0011

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Mine Number: C/007/004

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SCHOOLHOUSE CANYON REFUSE DISPOSAL FACILITY
SUPPLEMENTARY HYDROLOGIC ANALYSIS

1.0 INTRODUCTION

In a letter from Mr. H. E. Dolan of MESA to Mr. K. B. Hutchinson of Braztah Corporation on February 28, 1978, additional hydrologic information and analysis was requested in support of the proposed design for the Schoolhouse Canyon Refuse Disposal Facility. The analysis which follows supplements that given in Section 5 of Golder Associates' report entitled "Design of a Coal Refuse Disposal System", "Phase II, Detailed Design", and substantiates the adequacy of the Diversion Ditch and Spillway designs.

2.0 DISCUSSION

2.1 Derivation of Design Flow Rates

From table 6.6 of MESA's "Engineering and Design Manual, Coal Refuse Disposal Facilities" the proposed settling pond would be classified as "Intermediate" in size and "Moderate" in terms of its hazard potential. The recommended design storm criteria of 1/2 of the probably maximum precipitation (PMP) has therefore been adopted for design purposes. From Figures 17 and 20 of the 1975 U.S. Bureau of Reclamation publication, "Design of Small Dams" the following storm data has been obtained for the site vicinity:

Probable maximum precipitation (6 hours) PMP = 4 inches (1/2 PMP = 2 inches)

Probable maximum thunder storm (1 hour) PMTS = 6.5 inches (1/2 PMTS = 3.25 inches)

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Since the watershed and the proposed pond are both relatively small, the time of concentration and the routing time for a precipitation event occuring over the canyon catchment, are less than the duration of the design storms (1 hour and 6 hours for the PMTS and PMP respectively). The effect of short term storage is therefore insignificant, and the peak design flow for the hydraulic structures concerned can be considered equivalent to the peak run-off flow which would occur in response to the storm. Because of these factors it is obvious that the PMTS precipitation event will be the most critical in the design.

A 0.65 run-off coefficient was used in Golder Associates' Phase II Report. This was adopted however, for a storm of lower intensity and longer duration than that currently being considered. For the current studies a more conservative run-off coefficient of 0.8 is considered appropriate.

Conservative estimates of the peak flow rates to be carried by the diversion channel and the settling pond spillway have been obtained from a run-off hydrograph, derived from the precipitation hydrograph multiplied by the corresponding catchment area and the run-off coefficient. The peak flow rates anticipated on the basis of 1/2 PMTS are as follows:

a. <u>On the upper catchment (above the diversion ditch)</u>:

$$Q_p = 3.25 \frac{\text{in.}}{\text{hr.}} \times \frac{1 \text{ ft.}}{12 \text{ in.}} \times \frac{1 \text{ hr.}}{3{,}600 \text{ sec.}} \times 193 \text{ acres} \times 43{,}560 \frac{\text{sq. ft.}}{\text{acre}} \times 0.8$$

$$= 506 \text{ cfs}$$

b. On the lower catchment (below the diversion ditch):

$$Q_p = 3.25 \frac{in.}{hr.} \times \frac{1\ ft.}{12\ in.} \times \frac{1\ hr.}{3{,}600\ sec.} \times 63\ acres \times 43{,}560 \frac{sq.\ ft.}{acre} \times 0.8$$

$$= 165\ cfs$$

The design configurations of both the diversion ditch and the spillway will now be examined to determine their capacity to handle these design flow rates.

2.2 Diversion Ditch

The proposed diversion ditch channel comprises three distinct segments, each of which has a different trapezoidal cross-section configuration. These cross-sections were given as figure 5-3 of Golder Associates' Phase II Report and differ only in depth. The design calls for a minimum channel bottom width of 15 feet, 1:1 slopes on the down slope and 1/2:1 slopes on the upslope side of the ditch.

