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INTRODUCTION

Upon recommendation from Vaughn Hansen Associates [V.H.A.] a design storm of 25 years frequency, 6 hour duration was used to compute peak flows. This is the event used by V.H.A. in designing spillways for sediment ponds for UNC Plateau Mining [UNC P.M.C. - Runoff Control Plan, 1979]. It is felt that using this event for design purposes is adequate because most of the culverts are located in small canyons and drains with shallow cover and should they be washed out, they can easily be replaced. In the case of the 60"CMP culvert in drainage area G [refer to Exhibit G-3] the culvert is in a deep canyon with deep cover and therefore it was designed with a large safety factor to assure that it would not impound runoff.

RESULTS AND DISCUSSION

Drainage Area A

This area comprises the South half of Lion Deck and extends to the ridges on the North and South and to Areas D and H-2 on the East. Runoff from the entire Lion Deck located in Area A and in Area H-1 will be diverted thru a sediment pond located within this area. The runoff from the North half of Lion Deck was not added to the runoff from this area. Since the entire runoff from Lion Deck will be passing through a sediment pond, this runoff will be eliminated from the peak runoff flow as calculated until after the event is over and then released some 14 days later [V.H.A. Runoff Control Plan, 1979.]

The runoff from this area passes through an existing 30 inch CMP culver under the new road. Runoff was calculated to be 25.6 C.F.S. 42" CMP @ 25.6 C.F.S.= Headwater Depth of 0.64 diameter. From this 25.6 C.F.S. can be subtracted the runoff from the entire Lion Deck.

Drainage Area B

This area comprises the hillside above the North leg of the New Road. Runoff from this area will collect in a ditch on the up-hill side of the road and flow Eastward to the Switch Back at the East end. The ditch was constructed along with the road and is complete at this time. It is 1.5 feet deep with side slopes of 1.5:1 on the road side and 1:1 on the hill side. Runoff was calculated to be 8.3 C.F.S.

Drainage Area C

This area includes a small area between legs of the new road on the North side of the Main Canyon. Runoff from the area will collect in a ditch on the up-hill side of the road and run to an 18 inch diameter CMP under the new road. Runoff from this area was calculated to be 1.4 C.F.S. 18" CMP @ 1.4 C.F.S. = Headwater Depth of .49 diameter.

Drainage Area D

This area includes a small canyon above the road on the South side of the Main Canyon. Runoff will collect in a ditch on the up-hill side of the road and run into an existing 30 inch CMP under the road. Runoff was calculated to be 5.9 C.F.S. 30" CMP @ 5.9 C.F.S. = Headwater Depth of .43 diameter.

Drainage Area E

This area includes a small drainage above the road on the South side of the Main Canyon. Runoff will collect in a ditch on the up-hill side of the road and run into an existing 30 inch CMP under the road. Runoff was calculated to be 7.4 C.F.S. 30" CMP @ 7.4 C.F.S. = Headwater Depth of .52 diameter.

Drainage Area F

This area includes a small canyon above the road on the South side of the Main Canyon. Runoff will collect in a ditch on the up-hill side of the road and flow into an existing 30 inch CMP under the road. Runoff was calculated to be 2.8 C.F.S. 30" CMP @ 2.8 C.F.S. = Headwater Depth of .63 diameter.

Drainage Area G

This area includes the left fork of the Main Canyon which includes the No. 1 and No. 2 Mine Portal areas and the Raw Coal Truck Loading area. [Refer to Exhibit G-3] Runoff from these three areas just mentioned will pass through a sediment pond [V.H.A. Runoff Control Plan, 1979], and will be eliminated from the total runoff peak flow as calculated. After the event is over the runoff detained in the sediment pond will be released [14 days later]. The runoff will flow down the main drainage channel and into a ditch on the up-hill side of the road and into an existing 60 inch CMP under the road. Runoff was calculated to be 37.4 C.F.S. 60" CMP @ 37.4 C.F.S. = Headwater Depth .48 diameter. Because this culvert is in a deep fill it was oversized by more than three times the actual capacity required to assure that it would not impound water. The existing 60 inch CMP will pass a peak flow of 125 C.F.S.

