2.0 METHODS

2.1 Hydrograph Synthesis

Watershed boundaries used to determine runoff conditions at the site are shown on Plate 1. Data obtained from these watersheds were input to a computer code developed by Hawkins and Marshall (1979) to generate runoff hydrographs for the 10-year, 24-hour storm required for designing diversions. Inflow hydrographs to and outflow hydrographs from the sedimentation pond were developed for the 25-year, 24-hour storm using the hydrology and sedimentology model SEDIMOT II (Warner et al., 1980; Wilson et al., 1980). Both of these codes model runoff using the rainfall-runoff function and triangular unit hydrograph of the U.S. Soil Conservation Service (1972).

2.1.1 Runoff Volume. According to the U.S. Soil Conservation Service (1972), the algebraic and hydrologic relations between storm rainfall, soil moisture storage, and runoff can be expressed by the equations

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \tag{1}$$

and

$$S = \frac{1000}{CN} - 10 \tag{2}$$

where Q = direct runoff volume (inches)
S = watershed storage factor (inches)
P = rainfall depth (inches)
CN = runoff curve number (dimensionless)

It should be noted that (a) Equation (1) is valid only for $P>=0.2S$ (otherwise $Q=0$), (b) Equation (2), as stated, is in inches, with the values of 1000 and 10 carrying the dimensions of inches, although metric conversions are possible, and (c) CN is only a convinient transformation of S to establish a scale of 0 to 100 and has no intrinsic meaning.

The average curve number for undisturbed areas was obtained from the curves presented in Figure 2 using a measured cover densities as reported in Chapter 9 of the Permit Application Package for Tract 2). A curve number of 69 was thus obtained for the undisturbed areas, assuming a hydrologic soil group of C (see Appendix A).

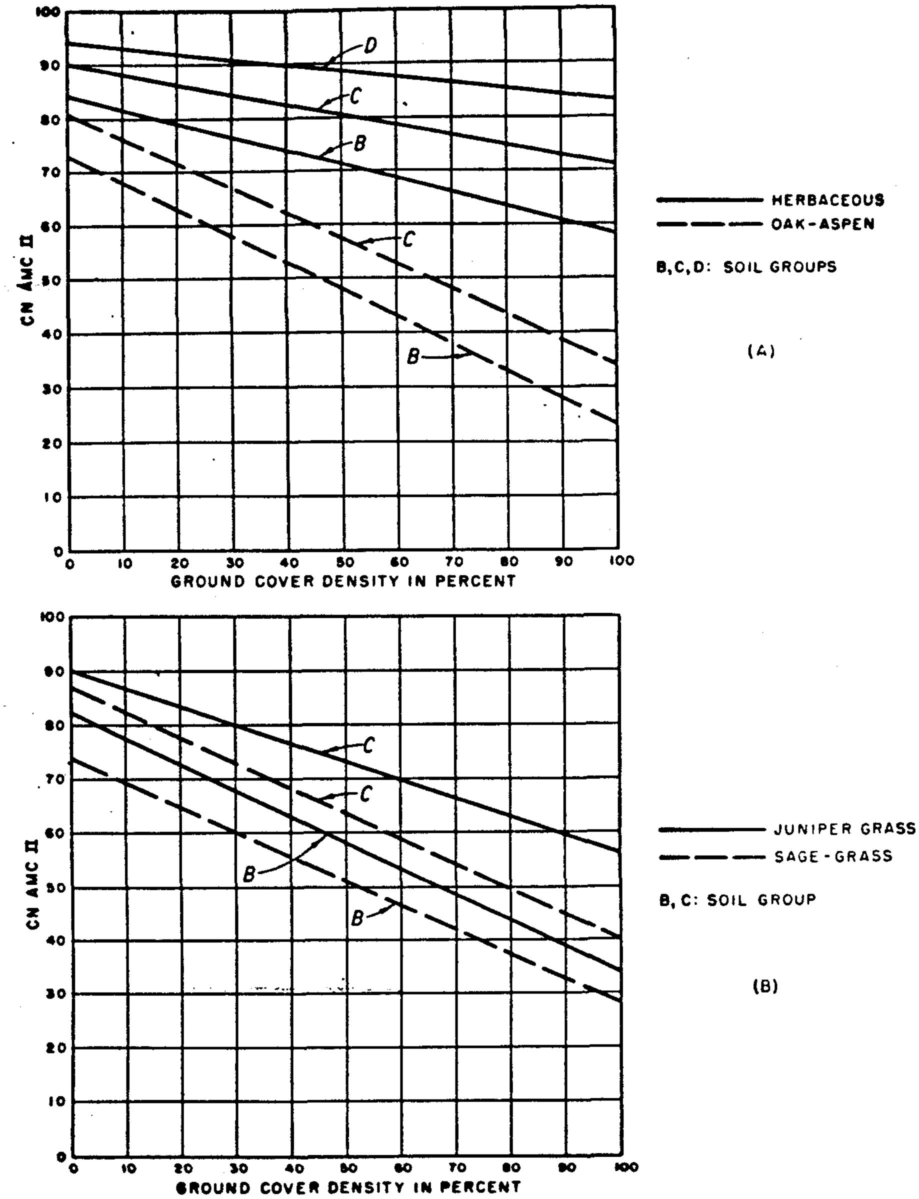

Figure 2. Runoff curve numbers for forest-range in the western U.S. (from U.S. Bureau of Reclamation, 1977).

