Mining and Reclamation Permit Application
Willow Creek Mine
Volume 13

Prepared For:

CYPRUS
Plateau Mining

P.O. Box PMC
Price, Utah 84501
(801) 637-2875

Prepared By:

TerraMatrix
Engineering & Environmental Services

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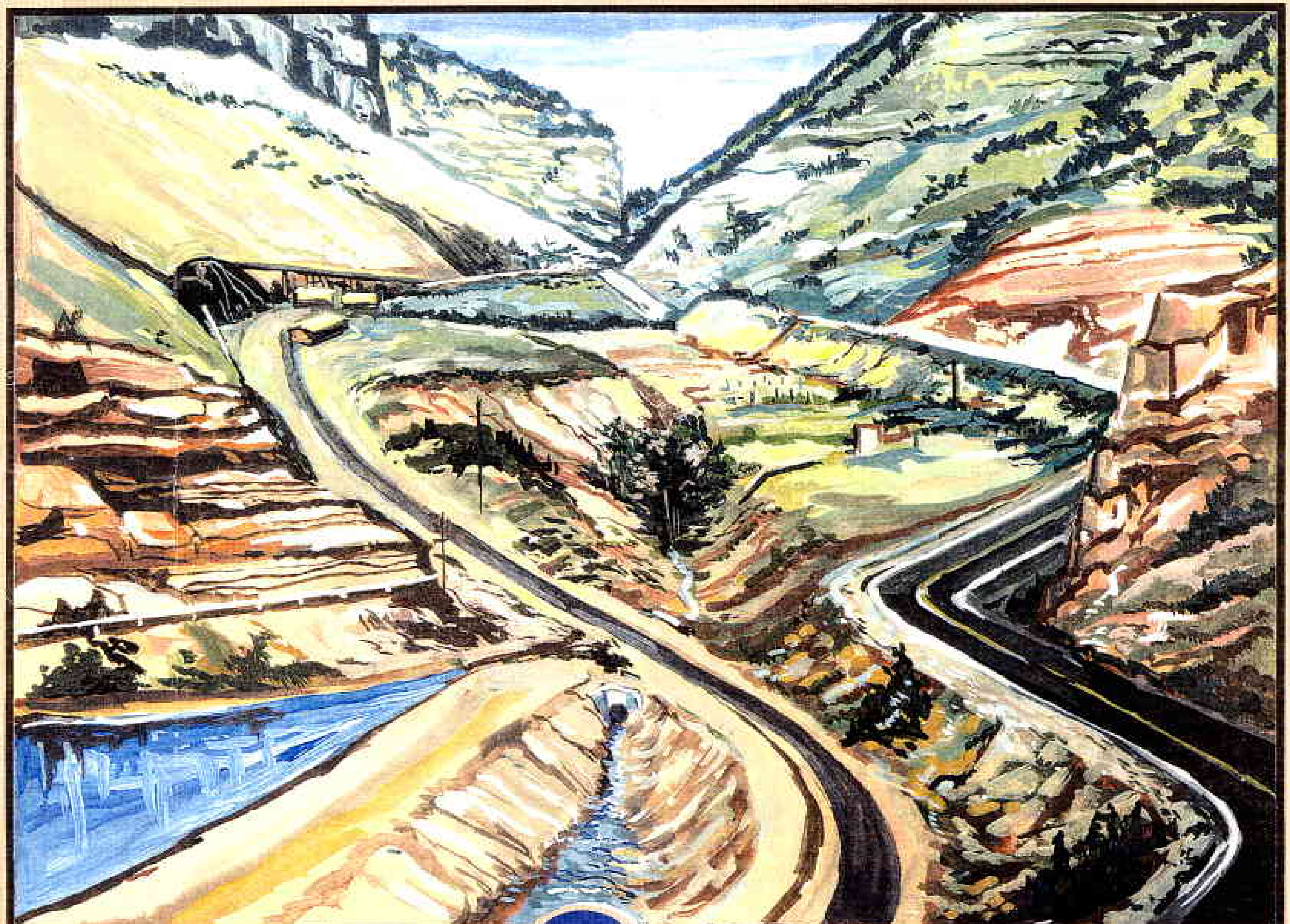

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VOLUME 13

EXHIBIT 19 - CASTLE GATE INFORMATION

This exhibit contains the Mining and Reclamation Plan (M&RP) for the Castle Gate Preparation Plant, formerly Section 3.4 of Chapter 3 of the Castle Gate Mine M&RP, Permit Number ACT/007/004. This section has been incorporated into the Willow Creek M&RP as this exhibit. Where the two M&RPs overlap, the information presented in the Willow Creek M&RP supersedes the document presented in this exhibit.

CASTLE GATE PREPARATION PLANT AND REFUSE DISPOSAL FACILITY CASTLE GATE MINE

CASTLE GATE MINE
AMAX COAL COMPANY
Carbon County, Utah

Prepared by

EARTHFAX ENGINEERING, INC.
Midvale, Utah

January 1995

SECTION 3.4

CASTLE GATE PREPARATION PLANT AND REFUSE DISPOSAL FACILITY

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3.4 CASTLE GATE PREPARATION PLANT AND REFUSE DISPOSAL FACILITY

3.4-1 General

The Castle Gate area is situated on the east bank of the Price River about 2 miles north of the city of Helper (see Exhibit 1.1). Approximately 74 acres are affected by coal preparation and disposal operations. As shown in Exhibit 3.4-1, about 44 acres are allocated for the preparation and 30 acres for the refuse disposal area.

The coal preparation plant can process 1,250 tons per hour of 4" x 0 raw coal. The circuit is composed of heavy media washers, fine coal cleaning, froth flotation, centrifugal drying, vacuum filtration, thickening and crushing. Adequate environmental controls to contain dust and effluent have been incorporated and the plant operates with a closed loop water system. Occasionally the plant needs to purge the thickener by pumping water into the overflow pond or injection well. (See Section 3.10)

Run of mine coal is reduced to 4" x 0 in the breaker building. Heavy media vessels operating at 1.40 - 1.60 specific gravity process the +3/8" wet screened plant feed, plus 1-1/4" clean coal is reduced in size by the clean coal crusher. Run of mine coal minus 3/8" x 28 mesh, after de-slimming, is pumped through heavy media cyclones, dewatered and delivered to the clean coal conveyor. Minus 28 mesh is beneficiated by froth flotation, filter dried and joins other clean coal circuits at the clean coal conveyor. Refuse from both heavy media circuits is combined with the minus 28 mesh filtered refuse on the refuse conveyor located on the basement floor. The refuse is conveyed to a 300 ton bin from which it is transferred by truck to the disposal area in School House Canyon.

3.4-2 Description of the Facility

The affected areas are delineated on Exhibit 3.4-1. The preparation plant has been constructed on the site of the former town of Castle Gate. The area is relatively wide and gently sloping and is covered to a large extent with fill resulting from the regrading of the townsite. Two tributaries to the Price River, Barn and School House Canyons intersect the preparation plant; the refuse disposal area is located in School House Canyon.

The Castle Gate area has been historically related to coal mining operations. Most of the miners and their families that worked the Utah Fuel No. 1, No. 2, No. 3, and No. 4 Mines lived in the old town of Castle Gate. Two mines and a coal preparation plant were located on or near the area of current Castle Gate Coal Company (CGCC) usage. The Ketchum Mine, located in the draw to the northeast of our guard shack, was operated from near the turn of the century to the early 1930's. The Utah Fuel No. 3 Mine, accessing the D Seam, was located just north of our water settlement pond. It opened in the early 1920's and then closed in 1937, due to flooding from the Price River. The old Utah Fuel Coal Plant, situated at the mouth of School House Canyon, began processing coal in 1938. The North American Coal Company, the owners of the facility closed the plant in 1972. In 1974, the old plant was demolished by McCulloch Oil Company.

The design of the current preparation plant was completed before the promulgation of current regulations, and the design of the refuse disposal area was completed about the time of issuance of the OSM Final Interim Regulations.

The runoff from the Castle Gate preparation plant disturbed area is channelled to one of four Sediment Ponds 011, 012A, 012B, 013. Three additional ponds on site relate to preparation facility operations. The north raw water pond is used for plant makeup and potable supply (see Exhibit 3.4-5). The two south ponds are used to clarify water used in the coal preparation process before returning it to the system. The larger settlement pond will also be used as an emergency holding pond for material from the thickener (see Exhibits 3.4-4 and 3.4-7). These three water processing ponds have little or no surface water runoff flow into them. They are non-discharging and do not require emergency spillways. In the event that the ponds must be drained for maintenance, the water will be pumped from them and channelled to one of the sediment ponds for processing before it is discharged into the Waters of the State.

The refuse disposal area is located as shown on Exhibit 3.4-1, and is designed for a capacity of about 3-1/2 million tons. As described in the Golder Associated report of January, 1978, on the detailed design of the facility, it was intended to meet applicable regulations of MESA (now MSHA), EPA, Utah Department of Natural Resources, Utah Division of Health, and with OSM Interim Regulations. The Golder report is included in this application as Appendix 3.4A.

Surface drainage from the refuse disposal area and associated affected areas is routed to a sedimentation pond in compliance with current regulations. Similarly, drainage from the haul road and associated affected area is run through the preparation plant pond system. Surface drainage from unaffected areas above the disposal area is permanently diverted into Barn Canyon.

