversion ditches, shoulder berms or slope drains

   c) Removal of Sediment from Water - silt fences, check dams or sediment basins

2. The NCHRP publication entitled "Erosion Control During Highway Construction" and the AASHTO Guideline "Erosion and Sediment Control in Highway Construction" may be used as guides for evaluation of measures.

3. Permanent erosion control due to erosive velocities shall be in accordance with criteria set forth in the Hydraulic Engineering Circular No. 15.

102.9 SCOUR AND BANK PROTECTION

1. Bank protection shall be provided to direct flows and protect banks, channels and roadway embankments from scour. Bank protection shall not be used to protect new bridge foundations. Remedial bank protection may be necessary at existing bridges.

2. The bank protection shall accommodate the design flow for the following conditions:

   a) Scour and velocity shall be computed for conditions which can cause the largest value of each considering the site conditions.

   b) The bank protection shall extend below the channel thalweg sufficient to be stable for the anticipated scour.

3. Methods of bank protection shall be in accordance with HEC No. 15. Due to the more recent information utilized to develop HEC No. 15, the use of HEC No. 11 for design of bank protection is discouraged.

102.10 SUBSURFACE DRAINAGE

1. Use perforated metal pipe.

2. Install in a narrow trench backfilled with a filter material.

3. Obtain design data from "Handbook of Steel Drainage and Highway Construction Products", American Iron and Steel Institute, latest edition.

4. Due to the involvement of geology and soil mechanics, it is suggested that the design consultant cooperate with ADOT Materials Services and Drainage Design Services.

102.11 CATTLE AND/OR VEHICLE PASSES

1. The use of a cattle and/or vehicle pass for a drainage structure is permissible.

2. When the design is a multi-barrel structure and is used for drainage purposes, at least one barrel must meet the requirement of a passageway as follows:

TYPE	SIZE
Stock pass	Min. 120" dia. structural plate or min. 10' x 8" box culvert
Vehicle pass	Min. 144" dia. structural plate or min. 10' x 10' box culvert

3. The desirable maximum length for a stock pass is 250 feet.

4. Bridge design should incorporate the same requirements.

5. Design and details shall be in accordance with ADOT Standards.

APPENDIX B

- PRECIPITATION CALCULATIONS

Steps to be used to determine precipitation values for various durations and return periods.

STEP 1. From the precipitation maps in the manual "Hydrologic Design for Highway Drainage in Arizona", determine the precipitation values for the 6 and 24 hour duration storms for return periods of 2, 5, 10, 25, 50 and 100 years. Tabulate these values in Table 1 in the column headed 'Map Values'

TABLE 1

Return Period (Years)	Precipitation Values (inches)			
	6 hour duration		24 hour duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.15	1.15	1.4	1.4
5	1.7	1.65	2.0	2.0
10	2.0	2.0	2.25	2.4
25	2.4	2.4	3.0	2.9
50	2.8	2.8	3.4	3.35
100	3.2	3.2	3.8	3.8

NOTE: There is a possibility of making an error while reading the maps because, (1) a site is not easy to locate precisely on a series of 12 maps, (2) there may be some slight registration differences in printing, and (3) precise interpolation between isolines is difficult. In order to minimize any errors in reading the maps, these values should be plotted on the diagram "Precipitation Depth versus Return Period" Fig. 1.

STEP 2. Plot these values on the diagram "Precipitation Depth versus Return Period" Fig. 1.

STEP 3. Draw a line of best fit through the 6 hour precipitation values and another line through the 24 hour precipitation values.

STEP 4. Tabulate the values represented by the lines of best fit, obtained in Step 3, in the column of Table 1 entitled "Corrected Value".

NOTE: The 1 hour precipitation value is needed to determine the 2 and 3 hour values as well as the 5, 10, 15 and 30 minute values.

STEP 5. Using the 6 and 24 hour values for the 2 year and 100 year return periods, from Table 1 (corrected values), solve the following equations to determine the 1 hour values:

$$Y_2 = -0.011 + 0.942 \ (X_1^2/X_2)$$

$$Y_{100} = 0.494 + 0.755 \ (X_3^2/X_4)$$

where Y_2 = 2 year 1 hour value
Y_{100} = 100 year 1 hour value
X_1 = 2 year 6 hour value from Table 1
X_2 = 2 year 24 hour value from Table 1
X_3 = 100 year 6 hour value from Table 1
X_4 = 100 year 24 hour value from Table 1

STEP 6. To determine 1 hour precipitation values for the other return periods, first plot the 2 year 1 hour value and the 100 year 1 hour value on Figure 1. Connect the two points by a straight line. The values on this line will give the 1 hour precipitation values for the various return periods.

STEP 7. To determine the 2 and 3 hour precipitation values, use the following formulae with data for the appropriate return period from Table 1 (corrected values):

2 hour = 0.341 (6 hour) + 0.659 (1 hour)
3 hour = 0.569 (6 hour) + 0.431 (1 hour)

An alternate method in lieu of the above equations is to use Fig. 2-2 (revised 4-75).

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SUBJECT: DRAINAGE DESIGN CRITERIA — CEM

T_{ALL} = ALLOWABLE SPREAD

= 12' − 1.08' = 10.92 ⇒ SAY 11.'

(1.08' = ½ JERSEY MEDIAN BARRIER WIDTH)

30' = WIDTH OF H.O.V. LANE

C-10.12 TYPE II, TYP SECTION

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n = MANNINGS ROUGHNESS COEFF. FOR REINFORCED CONC. PIPE.

n = .013

BRATOL/KING. TABLE 6-16

n = FOR PAVEMENT USE .016 (CONC. LINED CHANNEL).

BRATOL & KING. 7-22.

C = RUNOFF COEFFICIENT USING RATIONAL METHOD

C = .95

ADOT MANUAL d-2

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i = RAINFALL INTENSITY VALUES.

ADOT MANUAL ADDENDUM PROCEDURE

$Y_2 = -0.011 + 0.942\,(X_1^2/X_2)$

$Y_{100} = 0.494 + 0.755\,(X_3^2/X_4)$

Y_2 = 2yr − 1 HR

Y_{100} = 100yr − 1HR

X_1 = 2yr − 6 HR

X_2 = 2yr − 24 HR

X_3 = 100yr − 6 HR

X_4 = 100yr − 24HR.

$Y_2 = -0.011 + 0.942\,(1.15^2/1.4) = .88'$

$Y_{100} = 0.494 + .755\,(3.2^2/3.8) = 2.53'$

DMJM	PROJECT	PROJECT NO :	PAGE
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SUBJECT DRAINAGE DESIGN CRITERIA CONT. CEM

i = RAINFALL INTENSITY.