The curve number for disturbed areas was chosen from professional judgement and tabulated values presented by the U.S. Soil Conservation Service (1972). Accordingly, a value of 90 was used for the pad and road areas.

2.1.2 Unit Hydrograph. The translation of the runoff depth to an outflow hydrograph is accomplished in the codes using the triangular unit hydrograph of the U.S. Soil Conservation Service (1972). This unit hydrograph is shown in Figure 3 along with a typical curvilinear hydrograph. It is characterized by its time to peak (T_p), recession time (T_r), time of base (T_b), and the relations between these parameters (i.e., $T_r=1.67T_p$; $T_b=2.67T_p$). Thus, from the geometry of a triangle, the incremental runoff (Q) can be defined by the equation

$$Q = \frac{(2.67T_p)(q_p)}{2} \qquad (3)$$

or

$$q_p = \frac{0.75 \quad Q}{T_p} \qquad (4)$$

where q_p = peak flow rate (dimensioned according to Q and T) and other parameters have been previously defined.

When Q is expressed in inches and T_p in hours, q_p will be in inches per hour. The flow at any time $0<t<T_r$ may be determined by simple linear proportioning of the triangular unit hydrograph. The time to peak is related to the familiar expression time of concentration (T_c) by the equation

$$T_c + t = 1.7T_p \qquad (5)$$

in which the factor 1.7 is an empirical finding cited by the U.S. Soil Conservation Service (1972).

The time of concentration may be estimated by several formulas. For this report, T_c was determined from the following equations (U.S. Soil Conservation Service, 1972):

$$L = \frac{0.8 \quad (S+1)^{0.7}}{1900 \; Y^{0.5}} \qquad (6)$$

and

$$T_c = 1.67L \qquad (7)$$

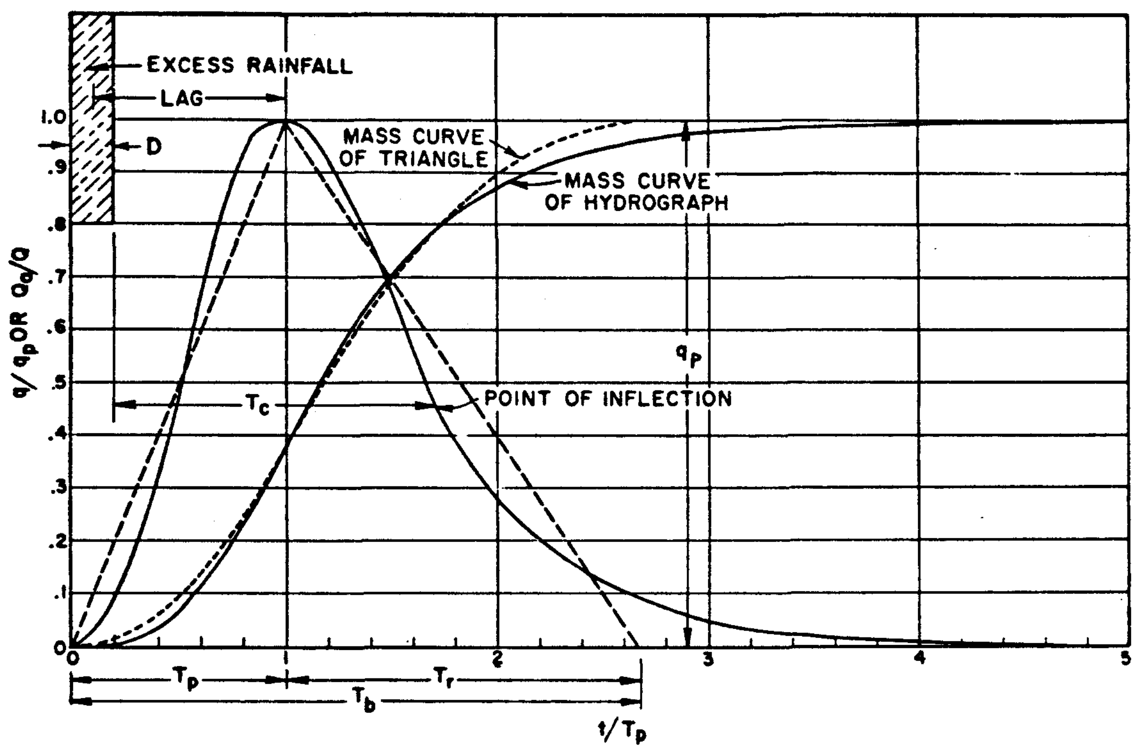

Figure 3. Curvilinear and triangular unit hydrographs (from U.S. Soil Conservation Service, 1972).