The refuse disposal area in School House Canyon originally had an estimated life of about 7 years. Actual refuse production figures, since the design phase, lead to slightly expanded estimate. The present designed storage may be adequate until 1996. The location was chosen after a study of many possible sites in the Castle Gate area with the feasibility of 15 sites examined in considerable detail (Golder Associated Report on "Design of a Coal Refuse Disposal System, Phase I, Site Feasibility Study", September, 1977; pertinent excerpts included as Appendix 3.4B). Design of the disposal area and its associated facilities, such as the sedimentation pond and embankment, was based on accepted engineering practice and, as noted above, to comply with state and federal regulations in force at the time. The MSHA review of the facility was completed on November 17, 1977, and I.D. No. 12-1-UT-9-0027 was assigned. The details of the designs are given in the Golder Associated Report on "Design of a Coal Refuse Disposal System, Phase II: Detailed Design, School House Canyon Refuse Disposal Facility", January, 1978 (Appendix 3.4A). In actuality, the refuse material is being placed and compacted in lifts of less than 2 feet in thickness.

The Golder Report recommended that additional stability analyses be performed on actual refuse materials sometime during the early stages of pile construction. Such analyses were performed during March of 1983 by Horrocks Engineers. Their report is included as Appendix 3.4C.

Access to the area is along ramps constructed on the face. Inter-ramp slopes will be constructed at angles of 2:1, which means that the overall slope of the face of the dump will be somewhat flatter than 2:1.

Inspections of the refuse pile will be made quarterly by an Professional Engineer, or specialist who is qualified to perform inspections on refuse piles. These inspections will continue until the refuse pile is finally graded and revegetated. These inspections will check for: signs of instability, proper drainage, combustible material, and check piezometers (2) for depth of water. Quarterly inspection reports are kept at the mine site.

Permanent survey monuments will be installed as the refuse pile is constructed. These monuments will be checked annually to detect any movement of the refuse pile.

A report will be submitted to the Division of Oil, Gas and Mining (DOGM) on an annual basis and certified by a registered Professional Engineer that the refuse pile is stable, not burning, and is being constructed according to the approved plan.

3.4-3 <u>Environmental Protection: Drainage Controls and Sanitary Facilities</u>

3.4-3(1) <u>Drainage Controls</u>

The existing facilities within the Castle Gate area were constructed in a manner which minimizes changes to the prevailing hydrologic balance. Effluent limitations set by R645-301-742.220 and present NPDES Permit limitations will not be exceeded if the discharge is the result of a precipitation event from the 10-year 24-hour storm or smaller.

Contributions of sediment to the Price River are minimized by diverting drainage from undisturbed areas away from the site. In addition, existing sedimentation ponds collect disturbed area surface runoff, and a system of berms and ditches around and within the disturbed areas ensure that disturbed-area flows do not mix with undisturbed-area flows.

Design criteria for sediment control structures, diversions, and culverts comply with the requirements set forth in R645-301-742. Methods used in hydrologic calculations are described in Section 7.2.2.

3.4-3(2) Storm Runoff Calculations

Peak discharge rates from the undisturbed and disturbed area drainage of the Castle Gate area were calculated for use in determining the adequacy of the existing diversion ditches and culverts. As described in Chapter 7, the storm runoff calculations for the temporary diversion structures were based on the 10-year 6-hour storm event of 1.4 inches of precipitation (Miller et. al., 1973).

The disturbed and undisturbed drainage areas for the Castle Gate area are presented on Exhibit 3.4-2. Those drainage areas too large to fit on Exhibit 3.4-2 can be found on Exhibit 7-3. Each drainage area is labeled according to the mine area, watershed, and whether it is disturbed or undisturbed. Any watershed contributing to a sedimentation pond was labeled as being disturbed.

Curve numbers were estimated from vegetation data presented on Exhibits 9-1 and by field observations. The north-facing slopes of the Castle Gate area are primarily vegetated with conifers and mixed brush. South-facing slopes are primarily vegetated with juniper and pinion, and mixed brush. Approximate vegetation cover densities are estimated from values contained in Chapter 9. Based on this information, tables provided by the U.S. Soil Conservation Service (1972), and professional judgement, curve numbers were estimated to vary from 75 to 82 for the undisturbed areas. A curve number of 90 was typically assumed for completely disturbed areas.

A summary of the runoff calculations is presented in Table 3.4-1. All runoff calculations are contained in Appendix 3.4D.

3.4-3(3) Diversion Structures

Diversion structures within the Castle Gate area include drainage ditches and culverts to convey storm runoff from disturbed and undisturbed drainage areas, and berms to contain disturbed-area drainage. These diversion structures are located on Exhibit 3.4-2.

The diversion cross sections approximate either a trapezoidal or triangular shape. Calculations supporting the design of the diversions identified on Exhibit 3.4-2 are contained in Appendix 3.4E. In addition, a summary of ditch geometry is presented in Table 3.4-2, and a summary of berm geometry is presented in Table 3.4-3.

The capacity of the diversion ditches was determined by calculating the normal depth of flow based on a minimum ditch slope. The maximum flow velocity and riprap D_{50} was calculated based on the maximum ditch slope. Ditch slopes were measured in the field or from a contour map of the Castle Gate area with a scale of 1 inch = 200 feet. A summary of minimum ditch geometries and riprap D_{50} is presented in Table 3.4-2. All ditch calculations are contained in Appendix 3.4E.

Nine culverts were installed in the Castle Gate area to divert storm runoff from the disturbed and undisturbed drainage areas. These culverts were located in the field and are identified on Exhibit 3.4-2.

The adequacy of the existing culverts to pass the design flow rate was determined using the methods defined in Chapter 7. Table 3.4-4 summarizes the peak flows associated with each culvert. All existing culverts will adequately pass the 10-year 6-hour storm. Culvert calculations are presented in Appendix 3.4F.

The slope of each culvert was measured in the field. Calculations were performed to determine the exit velocities at each culvert and the minimum riprap requirements. A summary of the culvert flow velocities and riprap sizing calculations are presented in Table 3.4-5. Culvert flow velocity computations are presented in Appendix 3.4F.

School House Canyon - Refuse Site Drainage Control - The drainage control plan for the School House Canyon Refuse Area is divided into three phases: current operation, final operation, and final reclamation. Diversions, culverts and watersheds associated with these phases are shown on Exhibits 3.4-2, 3.4-2B, 3.4-2C, and 3.4-3, respectively. Peak discharge values were calculated for each diversion for each phase. The maximum peak discharge value was then used to design each diversion channel so that each one would be adequately designed for all three phases of the mine plan. A comparison of peak discharge values, along with the maximum design discharge value for each diversion is presented in Table 3.4-20.

Peak discharge rates used to determine channel capacities and riprap sizing for the refuse area channels were calculated based on the 100-year, 6-hour precipitation event of 2.1 inches, in accordance with R645-301-746-212. The permanent channels are identified by both operational and reclamation labels. Diversion geometries are presented in Table 3.4-21. All necessary hydrologic calculations and design information for the three phases of School House Canyon are included in Appendix 3.4J.

The drainage areas used to calculate peak discharge values for the current operation phase are shown on Exhibit 3.4-2. The areas that extend beyond the borders of 3.4-2 are shown on Exhibit 7-3. Curve numbers for the current operation phase are presented in Appendix 3.4J. As feasible during the placement of refuse, the pile will be crowned near its center to minimize the drainage area to the ditches on either side of the refuse pile. However, drainage ditches CGD-6 (upper) and CGD-7 (upper) on top of the refuse pile have each been designed to handle under worst-case conditions all the flow from the top of the pile. Furthermore, the channels were designed to safely convey the peak flow calculated for the worst-case situation between current operational hydrology and final operational hydrology. Hence, CGD-7 (upper) was designed assuming final operational conditions and CGD-6 (upper) was designed assuming current operational conditions. Locations of the refuse-pile drainage ditches during current and final operational periods are shown on Exhibits 3.4-2 and 3.4-2C, respectively.

The designs for the refuse pile diversions will allow for variations in grading as additional refuse is placed on the top. However, in no case will water be allowed to form an impoundment on top of the pile.

As referred to in Appendix 3.4.A, 5.3.4 Outlet, Diversion CGD-5 has been designed and constructed to route flow around the Schoolhouse Refuse Fill as required by state and federal regulations. To minimize adverse impact outside the permit area, the discharge point has been located to route flow into an existing "gully" in Barn Canyon. Discharge at any other point within Barn Canyon would require significant amounts of surface disturbance and increase the likelihood of adverse environmental impacts to Barn Canyon.

The outlet and flowpath will be visually monitored quarterly and after significant precipitation events to evaluate the condition of the Diversion CGD-5, the discharge point and flow path in Barn Canyon. A professional engineer will establish points of reference at the discharge point and along the flow path in Barn Canyon to evaluate the hydrologic impact to Barn Canyon. Bench marks, cross sections, or other accepted engineering methods will be used to measure, record, and evaluate channel, discharge point, and flowpath conditions. Field observations will be recorded and maintained. If excessive erosion (determined on a case by case basis) occurs, vegetation, riprap, erosion netting or other methods will be implemented to provide channel protection.

Currently, there are two drainage diversions on the edges of the face of the Refuse Pile that are performing adequately, although they are not constructed to meet the design requirements for the final operation and reclamation phases. Since the mine operation is currently (1994) dormant, it is not reasonable to replace these diversions until the Preparation Plant starts processing coal again. Calculations verifying that the upper sections of diversions CGD-7 (lower) and CGD-6 (lower) are adequate to pass the 100-year 6-hour storm given the current Refuse Pile topography are presented in a supplement to Appendix 3.4J. Both of these diversions are grouted to hold the riprap in place and prevent erosion. The upper section of CGD-7 (lower) transitions into the permanent diversion CGD-7 (lower)/CGRD-3A as shown in Figure 3.4-12.

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During construction of the refuse pile, as refuse is added, ditches CGD-6 (upper) and CDG-7 (upper) will be graded or cut with a dozer as necessary near the canyon walls to meet the design requirements. The slopes of these ditches will generally be at a grade of approximately 2%. However, according to the designs provided in Appendix 3.4J, these slopes can vary from 1% to 2.5% for CGD-6 (upper) and 1% to 3% for CGD-7 (upper) and still meet the design criteria without the need for riprap. The alignment of both ditches will generally follow the canyon walls, with exact locations determined by equipment-operator needs and the elevation of the pile. The area between the ditches and the canyon wall will be graded to flow to the ditches, thereby preventing ponding to the degree practical.