T_c = TIME OF CONCENTRATION

MINIMUM = 10 MINUTES

ADOT MANUAL PAGE F-24

∴ FOR 10 YR STORM 24 HR = 2.4

6 HR = 2.0

1 HR = 1.5

FOR 50YR STORM 24 HR = 3.35

6 HR = 2.8

1 HR = 2.2

10 YR - t_c = 10 MIN i = 4.20 IN/HR.

ADOT FIG 3-2 RAINFALL INTEN CURVE

50 YR - t_c = 10 MIN i = 6.0 IN/HR.

Project No. _____ Station _____

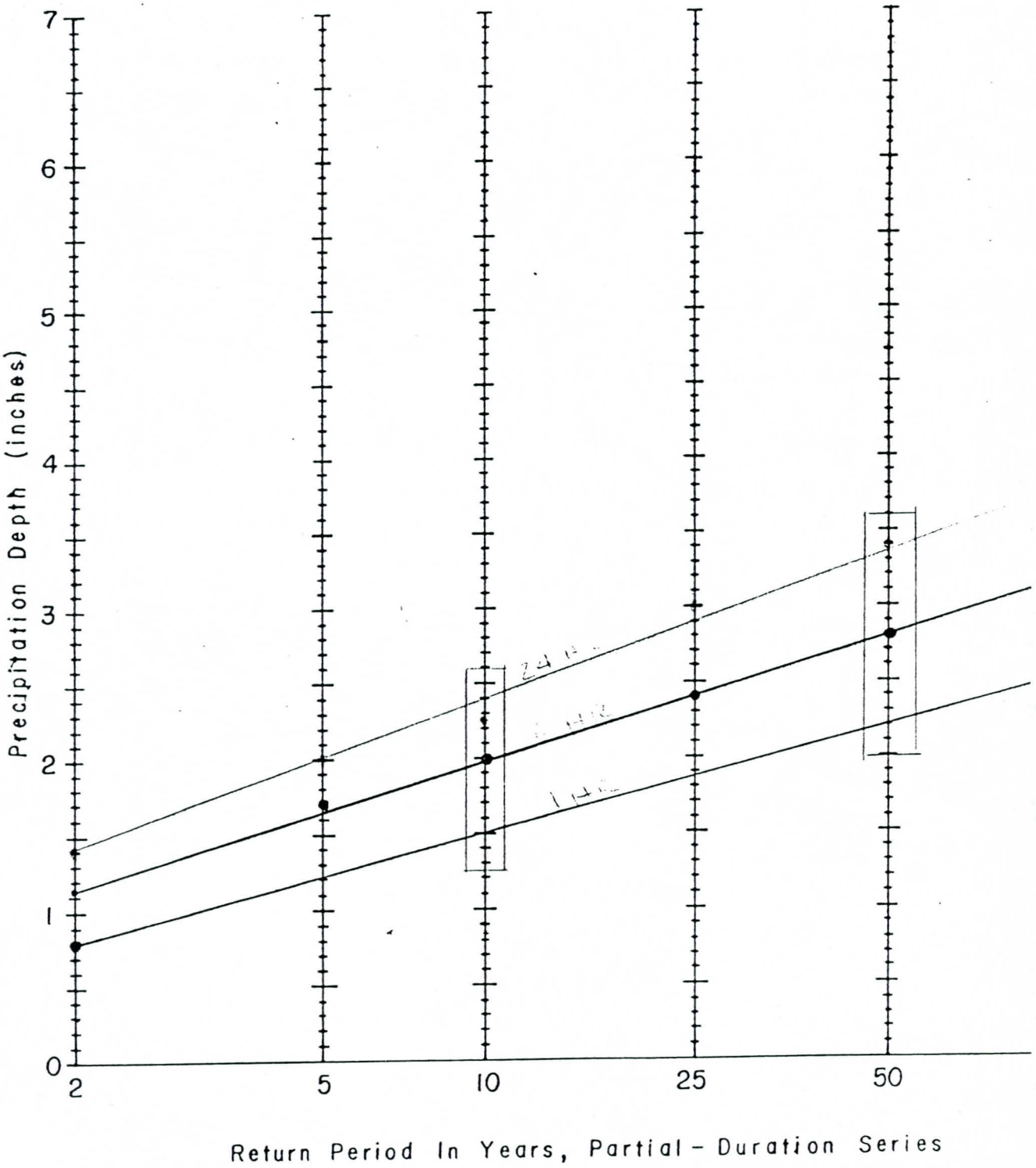

Figure 1 Precipitation Depth Versus Return Period for Partial – Duration Series

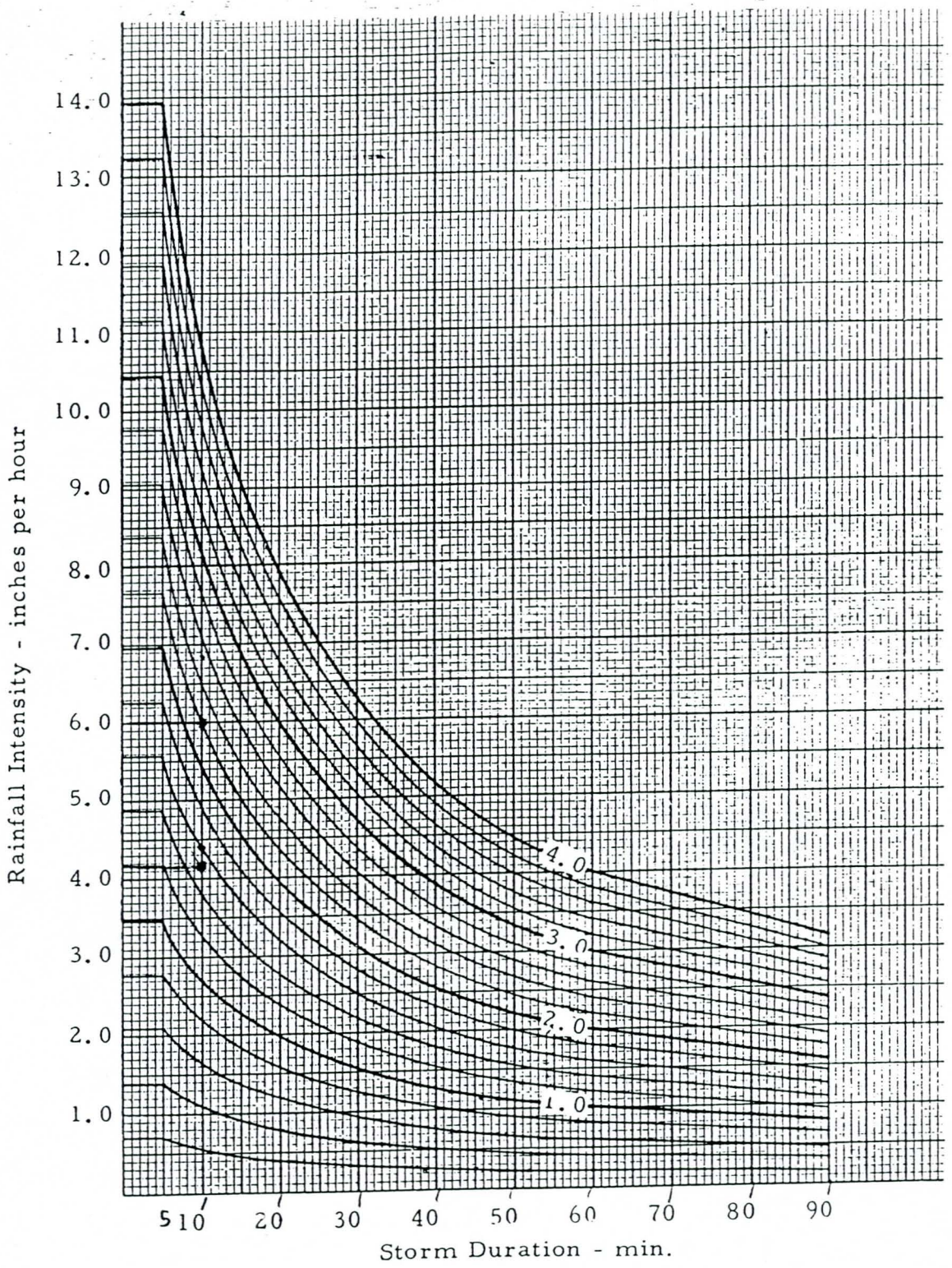

Fig. 3-2

STANDARD DURATION RAINFALL - INTENSITY CURVES

Based on:
Weather Bureau
Technical Paper No. 4

APPENDIX C

- SUPPORTING CALCULATIONS

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.'s	PROJECT NO: 5029.04	PAGE 28
		BY: DNJ / DATE: APR. 3, 88	

SUBJECT DETENTION FACILITY 'A' - DRAINAGE AREA D-100 & D-114

(100)

I. TOTAL AREA = 227,580 FT² (INCLUDES BOTH DRAINAGE AREA 110 & DRAINAGE AREA 114)

TYPES OF LAND USES:

PAV'T = 51,620 FT²

R.O.W = 175,960 FT²

II. WEIGHTED 'C' COEFFICIENT

PAV'T = 0.90

R.O.W. = 0.50

$$C_w = \frac{0.9\,(51,620) + 0.5\,(175,960)}{227,580}$$

$$= 0.59$$

III. VOLUME OF RUNOFF

STORM EVENT = 50-YR. 1-HR.
PRECIPITATION = 2.2 INCHES

$$\text{VOLUME} = \frac{2.2 \text{ INCHES}}{12 \text{ IN/FT}} \times 0.59 \times 227,580 \text{ FT}^2$$

$$= 24,620 \text{ FT}^3$$

IV AVAILABLE STORAGE AT DETENTION FACILITY 'A'

ELEVATION	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	ΣVOLUME (FT³)
1136	28,250				0
		29,700	1.0	29,700	
1137	31,150				29,700

V Summary

DETENTION FACILITY 'A' HAS SUFFICIENT AVAILABLE STORAGE TO RETAIN THE 50-YEAR-HOUR STORM EVENT AT A PONDING ELEVATION OF APPROXIMATELY 1137 WHICH WILL NOT AFFECT TRAFFIC ON I-10 EASTBOUND LANES.

(100) CATCH BASIN LOCATED AT DETENTION FACILITY 'A'

DRAINAGE AREAS D-100 & D-114
PROJECTED MAXIMUM PONDING ELEV = 1137.0
GRATE ELEV = 1135.67
PROJECTED MAXIMUM PONDING DEPTH IS 1.33 FT

MAXIMUM FLOW FROM DETENTION FACILITY 'A' THROUGH CATCH BASIN 100 (SUMP CONDITION)

TYPE OF CATCH BASIN: TYPE IV - SINGLE
C-15.30

GRATE: WIDTH = 2.0 FT
LENGTH = 2'-11 3/4" ≈ 3.0 FT

CLOGGING FACTOR = 2.0 (SUMP CONDITION)

CATCH BASIN HAS A CONCRETE APRON.