Drainage Area H-1

This area includes the hillside above the North half of Lion Deck and a small portion of the New Road. A diversion ditch has been constructed above Lion Deck to divert runoff from the hillside above the Deck away from the Deck. This runoff according to V.H.A. Runoff Control Plan, 1979, is to be diverted into the main drainage and would pass through the 42 inch CMP for Drainage Area A; this 42 inch CMP has the capacity to accept this additional flow - Area A runoff 25.6 C.F.S. plus Area H-1 runoff [65% only would be diverted to the 42 inch CMP] 2.73 equals 28.33 C.F.S. 42 inch CMP @ 28.33 C.F.S. = .69 diameter. An altervative to doing this would be to divert the entire runoff through the proposed 18 inch CMP for Area H-1 located at the East end of the drainage area. Runoff was calculated to be 4.2 C.F.S. 18 inch CMP @ 4.2 C.F.S. = Headwater Depth .85 diameter.

Drainage Area H-2

This area includes a portion of the hillside between the upper leg and the middle leg of the New Road. Runoff will pass through a 24 inch CMP under the road at the West end of the area. Runoff was calculated to be 6.0 C.F.S. The entire runoff from Area H-1 was added to the runoff from this area in sizing the culvert for this area. Runoff Area H-2 = 6.0 C.F.S. plus runoff Area H-1 - 4.2 C.F.S. equals 10.2 C.F.S. 24 inch CMP @ 10.2 C.F.S. = Headwater Depth .85 diameter.

METHODS

The runoff volume resulting from a particular rainfall depth was determined using the runoff curve number technique, as defined by the U.S. Soil Conservation Service [1972]. According to the curve number methodology, the algebraic and hydrologic relations between storm rainfall, soil moisture storage, and runoff can be expressed by the equations

$$Q = \frac{[P-0.2S]^2}{P+0.8S} \qquad [1]$$

and

$$CN = \frac{1000}{10+S} \qquad [2]$$

where Q is the direct runoff volume, in inches; P is the storm rainfall depth, in inches; S is a watershed storage factor, in inches, defined as the maximum possible difference between P and Q; and CN is a dimensionless expression of S referred to as the curve number. Curve number values were chosen using information supplied by the U.S. Soil Conservation Service [1972], Hawkins [1973], and personal hydrologic judgement following field observations. Values of P were obtained for selected durations and return periods from Miller et. al. [1973].

Estimates of the peak discharge to be expected from various precipitation events were made using the dimensionless hydrograph method illustrated in Figure 1 which was developed by the U.S. Soil Conservation Service [1972]. In this figure, D is the duration of excess rainfall; T_c is the time of conventration, T_p is the time of peak; T_r is the time of recession; T_b is the time of base, with all time units in hours; and q_p is the peak discharge, in cubic feet per second. Five separate hydrograph families have been developed by the U.S. Soil Conservation Service [1972], with the selection of the family of curves to be used based on the curve number and rainfall depth as given in Figure 2. According to the dimensionless hydrograph method,