As topsoil is removed from the canyon wall, the location of CGD-7 may be moved to accomodate construction equipment. This may occur for periods of up to 2 weeks as necessary. However, the ditch will be maintained in functional condition during such work.

The final operation phase incorporates a drainage plan for School House Canyon when the refuse pile reaches its design capacity, at the approximate elevation of 6550 feet. The drainage areas used to calculate the peak discharge values for the final operation phase are shown on Exhibit 3.4-2C. Those watersheds that extend beyond the borders of Exhibit 3.4-2C are shown in their entirety on Exhibit 3.4-2D. Curve numbers for the final operation phase are presented in Appendix 3.4J. Again, as a worst-case condition, drainage ditches CGD-6 (upper) and CGD-7 (upper) have each been designed to accommodate all of the flow from the top of refuse pile, even though the pile will be crowned in the center (see Exhibit 3.4-2C). Ditches CGD-6 (lower) and CGD-7 (lower) have likewise been designed to handle all of the flow from the face of the refuse pile, in addition to the flow from the top of the refuse pile and the adjacent watersheds. As the Refuse Pile grows, the drainage diversions on the face of the refuse will be extended after each ten foot vertical increase in pile elevation.

The final reclamation phase is based on the assumption that the refuse pile is full to design capacity, and that the disturbed area has been graded to drain, topsoiled and seeded.

The top of the refuse pile will be graded so that approximately 50% of the precipitation runoff will be conveyed to CGRD-7, and 50% to CGRD-8. The haul road will be removed during this phase, and CGRD-9 (lower) will be constructed. The drainage areas used to calculate the peak discharge values for the final reclamation phase are shown on Exhibit 3.4-3. Those areas that extend beyond the borders of Exhibit 3.4-3 are delineated on Exhibit 3.4-8. Curve numbers for the final reclamation phase are presented in Appendix 3.4J.

Appendix 3.4J also contains calculations for riprap and filter blanket volumes for permanent stream channels. The thickness, and thus the volume, of the riprap for each channel is related to the average proposed riprap stone diameter. For channels with maximum longitudinal slopes of less than 10%, the method developed by the U.S. Department of Transportation (1967) was used to determine the average riprap particle size (D_{50}). The proposed thickness of the riprap in these channels is twice the D_{50} dimension, as recommended by Barfield et al. (1981). Riprap for permanent channels with slopes exceeding 10% was sized based on the steep slope channel design methodology presented by Simons, Li & Associates (OSM/TR-82/2, 1982). In these cases, the riprap volume is based on a thickness of 1.25 times the calculated D_{50}. Filter blanket volumes are based on a thickness equal to one half the riprap thickness, but not less than six inches (Barfield et al., 1981).

The reclamation channels along the edge of the Refuse Pile will cross numerous terraces planned for the face of the pile. In these locations, the channel slope will transition from steep to mild, and then back to steep. To prevent scouring at channel transitions, Simons, Li & Associates (OSM/TR-82/2, 1982) recommends that steep slope riprap extend a minimum of 15 feet beyond the transition to a mild slope, and be placed a minimum of 15 feet above the start of a steep slope section of a channel. Since the terraces on the face of the pile are only about 40 feet wide, the riprap sized for the steep slopes will be used along the entire length of these channels.

A summary of riprap and filter blanket volumes for permanent School House Canyon channels is presented in Table 3.4-22. The riprap and filter blanket gradation designs for diversion CGRD-3a are presented in Appendix 3.4J. Methodologies used to develop these design gradations are explained in Chapter 7.

Only one culvert in the School House Canyon refuse area will be used throughout the current and final operation phases of the mine. As shown on Exhibit 3.4-2, culvert CGC-4 conveys runoff from diversion CGD-19 under the Refuse Haul Road to Pond 013. Design calculations using the 100-year 6-hour storm event (R645-301-746.212) indicate that a 24 inch corrugated metal pipe (CMP) culvert with an improved inlet will pass the 100-year 6-hour design flow. The improved concrete inlet was constructed in August 1994. To eliminate erosion of the steep slope at the outlet of the 24 inch CMP, an 18 inch diameter high density polyethylene (HDPE) culvert was attached to the CMP culvert using a 45° CMP elbow and CMP transition section. The culvert extension terminates approximately at the 60% sediment cleanout level in Pond 013 (elevation 6245.5, as shown on Exhibit 3.4-13). Several acceptable options for addressing erosion at the base of the HDPE culvert were evaluated, including the use of a 30 inch diameter half-round CMP culvert, a stilling basin, and large riprap. Large riprap (2 feet to 4 feet in diameter) has been placed at the base of the HDPE culvert. If the large riprap does not prevent appreciable erosion, then one of the other previously evaluated options, or another appropriate solution, will be implemented to minimize erosion at the outlet of the HDPE culvert. Depending on the water level in the pond, the water itself will dissipate the energy in the flow exiting the HDPE, thereby preventing erosion. However, if any scouring of the sediment in the base of the pond does occur, it will not affect the stability of the embankment or inslopes of the pond. Erosion will also not adversely affect water quality downstream of the pond, since the pond is not likely to discharge naturally. Tables 3.4-23, 3.4-24, and 3.4-25 summarize the design parameters associated with culvert CGC-4.

3.4-3(4) Sedimentation Ponds

Sedimentation Ponds 011, 012A, 012B, and 013 are located in the Castle Gate area and control the storm runoff from the disturbed drainage areas at the site. Survey of Pond 013 was conducted in April 1990 by Bruce Ware (Registered Land Surveyor) of Price, Utah. Horizontal and vertical control bench marks were not available for the survey, so approximations of actual coordinates and elevations were made. The other three ponds, 011, 012A and 012B were reconstructed in September-October of 1991 and resurveyed by a Professional Engineer. A description of the construction methods and the certification of the as-built surveys of Ponds 011, 012A and 012B are contained in Appendix 3.40. Horizontal and vertical control bench marks were not available, so initial coordinates and elevations were assumed, relative to an assumed elevation of the dam. The existing topography and cross sections for Ponds 011, 012 (A and B), and 013 are shown on Exhibits 3.4-11, 3.4-12, and 3.4-13, respectively. Sediment removal from the sedimentation ponds will be performed when the sediment reaches the 60% cleanout level. Prior to sediment transport, the sediment will be tested to determine if it contains any acid and/or toxic forming compounds. The sediment will then be transported to the Refuse Pile and deposited.

3.4-3(4)A <u>Pond 011</u>

The sediment storage volume of 1,193 cubic feet (0.027 acre-feet) was calculated as indicated in Appendix 3.4G using methods described in Chapter 7. The storm runoff volume from the 10-year 24-hour storm event is 42,370 cubic feet (0.973 acre-feet). The computation of the runoff volume assumed a drainage area of 12.6 acres and a curve number of 90 for the disturbed area. No undisturbed areas contributed to the pond.

From the stage-capacity curve for the pond structure contained in Appendix 3.4G, the allowable storage at the primary spillway elevation (97.0 ft) is approximately 43,563 cubic feet. Therefore the pond will fully contain the runoff from the 10-year, 24-hour storm event, as required by R645-301-742.221.33 (DOGM, 1992), and allow for sediment storage.

The pond topography and cross sections are presented in Exhibit 3.4-11. A summary of the stage-area and stage-capacity data for the pond are contained in Table 3.4-6. The stage-capacity curve for the pond design is presented in Appendix 3.4G.

The 25-year 6-hour storm was routed through the primary spillway to determine the maximum stage and flow rate. Computations were conducted assuming that the pond contained the maximum allowable sediment volume of 1,193 cubic feet (0.81 years). It was further assumed that the pond was full of water up to the spillway flowline prior to the start of the design runoff event. This results in a conservative estimation of the maximum stage since, in general, the pond can be assumed to be empty at the beginning of a storm event.

From the analysis of the 25-year 6-hour storm event, the maximum inflow rate to the pond structure is 8.37 cubic feet per second (cfs) and the maximum outflow rate is 5.25 cfs. The corresponding high water elevation is 97.8, 1.2 foot below the minimum embankment elevation of 99.0 feet. Thus, Pond 011 will adequately pass the 25-year 6-hour peak flow.

An emergency spillway has been added to Pond 011 during reconstruction based on R645-301-742.223 (DOGM, 1990). The crest of the emergency spillway is located one foot above the primary spillway flowline. The spillway has a 6-foot bottom width and 2H:1V side slopes. A typical section of the emergency spillway is presented in Exhibit 3.4-11.

The performance of the emergency spillway was evaluated in the event the primary spillway becomes inoperative. The 25-year 6-hour storm was routed through the emergency spillway assuming that the pond was initially full of water to the elevation of the emergency spillway when the storm occurred. A stage-discharge curve was calculated by SEDCAD for the emergency spillway. The SEDCAD input and output is contained in Appendix 3.4G. From the final (emergency spillway only) analysis of the 25-year 6-hour storm event, the maximum discharge out of the emergency spillway is 6.59 cfs with a maximum flow elevation of 98.6 (0.4 foot below the minimum embankment elevation).

The outlet of the primary spillway was evaluated to determine the suitability of the existing riprap. With a culvert slope of 1.5% and a peak discharge rate of 5.25 cfs during the 25-year 6-hour storm, the exit velocity was calculated to be 5.96 feet per second (fps). Riprap with a median diameter of 6 inches is necessary to prevent erosion at the outlet of the CMP spillway. The flow velocity and riprap sizing calculations are presented in Appendix 3.4G.