HEC-12 ANALYSIS

$$\text{PERIMETER } (P) = \frac{2(2)+3(2)}{2} = 5.0 \text{ LIN. FT.}$$

2 ← CLOGGING FACTOR

EQ. 17: GRATE INLET OPERATING AS A WEIR

$Q_i = C_w P d^{1.5}$ WHERE: $C_w = 3.0$
$P = 5.0$ LIN. FT.
$d = 1.33$ FT

$Q_i = 3.0(5)(1.33)^{1.5}$
$= 22.0$ CFS

EQ. 18: GRATE INLET OPERATING AS A ORIFICE

$Q_i = C_o A (2gd)^{1.5}$ WHERE $C_o = 0.67$
$A = 4.84/2 = 2.42$ FT²
↑ CLOGGING FACTOR

$Q_i = 0.67(2.42)[2(32.2)(1.33)]^{0.5}$
$= 15.0$ CFS

∴ ORIFICE CONDITION CONTROLS
$Q = 15$ CFS (50-yr 1-HR STORM EVENT)

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SUBJECT DETENTION FACILITY '6' (STA. 8080+14 TO 8083+00) - DRAINAGE AREA D-102

(101) DETENTION FACILITY '6' IS DESIGN TO PROVIDE RETENTION FOR THE 50-YR. 1-HR. STORM EVENT.

I. TOTAL AREA = 39,180 FT²

PAV'T = 25,460 FT²

R.O.W. = 13,720 FT²

II WEIGHTED 'C' COEFFICIENT

PAV'T = 0.90

R.O.W. = 0.50

$$C_w = \frac{0.9(25{,}460) + 0.5(13720)}{39180}$$

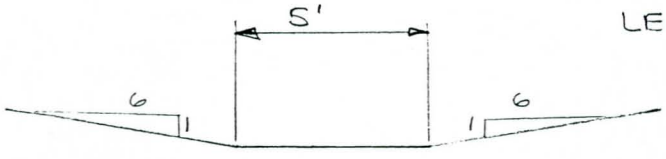
$$= 0.76$$

III VOLUME OF RUNOFF

STORM EVENT = 50-YR. 1-HR.
2.2 INCHES OF PRECIP.

$$\text{VOLUME} = \frac{2.2 \text{ IN}}{12 \text{ IN/FT}} \times 0.76 \times 39{,}180 \text{ FT}^2 = 5460 \text{ FT}^3$$

IV AVAILABLE STORAGE AT DRAINAGE DITCH



LENGTH = 275'

TOP OF APRON = 37.67
ELEV. AT GRATE = 1137.37
ELEV. AT END OF DRAINAGE DITCH IS 36.0

DEPTH (FT)	CROSS SECTIONAL AREA INLET (FT²)	CROSS SECTIONAL AREA UPSTREAM (FT²)	AVERAGE AREA (FT²)	LENGTH (FT)	VOLUME (FT³)
1.0	11.0	7.34	9.1	275	2521
1.5	16.5	12.87	14.7	275	4040
1.7	18.7	15.07	16.9	275	4640
2.0	22.0	18.37	20.2	275	5550 ← ELEV = 1137.7

V SUMMARY

DETENTION FACILITY '6' HAS SUFFICIENT CAPACITY TO RETAIN THE 50-YR. 1-HR. STORM EVENT AT A PONDING ELEVATION OF APPROXIMATELY 1137.7.

(101) CATCH BASIN LOCATED AT STA. 3080+14

DRAINAGE AREA D-102
PROJECTED MAXIMUM PONDING ELEV. = 1137.7
GRATE ELEV. = 1135.37
PROJECTED MAXIMUM PONDING DEPTH IS: 2.33 FT.

MAXIMUM FLOW FROM DRAINAGE DITCH THROUGH CATCH BASIN 100 (SUMP CONDITION)

TYPE OF CATCH BASIN: TYPE IV – SINGLE
C = 15.30

GRATE: WIDTH = 2.0 FT
LENGTH = 2'-11 3/4" ≈ 3.0'

CLOGGING FACTOR = 2.0 (SUMP CONDITION)

CATCH BASIN HAS A CONCRETE APRON

HEC-12 ANALYSIS

PERIMETER (P): SIDE FACING UPSTREAM
3/2 ≈ CLOGGING FACTOR
1.5 FT.

EQ. 17: GRATE INLET OPERATING AS A WEIR

$Q_i = C_w P d^{1.5}$ $C_w = 3.0$, $P = 1.5$ FT, $d = 2.33$ FT

$Q_i = 3.0(1.5)(2.33)^{1.5}$
$= 16.0$ CFS

EQ. 18: GRATE INLET OPERATING AS A ORIFICE

$Q_i = C_o A (2gd)^{0.5}$ $C_o = 0.67$

$A = 4.84/2 \approx$ CLOGGING FACTOR $= 2.42$ FT^2

$Q_i = 0.67(2.42)[2(32.2)(2.33)]^{0.5}$
$= 19.9$ CFS

∴ WEIR CONDITION CONTROLS
$Q = 12.7$ CFS
(50-YR 1-HR STORM EVENT)

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SUBJECT DETENTION FACILITY 'H' (STA 8083+49 TO STA. 8086+90) - DRAINAGE AREA D-104

(102) DETENTION FACILITY 'H' IS TO FUNCTION AS A RETENTION FACILITY, SIZED FOR THE 50-YR 1-HR STORM EVENT.

I. TOTAL AREA = 40,120 FT²

TYPES OF LAND USES

PAV'T = 26,120 FT²

R.O.W = 14,000 FT²

II. WEIGHTED 'C' COEFFICIENT

PAV'T 'C' = 0.90

R.O.W. 'C' = 0.50

$$C_w = \frac{0.9(26120) + 0.50(14,000)}{40120}$$

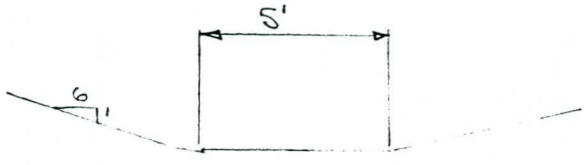
$$= 0.76$$

III. VOLUME OF RUNOFF

STORM EVENT = 50-YR. 1-HR
2.2 INCHES OF PRECIPITATION

$$\text{VOLUME} = \frac{2.2\ \text{IN}}{12\ \text{IN/HR}} \times 0.76 \times 40,120 = 5600\ \text{FT}^3$$

IV AVAILABLE STORAGE AT DRAINAGE DITCH



LENGTH = 280'

ELEV. AT GRATE = 35.77
APRON = 36.10
ELEV AT UPSTREAM END = 36.2

DEPTH (FT)	CROSS SECTIONAL AREA INLET (FT²)	CROSS SECTIONAL AREA UPSTREAM (FT²)	AVERAGE AREA (FT²)	LENGTH (FT)	VOLUME (FT)
1.0	11.0	8.8	9.9	280	2772
1.5	16.5	14.3	15.4	280	4312
1.8	19.8	17.6	18.6	280	5210
1.9	20.9	18.7	19.8	280	5544

ELEV = 1138

V SUMMARY

DETENTION FACILITY 'H' HAS SUFFICIENT STORAGE CAPACITY TO RETAIN THE 50-YR 1-HR STORM STORM EVENT AT A PONDING ELEVATION OF APPROXIMATELY 1138.

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SUBJECT			cm

(102) CATCH BASIN LOCATED AT STA. 8083+49

DRAINAGE AREA D-104
PROJECTED MAXIMUM PONDING ELEV = 1138
GRATE ELEV = 1135.77
PROJECTED MAXIMUM PONDING ELEV DEPTH IS: 2.23 FT.

MAXIMUM FLOW FROM DRAINAGE DITCH THROUGH CATCH BASIN 102 (SUMP CONDITION)

TYPE OF CATCH BASIN: TYPE IV - SINGLE
C-15.30

GRATE: WIDTH = 2.0 FT
LENGTH = 2'-11 3/4" ≈ 3.0'

CLOGGING FACTOR = 2.0 (SUMP CONDITION)

CATCH BASIN HAS A CONCRETE APRON

HEC-12 ANALYSIS

PERIMETER (P): SIDE OF CATCH BASIN FACING UPSTREAM
3/2 = 1.5 FT
└ CLOGGING FACTOR

EQ. 17 - GRATE INLET OPERATING AS A WEIR

$Q_i = C_w P d^{1.5}$ $C_w = 3.0$, $P = 1.5$ FT, $d = 2.23$ FT

$Q_i = 3.0(1.5)(2.23)^{1.5}$
$= 15.0$ CFS

EQ. 18 - GRATE INLET OPERATING AS AN ORIFICE

$Q_1 = C_o A (2gd)^{0.5}$ $C_o = 0.67$

$A = 4.84/2 = 2.42$ FT^2
└ CLOGGING FACTOR

$d = 2.23$ FT

$Q_1 = 0.6(2.42)[2(32.2)(2.23)]^{0.5}$
$= 17.4$ CFS

∴ WEIR CONDITION CONTROLS
$Q = 11.8$ CFS
(50-yr 1-HR STORM EVENT)

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SUBJECT DETENTION FACILITY 'B' - DRAINAGE AREA D-106 cmc

103

I. TOTAL AREA = 247,850 FT²

TYPES OF LAND USES

PAV'T = 81,600 FT²

R.O.W. = 166,250 FT²

II WEIGHTED 'C' COEFFICIENT

PAV'T 'C' = 0.90

R.O.W. 'C' = 0.50

$$C_w = \frac{0.9\,(81600) + 0.5\,(166,250)}{247,850}$$

$$= 0.63$$

III VOLUME OF RUNOFF

STORM EVENT = 50-YR. 1-HR

PRECIPITATION = 2.2 INCHES

$$\text{VOLUME} = \frac{2.2 \text{ INCHES}}{12 \text{ IN/FT}} \times 0.63 \times 247,800 \text{ FT}^2$$

$$= 28,620 \text{ FT}^2$$

IV AVAILABLE STORAGE IN DETENTION FACILITY 'B'

ELEVATION	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	ΣVOLUME (FT³)
1136	16,740				0
		18,000	1.0	18,000	
1137	19,260				18,000
		19,990	0.5	11,990	
1137.6	20,730				29,990

V SUMMARY.

