November 6, 1978

Mr. John L. Bell
Trail Mountain Coal Company
P.O. Box 356
Orangeville, Utah 84537

Re: Culvert design

Dear Mr. Bell:

Enclosed is an estimate of peak flow for the small watershed which is presently diverted away from your portal and discharges near the bath-house. Estimated peak flow from the 10 year 24 hour storm for this water shed is 91 cubic feet per second.

Any culvert design that can handle this flow that also discharges in such a way that prevents erosion is acceptable. An example would be a 4 foot diameter culvert, that has a headwall at the entrance with at least 4.8 feet of head as measured from the bottom of the pipe, and a rip rapped discharge. A trash rack must also be installed at the entrance.

I hope this will be of some help to your operation.

Sincerely,

K. MICHAEL THOMPSON
ENGINEERING GEOLOGIST

KMT/sp
enc: peak flow estimate

Temporary Diversion Located Near Bath-House
Trail Mountain Mine

Time of Concentration (Tc)

$$Tc\ (hrs) = 0.00013 \frac{L^{0.77}}{S^{0.385}} \quad \text{(Kirpitch)}$$

Where:

L= Length of basin area in feet, measured along the watercourse and in a direct line from the discharge point to the farthest point on the basin:
  2.25 inches *62,500* ½ft/in = 11,719 ft.

S= Ratio in feet to "L" of the fall of the basin to the length, or approximately the average slope of the basin in dimentionless ratio:
  (9600-7250)/11,719 = 0.20

L should be 6510', 1.25 in on map not 2.25 KMP 11/22

$$Tc\ (hrs) = 0.00013 \frac{11,719^{0.77}}{0.20^{0.385}}$$

$$= 0.33 \text{ hrs.} = 20 \text{ minutes}$$

Peak Flow Estimate

Qp (cfs.) = CiA Rational Formula

Where:

C= Rational Coefficient estimate at 0.20, equivalent to curve number of 72.

i= Rainfall intensity (in/hr) for period equal to time of concentration.
  Tc= 20 minutes, i= 0.36 in/20 min.
  = 1.08 in/hr.

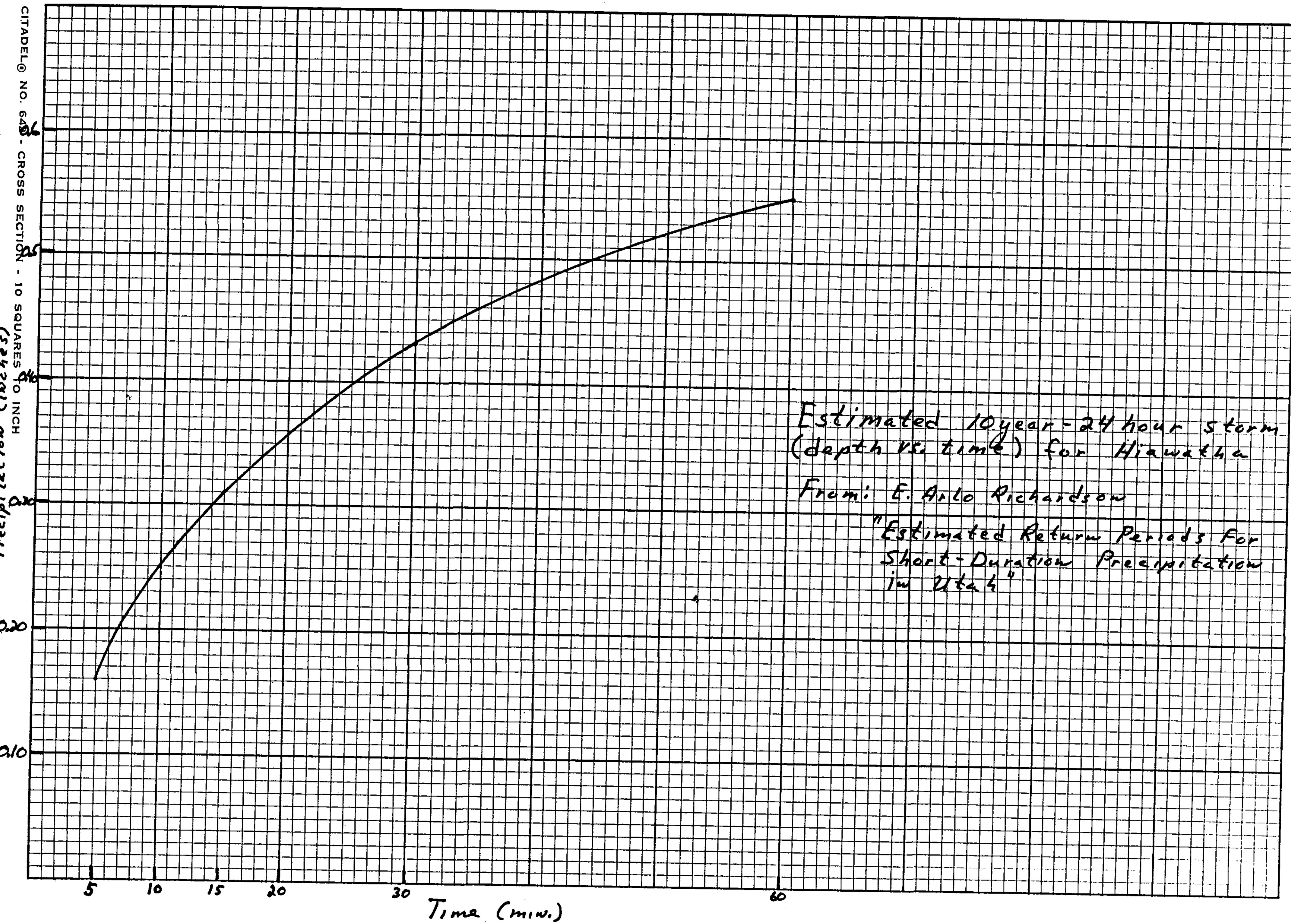
A= Area in acres.
  0.66 mi^2 *640 ac/mi^2 = 422.4 acres