The emergency spillway was evaluated to determine the necessity of riprap on the outlet slope. With a channel slope of 0.33 ft/ft, a Manning's roughness coefficient of 0.035 and a maximum discharge rate of 6.59 cfs during the 25-year 6-hour storm (emergency spillway only outflow), the flow velocity was calculated to be 6.76 fps. An average riprap diameter of 5 inches is required for this flow velocity.

The inlet channels to Pond 011 were evaluated to determine the adequacy of the existing riprap and capacity of the channels during the 25-year 6-hour storm event. The calculations for the inlet channels are presented in Appendix 3.4G. Based on the minimum channel slopes, the two channels have adequate capacity. Based on the maximum channel slopes, the flow velocity is 7.9 fps in the north inlet channel and 8.8 fps in the south inlet channel. These velocities require median riprap diameters of 6 inches and 9 inches, respectively.

According to R645-301-742.221.34 (DOGM, 1992), ponds sedimentation ponds require a non-clogging dewatering device. Because the pond is incised, the elevation of the flowline of the dewatering device would be below the adjacent topography and the water would not drain. Therefore, the pond will be dewatered using a portable pump system. The inlet structure to the pump will float on the surface of the water. The pump system will include an oil skimmer to prevent floating matter from being discharged from the pond during dewatering. The pond will be dewatered to elevation 82.6, the maximum sediment storage elevation. Prior to dewatering, the impounded water will be sampled and tested to insure that it meets NPDES discharge requirements.

Sediment removal will be performed when the sediment reaches an elevation of 82.0, which corresponds to 60% of the maximum design sediment volume.

3.4-3(4)B **<u>Ponds 012A and 012B</u>**

The sediment storage volume for Pond 012A of 3,812 cubic feet (0.088 acre-feet) was calculated as indicated in Appendix 3.4H using methods described in Chapter 7. The storm runoff volume from the 10-year 24-hour storm event is 52,393 cubic feet (1.203 acre-feet). Thus, the minimum required capacity of the pond at the elevation of the primary spillway must be 56,205 cubic feet (assuming the spillway does not spill during the 10-year 24-hour storm).

From the stage-capacity curve for Pond 012A contained in Appendix 3.4H, the allowable storage at the primary spillway elevation (97.3 feet) is 56,205 cubic feet. Therefore, the pond will fully contain the 10-year 24-hour storm event.

The elevation of the maximum sediment storage level for Pond 012A is 92.6 feet (4.7 feet below the spillway flowline).

The sediment storage volume for Pond 012B of 9,518 cubic feet (0.219 acre-feet) was calculated as indicated in Appendix 3.4H using methods described in Chapter 7. The storm runoff volume from the 10-year 24-hour storm event is 43,605 cubic feet (1.001 acre-feet). Thus, the minimum required capacity of the pond at the elevation of the primary spillway should be 53,123 cubic feet (assuming the spillway does not spill during the 10-year 24-hour storm).

From the stage-capacity curve for Pond 012B structure contained in Appendix 3.4H, the allowable storage at the primary spillway elevation (91.0 feet) is 53,123 cubic feet. Therefore the pond will fully contain the 10-year 24-hour storm event.

The pond topography and cross sections are presented in Exhibit 3.4-12. A summary of the stage-area and stage-capacity data for Ponds 012A and 012B are contained in Table 3.4-7A and 7B, respectively. The stage-capacity curves for the two ponds are presented in Appendix 3.4H.

A riprap lined open channel spillway was constructed for Pond 012B. Based on R645-301-742.223 (DOGM, 1992) only one spillway is required. The spillway has a bottom width of 7 feet and 2H:1V side slopes. The spillway crest elevation is 91.0 feet. The spillway location is presented on Exhibit 3.4-12.

The 25-year 6-hour storm event (1.6 inches of precipitation) was used to determine the adequacy of the primary spillways of both Ponds 012A and 012B. The calculation methods used are described in Chapter 7. The calculations for sedimentation Ponds 012A and 012B are contained in Appendix 3.4H.

The 25-year 6-hour storm was routed through the primary spillways to determine the maximum stage and flow rate. Computations were conducted assuming that the pond contained the maximum allowable sediment volume in each pond. In addition, the computer software program SEDCAD assumes that the ponds are full of water up to the spillway elevation at the beginning of the storm event. This results in a conservative estimation of the maximum stage since, in general, the pond can be assumed to be empty at the beginning of a storm event.

Using the above assumptions, SEDCAD calculated a maximum inflow rate of 9.85 cfs and a maximum outflow rate of 5.85 cfs for Pond 012A (see Appendix 3.4H). The corresponding high water elevation is 97.9 FEET, 0.6 foot above the primary spillway flowline and 2.1 feet below the minimum embankment elevation of 100.0 feet. Therefore, the pond and the primary spillway on Pond 012A are adequate to pass the 25-year 6-hour storm event. The pond is considered adequate to meet the requirements of R645-301-742.220.

An emergency spillway was constructed at the downstream end of Pond 012A in accordance with R645-301-742.223. The emergency spillway is a riprap lined open channel with a 6-foot bottom width and 2H:1V side slopes. The spillway crest elevation is 98.3.

The performance of the emergency spillway was evaluated in the event the primary spillway becomes inoperative. The 25-year 6-hour storm was routed through the emergency spillway assuming that the pond was initially full of water to the elevation of the emergency spillway when the storm occurred. A stage-discharge curve was calculated by SEDCAD for the emergency spillway. The SEDCAD input and output is contained in Appendix 3.4H. From the final (emergency spillway only) analysis of the 25-year 6-hour storm event, the maximum discharge out of the emergency spillway is 5.70 cfs with a maximum flow elevation of 98.8 (1.2 feet below the minimum embankment elevation).

As indicated in Appendix 3.4H, SEDCAD calculated a maximum inflow rate of 14.32 cfs and a maximum outflow rate of 12.30 cfs for the Pond 012B structure. The corresponding high water elevation is 91.8 feet, 1.0 feet below the minimum embankment elevation of 92.8 feet. Therefore, the Pond 012B and primary spillway are adequate to pass the 25-year 6-hour storm event (R645-301-742.220).

Calculations using the 25-year 6-hour storm to determine the minimum size riprap required for the inlet and outlet channels of Pond 012A are presented in Appendix 3.4H. Pond 012A has two inlets, a one foot diameter CMP culvert and an open trapezoidal channel. The steep slope of the culvert, and the steep slope at the end of the culvert, necessitates riprap with an average diameter of 16 inches. The open channel inlet carries the majority of the water to Pond 012A (9.85 cfs) during a 25-year 6-hour storm, but the shallow slope of 3% requires riprap of only one inch average diameter. In fact, if the inlet channel is reasonably vegetated, no riprap is necessary. The primary spillway is an 18 inch culvert that slopes at

10.0% between Pond 012A and Pond 012B. The peak design discharge rate of 5.85 cfs results in an exit velocity of 9.1 fps. Riprap with a median diameter of 7 inches is required to prevent erosion at the end of the spillway culvert.

The emergency spillway on Pond 012A has a bottom width of 6 feet side slopes of 2H:1V, and a channel slope of 7%. The peak discharge rate of 5.70 cfs (as determined by the "emergency spillway only" SEDCAD run) results in a velocity of 4.2 fps. Based on the calculations presented in Appendix 3.4H, riprap with a median diameter of 2 inches is required.

The inlet channel to Pond 012B conveys 14.32 cfs during a 25-year 6-hour storm event. The slope of the channel is only 3%, resulting in a flow velocity of 5.0 fps. Two inch diameter riprap is required to prevent erosion along the base of the channel. The open channel spillway on Pond 012B has a bottom width of 7 feet, side slopes of 2H:1V, and a channel slope of 50%. The peak discharge rate of 12.30 cfs results in a peak velocity of 8.77 cfs. Based on calculations presented in Appendix 3.4H, a median riprap size of 9 inches is required for this spillway structure.

In accordance with R645-301-742.221.34 (DOGM, 1992), Ponds 012A and 012B each have a non-clogging dewatering device. The flowline of the dewatering device in Pond 012A was installed at elevation 93.0, 0.4 feet above the maximum sediment storage elevation. The remaining 0.4 foot of water in Pond 012A will be dewatered using a pump system. The inlet structure to the portable pump will float on the surface of the water. An oil skimmer will be attached to the float to prevent floating matter from being discharged from the pond during dewatering. The flowline of the dewatering device in Pond 012B was installed at elevation 86.1, the maximum sediment storage elevation. Refer to Exhibit 3.4-12 for a typical section of the decant system.

Sediment removal will be performed when the sediment level in Pond 012A reaches an elevation of 92.4, and at elevation 85.4 in Pond 012B. These elevations correspond to 60% of the maximum design sediment volume.

3.4-3(4)C Pond 013

The stage-area and stage-capacity data for Pond 013 were determined from the pond topography contained in Exhibit 3.4-13. A summary of these data is contained in Table 3.4-8. The stage-area and stage-capacity curves for Pond 013 are presented in Appendix 3.4I.

The required 3-year sediment storage volume of 72,235 cubic feet (1.658 acre-feet) was calculated as indicated in Appendix 3.4I using methods described in Chapter 7. The storm runoff volume from the 10-year 24-hour storm event is 138,595 cubic feet (3.182 acre-feet). The computation of the runoff volume assumed a drainage area of 79.4 acres and a weighted curve number of 81 for the disturbed and undisturbed areas. Thus, the minimum capacity of the pond at the elevation of the spillway must be 210,830 cubic feet (assuming the spillway does not spill during the 10-year 24-hour storm).

From the stage-capacity curve contained in Appendix 3.4I, the allowable storage at the spillway elevation (6,255.0 ft) is approximately 396,000 cubic feet. Therefore, additional volume is available for sediment storage. Subtracting the runoff volume from the existing pond capacity at the spillway results in a maximum sediment storage capacity of 257,405 cubic feet (5.909 acre-feet). The elevation of the maximum sediment storage level at this capacity is 6,250.2 feet (4.8 feet below the spillway). Based on this storage volume, the 60% clean-out volume for Pond 013 is 154,443 cubic feet (3.546 acre-feet). The 60% clean-out elevation is 6,245.5 feet (9.5 feet below the spillway).