STORAGE PROVIDED AT DETENTION FACILITY 'B' IS PROJECTED AT 29,990 CU. FT. AT A PONDING DEPTH ELEVATION OF 1137.6. WHICH EXCEEDS THE MINIMUM STORAGE OF 28,620 CU. FT. FOR THE 50-YR 1-HR. STORM EVENT.

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SUBJECT			cm

(103) CATCH BASIN LOCATED AT DETENTION FACILITY 'B'

DRAINAGE AREA D-106
PROJECTED MAXIMUM PONDING ELEV = 1137.6
GRATE ELEV = 1135.67
PROJECTED MAXIMUM PONDING DEPTH IS = 1.93 FT.

MAXIMUM FLOW FROM DETENTION FACILITY 'B' THROUGH CATCH BASIN 106 (SUMP CONDITION).

TYPE OF CATCH BASIN: TYPE IV - SINGLE
G-15.30

GRATE: WIDTH = 2.0 FT
LENGTH = 2'-11 3/4" ≈ 3.0'

CLOGGING FACTOR = 2.0 (SUMP CONDITION)

CARH BASIN HAS A CONCRETE APRON.

HEC-12 ANALYSIS

PERIMETER (P) = $\frac{2(2)+3(2)}{2 = \text{CLOGGING FACTOR}}$ = 5 LIN. FT.

EQ 17: GRATE INLET OPERATING AS WEIR

$Q_i = C_w P d^{1.5}$ WHERE: $C_w = 3.0$
$P = 5$ LIN. FT.
$d = 1.92$ FT

$Q_i = 3.0(5)(1.93)^{1.5}$
$= 40.2$ CFS

EQ. 18: GRATE INLET OPERATING AS OFIFICE

$Q_i = C_o A (2gd)^{0.5}$ WHERE: $C_o = 0.67$
$A = 4.84/2 = 2.42$ FT^2
0.66 FACTOR
$d = 1.92$ FT.

$Q_i = 0.6(2.42)\left[2(32.2)(1.93)\right]^{0.5}$
$= 16.2$ CFS

∴ ORIFICE CONDITION CONTROLS: Q = 16.2 CFS
(50-YR 1-HR STORM EVENT)

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SUBJECT DETENTION FACILITY 'C' - DRAINAGE AREA D-108 & D-118 cm

(104)

I. TOTAL AREA = 141,170 FT²

PAV'T. = 32,550 FT²

R.O.W. = 108,620 FT²

CONTRIBUTING AREA FROM DETENTION FACILITY 'D'

PAV'T = 80,290 FT²

ROW = 51,380 FT²

* TOTAL AREA = 272,840 FT²

PAV'T = 112,840 FT²

R.O.W = 160,000 FT²

II WEIGHTED 'C' COEFFICIENT

PAV'T C = 0.90

R.O.W C = 0.50

$$C_w = \frac{0.9(112,840) + 0.5(160,000)}{272,840}$$

$$= 0.67$$

III VOLUME OF RUNOFF

STORM EVENT = 50-YR. 1-HR.

PRECIPITATION = 2.2 INCHES

VOLUME =

$$\frac{2.2 \text{ IN.}}{12 \text{ IN./FT}} \times 0.67 \times 272,840 \text{ FT}^2 = 33,510 \text{ FT}^3$$

IV AVAILABLE STORAGE IN DETENTION FACILITY 'C'

ELEVATION	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	ΣVOLUME (FT³)
1138	34,130				0
		36,150	1.0	36,150	
1139	38,170				36,150

V SUMMARY

DETENTION FACILITY 'C' HAS SUFFICIENT STORAGE CAPACITY TO RETAIN THE 50-YR 1-HR STORM EVENT AT A PONDING ELEVATION OF APPROXIMATELY 1139.0.

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SUBJECT:			cm

(104)

CATCH BASIN LOCATED AT DETENTION FACILITY 'C'

DRAINAGE AREA D-108
PROJECTED MAXIMUM PONDING ELEV = 1139
GRATE ELEV = 1137.67
PROJECTED MAXIMUM PONDING DEPTH IS: 1.33 FT.

MAXIMUM FLOW FROM DETENTION FACILITY 'C' THROUGH CATCH BASIN 108 (SUMP CONDITION):

TYPE OF CATCH BASIN: TYPE 4 - SINGLE
C-15.30.

GRATE: WIDTH = 2.0 FT
LENGTH = 2 - 11 3/4" ≈ 2.989' ≈ 3.0'.

CLOGGING FACTOR = 2.0 (SUMP CONDITION).

CATCH BASIN HAS A CONCRETE APRON.

HEC-12 ANALYSIS

$$\text{PERIMETER } (P) = \frac{2(2) + 3(2)}{2} = 5 \text{ LIN. FT.}$$
2 ← CLOGGING FACTOR

EQ. 17 - GRATE INLETS OPERATING AS WEIRS:

$Q_i = C_w P d^{1.5}$ WHERE $C_w = 3.0$
$P = 5$ LIN. FT.
$d = 1.33$ FT

$Q_i = 3.0(5)(1.33)^{1.5}$
$= 23.0$ CFS

EQ. 18 - GRATE INLETS OPERATING AS A ORIFICE

$Q_i = C_o A (2gd)^{0.5}$ $C_o = 0.67$
A = CLEAR OPENING AREA
$= 4.84 \text{ FT}^2 / 2 = 2.42 \text{ FT}^2$ ← CLOGGING FACTOR
$d = 1.33 \text{ FT}^2$

$Q_i = 0.67(2.42)[2(32.2)(1.33)]^{0.5}$
$= 15.0$ CFS.

∴ ORIFICE CONDITION CONTROLS ⇒ Q = 15.0 CFS
(50-YR 1-HR STORM EVENT)

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SUBJECT DETENTION FACILITY 'D' - DRAINAGE AREA 110 Lem

(105) I. TOTAL AREA = 115,780 FT²

TYPES OF LAND USE

PAV'T = 51,190 FT²

ROW = 64,590 FT²

II WEIGHTED 'C' COEFFICIENT

PAV'T 'C' = 0.90

R.O.W 'C' = 0.50

$$C_w = \frac{0.9(51,190) + 0.5(64,590)}{115,780}$$

$$= 0.68$$

III VOLUME OF RUNOFF

STORM EVENT 50-YR, 1-YR

PRECIPITATION 2.2 INCHES

VOLUME =

$$\frac{2.2 \text{ INCHES}}{12 \text{ IN./FT}} \times 0.68 \times 115780 \text{ FT}^2 = 14,430 \text{ FT}^3$$

III AVAILABLE STORAGE AT

ELEVATION	AREA (FT²)	AVG. AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	Σ VOLUME (FT³)
1138.0	0				0
		11,560	1.0	11,560	
1139.0	23,120				11,560
		23,390	0.2	4,670	
1139.2	23,640				16,230

∴ STORAGE PROVIDED IN DETENTION FACILITY 'D' IS BETWEEN ELEVATIONS 1139.0 AND 1139.0 & EXCEEDS THE 50-YR 1-HR STORM EVENT REQUIRED STORAGE OF 14,430 FT³.

(105) CATCH BASIN LOCATED AT DETENTION BASIN 'D'

DRAINAGE AREA D-110
PROJECTED MAXIMUM PONDING ELEV. = 1139.2
GRATE ELEV. = 1137.67
PROJECTED MAXIMUM PONDING DEPTH = 1.53

MAXIMUM FLOW FROM DETENTION FACILITY 'C' THROUGH CATCH BASIN 110 (SUMP CONDITION)

TYPE OF CATCH BASIN: TYPE 4 - SINGLE
C-15.30

GRATE: WIDTH = 2.0 FT
LENGTH = 2'-11¾" ≈ 3.0'

CLOGGING FACTOR = 2.0 (SUMP CONDITION)

CATCH BASIN HAS A CONCRETE APRON

HEC-12 ANALYSIS

PERIMETER(P): $\frac{2(2)+3(2)}{2}$ = 5 LIN. FT.
2 ← CLOGGING FACTOR

EQ. 17: GRATE INLETS OPERATING AS WEIR

$Q_i = C_w P d^{1.5}$ WHERE C_w = 3.0
P = 5 LIN. FT.
d = 1.53 FT

$Q_i = 3.0(5)(1.53)^{1.5}$
= 28.4 CFS

EQ. 18: GRATE INLETS OPERATING AS AN ORIFICE

$Q_i = C_o A(2gd)^{0.5}$ WHERE: C_o = 0.67
$A = \frac{4.84}{2 - \text{CLOGGING FACTOR}} = 2.42$ FT²
d = 1.53 FT.