Peak Flow (cfs) = 0.20 * 1.08 in/hr * 422.4 acres
= 91 cfs.

Culvert Size Estimate

A 48 inch diameter culvert will pas 91 cubic feet per second if the entrance has a headwall and there is 4.8 feet of head measured from the bottom of the pipe.

This culvert size and design is one of numerous possibilities. Any design that will pass 91 cfs will be acceptable.

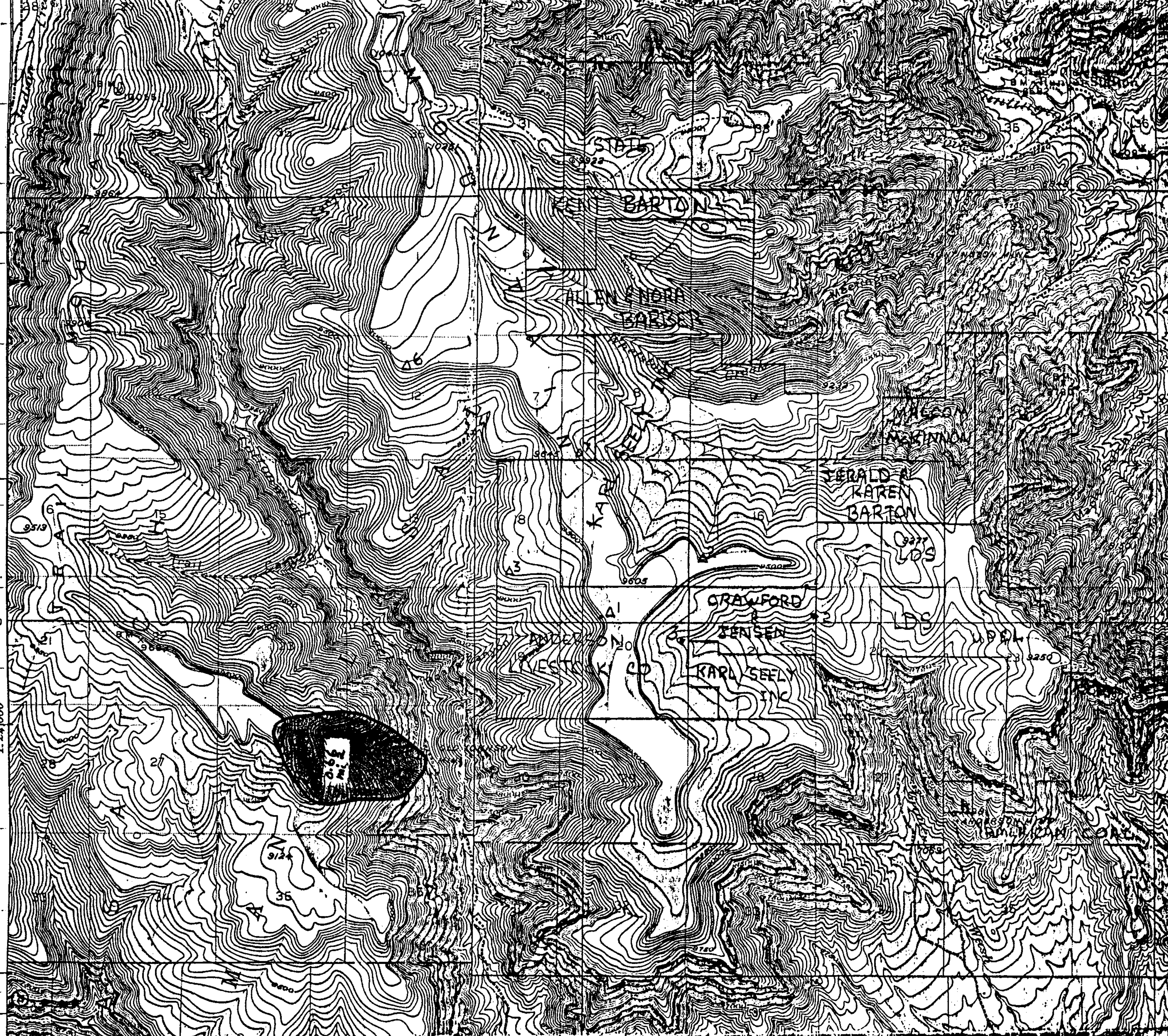

0.6
0.5
0.40
0.30
0.20
0.10
5
10
15
20
30
60
Time (min.)
Estimated 10 year - 24 hour storm
(depth vs. time) for Hiawatha
From: E. Arlo Richardson
"Estimated Return Periods for
Short-Duration Precipitation
in Utah"

T. 17 S.

20

(Joes Valley Reservoir)
1:24,000

PROVO 99 MI.
EPHRAIM (U.S. 89) 25 MI.



some influencing factors, such as storm frequency, initial soil-moisture condition, storm duration, and time of year. Using mean annual temperature as a parameter, for example, Langbein and others [3] have shown a satisfactory relationship between mean annual precipitation and runoff.

The initial soil-moisture condition, or *antecedent moisture condition*, is a parameter which cannot be determined directly and used reliably. For practical purposes, this parameter is usually expressed by an index which is so defined that it can be roughly representative of the initial soil-moisture condition and also can be easily measured. The following are some such indexes that have been proposed: groundwater flow at the beginning of the storm, basin evaporation, and antecedent precipitation.

When groundwater flow at the beginning of the storm is used as an index, it should be supplemented by a weighted index of the rainfall for several days preceding because recent rains affect current moisture content.