The 25-year 24-hour storm event (2.3 inches of precipitation (Miller, et. al., 1973)) was used to determine the adequacy of the spillway (a riprap lined trapezoidal channel spillway). These calculations are presented in Appendix 3.4I. The calculation methods used are described in Chapter 7.

The 25-year 24-hour storm was routed through the spillway to determine the maximum stage and flow rate. Computations were conducted assuming that the pond contained the maximum allowable sediment volume of 257,405 cubic feet. In addition, the computer software program SEDIMOT II assumes that the pond is full of water up to the spillway elevation at the beginning of the storm event. This results in a conservative estimation of the maximum stage since, in general, the pond can be assumed to be empty at the beginning of a storm event.

From the analysis of the 25-year 24-hour storm event, the maximum inflow rate to the pond is 49.37 cfs and the maximum outflow rate is 31.78 cfs. The corresponding high water elevation is 6,256.3, 2.7 feet below the minimum embankment elevation of 6,259.0 feet. Thus, Pond 013 will meet the storage and flow requirements of R645-301-742.200.

Based on R645-301-742.22 (DOGM, 1990), the pond must also pass the 100-year 6-hour storm event. This storm event of 2.0 inches was routed through the spillway to determine the maximum stage and flow rate. The pond was assumed full of sediment up to the maximum sediment level, and full of water up to the spillway flowline. From the analysis of the 100-year 6-hour storm event, the maximum inflow rate to the pond is 35.95 cfs and maximum outflow rate is 19.5 cfs. The corresponding high water elevation is 6,256.1, 2.9 feet below the minimum embankment elevation of 6,259.0 feet. Thus, Pond 013 will adequately pass the 100-year 6-hour precipitation event.

The inlet channels to Pond 013 were evaluated to determine the adequacy of the existing riprap and capacity of the channels during the 25-year 24-hour storm event. The calculations for the inlet channels are presented in Appendix 3.4I. Based on the minimum channel slopes, the two channels have adequate capacity. Based on the maximum channel slopes, the flow velocity is 12.9 fps in the west inlet channel and 8.1 fps in the east inlet channel. These velocities require median riprap diameters of 16.8 inches and 7.2 inches, respectively. The existing median riprap size of 12 inches is adequate for the east inlet channel. The 12-inch median riprap diameter in the west inlet channel is undersized based on the 25-year 24-hour storm. The flow velocity of the west inlet channel was reevaluated, based on the 25-year 6-hour storm, to be 10 fps. This flow velocity requires a median riprap diameter of 9.6 inches. Therefore, the existing riprap for the west inlet channel is adequate.

The outlet of the primary spillway was evaluated to determine the suitability of the existing riprap. With a maximum channel slope of 47% and a peak discharge rate of 31.78 cfs during the 25-year 24-hour storm, the exit velocity was calculated to be 11.4 fps. The existing median riprap diameter of 18 inches is adequate for this flow velocity. The flow velocity and riprap sizing calculations are presented in Appendix 3.4I.

According to R645-301-742.221.34 (DOGM, 1990), a non-clogging dewatering device must be installed in the pond. Because the pond does not require reconstruction, it will be dewatered using a pump system. The inlet structure to the portable pump will float on the surface of the water. The system will include an oil skimmer to prevent floating matter form being discharged from the pond during dewatering. The pond will be dewatered to elevation 6250.2, the maximum sediment storage elevation.

Sediment removal will be performed when the sediment reaches an elevation of 6250.2, which corresponds to 60% of the maximum design sediment volume.

3.4-3(5) Pond Embankment Stability Analyses

3.4-3(5)A General

Both the inslopes and outslopes of the embankments of Ponds 011 and 012 at the Preparation Plant of the Castle Gate Mine were analyzed for long term stability. These analyses was performed to address the requirements of R645-301-733.210 and R645-301-533.100, which stipulate that all embankments not under the jurisdiction of the Mine Safety and Health Administration (MSHA) shall have a minimum static factor of safety of 1.3.

A field survey of the pond embankments at the Preparation Plant was conducted to ascertain the most likely location of possible embankment failure. The field survey consisted of visually evaluating the embankments and noting specific slope geometry characteristics. Soil samples were taken from the embankments for later visual classification.

Since lab testing of soil sampled from the embankments is not included in the scope of these analyses, soil properties were assumed. The bases for those assumptions were visual classification of soil samples and typical soil properties presented by Hoek (1981) and NAVFAC DM-7 (1971). Soil parameter assumptions made in this analysis are generally conservative because of the absence of lab data.

Based on information gathered during the field survey and the results of visual classification of the soil samples, the slope stability computer software program GEOSLOPE (GEOCOMP, Inc.) was utilized to determine an in-situ factor of safety for each of the embankments. The resulting computer output is contained in Appendices 3.4P-1, 3.4P-2 and 3.4P-3.

GEOSLOPE is a computer program based on the FORTRAN program STABL3 which was developed at Purdue University. GEOSLOPE utilizes the limit equilibrium procedure of slices to determine the safety factor of potential circular failure surfaces by the Modified Bishop's Method. Both deep failure surfaces and surfaces that generally pass through the toe of the embankments were analyzed. Only the analysis that produced the lowest factor of safety for each embankment is included in Appendix 3.4P-1 through 3.4P-3

3.4-3(5)B <u>Pond 011</u>

Pond 011 is located near the west end of the Preparation Plant site. The pond is primarily incised, although it does have a small embankment on the side of the pond closest to diversion CGD-3. The critical sections that were analyzed are shown on Exhibit 3.4-11.

The geometry of the Pond 011 outslope embankment was modeled with a 100 foot section consisting of a 15° outslope from the centerline of diversion CGD-3, an embankment 19 feet in width at the top, and an inslope of 43° (Section C - C′ on Exhibit 3.4-11). The embankment is composed primarily of silty sand. The assumed soil strength parameters are identified in Table 3.4-19. The phreatic surface was assumed to be at the ground surface at the toe of the outslope, and at 2.0 feet below the top of the embankment on the inside of the embankment. This corresponds to the maximum water level in the pond during a 25-year 6-hour storm event, assuming the pond is full of water up to the level of the spillway flowline at the beginning of the storm. See Section 3.4-3(4) for a description of the methods used to determine that water surface elevation.

The existing embankment is stable with a factor of safety of 4.04. See Appendix 3.4P-1 for GEOSLOPE computer results.

A 50 foot, 40° section was analyzed for inslope stability, as depicted by section cut D - D′ on Exhibit 3.4-11. The phreatic surface was assumed to be horizontal at the maximum 25-year 6-hour storm event level of 97.8. The pore pressure parameters were assumed to be equal to zero since it is anticipated that the pore pressures will dissipate quickly during pond dewatering, due to the granular nature of the soil. A summary to the soil strength parameters are listed in Table 3.4-19, and a sketch of the section geometry is included in Appendix 3.4P-1.

The calculated factor of safety for Section D - D' is 1.16. This is less than the factor of safety of 1.30 required by R645-301. There are several constraints, such as an existing road and channel diversions, in the immediate vicinity of Pond 11 which preclude the relaxation of the steep interior slopes to achieve a larger factor of safety. In the event of any sloughing of material on the inside of the pond, the material will be removed so as to maintain the design volume capacity. See Appendix 3.4P-1 for GEOSLOPE computer output results.

3.4-3(5)C <u>Pond 012A</u>

Pond 012A is located toward the east end of the Preparation Plant site. The pond is entirely incised and thus no outslope stability analysis was performed on this pond.

A 65 foot, 49.6° section was analyzed for inslope stability, as depicted by section cut G - G' on Exhibit 3.4-12. The phreatic surface was assumed to be horizontal at the maximum 25-year 6-hour storm event level of 97.9. The pore pressure parameters were assumed to be equal to zero since it is anticipated that the pore pressures will dissipate quickly during pond dewatering, due to the granular nature of the soil. A summary to the soil strength parameters are listed in Table 3.4-19, and a sketch of the section geometry is included in Appendix 3.4P-2.

The calculated factor of safety for Section G - G' is 1.20. This is less than the factor of safety of 1.30 required by R645-301. The disturbed area of the Preparation Complex is quite narrow in the vicinity of Pond 12A, and the road adjacent to the pond will not allow for a relaxation of the steep interior slope to achieve a larger factor of safety. In the event of any sloughing of material on the inside of the pond, the material will be removed so as to maintain the design volume capacity. See Appendix 3.4P-2 for GEOSLOPE computer output results.

3.4-3(5)D Pond 012B

Pond 012B is also located toward the east end of the disturbed area of the Preparation Plant site. A ten foot high embankment forms the entire south side of Pond 012B. The critical sections that were analyzed are shown on Exhibit 3.4-12.

The geometry of the Pond 012B outslope embankment was modeled by a 70 foot section through a 45° outslope on the south side of the pond, an embankment 9 feet in width at the top, and an inslope of 39°. The embankment is composed primarily of silty sand with some gravel. The selected soil strength parameters are identified in Table 3.4-19. The phreatic surface was assumed to be at the ground surface at the toe of the outslope, and at 1.0 feet below the top of the embankment on the inside of the embankment. This corresponds to the maximum water level in the pond during a 25-year 6-hour storm event, assuming the pond is full of water at the beginning of the storm. See Section 3.4-3(4) for a description of the methods used to determine that water surface elevation.

The existing embankment is stable with a factor of safety of 1.68. See Appendix 3.4P-3 for GEOSLOPE computer results.