$Q_i = 0.67(2.42)[2(32.2)(1.53)]^{0.5}$
= 16.0 CFS

∴ ORIFICE CONDITION CONTROLS ⇒ Q = 16.0 CFS

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SUBJECT: DETENTION FACILITY 'E' - DRAINAGE AREA D-118				

I. TOTAL AREA = 131,670 FT²

TYPES OF LAND USE:

PAVT. = 80,290 FT²

R.O.W. = 51,380 FT²

II WEIGHTED 'C' COEFFICIENT

PAV'T 'C' = 0.90
ROW 'C' = 0.50

$$C_{WEIGHTED} = \frac{0.90(80290) + 0.50(51380)}{131\,670}$$

$$= 0.74$$

III. VOLUME OF RUNOFF

STORM EVENT = 50-YR 1-HR

PRECIPITATION = 2.2 INCHES

$$\text{VOLUME} = \frac{2.2 \text{ INCHES}}{12 \text{ INCHES/FT}} \times 0.74 \times 131{,}670 \text{ FT}^2$$

$$= 17{,}860 \text{ FT}^2$$

IV AVAILABLE STORAGE IN DETENTION FACILITY 'E'

ELEV.	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	Σ VOLUME (FT³)
1140	4380			0	0
		5250	1.0	5250	
1141	6120				5250
		6860	1.0	6860	
1142	7600				12110
		7600	0.2	1520	
1142.2	7600				13630

V SUMMARY

ALTHOUGH DETENTION FACILITY 'E' DOES NOT PROVIDE SUFFICIENT STORAGE CAPACITY FOR THE 50-YR 1-HR STORM EVENT. THE OUTLET PIPE (18"ϕ) GENERATES SUFFICIENT FLOW IN THAT THE STORAGE WILL NOT BE REQUIRE

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SUBJECT			cm

PROJECTED INFLOW INTO DETENTION FACILITY 'E'

AREA = 2.0 ACRES

'C' COEFFICIENT = 0.74

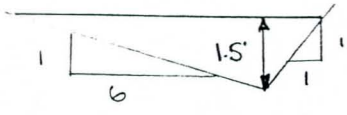
TIME OF CONCENTRATION

A. OVERLAND = 95 LIN. FT. (PAVEMENT)
SLOPE = 2%
KIRPICH METHOD: $T_c = \frac{.04593\,L^{0.77}}{S^{0.385}}$

$T_c = \frac{.04593\,(95)^{0.77}}{2^{0.385}}$

= 1.17 MINUTES

B. CHANNEL TO DETENTION FACILITY 'E'
LENGTH = 1140 L.F.
AVERAGE SLOPE = $\frac{63.8 - 40.6}{1140} = 0.020 \approx 2\%$
ASSUME DEPTH OF FLOW @ 1.5'



AREA = 7.87 FT²
W.P. = 11.2
R = 0.70

USE n = 0.04

MANNING'S EQUATION

$V = \frac{1.49}{0.04}(0.70)^{2/3}(0.02)^{1/2}$

= 4.2 fps

TRAVEL TIME = $\frac{1140 \text{ LIN. FT}}{4.2 \text{ FPS}}$ = 271 SECS = 4.5 MIN

C. TOTAL TRAVEL TIME = 1.17 + 4.5 = 5.7 MIN
USE 10 MIN.

FLOW ENTERING DETENTION FACILITY 'E'

RATIONAL METHOD

A. 10-YR STORM EVENT

T_c = 10 MIN ⇒ I = 4.2 IN/HR

Q = CIA
= 0.74 (4.2) (3.0)
= 9.3 CFS

B. 50-YR STORM EVENT

T_c = 10 MIN ⇒ I = 6.0 IN/HR

Q = CIA
= 0.74 (6.0)(3.0)
= 13.3 CFS

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SUBJECT			cm

EXISTING 18" DIA. OUTLET PIPE FROM DETENTION FACILITY 'E'

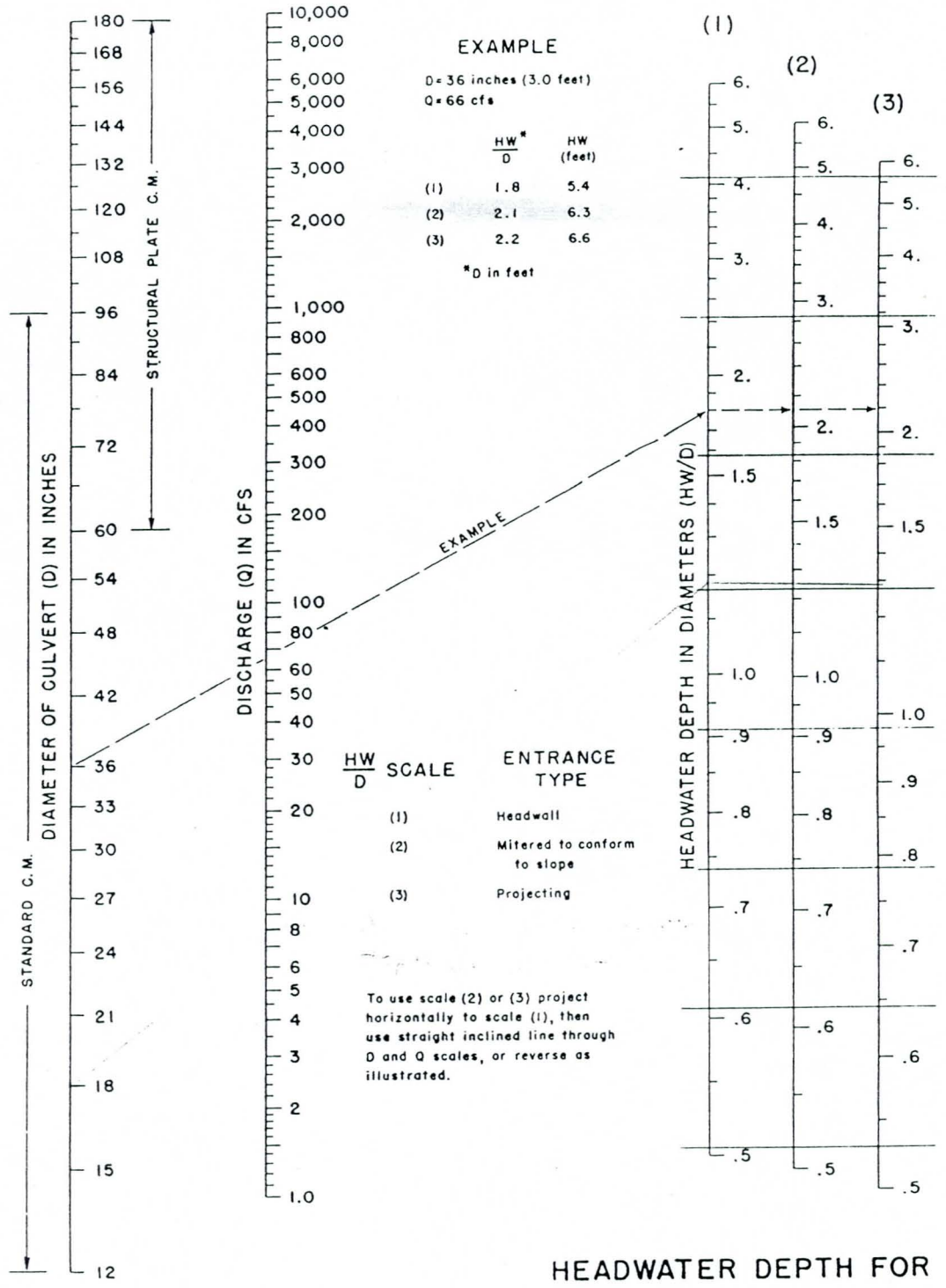
DRAINAGE AREA D-112
PROJECTED PONDING ELEV = 1142.2
OUTLET PIPE ELEV = 1140.26
PROJECTED PONDING DEPTH = 1.94 FT

HEC-5 ANALYSIS

CHART 5
ASSUME INLET CONTROL
ENTRANCE TYPE: GROOVED END PROJECTING
TYPE 3
OUTLET PIPE DIAMETER IS 18"
HEADWATER DEPTH / DIA = 1.29

∴ PROJECTED MAXIMUM FLOW IS ESTIMATED AT 8.0 CFS.
FLOW TRANSFERRED TO DETENTION FACILITY 'C'.