The Chezy-Manning equation is the governing equation for open channel flow and is written as follows:

$$Q = \frac{1.49}{n} A R_h^{\ 2/3} \quad S^{1/2}$$

Where Q = flow in cfs

n = roughness coefficient (conservatively estimated at 0.05)

A = cross-sectional area of fluid perpendicular to flow direction

R_h = hydraulic radius (ratio of area to wetted perimeter)

S = slope of channel

Area wetted perimeter (P) and hydraulic radius can be written in terms of water depth (y) as follows:

$$A = 15\,y + 0.75\,y^2 \;,\; P = 15 + \sqrt{2}\,y + \frac{\sqrt{5}}{2}\,y$$

$$R_h = \frac{A}{P} = \frac{y + 0.05\,y^2}{1 + 0.169y}$$

The Chezy-Manning equation can then be written in terms of depth (y) as follows:

$$Q = \frac{1.49}{0.05} \cdot 15 \cdot (y + 0.05y^2) \cdot \left(\frac{y + 0.05y^2}{1 + 0.169y}\right)^{2/3} \cdot (S)^{1/2}$$

Using a design flow rate of 506 cfs the following data has been derived regarding the available freeboard, given the respective cross-section depths and channel gradients.

Channel Section	Total Channel Section Depth (ft.)	Gradient	Water Depth at Peak Flow (ft.)	Available Freeboard (ft.)
A	4.5	0.04	2.90	1.60
B	5.0	0.02	3.57	1.43
C	6.0	0.01	4.37	1.63

From MESA, 1975, "Design Guidelines for Coal Waste Structures", the following formula has been obtained to determine the required freeboard:

$$H = 1 + 0.025\,V\,(y)^{1/3}$$

where H = design freeboard (ft.)

V = flow velocity (ft./sec)

y = water depth (ft.)

Again for each of the sections concerned the design freeboard is as follows:

Channel Section	Flow Depth (ft)	Velocity flow ÷ area (ft/sec)	Required Freeboard (ft)
A	2.90	10.20	1.36
B	3.57	8.02	1.31
C	4.37	6.34	1.26

From a comparison of the data tabulated, it is concluded that the diversion ditch can safely pass the peak flow attributable to 1/2 PMTS.

2.3 Settling Pond Spillway

As stated in Section 2.1, the spillway design must be capable of carrying the peak run-off flow generated by half the PMTS over the lower catchment area. This flow from the previous calculations was estimated at 165 cfs.

Consistent with Mr. Dolan's letter, the spillway is modelled as a broad crested weir with a weir coefficient of 3 (page 6.136, Engineering Design Manual, Coal Refuse Disposal Facilities, MESA) The formula for flow over a broad crested weir is as follows:

$$Q = CLH^{1.5}$$

where Q = flow in cfs (165 cfs required)

C = weir coefficient (3)

L = weir width (15 feet)

H = height of water above the weir behind the dam.

Solving for H

$$H = \left(\frac{Q}{CL}\right)^{2/3} = 2.38 \text{ feet}$$

The current design provides a 5 foot difference in elevation between the embankment crest and the highest point in the spillway. A 2.62 ft. freeboard allowance is therefore available at peak flow.

2.4 Culvert Discharge System

The existing culvert system, into which the spillway flow will discharge, was designed by Martin Berkowitz and Associates. It comprises 555 feet of galvanized corrugated steel pipe, 4 feet in diameter (at its minimum), with an elevation difference of 156 feet from the points of entry to the Price River.

The equation for flow in a pipe is

$$Q = \frac{h_L d \quad (2g)}{Fl} \cdot \pi \cdot \frac{d^2}{4}$$

where h_L = head loss (156 ft.)

d = diameter (4 ft.)

g = gravity (32.2 ft./sec^2)

l = length (555 ft.)

f = friction factor (assumed 0.1)

Solving for Q

$$Q = \frac{(156)\ (4)\ (2)\ (32.2)\ (\pi)\ (16)}{(0.1)\ (555)\ (4)}$$

$$= 9100 \text{ cfs}$$

Hence the culvert system is more than adequate to carry the design flood of 165 cfs.

3.0 CONCLUSIONS

The hydrologic analysis presented above has shown that all components of the hydrologic system including the diversion ditch, the spillway and the culvert meet with current MESA regulations and are adequately designed to carry the appropriate design flood.