An 80 foot, 41° section was analyzed for inslope stability, as depicted by section cut H - H' on Exhibit 3.4-12. The phreatic surface was assumed to be horizontal at the maximum 25-year 6-hour storm event level of 91.8. The pore pressure parameters were assumed to be equal to zero since it is anticipated that the pore pressures will dissipate quickly during pond dewatering, due to the granular nature of the soil. A summary of the soil strength parameters are listed in Table 3.4-19, and a sketch of the section geometry is included in Appendix 3.4P-3.

The inslope is stable with a factor of safety of 1.46. See Appendix 3.4P-3 for GEOSLOPE computer output results.

3.4-3(5)E Pond 013

In 1977, Golder Associates of Kirkland, Washington analyzed the embankment of Pond 013 and deemed it "stable under all conditions of operation." MSHA subsequently assigned the entire refuse disposal facility, including Pond 013, an identification number of 12-1-UT-9-0027.

3.4-3(6) Sanitary Facilities

Waste water from all site buildings and the bathhouse is connected to the PRWID sewer line passing through the site.

3.4-3(7) Alternative Sediment Controls

Exhibit 3.4-2, Existing Drainage Pattern and Control Structures, identifies the areas within the disturbed area boundary which do not report to the sedimentation ponds. By definition, these areas are referred to as alternative sediment control areas. These areas and the controls utilized to control erosion are explained below, and are summarized in Table 7-8.

Rainfall runoff across the road and adjacent areas in the vicinity of the truck scale (1.6 acres) flows to a small depression alongside the railroad tracks. The majority of the Unit Train Loadout Area (0.9 acres) is naturally revegetated with grasses. In addition, rainfall landing on the structures is diverted to a storm runoff tank, as explained in Section 3.8. The area immediately adjacent to the Raw Water Pond is also naturally revegetated. Any erosion within the pond embankments is trapped within those embankments (1.75 acres). Drainage is properly controlled along the road north of the Raw Water Pond (0.6 acres), in accordance with R645-301-742.400.

3.4-4 Reclamation Plan

3.4-4(1) Reclamation Work

The preparation plant is designed to remain in use until the minable reserve base is depleted, a minimum of 25 years. Reclamation on the 74 acre site could potentially begin as early as 2015 depending on the depletion of the reserves. Reclamation of the School House Canyon refuse site will begin as soon as the canyon is filled to its design capacity, which will not be during this first renewal period of 1989-1994.

The postmining reclamation topography plan for the Castle Gate area is shown on Exhibit 3.4-3. The reclamation work consists of the following:

Phase I Reclamation

Demolition - All the existing structures which lie within the disturbed area boundary will be removed, including the beltline structures, as explained in Adit No. 1 Section 3.5-4(1). However, utilities within the utility corridor, along with a buried telephone cable parallel with the utility corridor, will remain. Water supply intakes serving the Preparation Plant outside the disturbed area boundary (Exhibit 1-1) will remain, while the piping within the disturbed area boundary and outside the utility corridor will be removed. In addition, the culverts identified on Exhibit 3.4-3 will remain. Removing these culverts and replacing them with permanent reclamation stream channels could possibly leave sections of the underground utilities in the utility corridor exposed. Since this is not acceptable, these culverts must remain in place indefinitely.

Portal Sealing - There are no portals to seal at the Castle Gate Plant.

Grading - Grading work will be done in order to establish overland flow drainage and approximate the original contour. Approximate original contour is achieved by blending the spoil material into the adjacent area and creating landforms which resemble the surrounding topography. The mass balance calculations associated with the grading are presented in Table 3.4-9. Exhibit 3.4-10 indicates the distribution of cuts and fills related to the grading plan.

Although several of the cuts slopes will be backfilled, a few cut slopes will not. The cut slopes to remain indefinitely are identified on Exhibit 3.4-3A. The cut slopes were analyzed in their present configuration by a consulting firm, EarthFax Engineering, Inc. (EarthFax), for stability and retention as approximate original contour. The cross sections used to analyze the slopes are shown on Exhibit 3.4-2A. The analysis was prepared in conjunction with the postmining reclamation plan. A copy of the EarthFax report is located in Appendix 3.4K. Section 4.0 of the EarthFax report documents that the calculated factor of safety for the retained cut slopes exceeds the minimum static factor of safety of 1.3 stipulated by R645-301-553.130. In addition, Section 3.6. of the EarthFax report documents the existence of natural cliffs and ledges in the Castle Gate Area. The conclusion presented in Section 5.0 states that the cut slopes are similar in structural composition and geometry to the naturally existing cliff/ledge formations and thus are compatible with the surrounding topography.

During the Phase I grading process, the following work will be performed:

1) Elimination of berms and temporary diversions, except where noted.
2) Grading to establish overland flow drainage where possible.
3) Construction of permanent stream channels.
4) Removal of existing culverts, except as noted.
5) Removal of Pond 012A.
6) Enlargement of Ponds 011 and 012B (renamed 012).
7) Installation of silt fences.
8) Soil preparation, seeding, fertilizing and mulching.

During Phase I of reclamation, several berms and ditches which direct flow to the sediment ponds will be retained. However, many of the diversions that collect precipitation runoff from undisturbed areas will be eliminated. This necessitates enlargement of several existing ponds to function as primary sediment control structures during Phase I of reclamation. Specifically, Sediment Ponds 011 and 012B will be enlarged, while Pond 013 will remain at its current size for Phase I of reclamation. The enlarged Pond 012B has been renamed Pond 012 for Phase I of reclamation. Pond 012A will be eliminated. See Exhibit 3.4-3A for the Phase I plan for sediment ponds. Exhibits 3.4-9A and 3.4-9B consist of more detailed plans to enlarge Ponds 011 and 012B (now Pond 012), along with pertinent design data. Appendix 3.4M contains the engineering calculations supporting the need to enlarge Ponds 011 and 012. Ponds 011, 012, and 013 will be retained for two years or until adequate vegetation is established to control erosion.

The reclamation of the Castle Gate Preparation Plant area will take place over the area which was the old town site of Castle Gate. Old utilities, foundations and debris may be uncovered during the grading operation. This may result in the alteration of the contours shown on map 3.4-3 by as many as two contour intervals in order to keep from uncovering the old town site. Much of the foundation debris will be used as deep fill layers against the cut slope just east of the existing Thickener Ponds.

Phase I of reclamation will also include the removal of all roads and culverts, except as noted, and the establishment of permanent stream relocations. Prior to removal of the asphalt covered roads, the asphalt will be collected and properly disposed of beyond the boundaries of the Castle Gate permit.

Several wells exist within the Preparation Plant disturbed area boundary. The slurry injection wells shown on Exhibit 3.10-1 will be sealed, and the area in the immediate vicinity of the wells reclaimed in accordance with the slurry injection well reclamation plan contained in Section 3.10 of this permit. Two piezometer wells below Pond 013 will be monitored during Phase I of reclamation and then sealed at the beginning of Phase II of reclamation. No other unsealed monitoring or exploration wells exist on the property.

The reclamation topography plan for the Unit Train Loadout area is shown on Exhibit 3.4-3. A discussion of the reclamation plan is included in Section 3.8.

The backfill and grading topography shown on Exhibit 3.4-3 is compatible with the postmining land use of wildlife habitat and grazing, and provides adequate drainage and long term stability as required by R645-301-553.522.

The final configuration for the refuse pile is also suitable for the approved postmining land use of wildlife habit and grazing. Terraces will be constructed on the outslope of the refuse pile which increase stability, control erosion, and conserve soil moisture. The grade on the outslope between the terrace benches will not be steeper than 2H:1V. The terraces will be approximately 40 feet wide, and slope at approximately 10%. A profile and cross-section of the face of the Refuse Pile are presented as Figures 3.4-10 and 3.4-11, respectively.

Resoiling - The 74 acres in Castle Gate which will be reclaimed were disturbed by mining activities prior to the enactment of SMCRA. No topsoil was salvaged from the site. The existing soils at the site will be used as resoiling material except at the refuse pile.

The existing soils at the Preparation Plant site have been analyzed for the parameters listed below. Sampling locations are depicted on Exhibit 8-4. Subsequent to the reclamation grading, the resoiling materials will be sampled again and retested for the same parameters. Appropriate soil amendments will be added according to results of these tests. Areas which are not anticipated to revegetate to support the intended land use once soil amendments have been added will be covered with 6" of resoiling material from the Gravel Canyon Storage Site.

The refuse pile will be covered with 24" of soil from Gravel Canyon. Approximately 96,000 cubic yards of material will be needed for this purpose. Justification for use of less than 4' of cover on the refuse pile is the nontoxic nature of the refuse. Approximately one year prior to placement of substitute topsoil from Gravel Canyon on the refuse pile, the following parameters will be evaluated on both refuse and substitute topsoil in order to prove non-toxicity and assess the necessity to add appropriate soil amendments: Ph, electrical conductivity, saturation percentage, particle size analysis, soluble Ca,Mg and Na, sodium absorption ratio, selenium, total N, nitrate-N, boron, maximum acid potential, neutralization potential, organic carbon, exchangeable sodium, available water capacity and rock fragments. The above parameters will also be checked on the existing resoiling materials throughout the site. The rate of testing will be 1 analysis for every 2.5 acres of disturbance at various depths, 0-6",6"-12", 1'-2', 2'-3', 3'-4'. Results of the tests will be forwarded to DOGM for review.

Any acid forming or toxic materials exposed during the grading operation, which may adversely affect water quality or vegetation, will be excavated and transported to the Refuse Pile, if this is feasible. Where acid and/or toxic soil cannot be readily removed, the toxic soil will be buried under four feet of topsoil. Any other methods of disposal are subject to DOGM approval prior to implementation.