CHART 5



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.'s	PROJECT NO: 5029.05	PAGE: 44
		BY: DMJ / DATE: APR. 88	

SUBJECT: DETENTION FACILITY 'F' - DRAINAGE AREA D-112 — lcm

I. TOTAL AREA = 69,750 FT²

TYPES OF LAND USES:

PAV'T = 13,400 FT²

R.O.W. = 56,350 FT

II. WEIGHTED 'C' COEFFICIENT

PAV'T 'C' = 0.90

R.O.W. 'C' = 0.50

$$C_{weighted} = \frac{0.9(13,400) + 0.5(56,350)}{69,750}$$

$$= 0.58$$

III. VOLUME OF RUNOFF

STORM EVENT = 50-YR. 1-HR.
PRECIPITATION = 2.2 INCHES

$$\text{VOLUME} = \frac{2.2 \text{ INCHES}}{12 \text{ IN/FT.}} \times 0.58 \times 69,750 = 7420 \text{ FT}^2$$

IV AVAILABLE STORAGE IN DETENTION FACILITY 'F'

ELEV.	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	Σ VOLUME (FT³)
1138	660				0
		750	0.4	300	
1138.4	850 (9,200)				300
		10,150	0.6	6,090	
1139.0	11,000				6390
		11,140	0.1	1,110	
1139.1	11,280				7500

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.'s	PROJECT NO: 5029.04	PAGE: 45
		BY: DNJ / DATE: APR.	
SUBJECT:			

EXISTING 12" DIA. OUTLET PIPE FROM DETENTION FACILITY 'F'

DRAINAGE AREA D-
PROJECTED MAXIMUM PONDING ELEV = 1139.1
OUTLET PIPE ELEV.= 1138.26
PROJECTED MAXIMUM PONDING DEPTH IS = 0.84 FT.

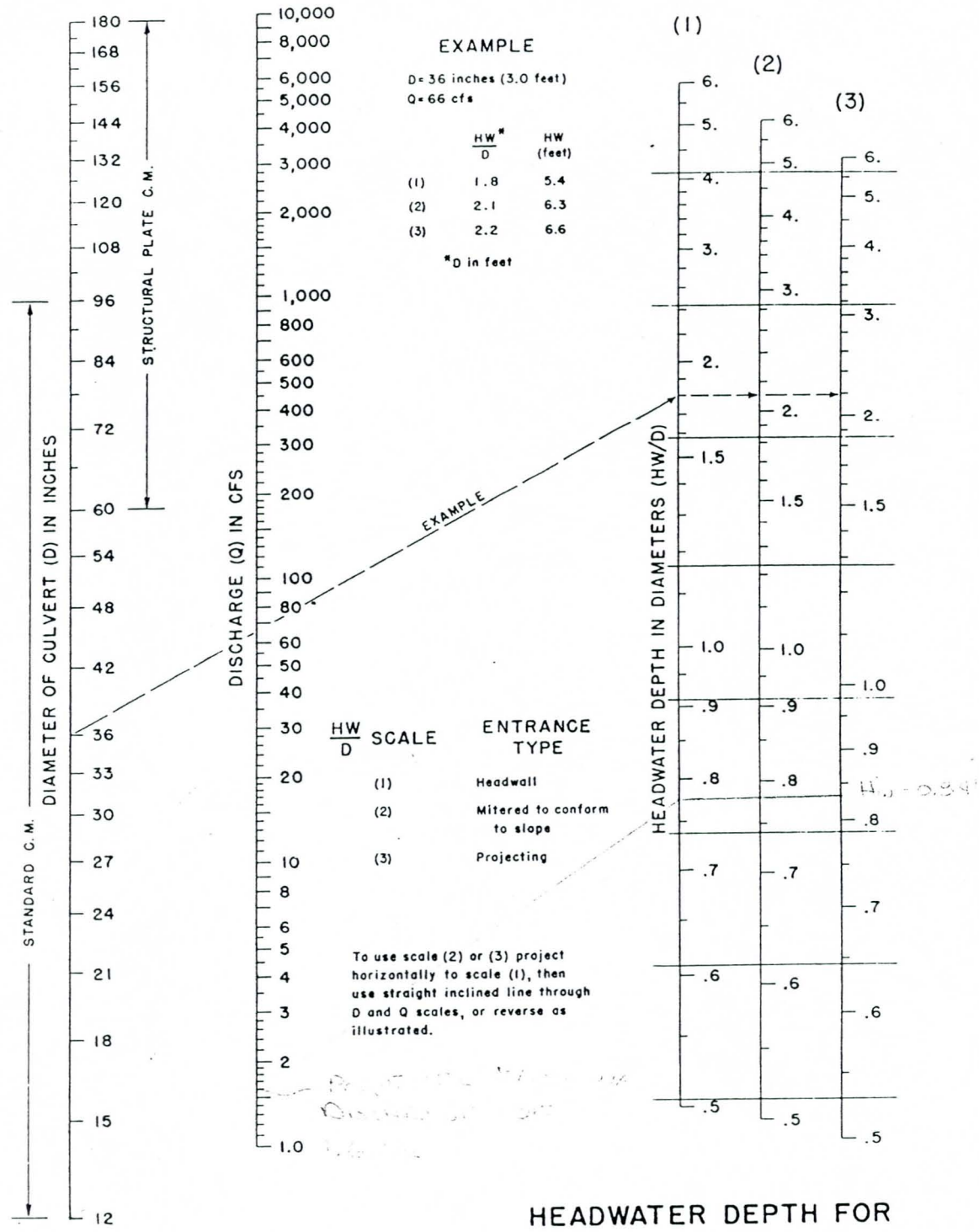
MAXIMUM FLOW FROM DETENTION FACILITY 'F' THROUGH EXISTING 12" DIA. OUTLET PIPE.

HEC-5 ANALYSIS

CHART 5
ASSUME INLET CONTROL
ENTRANCE TYPE: GROOVE END PROJECTING
TYPE 2
OUTLET PIPE DIAMETER IS 12"
HEADWATER DEPTH/DIA. IS 0.84 FT

∴ PROJECTED MAXIMUM FLOW OF 1.6 CFS.
REFER TO NOMOGRAPH ON FOLLOW PAGE

CHART 5



HEADWATER DEPTH FOR C. M. PIPE CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.	PROJECT NO: 5029.04	PAGE: 47
		BY: DJJ / DATE: June 96	
SUBJECT			cont

(107) CATCH BASIN ALONG 48th STREET AT STA. 15+97

DRAINAGE AREA D-114

AREA = 380' X 40' = 15,200 FT² = 0.35 ACRES

PAVEMENT = 15,200 FT²

$C_w = 0.90$

RUNOFF CALCULATIONS:

t_c = 10 MINUTES

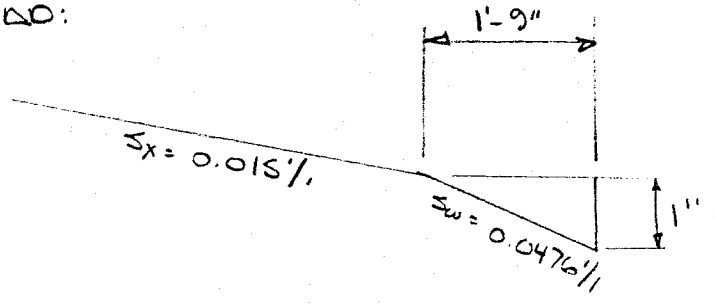
I = 4.20 IN/HR (CATCH BASIN ON GRADE - 10-YR FREQ.)

$Q = CIA$

$= 0.9(4.2)(0.35)$

$= 1.32$ CFS

SPREAD:



HEC-12

CHART 5

$Q_n = 1.32(0.016) = 0.021$

$S_o = 0.0422\%$

W = 1'-9"

$S_x = 0.015\%$

$S_w/S_x = 0.0476/0.015 = 3.17$

$T/W = 4.6 \Rightarrow 4.6(1.75) = 8.05'$

DEPTH:

$8.05 - 1.75 = 6.3$

$6.3\ (0.015) = 0.095$ FT

DEPTH = $1"/12 + 0.095' = 0.18$ FT = d

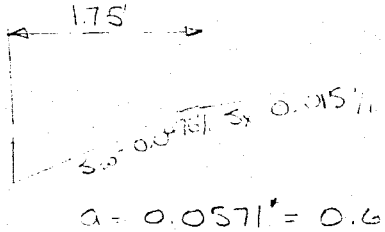
LENGTH OF SLOTTED DRAIN

HEC-12 (eq. 15)

$S_e = S_x + S'_w E_o$

$S'_w = \frac{a}{12w} = \frac{0.6846}{12(1.75)}$

$S'_w = 0.0326$



$a = 0.0571' = 0.6845"$

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.'s	PROJECT NO: 502904	PAGE: 48
		BY: DNJ / DATE: JUNE 98	
SUBJECT			cmc

(107)

E_o = CHART 4 (HEC-12)

$\frac{W}{T} = \frac{1.75}{8.05} = 0.2174$

$S_w/S_x = \frac{0.0476}{0.015} = 3.17$

$E_o = 0.52$

$S_e = 0.015 + 0.0326(0.52)$

$= 0.032$ (EQUIVALENT CROSS SLOPE)

EQ. 16 (HEC-12)