Accumulated evaporation from a standard evaporation pan has been used as an index of field-moisture deficiency. Linsley and Ackermann [4] found that the deficiency at any time was equal to 0.9 times the accumulated pan evaporation since the ground was last saturated minus any additions due to intervening rains.

The antecedent precipitation can be used as an index because it affects the soil-moisture condition. For the correlation of annual runoff with rainfall, Butler [5] showed an *antecedent precipitation index* (API), P_a, which can be expressed essentially as

$$P_a = aP_0 + bP_1 + cP_2 \tag{14-1}$$

where P_0, P_1 and P_2 are the annual rainfalls for the current year, the antecedent year, and the second antecedent year, respectively. The weighting coefficients a, b, and c, with their sum equal to unity, are determned by trial and error in order to obtain a best correlation between the runoff and the weighted API. For individual storms, Kohler and Linsley [6] proposed a similar API:

$$P_a = b_1P_1 + b_2P_2 + \cdots + b_tP_t \tag{14-2}$$

where b_t is a constant less than unity, and P_t is the amount of precipitation which occurred t days prior to the storm under consideration. The constant b_t is commonly assumed as a function of t. If a day-by-day value of the index is required, b_t may be assumed to decrease exponentially with t, or $b_t = k^t$, where k is a *recession constant*. Thus,

$$P_{at} = P_{a0}k^t \tag{14-3}$$

where P_{a0} is the initial value of API and P_{at} is the reduced value t days after. The index after any day is related to the index of the day before as $P_{a1} = kP_{a0}$ since $t = 1$. The value of k normally ranges between 0.85 and 0.98.