Prior to placement of any borrowed material, the area will be dry and scarified to a depth of 4". After reclamation grading but prior to seeding, the soil on all slopes with grades less than 20% will be ripped to a depth of 18 to 24 inches parallel to the contours. This procedure will encourage moisture retention and reduce the surface compaction to allow for a more favorable germination environment for the vegetation. Soil ripping will not be performed on the Refuse Pile.

Seeding and Mulching - Castle Gate preparation plant will use two species mixes listed in Chapter 9. The majority of the site will be seeded with species list #1, as it is a pre-SMCRA site. The riparian areas shown on the reclamation plan (Exhibit 3.4-3A) will be seeded with species list #3. In both cases, the seed will be mixed with a small amount of wood fiber mulch, used as a tracer, and water to form slurry. The slurry will be applied to the reclaimed surfaces using a hydroseeder. The balance of the mulch, mixed with a tackifier and the fertilizer also in a slurry, will then be sprayed over the same area. The total coverage of the mulch will be a the rate of 2,000 pounds per acre. In areas inaccessible to the hydroseeder, the seed will be broadcast by mechanical means. Areas inaccessible to the hydromulch equipment will be mulched with straw and tacked with nylon or other suitable netting. The rate of application for straw will be 2,000 pounds per acre.

Phase II Reclamation

Phase II reclamation will commence once the vegetation is adequately established based on the criteria presented in Chapter 9. This phase of reclamation will consist of filling in the three sediment ponds (011, 012 and 013) and removing silt fences and accumulated soil in the vicinity of the fences. However, where removal of the silt fence fabric will substantially disrupt the established vegetation adjacent to the fence, the fabric may be cut at ground level and the buried fabric abandoned in place. All the temporary diversions and berms which were left to control runoff during Phase I reclamation will be removed. Grading will be performed to bring the site within tolerance of the postmining reclamation plan depicted in Exhibit 3.4-3.

The areas disturbed during Phase II reclamation will be seeded and mulched according to the plan described above and in accordance with Chapter 9 of this permit.

All piezometer wells will be sealed in accordance with R645-301-731.400, R645-301-631, and R645-301-765.

Phase III Reclamation

Phase III reclamation will consist of water and vegetation monitoring until bond release.

3.4-4(2) Reclamation Hydrology

Reclamation Channel Design - The reclamation channels for the Castle Gate Preparation Plant area were designed to approximate the geometry of the existing natural stream channels. The natural channel sections were measured in the field and approximated with a trapezoidal cross section. The reclamation channels were designed with a 3H:1V side slope to ensure channel stability. However, three existing stream channels, Castle Gate Reclamation Ditches CGRD-4, CGRD-5, and CGRD-10 were constructed with 1H:1V, 1.5H:1V, and 1.2H:1V side slopes, respectfully. These three ditches were previously designed for the operational hydrology of the Castle Gate area and were determined to be adequately designed for reclamation hydrology. While CGRD-4 and CGRD-5 are permanent reclamation channels, CGRD-10 will be removed when Pond 013 is removed at the end of Phase I of reclamation. The hydraulic slope of each channel was measured from a postmining topographic map (scale: 1" = 100'), presented as Exhibit 3.4-3.

All calculations supporting the designs of the reclamation hydrology structures outside of School House Canyon and CGRD-4 are presented in Appendix 3.4L. The design assumptions for the permanent School House Canyon channels are discussed in Section 3.4-3(3), and those supporting calculations are contained in Appendix 3.4J.

Curve numbers for the undisturbed drainage areas were taken from Appendix 3.4D. The reclaimed areas (CGRWS-R1, R2, R3, & R4) were assumed to have a curve number of 80. The reclamation channel drainage areas for the Castle Gate Preparation Plant Area are presented on Exhibits 3.4-3 and 3.4-8.

Peak discharge rates used to determine channel capacities and riprap sizing for the reclamation channels were calculated based on the 100-year 6-hour precipitation event of 2.0 inches for perennial and intermittent channels. All other channels were designed for the 10-year 6-hour storm event of 1.4 inches (Miller et.al, 1973). A summary of the runoff calculations is presented in Table 3.4-10. The peak discharge rates for each diversion are presented in Table 3.4-11. The reclamation channel geometries and minimum riprap sizes are presented in Table 3.4-12.

Appendix 3.4L contains calculations for riprap and filter blanket volumes for permanent stream channels. The thickness, and thus the volume, of the riprap for each channel is related to the average proposed riprap stone diameter. For channels with maximum longitudinal slopes of less than 10%, the method developed by the U.S. Department of Transportation (1967), was used to determine the average riprap particle size (D_{50}). The proposed thickness of the riprap in these channels is twice the D_{50} dimension, as recommended by Barfield et al. (1981). Riprap for permanent channels with slopes exceeding 10% was sized based on the steep slope channel design methodology presented by Simons, Li & Associates (OSM/TR-82/2, 1982). In these cases, the riprap volume is based on a thickness of 1.25 times the calculated

D_{50} (Simons, Li & Associates, 1982). Filter blanket volumes are based on a thickness equal to one half the riprap thickness, but not less than six inches (Barfield et al., 1981).

The following general approach was used during design of the reclamation channels:

- The design capacity of the perennial and intermittent reclamation channels was based on the 100-year 6-hour storm and the minimum channel slope.

- The design capacity of the ephemeral reclamation channels was based on the 10-year 6-hour storm and the minimum channel slope.

- Riprap was sized based on the 100-year 6-hour storm and the maximum channel slope for perennial and intermittent channels.

- Riprap was sized based on the 10-year 6-hour storm and the maximum channel slope for ephemeral drainage channels.

- The roughness coefficient (Manning's "n") for riprapped channels was determined according to the equation (Barfield et al., 1981):

$$n = 0.0395 D_{50}^{1/6}$$

  where, n = Manning's roughness coefficient
  D_{50} = median riprap diameter (ft)

- Designs are based on channel construction on fill. Where the reclamation channel construction occurs on rock, riprap quantities will be reduced or eliminated (depending on the competency of the rock).

- When transitioning downstream from a steep channel slope to a flat channel slope, the larger riprap from the steep section will be extended into the channel section with the flatter slope for at least 15 feet to minimize erosion (Simons, Li & Associates, 1982).

- The reclamation channels are designed to pass the peak discharge with a minimum freeboard of 1 foot.

- Where channel slopes exceed 20 percent, a small plunge pool will be constructed at the grade break to dissipate energy. This plunge pool will be lined with riprap to provide erosion protection. Refer to Figure 3.4L-3 in Appendix 3.4L for a typical plunge pool design.

A detailed riprap and filter blanket design is not presented in this text since adequate soil samples were not available. Castle Gate Coal is committed to preparing a detailed design for the riprap and filter blanket gradations. Samples will be taken once the reclamation grading has progressed sufficiently to expose the base of the reclamation channels. The riprap and filter blanket gradations for the mild slope sections of the channels will be engineered based on methods presented in Barfield et al. (1981). The procedure presented by Simons, Li & Associates (1982) will be used to design the riprap gradation for steep slope channels. This design procedure assumes that the riprap is predominately angular in shape. The filter blanket will consist of a properly graded coarse grained soil; a synthetic fabric will not be used. The detailed designs will be submitted to DOGM for approval prior to delivery of filter blanket and riprap materials to the site.

Table 3.4-15 summarizes the required riprap and filter blanket volumes for the reclamation channels located outside of School House Canyon. Total volumes and tonnage reported in Table 3.4-15 do not account for the riprap required at the base of the reclamation culverts.

Reclamation Culvert Design - Three culverts will remain for the Castle Gate reclamation plan. Castle Gate reclamation culvert (CGRC-1), is an existing 60 inch x 120 inch box culvert which will remain for Phase I final reclamation. CGRC-1 will subsequently be removed when Phase I reclamation is completed. The average riprap size required at the CGRC-1 outlet is 1 inch. CGRC-2 is an adequately designed existing 60-inch concrete culvert located under the D&RGW Railroad tracks. CGRC-2 (Operations Hydrology CGC-5) extends to the Price River and will be shortened for Phase I reclamation as shown on Exhibit 3.4-3A. An average riprap size of 39 inches will be required at the outlet. Finally CGRC-3 consists of two 84 inch CMP culverts that require an average riprap size of 30 inches at the outlet. Calculations regarding design of the Castle Gate Preparation Plant reclamation culverts are presented in Appendix 3.4L. Summaries of the reclamation culvert discharges and designs are presented in Tables 3.4-13 and 3.4-14, respectively.

3.4-4(3) Reclamation Sedimentation Ponds

During Phase I of reclamation, sedimentation ponds will be the primary means of capturing sediment erosion from the reclaimed areas designated on Exhibit 3.4-3A. Since several of the diversions channelling undisturbed area runoff around the disturbed area will be removed, some undisturbed area runoff will now contribute to the ponds. Consequently, Ponds 011 and 012B are currently undersized to accommodate approximately three years of sediment storage as well as the storm runoff from the 10-year 24-hour storm event. Those two ponds will be expanded from their current operational hydrology size, while Pond 013 need not be modified. Since Pond 012A will be removed during Phase I of reclamation, Pond 012B is henceforth referred to as Pond 012 once it is enlarged. All pond sizing calculations are contained in Appendix 3.4M. Alternative sediment controls will be utilized to trap sediment where grading does not allow the runoff to flow to a sediment pond.

Curve numbers for the undisturbed drainage areas contributing to the ponds were estimated from vegetation data presented on Exhibit 9-1, and by field observations. Cover densities for each vegetative group were estimated from information presented in Chapter 9. Curve numbers varied from 75 to 78 for the undisturbed drainage areas which contribute to the sedimentation ponds. A summary of curve numbers for those areas is presented in Appendix 3.4L. A curve number of 80 for the reclaimed areas was chosen from professional judgement and tabulated values presented by the U.S. Soil Conservation Service (1972).