$L_T = KQ^{0.42} S^{0.2} \left(\frac{1}{n S_e}\right)^{0.6}$

$= 0.6 (1.32)^{0.42} (0.0422)^{0.3} \left(\frac{1}{0.016(0.032)}\right)^{0.6}$

$L_T = 24.6$ FT

ACTUAL LENGTH = $1.25 \times L_T$

$= 1.25 \times 24.6$

$= 30.8$ FT

USE 20 FT SLOTTED DRAIN

EFFICIENCY: HEC-12 (CHART 10)

$E = 1 - (1 - L/L_T)^{1.8}$

$= 1 - (1 - 20/30.8)^{1.8}$

$= 0.85$

Q INTERCEPT = $Q_i = 1.32(0.85) = 1.12$ CFS

BYPASS FLOW: $Q_b = Q - Q_i$

$= 1.32 - 1.12$

$= 0.2$ CFS

CATCH BASIN DESIGN

$Q = 0.2$ CFS

SPREAD

HEC-12 CHART 5

$Q_n = 0.2(0.016) = 0.032$

$S_o = 0.0422\%$

$S_x = 0.015\%$

$S_w/S_x = 0.0476/0.015 = 3.17$

$T/W = 0.9 \Rightarrow T = 0.9(1.75') = 1.58$ FT (SPREAD REMAINS IN GUTTER)

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.'s	PROJECT NO: 5029.04	PAGE: 49
		BY: DNJ / DATE: JUNE 88	
SUBJECT			Elm

(107)

DEPTH:

1.58' x 0.0476' = 0.0752 FT

FRONTAL FLOW

100% FRONTAL FLOW → NO SIDE FLOW

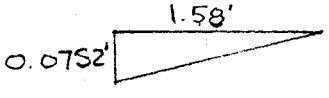
FRONTAL FLOW INTERCEPTION

HEC-12 EQUATION 9 & CHART 7

$R_f = 1 - 0.09(V - V_o)$

USE P-1-7/8 IN CHART 7

VELOCITY IN GUTTER



AREA = 1/2(1.58)(0.0752)
= 0.0594 FT²

$V = \frac{Q}{A} = \frac{0.2}{0.0594} = 3.4$ ft

CHART 7

USE P-1-7/8

L = 3.0'

⇒ $R_f = 1.0$ ⇒ 100% INTERCEPTION

∴ THE CATCH BASIN AND 20 LINEAR FEET OF SLOTTED DRAIN PROVIDES 100% INTERCEPTION.

DMJM	PROJECT: 48th STREET / BROADWAY ROAD T.I.s	PROJECT NO: 5029.04		PAGE: 50
		BY: DNJ	DATE: JUNE 88	
SUBJECT:				

(108) CATCH BASIN AT STA. 14+52.4 BROADWAY ROAD.

CATCH BASIN 108 IS RELOCATED, BUT THE RELOCATION DOES NOT AFFECT FLOW TO THE CATCH BASIN. THEREFORE, CALCULATIONS ARE NOT REQUIRED AND HAVE NOT BEEN INCLUDED.

DMJM	PROJECT: 48 TH STREET/ BROADWAY RD T.I.s	PROJECT NO: 5029.04	PAGE: 51
		BY: DMJ / DATE: JUNE 88	
SUBJECT			lem

(109) CATCH BASIN ALONG 48TH STREET AT STA. 14+74.7

DRAINAGE AREA D-124

AREA: 42'X255' = 10,710 FT² ≈ 0.25 ACRES

PAVEMENT = 10,710 FT

$C_w = 0.90$

RUNOFF CALCULATIONS:

t_c = 10 MINUTES

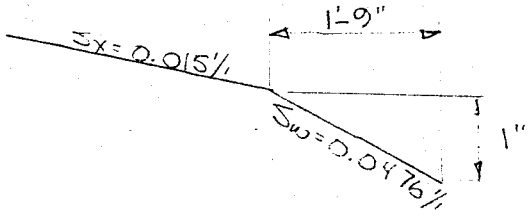
I = 4.20 IN/HR (CATCH BASIN ON GRADE -10-YR FREQ.)

$Q = CIA$

$= 0.9(4.20)(0.25)$

$= 0.94$ CFS

SPREAD:



HEC-12

CHART 5

$Q_K = 0.94(0.016) = 0.0150$

$S_o = 0.0422\%$

$W = 1'-9"$

$S_x = 0.015\%$

$S_w/S_x = \frac{0.0476}{0.015} = 3.17$

$T/W = 3.6 \Rightarrow T = 3.6(1.75) = 6.3$

DEPTH:

$6.3' - 1.75' = 4.55'$

$4.55(0.015) = 0.068'$

DEPTH: $1"/12 + 0.068' = 0.15' = d$

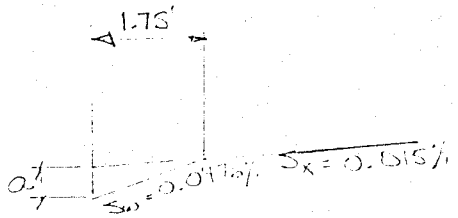
LENGTH OF SLOTTED DRAIN

HEC-12 (EQ. 13)

$S_e = S_x + S_w E_o$

$S_w' = \frac{a}{12W} = \frac{0.6345}{12(1.75)}$

$= 0.0326$



$a = 0.00571' = 0.6345"$

DMJM	PROJECT: 48TH STREET/BROADWAY T.I.'s	PROJECT NO: 5029.04	PAGE 52
		BY: DNJ	DATE: JUNE 93

SUBJECT

(109)

E_o = CHART 4 (HEC-12)

$W/T = 1.75/6.3 = 0.2778$

$S_w/S_x = 0.0476/0.015 = 3.17$

$E_o = 0.68$

$S_e = 0.015 + 0.0326(0.68)$

$= 0.0372$ EQUIVALENT CROSS SLOPE

EQ. 16 (HEC-12)

$$L_T = KQ^{0.42} S^{0.3} \left(\frac{1}{n S_e}\right)^{0.6}$$

$$= 0.6(0.94)^{0.42}(0.0422)^{0.3}\left(\frac{1}{0.016(0.0372)}\right)^{0.6}$$

$= 19.5$ FT

ACTUAL LENGTH

$= L_T \times 1.25$

$= 19.5 \times 1.25$

$= 24.4'$

USE 20 LINEAR FT. OF SLOTTED DRAIN)

EFFICIENCY OF SLOTTED DRAIN): HEC-12 (CHART 10)

$$E = 1 - (1 - L/L_T)^{1.8}$$

$$= 1 - (1 - 20/24.4)^{1.8}$$

$= 0.95.$

$Q_{INT.} = QE = 0.94(0.95) = 0.89$ CFS

$Q_{bypass} = 0.94 - 0.89 = 0.05$ CFS

CATCH BASIN DESIGN

$Q = 0.05$ CFS

SPRELC

HEC-12 (CHART 5)

$Q_n = 0.05(0.016)$

$= 0.0008$

$S_o = 0.0422\%$

$S_x = 0.015\%$

$S_w/S_x = 0.047/0.015 = 3.17$

$T/W = 0.5 \Rightarrow T = 0.5(1.75') = 0.88$ FT (SPRELC CEILING IN GUTTER)

DMJM	PROJECT: 48TH STREET/BROADWAY ROAD T.I.S	PROJECT NO: 5029.04	PAGE: 53
		BY: DNJ / DATE: JUNE 98	

SUBJECT: Cont

(109)

DEPTH

$0.88' \times 0.0476 = 0.0419$ FT

FRONTAL FLOW

100% → NO SIDE FLOW

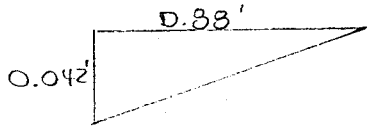
FRONTAL FLOW INTERCEPTION

HEC-12 EQ. 9 & CHART 7

$R_f = 1 - 0.09(V - V_o)$

USE P-1-7/8 IN CHART 7

VELOCITY IN GUTTER



$\text{AREA} = \frac{1}{2}(0.042)(0.88) = 0.0185 \text{ FT}^2$

$V = \frac{Q}{A} = \frac{0.05}{0.0185} = 2.7 \text{ fps}$

CHART 7

USE P-1-7/8

$L = 3'$

$\Rightarrow R_f = 1.0 \Rightarrow R_f = 100\%$ INTERCEPTION

1. THE CATCH BASIN AND 20 LIN. FT. OF SLOTTED DRAIN PROVIDES 100% INTERCEPTION FOR THE 10-YR. STORM EVENT.

DMJM	PROJECT: 48 TH STREET/BROADWAY RC. T.I.s	PROJECT NO: 5029.04	PAGE: 54
		BY: DMJ / DATE: JUNE 88	

SUBJECT

(110) CATCH BASIN AT STA. 14+49.2 BROADWAY ROAD

THE EXISTING CATCH BASIN AT STA. 14+49.2 BROADWAY ROAD IS REMOVED AND REPLACED BY A CATCH BASIN TO FIT THE SPECIAL CURB AND GUTTER. SINCE THE DRAINAGE AREA IS NOT AFFECTED BY THIS CHANGE, CALCULATIONS ARE NOT REQUIRED NOR INCLUDED.