By using API, week of year, and storm duration as parameters, Kohler and Linsley [6] developed a relationship between the storm runoff and precipitation by a graphical method of coaxial correlation (see Figs. 8-II-3 and 25-IV-2). Hopkins and Hackett [7] extended such analyses by including an index of antecedent temperature (ATI) and the average annual basin temperature as additional parameters.

2. The Rational Formula. The relation between rainfall and peak runoff has been represented by many empirical or semiempirical formulas [8]. The *rational formula* can be taken as a representative of such formulas (see also Secs. 20, 21, and 25-I). Although this formula is based on a number of assumptions which cannot be readily satisfied under actual circumstances, its simplicity has won it popularity. The origin of this formula is somewhat obscure. In American literature, the formula was first mentioned in 1889 by Kuichling [9] for a determination of peak runoff for sewer design in Rochester, New York, during the period from 1877 to 1888. Some authors believe that the principles of the formula were explicit in the work of Mulvaney [10] in 1851. In England, the method using the rational formula is often referred to as the *Lloyd-Davis method* owing to the implication ascribed to a paper of 1906 [11].

$$Q = CIA \tag{14-4}$$

where Q is the peak discharge in cfs, C is a runoff coefficient depending on characteristics of the drainage basin, I is the rainfall intensity in in. per hr, and A is the drainage area in acres. The formula is called rational because the units of the quantities involved are numerically consistent approximately.

When using the rational formula, one must assume that the maximum rate of flow, owing to a certain rainfall intensity over the drainage area, is produced by that rainfall which is maintained for a time equal to the period of concentration of flow at the point under consideration. Theoretically, this is the *time of concentration*, which is the time required for the surface runoff from the remotest part of the drainage basin to reach the point being considered. For uniform rainfall intensity, this would be the *time of equilibrium* at which the rate of runoff is equal to the rate of rainfall supply. For natural drainage basins of large size and complex drainage pattern, runoff water originating in the most remote portion may arrive at the outlet too late to contribute to the peak flow. Accordingly, the time of concentration is generally greater than the lag time of the peak flow. For small drainage basins with simple drainage patterns, the time of concentration may be very close to the lag time of the peak flow. For small agricultural drainage basins, Ramser [12] has determined the time of concentration by noting the time required for the water in the channel at the gaging station to rise from the low to the maximum stage as recorded by the water-stage recorder. An empirical formula for the time of concentration in hours thus determined by Kirpich [13] is

$$t_c = 0.00013 \frac{L^{0.77}}{S^{0.385}} \tag{14-5}$$

where L is the length of the basin area in feet, measured along the watercourse from the gaging station and in a direct line from the upper end of the watercourse to the farthest point on the drainage basin; and S is the ratio in feet to L of the fall of the basin from the farthest point on the basin to the outlet of runoff, or approximately the average slope of the basin in dimensionless ratio.

Some values of the runoff coefficient C are reported by a joint committee of the American Society of Civil Engineers and the Water Pollution Control Federation [14] as given in Table 14-1. These values are applicable for storms of 5 to 10-year frequencies. Less frequent higher-intensity storms will require the use of higher coefficients because infiltration and other abstractions have a proportionally smaller effect on peak runoff. Average values of C for agricultural lands are also given in Sec. 21.

According to Krimgold [15], the assumptions involved in the rational formula are:

(1) The rate of runoff resulting from any rainfall intensity is a maximum when this rainfall intensity lasts as long or longer than the time of concentration.

(2) The maximum runoff resulting from a rainfall intensity, with a duration equal to or greater than the time of concentration, is a simple fraction of such rainfall intensity; that is, it assumes a straight line relation between Q and I, and $Q = 0$ when $I = 0$.

(3) The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.

(4) The relationship between peak discharges and size of drainage area is the same as the relationship between duration and intensity of rainfall.

(5) The coefficient of runoff is the same for storms of various frequencies.