where L = watershed lage (hours)
= hydraulic length of the watershed, or distance along the main channel to the watershed divide (feet)
S = watershed storage factor defined in Equation (2)
Y = average watershed slope (percent)
T_c = time of concentration (hours)

2.2 Diversion Channel Calculations

A diversion channel was designed to convey runoff from an undisturbed area away from the disturbed site using the Manning and continuity equations:

$$V = \frac{1.486}{n} R^{0.67} S^{0.50} \tag{8}$$

and

$$Q = AV \tag{9}$$

where V = velocity (feet per second)
R = hydraulic radius (feet)
S = hydraulic slope (feet per foot)
n = roughness coefficient
Q = discharge (cubic feet per second)
A = flow area (square feet)

Values of the roughness coefficient required for the solution of Equation (8) were obtained by comparing local conditions with tabulated values provided by the U.S. Soil Conservation Service (1956). An empirical formula developed by Anderson et al. (1970) was used to determine the roughness coefficient for riprap linings.

Calculations with Equations (8) and (9) were performed using an iterative computer code entitled TRAP1 as obtained from the U.S. Office of Surface Mining and outlined by Weider et al. (1983). This code was used to determine flow conditions in the diversion channel at the design flow rate.

2.3 Spillway Hydraulics

The sedimentation pond at the downstream edge of the site has been designed with a primary and emergency spillway. The primary spillway consists of a CMP riser and pipe through the embankment while the emergency spillway consists of a riprapped overflow at the corner of the embankment.

At low heads, the hydraulic capacity of the primary spillway behaves as a weir. According to Barfield et al. (1981), the equation for weir-controlled flow is

$$Q = CLH^{1.5} \tag{10}$$

where Q = discharge (cubic feet per second)
C = weir coefficient
L = length of the weir (feet)
H = depth of water above the weir crest (feet)

A value of the weir coefficient equal to 3.1 was selected since the structure will act as a broad-crested weir (Barfield et al., 1981). The length of the weir is equal to the circumference of the CMP riser.

As the depth of water increases above the riser, the riser acts like an orifice. The equation for orifice flow is (Barfield et al., 1981)

$$Q = C'A(2gH)^{0.5} \tag{11}$$

where C' = orifice coefficient
A = cross-sectional area of the inlet (square feet)
g = gravitational constant (feet per second squared)

and other parameters have been previously defined. A value of 0.60 was selected for the orifice coefficient based on guidelines presented by Barfield et al. (1981).

Pipe flow occurs when the head increases sufficiently to cause the outlet of the discharge pie leading from the riser to flow full. The discharge capacity of the culverts under pipe flow conditions was determined using the equation

$$Q = A(2gH')^{0.5}/(1+K_e+K_b+K_cL)^{0.5} \tag{12}$$

where H' = head on the pipe (feet)
K_e = entrance loss coefficient
K_b = bend loss coefficient
K_c = friction loss coefficient

and all other parameters have been previously defined. Values of 1.0, 0.5, and 0.062 were used for K_e, K_b, and K_c, respectively based on information provided by Barfield et al. (1981).

The discharge capacity of the emergency spillway was determined using a method developed by the U.S. Soil Conservation Service (1968) and expanded by Barfield et al. (1981) for broad-crested weirs. According to this methodology, the critical specific energy head (H_{ec}) is determined for selected values of the energy head of water in the pond (H_p) from Figure 4. The