DMJM	PROJECT: 48th STREET/BROADWAY ROAD T.I.'s	PROJECT NO: 5029.04	PAGE: 55
		BY: DNJ / DATE: JUNE 88	

SUBJECT RETENTION FACILITY 'A'

RETENTION FACILITY 'A'

DRAINAGE AREA D-120

I. TOTAL AREA = 16,040 FT²
PAV'T = 5460 FT²
R.O.W. (INFIELD AREA) = 10,580 FT²

II WEIGHT 'C' COEFFICIENT
PAV'T. 'C' = 0.9
ROW 'C' = 0.5

$$C_w = \frac{0.9(5460) + 0.5(10,580)}{16,040}$$
$$= 0.64$$

III VOLUME OF RUNOFF

STORM EVENT = 50-YR 1-HR
PRECIPITATION = 2.2 INCHES

VOLUME = $\frac{2.2 \text{ INCH}}{12 \text{ IN/FT}} \times 0.63 \times 16,040 \text{ FT}^2 = 1,850 \text{ FT}^3$

IV AVAILABLE STORAGE A RETENTION FACILITY 'A'

ELEVATION	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	Σ VOLUME (FT²)
1142.0	2750				0
		3430	1.0	3480	
1143.0	4220				3480

V SUMMARY

RETENTION FACILITY 'A' IS PROJECTED TO PROVIDE 3480 FT³ OF STORAGE AT A HOLDING ELEVATION OF 1143 WHICH EXCEED THE REQUIRED 50-YR 1-HR STORM EVENT STORAGE REQUIREMENT OF 1,850 FT³. ADEQUATE FREEBOARD BELOW ROADWAY IS PROVIDED.

DMJM	PROJECT: 48th STREET/BROADWAY ROAD	PROJECT NO: 5029.04	PAGE: 56
		BY: DMJ / DATE: June 86	

SUBJECT: RETENTION FACILITIES 'B' — lcm

RETENTION FACILITY 'B'
DRAINAGE AREAS D-122 & D-124

I. TOTAL AREA = 44,000 FT²
PAV'T. = 10,710 FT²
R.O.W. = 33,290 FT²

II. WEIGHTED 'C' COEFFICIENT

PAV'T 'C' = 0.9
R.O.W. 'C' = 0.5

$$C_w = \frac{0.5(33{,}290) + 0.9(10{,}710)}{44000} = 0.60$$

III VOLUME OF RUNOFF

STORM EVENT = 50-YR 1-HR.
PRECIPITATION = 2.2 INCHES

VOLUME $\frac{2.2 \text{ IN.}}{12 \text{ IN./FT.}} \times 0.60 \times 44{,}000 \text{ FT}^2 = 4{,}840 \text{ FT}^3$

IV AVAILABLE STORAGE WITHIN DRAINAGE AREA D-122

ELEVATION	AREA (FT²)	AVERAGE AREA (FT²)	DEPTH (FT)	VOLUME (FT³)	Σ VOLUME (FT³)
1139	2830				0
		4010	1.0	4010	
1140	5190				4010
		5250	0.2	1050	
1140.2	5310				5060

V SUMMARY

RETENTION FACILITY 'B' HAS ADEQUATE STORAGE (5060 FT³) AT ELEVATION 1140.2 STORAGE TO RETAIN THE 50-YR 1-HR STORM EVENT.

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway 48th Street/Broadway Rd. T.I.'s
Location PHOENIX
Project No. ACI-10-3(198)

DESIGN DATA
Frequency 10 years
P_6= in. P_{24}= in. P_1= in.
Outlet TW Elev. =
Pipe n = 0.012

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line			
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design (MIN)	in./hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft./ft.	Diam. in.	Q_{full} cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation Inlet	Elevation Outlet
100	100 to Exist. MH	D-100 D-119	3.08	3.08	10.0		10.0	4.20	12.9*	1132.62	1133.67	105	0.01	18	11.4					
101	101 to Exist. MH	D-102	0.68	0.68	10.0		10.0	4.20	2.9*	1132.62	1133.37	27.25	0.025	18	18.9					
102	102 to MH 104	D-104	0.70	0.70	10.0		10.0	4.20	2.9*	1133.29	1133.77	24	0.02	18	16.2					
103	103 to Exist. M.H.	D-106	3.58	3.58	10.0		10.0	4.20	15.0*	1133.48	1133.67	62.75	0.003	18	6.3					
104	104 to MH 108	D-108 D-118	2.17	2.17	10.0		10.0	4.20	9.1*	1135.17	1135.67	100.0	0.005	18	8.1					
105	105 to MH 108	D-107	2.66	2.66	10.0		10.0	4.2	11.2*	1135.09	1135.67	210	0.003	18	6.3					
107	107 to Outlet	D-114	0.31	0.31	10.0		10.0	4.2	1.3	1140.00	1149.83	140	0.07	18	16.1					
109	109 to Outlet	D-109	0.22	0.22	10.0		10.0	4.2	1.0	1140.5	1149.7	45.	0.023	18	34.9					

* MILDEST SLOPE

Computed by: DNJ Checked by: [illegible] Date: June 88

F-45

57

* FLOWS ARE NOT REPRESENTATIVE BECAUSE OF PEAK FLOW ATTENTUATION IN THE DETENTION FACILITIES. Outlet pipes are purposely undersized to provide additional peak flow attentuation.

ARIZONA HIGHWAY DEPARTMENT
STRUCTURES SECTION
HYDRAULICS BRANCH

STORM SEWER SYSTEM DESIGN: SEWER CALCULATION SHEET

LOCATION DATA
Highway 48th Street/Broadway Road T.I.'s
Location PHOENIX
Project No. ACT-10-3(198)

DESIGN DATA
Frequency 50 years
P_6= ____ in. P_{24}= ____ in. P_1= ____ in.
Outlet TW Elev. = ____
Pipe n = 0.012

SEWER CALCULATIONS

Location		Drainage Area			Time of Flow, Min.			I	Q	Sewer Profile				Sewer Design			Hydraulic Grade Line			
Line No.	From - To	No.	Increm. CA	Σ CA	Inlet	Sewer	Design (MIN)	in./hr	cfs	Crown Elevation Inlet	Crown Elevation Outlet	Length ft.	Slope ft./ft.	Diam. in.	Q full cfs	V f.p.s.	S_f ft./ft.	h_L ft.	Elevation Inlet	Elevation Outlet
100	100 to EXIST. MH	D-100 D-114	3.08	3.08	10.0		10.0	6.00	18.5	*1132.62	1133.67	105	0.01	18	11.4					
101	101 to Exist M.H.	D-102	0.68	0.68	10.0		10.0	6.00	4.1	*1132.62	1133.37	27.25	0.0275	18	18.9					
102	102 to MH 104	D-103	0.70	0.70	10.0		10.0	6.00	4.2	*1133.29	1133.77	24	0.02	18	16.2					
103	103 to Exist. MH	D-106	3.58	3.58	10.0		10.0	6.00	21.5	*1133.48	1133.67	62.75	0.003	18	6.3					
104	104 to MH 108	D-108 D-118	2.17	2.17	10.0		10.0	6.00	13.0	*1135.17	1135.67	100.0	0.005	18	8.1					
105	105 to MH 108	D-110	2.66	2.66	10.0		10.0	6.00	15.9	*1135.04	1135.67	210	0.003	18	6.3					
107	107 to outlet	D-114	0.31	0.31	10.00		10.0	6.00	1.9	1140.0	1149.83	140	0.02 ↑ mildest slope	18	16.1					
109	109 to outlet	D-109	0.22	0.22	10.00		10.00	6.00	1.3	1140.50	1144.70	45	0.093	18	34.9					

Computed by: DNJ Checked by: [illegible] Date: June 88

F-45

58

* FLOWS ARE NOT REPRESENTATIVE BECAUSE OF PEAK FLOW ATTENUATION IN THE DETENTION FACILITIES. Outlet pipes are purposely undersized to provide additional peak flow attenuation.

APPENDIX D

- SUPPORTING DOCUMENTATION

TELECON CONFIRMATION

5029.04

TO:	Dennis Jermeland	BY:	Rodney D. Smith RDS
FIRM:	DMJM	DATE:	April 4, 1988
PHONE:	(602) 264-1397	TIME:	
CONTRACT NO.	84-41	PROJECT NO.:	IR-10-3(222)PE,(198)C

SUBJECT: 48TH ST & BROADWAY TI'S

On-site retention is temporary.

DMJM can disregard 0.5' freeboard below pavement structural section.

Storm duration: 1 hr 50 yrs.

Keep ponding 1' below shoulder elevation. The adjacant pavement will be removed in Phase II construction.

RECEIVED

APR 12 1988

D M J M

DISTRIBUTION: ERS, RJT, RDS
FILE: 2B, 84-41

RDS/lcc
4/11/88

Sverdrup

ASMC

ARIZONA STATEWIDE MANAGEMENT CONSULTANT

APPENDIX E

- DRAINAGE AREAS PLAN
- STORM DRAINAGE SYSTEM