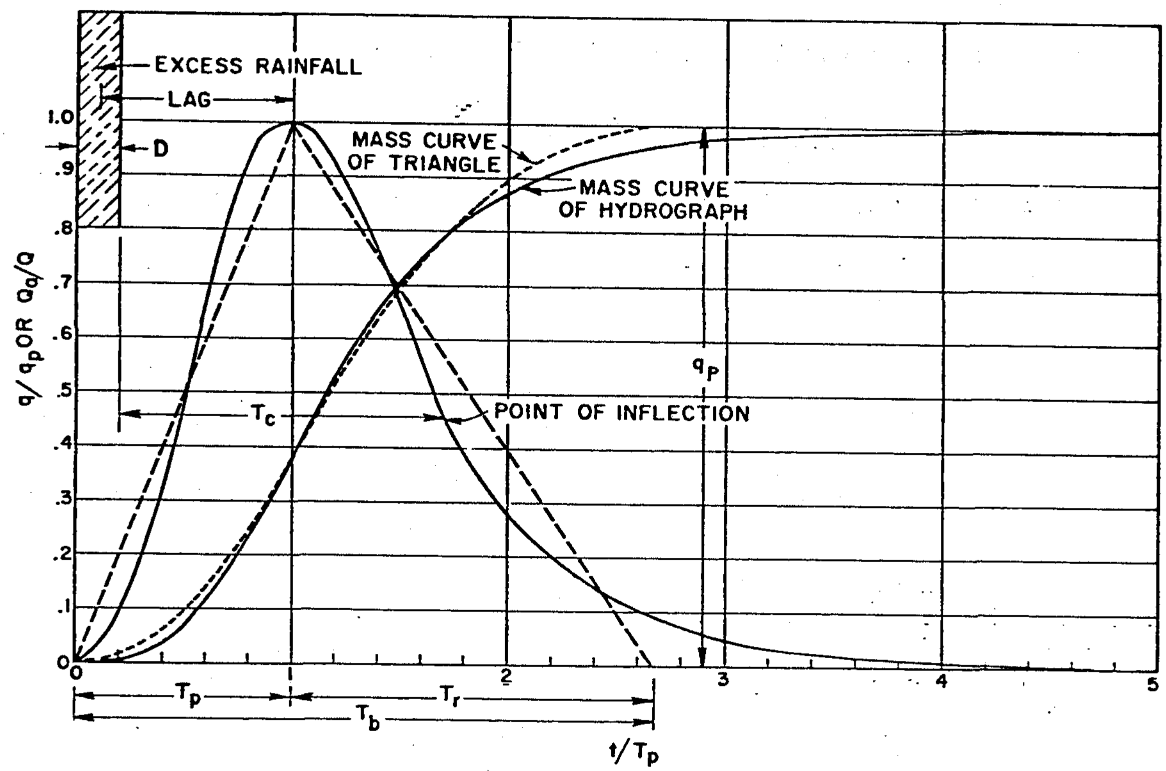

Figure 1. Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph [from U.S. Soil Conservation Service, 1972].

one discharge and two time constants are determined from empirical equations and storm distributions. The constants are multiplied by increments of discharge and time from the dimensionless hydrograph to obtain the plotting points of the synthetic hydrograph. Hydrographs were not plotted for this report because only peak flows are needed to design culver sizes.

The discharge constant used in the dimensionless hydrograph method is determined according to the equation

$$q = \frac{484 \, A \, Q}{T_p} \qquad [3]$$

where q is the peak discharge constant, in cubic feet per second; A is the drainage area, in square miles; Q is the runoff volume, in inches [as determined by equation 1]; T_p is the time elapsed from the beginning of runoff to the hydrograph peak, in hours; and 484 is a constant. T_p is assumed to be a function of watershed lag, which is determined according to the equation

$$L = \frac{[\ell^{0.8}] \, [S + 1]^{0.7}}{1900 \, Y^{0.5}} \qquad [4]$$

where L is the watershed lag, in hours; ℓ is the hydraulic length, or the length of the mainstream to the farthest divide, in feet; S is as previously defined; and Y is the average watershed slope, in percent. Values of Y were obtained by measuring the lengths [in feet] of selected contour lines within the drainage boundary, multiplying by the selected contour interval [in feet], dividing by the drainage area [in square feet], and multiplying by 100. The hydraulic length was taken from an appropriate topographic map while S was determined from equation 2 once the runoff curve number had been estimated.

According to the U.S. Soil Conservation Service [1972], the watershed lag is equal to 0.6 T_c and the time to peak is equal to 0.7 T_c. Combining these two expressions it can be seen that

$$T_p = 1.17L \qquad [5]$$

where both variables are as previously defined.

Following the determination of given peak discharge, design sizes for culverts used for runoff diversions and conveyance were determined using methods derived by the U.S. Bureau of Public Roads as presented by the U.S. Soil Conservation Service [1972] and illustrated in Figure 3. Inlet control was assumed in all cases.