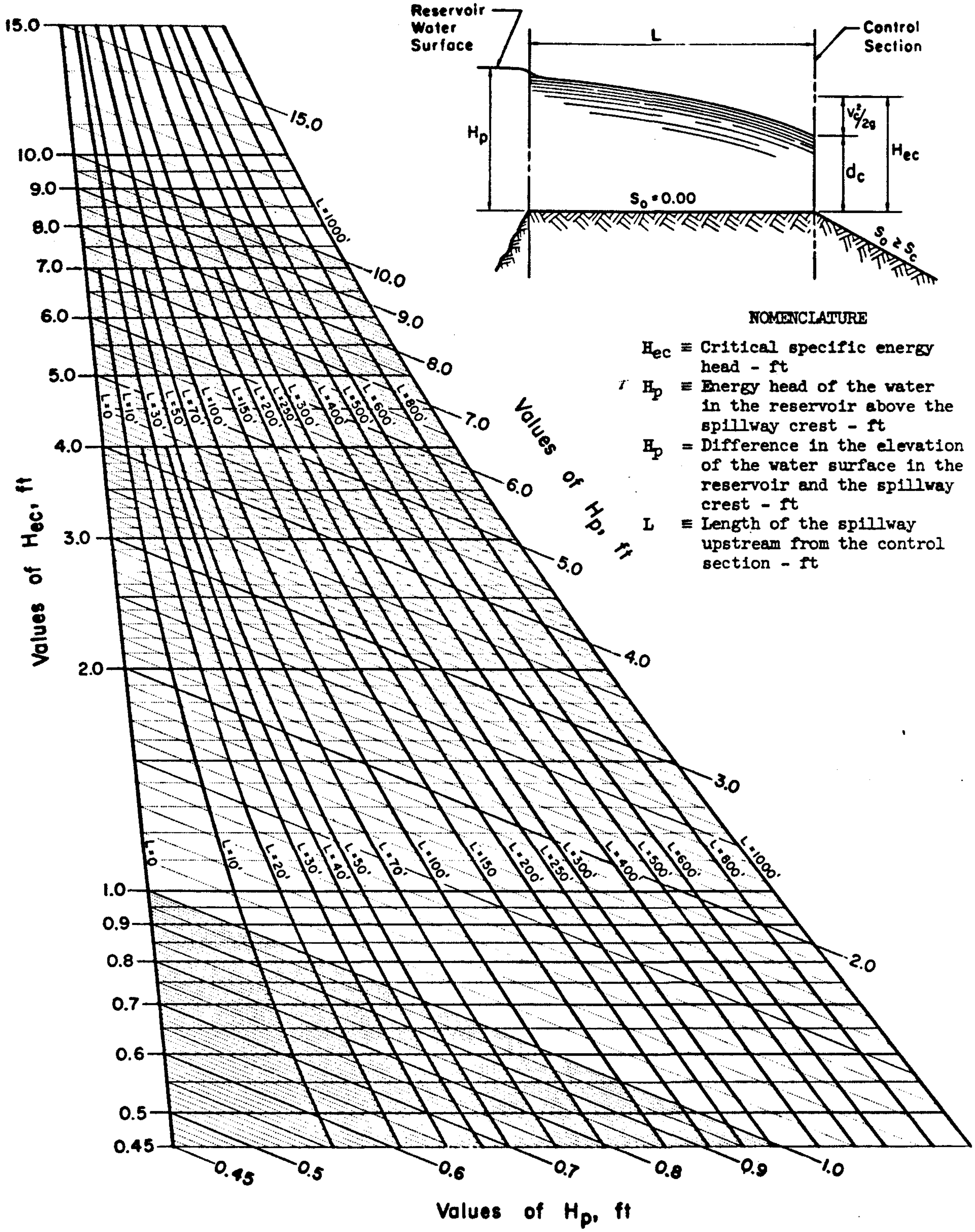

Figure 4. Head relationships for selected broad-crest weirs (from U.S. Soil Conservation Service, 1968)

discharge capacity of the spillway is then calculated for the standard 100-foot wide rectangular section from the equation

$$q_r = (0.544)(g^{0.5})(H_{ec}^{1.5})(100) \qquad (13)$$

where q_r = discharge for standard 100-foot rectangular section (cubic feet per second)

and all other parameters have been previously defined. The flow is then corrected for a trapzoidal section using the equation

$$q = ([1.5b + zH_{ec}]/150)(q_r) \qquad (14)$$

where q = corrected discharge (cubic feet per second)
b = bottom with of channel (feet)
z = channel side slope (run over rise - dimensionless)

The combined ouflow capacity of the two spillways was determined by summing the outflows from the primary and emergency spillways at specific heads. The hydraulics of the spillway system was determined by assuming the pond was dewatered to the top of the sediment storage level prior to inflow from the 25-year, 24-hour storm.

2.4 Stability Analyses

Due to space restrictions, the sediment pond for the mine site was designed with upstream and downstream slopes both equal to 2h:1v. Since UMC 817.46(m) requires a combines slope of 5h:1v, a stability analysis was conducted to ensure that the pond embankment, as designed, would be stable.

The stability analysis was conducted using a microcomputer version of the program entitled STABL2 (Siegel, 1978). The modified Bishop method was used to calculate the factor of safety under both static and seismic conditions. Seismic conditions were modeled using a horizontal acceleration coefficient of 0.080 and a vertical acceleration coefficient of 0.000. Stability was modeled assuming both full and empty ponds, both with and without the designed clay liner functioning.