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Golder Associates

CONSULTING MINING AND GEOTECHNICAL ENGINEERS

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E/78/213
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HELPER, UTAH

American Electric Power Service
Corporation
P.O. Box 629
Helper, Utah 84526

ATTN: MR. LEE McCLOSKEY RETN

RE: SCHOOLHOUSE CANYON REFUSE DISPOSAL FACILITY - REVISED SUPPLEMENTARY HYDROLOGIC ANALYSIS

Gentlemen:

Please find attached additional Hydrologic Analysis in support of the Diversion Ditch and Settling Pond spillway designs. This document has been extensively revised from that supplied to you on March 7, 1978 in the light of a further telephone conversation with Mr. Bob Petterson on March 10, 1978 and a complete review by our hydrological staff. In particular a time distribution of the six-hour PMP and one-hour PMTS precipitation events has been incorporated in accordance with the USBR publication, "Design of Small Dams", 1974.

The results of this revised study indicate that the Settling Pond Spillway design is adequate to pass the peak runoff under the appropriate storm criteria. For the Diversion Ditch, however, the design requires modification to increase the minimum depth by 0.5 feet, such that it will also pass the peak runoff from the design storm.

To satisfy this design modification, the following amendments should be made to Diversion E, "Technical Specifications" of your construction bid package.

1. In Section 2.0, "Scope: Schoolhouse Canyon Diversion Ditch and Improvement to Existing Channel" on page E2-2, the second paragraph, second sentence should read:

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"The ditch is designed to have a minimum bottom width of fifteen (15) feet and a minimum depth of six and one-half (6.5) feet."

2. On Drawing 3, "Typical Diversion Ditch Cross-Section", the depth should be modified to show "6.5 feet minimum."

As agreed in our telephone conversation yesterday, a copy of this letter and the attachment is also being forwarded to Mr. Bob Petterson of MESA, to expedite the approval process.

Should any further questions arise or clarification be required, please do not hesitate to contact us.

Sincerely,

GOLDER ASSOCIATES, INC.

Graham A. Mathieson
Project Engineer

A. Allen Gass
Principal in Charge

GAM/AAG:kw
S77212

Enclosure: Supplementary hydrological analysis (revised March 13, 1978).

cc: Bob Petterson

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SCHOOLHOUSE CANYON REFUSE DISPOSAL FACILITY
SUPPLEMENTARY HYDROLOGIC ANALYSIS

Revised March 13, 1978

1.0 INTRODUCTION

In a letter from Mr. H. E. Dolan of MESA to Mr. K. B. Huchinson 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 one-half of the probable maximum precipitation has therefore been adopted to 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)

The time distribution of the one-hour thunderstorm is as follows (USBR Design Small Dams, 1974, Table 2, p. 52):

0 - 15 min.	48% x 3.25" = 1.56"
15 - 30 min.	23% x 3.25" = .75"
30 - 45 min.	17% x 3.25" = .55"
46 - 60 min.	12% x 3.25" = .39"

For the six-hour PMP storm we have the following time distribution (USBR Design Small Dams, 1974, Fig. 18, p.51):

0 - 30 min.	17% x 2" = .34
30 - 60 min.	13% x 2" = .16
1 - 2 hr.	18% x 2" = .32
2 - 3 hr.	15% x 2" = .30
3 - 4 hr.	14% x 2" = .28
4 - 5 hr.	12% x 2" = .24
5 - 6 hr.	11% x 2" = .22

Since the watershed is relatively small and steep, the time of concentration for a precipitation event occurring over the catchment is less than the time increments used in the design storms (15 min. for PMTS, and 30 min. for PMP). Times of concentration were computed for the upper and lower catchment (above and below the diversion ditch) using the following equation (USBR Design Small Dams, 1974, P. 71):

$$T_c = \left(\frac{11.9\ L^3}{H}\right)^{0.385}$$

where T_c = time of concentration in hours

L = length from point of interest to most hydrologically remote point in miles

H = elevation difference in feet

Times of concentration for the upper and lower catchments were determined to be 8.5 minutes and 6.4 minutes respectively. Peak design flow for the diversion ditch is therefore equal to the peak runoff generated by the design storm. Routing is significant only for the settling pond and the lower catchment. Because of this, the peak design flow for the diversion ditch and the settling pond is equal to the runoff rate generated by the first 15-minute time increment of the PMTS.

A 0.65 runoff coefficient was used in Golder Associates' Phase II report to AEP. This was adopted for a storm of lower intensity and longer duration than the storms in use for the current analysis. A more conservative runoff coefficient of 0.8 was used for the PMTS event.

Peak flow rates for the diversion ditch and settling pond were determined using the Rational Method which is the accepted method in small watersheds such as Schoolhouse Canyon. In equational form:

$$Q_p = CiA$$

where Q_p = peak flow rate (cfs)
C = runoff coefficient
i = rainfall intensity (in/hr) of a storm whose duration is equal to the time of concentration*
A = area of the watershed (acres)

a. On the upper catchment (above the diversion ditch)

$C = 0.8$
$i = 6.24$ in/hr
$A = 193$ acres
$Q_p = 963$ cfs

*Note: Time distributions of rainfall for the PMTS are available for a minimum duration of 15 min. Thus the rainfall intensity in in/hr is based upon the 15 min. rainfall.