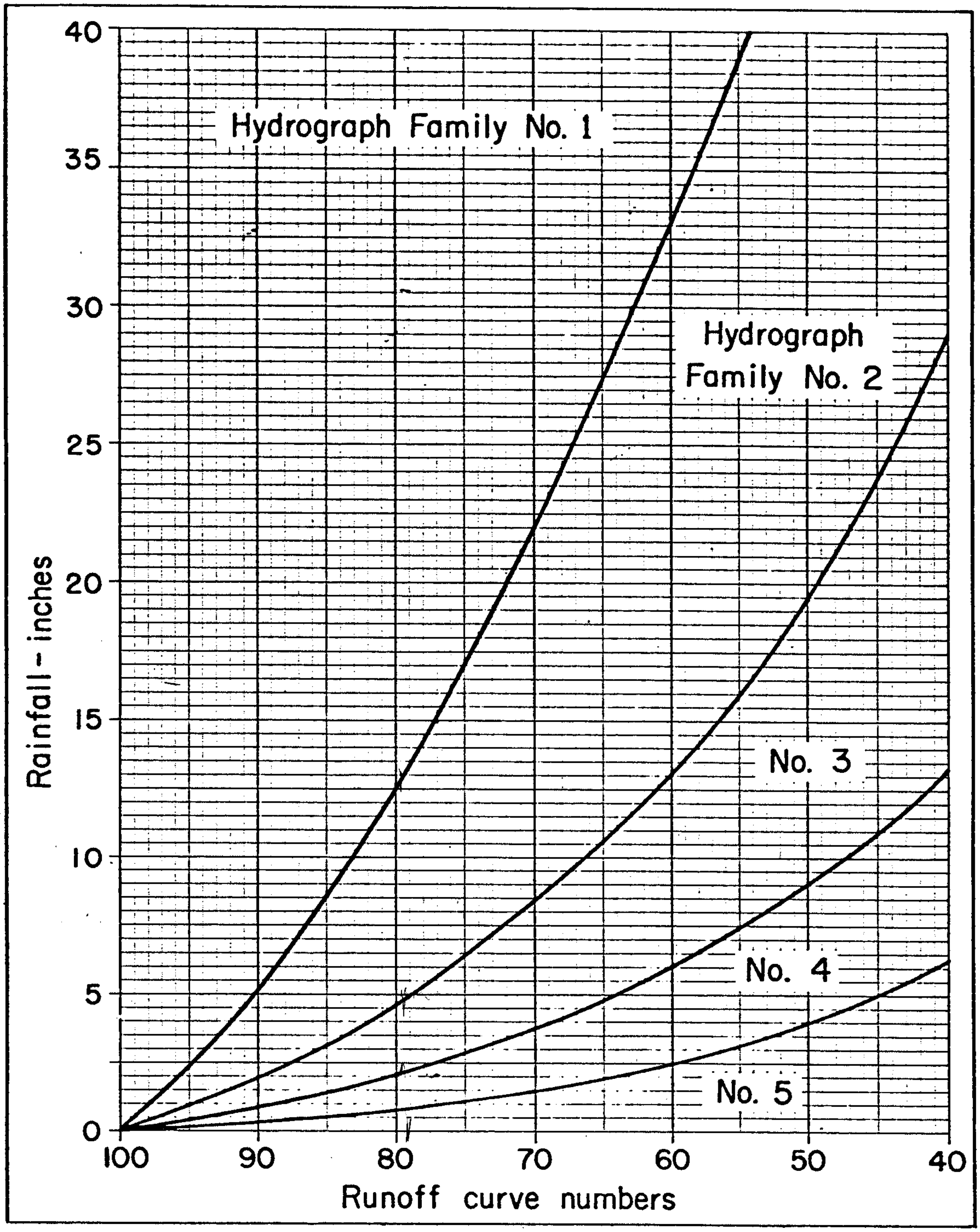

Figure 2. Chart for selecting a hydrograph family for a given rainfall and runoff curve number [from U.S. Soil Conservation Service, 1972].