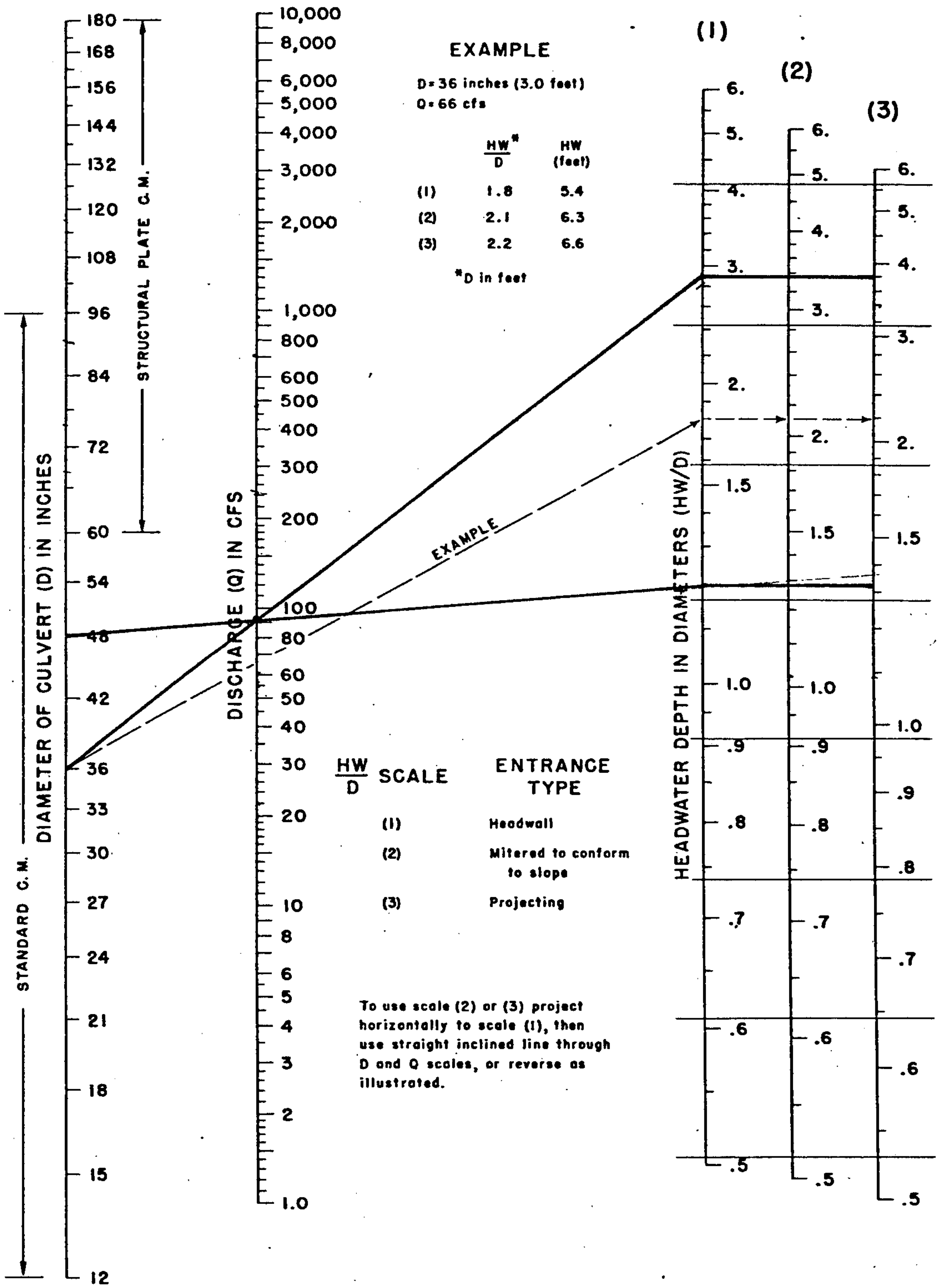
(6) The coefficient of runoff is the same for all storms on a given watershed.

It is believed that these assumptions might nearly hold for paved areas with gutters and sewers of fixed dimensions and hydraulic characteristics. The formula has thus been rather popular for the design of drainage systems in urban areas and airports. The exactness and satisfaction of these assumptions in application to other drainage basins, however, have been questioned. In fact, many hydrologists have called attention to the inadequacy of this method.

From Chow. Handbook of Applied Hydrology

If multiple culvert design, Divide Q(cfs) equally between them & use Q for 1 pipe to determine culvert Diameter.

Chart 2-53: HEADWATER DEPTH FOR C.M.P. CULVERTS WITH INLET CONTROL



BUREAU OF PUBLIC ROADS JAN. 1963

From "Utah State Dept. of Highways
"Manual of Instruction
Part 4
Roadway Drainage