The peak runoff for the upper catchment was also computed using the S.C.S. empirical method with a curve number of 93. This yielded a peak flow of 824 cfs, which was less than the flow of 963 cfs computed above by the Rational Method. The larger flow value was used in the design of the diversion ditch.

b. <u>On the lower catchment</u> (below the diversion ditch)

This is a slightly more complex case, since the storage of the pond must be considered. The design flow for the spillway will be the runoff occurring after the pond is full. The pond has a capacity of 11.1 acre-ft. (or 484,000 cubic feet).

Peak flow rates were calculated in the same manner as for the upper catchment, but for each of the 15-minute increments instead of just the first one. For the first time increment:

$C = 0.8$
$i = 6.24$ in/hr
$A = 63$ acres
$Q_{p1} = 315$ cfs

For the other time increments the following peak flow rates were obtained:

$Q_{p2} = 151$ cfs
$Q_{p3} = 111$ cfs
$Q_{p4} = 79$ cfs

It is assumed that the discharge during each time increment flows at the peak rate for the entire increment. Thus the runoff hydrograph will decrease in steps after the first

intense period of rainfall. Based upon this assumption the pond will be filled during the third time increment during which the required peak design flow for the spillway is 111 cfs.

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:

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

Where Q = flow in cfs
n = roughness coefficient (conservatively estimated at 0.04)
A = cross-sectional area of channel 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 (R_h) can be written in terms of water depth (y) for the given channel geometry as follows:

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$$A = 15y + 0.75\ y^2$$

$$P = 15 + \sqrt{2}y + \frac{\sqrt{5}}{2}y$$

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

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

$$Q = \frac{1.49}{0.04} \times 15\ (y + .05y^2) \times \left(\frac{y + .05y^2}{1 + 0.169y}\right)^{2/3} \times (S)^{1/2}$$

Using a design flow rate of 963 cfs, the following data has been obtained for the three channel sections:

Channel Section	Depth (ft)	Gradient	Water Depth At Q_p (ft)	Available Freeboard Present Design (ft)
A	4.5	0.04	3.75	0.75
B	5.0	0.02	4.61	0.39
C	6.0	0.01	5.66	0.34

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

$$H = 1 + 0.25\ V(y)^{1/3}$$

where H = design freeboard (ft)
V = Flow velocity (ft/sec)
y = water depth (ft)

In a telephone conversation with Mr. Bob Peterson of MESA on March 10, 1978 he indicated that since this channel is not in the vicinity of the settling pond embankment, the freeboard requirement could be reduced to 60% of that computed by the above method.

Channel Section	Flow Depth (Ft)	Velocity (Ft/Sec)	Required Freeboard (Ft)	Available Freeboard (Ft)	Increased Cut Required (Ft)
A	3.75	14.5	.94	.75	.19
B	4.61	11.4	.88	.39	.49
C	5.66	8.91	.84	.34	.50

2.3 Settling Pond Spillway

As stated in Section 2.1 the spillway must be capable of carrying the runoff generated by half of the PMTS after routing through the pond. This flow was estimated at 111 cfs from the above calculations.

Consistent with Mr. Dolan's letter, the spillway is modelled as a broad crested weir with a weir coefficient of 3.0 (P. 6.136, Eng. Design Manual, Gal Refuse Disposal Facilities, MESA). The formula for flow over a broad crested weir is as follows:

$$Q = CLH^{1.5}$$

where Q = flow (111 cfs)

C = weir coefficient (3.0)

L = weir width (15 ft)

H = height of pond above the weir (5 ft available)

Rearranging terms and solving for H

$$H = \left(\frac{Q}{CL}\right)^{2/3} = 1.84 \text{ ft. (freeboard = 3.16 ft)}$$

As an additional consideration, it is possible that for a variety of reasons that the pond would be full before PMTS begins. Taking this extreme situation, the spillway would be required to carry the peak flow generated by the first 15 minute increment of the PMTS, which was shown above to be 317 cfs. Solving the weir equation again

$$H = \left(\frac{317}{3 \times 15}\right)^{2/3} = 3.67 \text{ ft. (available freeboard = 1.33)}$$

2.4 Culvert Discharge System

The existing culvert discharge system, into which the spillway will discharge, was designed by Martin Berkowitz & Associates. It comprises 555 ft. of galvanized corrugated steel pipe, 4 ft. in diameter at its minimum and 5 ft. in diameter at the inlet. A brief review of this system indicates that the amount of water flowing in the pipe will be controlled by the amount which the inlet can accept under the designed head conditions.

The current design has a 10 ft. high embankment around the inlet which allows 10 ft. of head to drive the flow into the culvert.

Flow into a submerged culvert of this type is described by the following equation:

$$Q = C_o \, 2\pi R_s \, (H)^{1.5}$$

where Q = flow (cfs)

C_o = weir coefficient (1.0 for the given geometry, U.S.B.R. Design Small Dams, Fig. 283, P. 417)

R_s = culvert radius (2.5 ft)

H = head (10 ft)

The maximum flow into the culvert is estimated at 497 cfs which is sufficient to pass the design runoff from the lower section of Schoolhouse Canyon.

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3.0 CONCLUSIONS

The hydrologic analysis presented above has shown that the lower components of the system (pond, embankment, spillway, and culvert) are adequate to carry the flow generated by the design storms, and that with minor revision, the diversion ditch can be made to handle the design storm. The diversion ditch should have a design minimum overall depth of 6.5 ft. rather than 6 ft. as given in the specifications.