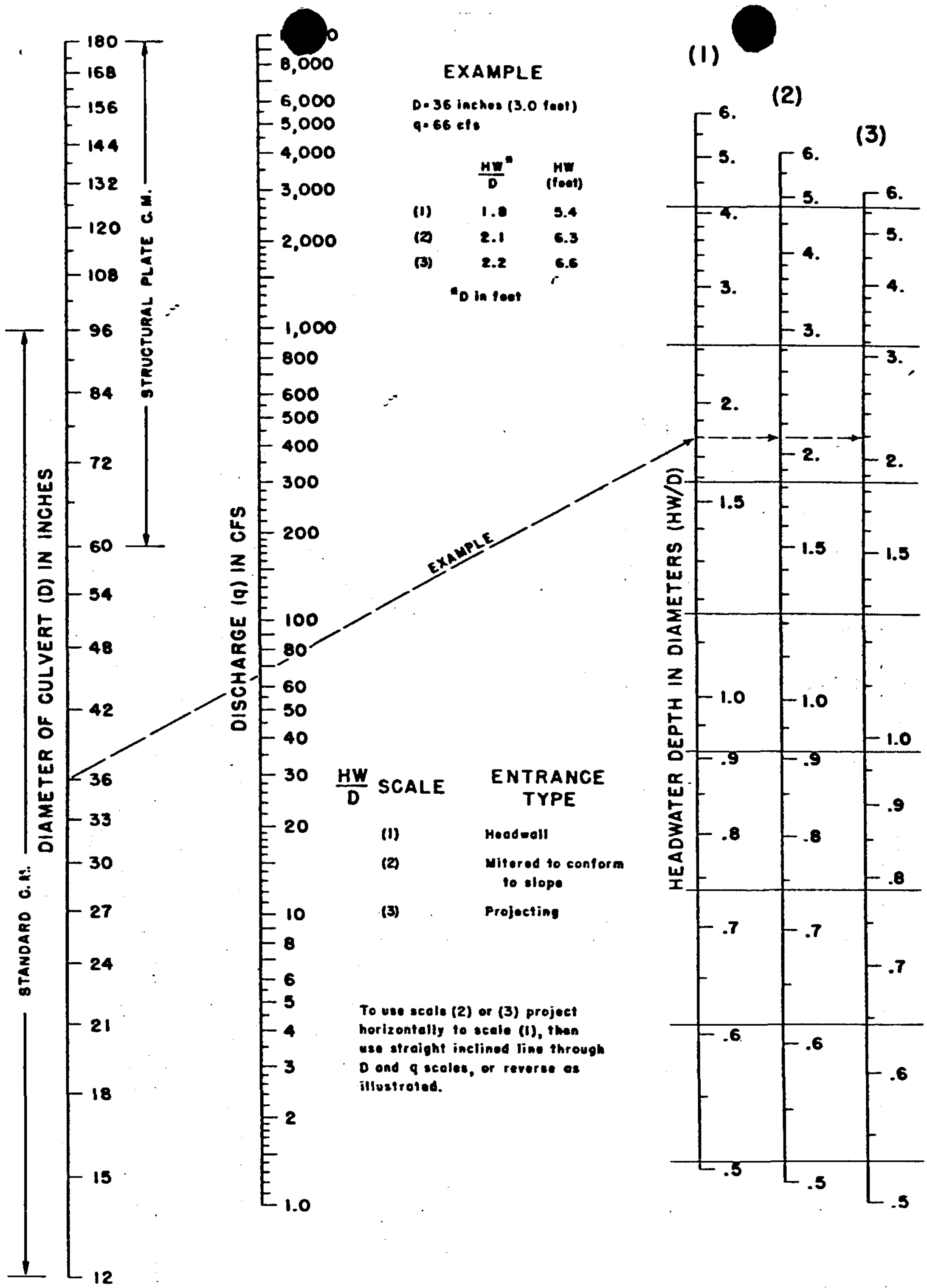

Figure 3. Headwater depth for C.M. pipe culverts with inlet control.

PEAK FLOW CALCULATIONS

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA A - EXISTING 42" CULVERT - CMP

Design Storm 25yr 6hr

Drainage Area .17 sq.mi. Runoff Condition No. II

Lag .0905 Hrs. T_c .15 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .105 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 36.95

(8) Revised T_o/T_p Ratio 36

(9) Revised T_p .1078 Hrs. $\frac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 763.26 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 381.63 C.F.S. $Q \times q_p$

(12) q_c/q_p from Hydrograph Family .0672

(13) Peak Flow 25.65 C.F.S. $q_c/q_p \times Qq_p$

W.C. 3050

$$\text{LAG} = \frac{3050^{0.8}\left(\frac{1000}{77} - 10 + 1\right)^{0.7}}{1900\ (88.10)^{0.5}} = \frac{1613.96}{17833.70} = .0905$$

$$T_c = .0905/.6 = .15$$

HEADWATER DEPTH — 42" CMP @ 26 CFS = 0.64 DIA.

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA B - NO CULVERT = DITCH FLOW

Design Storm 25 yr 6 hr

Drainage Area .06 sq.mi. Runoff Condition No. II

Lag .12 Hrs. T_c .21 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve ——

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. —— in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .15 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 26.39

(8) Revised T_o/T_p Ratio 25

(9) Revised T_p .16 Hrs. $\frac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 181.5 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 90.75 C.F.S. Qxq_p

(12) q_c/q_p from Hydrograph Family .092

(13) Peak Flow 8.35 C.F.S. $q_c/q_p \times Qq_p$

W.C. 3900'

2.63

$$LAG = \frac{3900^{0.8}\left(\frac{1000}{77}-10+1\right)^{0.7}}{1900\ (68.55)^{0.5}} = \frac{1964.74}{15731.04} = .12$$

$$TC = {.12}/{.6} = .21$$

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA C - CULVERT REQUIRED

Design Storm 25 yr 6hr

Drainage Area .01 sq.mi. Runoff Condition No. II

Lag .03 Hrs. T_c .06 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .04 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 97

(8) Revised T_o/T_p Ratio 50

(9) Revised T_p .08 Hrs. $\frac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 60.50 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 30.25 C.F.S. $Q \times q_p$

(12) q_c/q_p from Hydrograph Family .0464

(13) Peak Flow 1.40 C.F.S. $q_c/q_p \times Qq_p$

wc. 640'

LAG =

$$\frac{640^{0.8}\left(\frac{1000}{77}-10.41\right)^{0.7}}{1900\left(51.02\right)^{0.5}} = \frac{462.27}{13571.37} = .03$$

$$Tc = .03/.6 = .06$$

MINIMUM CULVERT SIZE REQ'D. 18" @ 2 C.F.S = 0.49 DIA HEADWATER DEPTH

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA D - EXISTING 30" CMP CULV.

Design Storm 25 yr 6 hr

Drainage Area .04 sq.mi. Runoff Condition No. II

Lag .08 Hrs. T_c .13 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .09 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 43.11

(8) Revised T_o/T_p Ratio 36

(9) Revised T_p .11 Hrs. $\frac{T_o}{(T_o/T_p)\text{ rev.}}$

(10) q_p 176.0 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 88 C.F.S. Qxq_p

(12) q_c/q_p from Hydrograph Family .0672

(13) Peak Flow 5.91 C.F.S. $q_c/q_p \times Qq_p$

WC. 2100'

$$LAG = \frac{2100^{0.8}\left(\frac{1000}{77}-10+1\right)^{0.7}}{1900\ (67.71)^{0.5}} = \frac{1196}{15634.36} = .08$$

$$Tc = \frac{.08}{.6} = .13$$

HEADWATER DEPTH - 30" CMP @ 6 CFS. = .43 DIA.

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA E - EXISTING 30" CMP CULV.

Design Storm 25yr 6hr

Drainage Area .05 sq.mi. Runoff Condition No. II

Lag .08 Hrs. T_c .13 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .09 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 43.11

(8) Revised T_o/T_p Ratio 36

(9) Revised T_p .11 Hrs. $\frac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 220 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 110.0 C.F.S. $Q \times q_p$

(12) q_c/q_p from Hydrograph Family .0672

(13) Peak Flow 7.39 C.F.S. $q_c/q_p \times Qq_p$

WC. 2100

$$LAG = \frac{2100^{0.8}\left(\frac{1000}{77}-10+1\right)^{0.7}}{1900\ (68.72)^{0.5}} = \frac{1196}{15750.53} = .08$$

$$T_c = .08/.6 = .13$$

HEADWATER DEPTH - 30" CMP @ 8 CFS. = .52 DIA.

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA F - EXISTING 30" CMP CULV.

Design Storm 25 yr 6hr

Drainage Area .02 sq.mi. Runoff Condition No. II

Lag .07 Hrs. T_c .11 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .08 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 50.66

(8) Revised T_o/T_p Ratio 50

(9) Revised T_p .08 Hrs. $\frac{T_o}{(T_o/T_p)\text{ rev.}}$

(10) q_p 121.0 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 60.5 C.F.S. Qxq_p

(12) q_c/q_p from Hydrograph Family .0464

(13) Peak Flow 2.81 C.F.S. $q_c/q_p \times Qq_p$

WC. 1500

$$\text{LAG} = \frac{1500^{0.8}\left(\frac{1000}{77} - 10 + 1\right)^{0.7}}{1900\left(53.67\right)^{0.5}} = \frac{913.75}{13919.36} = .07$$

$$T_c = .07/.6 = .11$$

HEADWATER DEPTH - 30" CMP @ 3 CFS = .63 DIA.

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA G - EXISTING 60" CMC CULV.

Design Storm 25 yr 6 hr

Drainage Area .29 sq.mi. 117 Runoff Condition No. II

Lag .17 Hrs. T_c .28 (high) Hrs.

Runoff Curve No. 77 ? Storm Distrib. Curve A

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in. ?

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .20 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 19.40

(8) Revised T_o/T_p Ratio 16

(9) Revised T_p .24 Hrs. $\dfrac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 584.83 C.F.S. $q_p = \dfrac{484A}{\text{Rev. } T_p}$

(11) Qq_p 292.42 C.F.S. Qxq_p

(12) q_c/q_p from Hydrograph Family .128

(13) Peak Flow 37.43 C.F.S. $q_c/q_p \times Qq_p$

WC. 5000

$$LAG = \frac{5000^{0.8}\left(\frac{1000}{77} - 10 + 1\right)^{0.7}}{1900\ (56.11)^{0.5}} = \frac{2394.04}{14232.26} = .17$$

$$T_c = \frac{.17}{.6} = .28$$

HEADWATER DEPTH - 60" @ 38 CFS = .48 DIA.

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA H1 - CULVERT REQUIRED

Design Storm 25 yr 6 hr

Drainage Area .03 sq.mi. 20.1 Acres Runoff Condition No. II

Lag .07 Hrs. T_c .11 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .08 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 50.39

(8) Revised T_o/T_p Ratio 50

(9) Revised T_p .08 Hrs. $\frac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 181.5 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 90.75 C.F.S. Qxq_p

(12) q_c/q_p from Hydrograph Family .0464

(13) Peak Flow 4.21 C.F.S. $q_c/q_p \times Qq_p$

W.C. 1860'

$$LAG = \frac{1860^{0.8}\left(\frac{1000}{77}-10+1\right)^{0.7}}{1900\,(69.67)^{0.5}} = \frac{1085.34}{15859.03} = .07$$

$$Tc = \frac{.07}{.6} = .11$$

HEADWATER DEPTH 18" CMP @ 5 CFS. = .85 DIA

18" CMP REQ'D.

PEAK FLOW COMPUTATION FORM

Location DRAINAGE AREA H2 - ALSO PASSES DRAINAGE FROM DR. AREA H1
CULVERT REQ'D.

Design Storm 25 yr 6 hr

Drainage Area .04 sq.mi. 27 ACRES Runoff Condition No. II

Lag .10 Hrs. T_c .16 Hrs.

Runoff Curve No. 77 Storm Distrib. Curve —

Hydrograph Family No. 4

Rainfall: (1) Point 2.1 in.

(2) Areal 2.1 in.

(3) Duration Adj. — in.

(4) Q .5 in.

(5) T_o 3.88 hrs.

(6) T_p .11 hrs. $T_p = 0.7\ T_c$

(7) T_o/T_p 35.27

(8) Revised T_o/T_p Ratio 36

(9) Revised T_p .108 Hrs. $\frac{T_o}{(T_o/T_p)\ \text{rev.}}$

(10) q_p 179.26 C.F.S. $q_p = \frac{484A}{\text{Rev. } T_p}$

(11) Qq_p 89.63 C.F.S. Qxq_p

(12) q_c/q_p from Hydrograph Family .0672

(13) Peak Flow 6.02 C.F.S. $q_c/q_p \times Qq_p$

W.C. 2500

$$\text{LAG} = \frac{2500^{0.8}\left(\frac{1000}{77}-10+1\right)^{0.7}}{1900\ (54.42)^{0.5}} = \frac{1375.02}{14016.28} = .10$$

$$T_c = \frac{.10}{.6} = .16$$

HEADWATER DEPTH - 24" CMP @ 6 CFS + 4.2 CFS FROM AREA H1 = 10.2 CFS = .85 DIA.

24" CMP REQ'D.