The 25-year 6-hour storm event was routed through reclamation Ponds 011 and 012 to determine the adequacy of the existing spillway under reclamation conditions. The computer software SEDIMOT II was used for the routing. SEDIMOT II assumes that the pond is full of water up to the spillway/overflow elevation at the beginning of the storm event. This results in a conservative estimation of the maximum stage since, in general, the pond can be assumed to be empty at the beginning of a storm. Overflow from Pond 011 discharges to reclamation ditch CGRD-6, while Pond 012 discharges to the railroad right-of-way and flows south toward Willow Creek.

A diversion/berm system was designed to convey the runoff and sediment from the reclaimed areas to the sedimentation ponds using the 10-year 6-hour storm event. Berms will be constructed in the locations indicated on Exhibit 3.4-3A. Adjacent to each berm will be a broad swale diversion which will convey the runoff. The height of the berm will be approximately 1.0 foot, with a 1.0 foot top width and side slopes of 2H:1V. The bottom width of the swales will be 8 feet alongside all the berms. Maximum flow velocities do not exceed 2.5 fps and, therefore, riprap protection will not be required. The last entry in Table 3.4-12 summarizes the geometry of a typical berm/swale. Calculations for the berm/swale system are provided in Appendix 3.4M.

Once Ponds 011 and 012 are enlarged for Phase I reclamation and all grading and seeding have been completed, Ponds 011, 012 and 013 will serve to collect sediment for a minimum of two years. The ponds will not be removed until the removal is authorized by DOGM, vegetation over the reclaimed area has been properly established in accordance with R645-301-763.100, and the water quality bond release standards of R645-301-880.320 are complied with. Sediment will be removed from the reclamation sedimentation ponds when the sediment reaches the 60% cleanout level, as determined by reading the sediment marker in each pond. The sediment will first be evaluated to determine if it contains acid and/or toxic forming compounds, and the results forwarded to DOGM. The sediment will then be transported to a location designated within the Castle Gate Mine permit boundary.

The following summaries are provided for the proposed reclamation sedimentation pond structures. The proposed topography and cross sections for Ponds 011 and 012 are presented on Exhibits 3.4-9A and 3.4-9B, respectively.

Reclamation Pond 011 - Pond 011 was modified in 1991 for the purposes of operational hydrology sediment control. An as-built survey was performed by Dan W. Guy in October 1991, and the results of the survey are summarized in Exhibit 3.4-11. The capacity of Pond 011 at the principle overflow is currently 43,563 cubic feet.

Using reclamation parameters, an annual sediment volume of 1,653 cubic feet was calculated for Pond 011 using the methods described in Chapter 7. The storm runoff volume from the 10-year 24-hour storm event is 53,600 cubic feet (1.23 acre-feet). The computation of the runoff volume assumed a reclaimed drainage area of 11.6 acres and a curve number of 80. A curve number of 75 was used for the undisturbed areas covering 33.4 acres. Thus, Pond 011, as currently constructed, is insufficient to contain the 10-year 24-hour storm during Phase I of reclamation.

The design size of the pond has been increased by expanding it to the southeast so that the existing overflows and spillways can be utilized during reclamation. Exhibit 3.4-9A contains a plan view and two section views of the modified pond. From the stage/storage capacity curve for the pond structure contained in Appendix 3.4M, the allowable storage at the principle overflow elevation (97.0 feet) is approximately 58,200 cubic feet (1.34 acre-feet). (Elevation is relative to the elevation of the top of the embankment next to the emergency spillway of 100.00 feet.) Therefore, the modified pond will contain the runoff from the 10-year 24-hour storm event and 2.7 years of sediment storage. Table 3.4-16 summarizes the stage-capacity data for Pond 011, once it is enlarged for reclamation.

The 25-year 6-hour storm event (1.6 inches of precipitation (Miller, et. al., 1973)) was used to assess the capacity of the existing principle overflow for Phase I of reclamation. The methods used to calculate the capacity of the overflow pipe are described in Chapter 7. Computations assumed that the pond contained the maximum allowable sediment volume of 4,600 cubic feet (2.7 years), and that the pond was full of water up to the overflow flowline prior to the start of the design runoff event.

From the analysis of the 25-year 6-hour storm event, the maximum combined inflow rate to the pond structure is 3.5 cfs and the maximum outflow rate is 2.82 cfs. The corresponding high water elevation is 97.48 feet, 1.52 feet below the minimum embankment elevation of 99.0 feet. Thus, Pond 011 will adequately pass the 25-year 6-hour peak flow, and the freeboard will be adequate. Since the emergency spillway is at elevation 98.0 feet, it will not pass water during the 25-year 6-hour storm event. The calculations for sedimentation Pond 011 are contained in Appendix 3.4M.

The maximum outflow rate was used to size the riprap at the end of the principle overflow culvert. Based on a culvert slope of 1.5%, a Manning's 'n' of 0.022, a culvert size of 18 inches, and a discharge of 2.82 cfs, the flow velocity against the riprap at the end of the culvert will be 4.0 fps. Riprap with an average diameter of 2 inches is required. Thus, the existing riprap of 18" average diameter is satisfactory for reclamation.

SEDCAD was used to route a 25-year 6-hour storm through the pond assuming that the pond is full of water at the beginning of the storm and that the principle overflow was plugged. The resulting maximum water level is 98.3 feet, with a depth of flow through the spillway of only 0.3 feet. Using the maximum discharge rate of 2.96 cfs, the emergency spillway outslope was evaluated to determine riprap requirements. The spillway has a 6 foot bottom width and 2H:1V side slopes. With a channel slope of 0.33 feet per foot, a bottom width of 6 feet, and a Manning's roughness coefficient of 0.035, the flow velocity was calculated to be 5.0 fps. An average riprap diameter of 3 inches is required for this flow velocity. The existing riprap, with a D_{50} size equal to 6 inches, is satisfactory. The spillway need not be modified for reclamation. A cross section of the emergency spillway is presented in Exhibit 3.4-11 and 3.4-9A.

The peak inflows into the north and south inlets were calculated to be 1.8 and 1.7 cfs, respectively. Flow velocities of 5.1 an 6.4 fps were calculated, as shown in Appendix 3.4M. These inlet channels require an average riprap size of 3 inches and 4 inches, respectively. Calculated channel flow depths will provide for over 1.5 feet of freeboard.

According to R645-301-742.221.34 (DOGM, 1990) a non-clogging dewatering device must be installed in the pond. Because the pond is incised, the elevation of the flowline of the dewatering device is below the adjacent topography, a decant is not feasible. Therefore, the pond will be dewatered using a pump system. The pond will be dewatered to elevation 84.5, the maximum sediment storage elevation. The inlet structure to the pump will float on the surface of the water. The pump system will include an oil skimmer to prevent floating matter from being discharged from the pond during dewatering. Prior to dewatering, the impounded water will be sampled and tested to insure that it meets NPDES discharge requirements.

Sediment removal will be performed when the sediment reaches an elevation of 83.0, which corresponds to 60% of the maximum design sediment volume.

Reclamation Pond 012 - Since the main access road traversing the Prep Plant site will be removed during Phase I of reclamation, it cannot serve as a berm to transmit disturbed area runoff to Pond 012A. Thus, Pond 012A will be removed under Phase I reclamation, as mentioned above. Pond 012B was evaluated to determine its capacity to contain the 10-year 24-hour storm event and sufficient sediment erosion from the watersheds identified on Exhibit 3.4-3A.

Pond 012B was modified in 1991 for the purposes of operational hydrology sediment control. An as-built survey was performed by Dan W. Guy in September 1991, and the results of the survey are summarized in Exhibit 3.4-12. The capacity of Pond 012B at the primary spillway is currently 53,123 cubic feet (1.22 acre feet).

The three year sediment storage volume for Pond 012B of 9573 cubic feet (0.22 acre-feet) was calculated using methods described in Chapter 7. The calculations are contained in Appendix 3.4M. The storm runoff volume from the 10-year 24-hour storm event is 60,025 cubic feet (1.38 acre-feet). These results are based on a curve number of 75 for the 9.91 acres of undisturbed area, and a curve number of 80 for the 28.94 acres of reclaimed area. The minimum necessary capacity of the pond at the elevation of the spillway must be 69,600 cubic feet (1.60 acre feet). Thus, the pond is undersized for its use during Phase I reclamation.

The design of Pond 012B was modified to increase its capacity without affecting the existing spillway structure. A plan view and two section views of the modified pond (referred to as Pond 012) are shown in Exhibit 3.4-9B. From the stage-capacity curve contained in Appendix 3.4M, the allowable storage at the spillway elevation (6097.5 feet) is 80,500 cubic feet. Therefore the pond will fully contain the 10-year 24-hour storm event and more than three years of sediment once it is enlarged. Table 3.4-17 summarizes the stage-capacity data for Pond 012.

The 25-year 6-hour storm event (1.6 inches of precipitation (Miller, et. al., 1973)) was used to assess the capacity of the existing Pond 012B spillway for use during Phase I of reclamation. The calculation methods used are described in Chapter 7, while the calculations are contained in Appendix 3.4M. Computations were conducted assuming that the pond contained the maximum allowable sediment volume of 20,500 cubic feet at elevation 6093.0. In addition, the computer software program SEDIMOT II assumes that the ponds are full of water up to the spillway elevation at the beginning of the storm event.

Using the above assumptions, SEDIMOT II calculated a maximum inflow rate of 5.94 cfs and a maximum outflow rate of 3.26 cfs for Pond 012. The corresponding high water elevation is 6097.88 feet, 0.38 feet above the primary spillway flowline and 1.12 feet below the minimum embankment elevation of 6099.0 feet. Therefore, Pond 012 as modified under this design and the existing spillway are adequate to pass the 25-year 6-hour storm event during Phase I reclamation. The pond is considered adequate to meet the requirements of R645-301-742.220.

Appendix 3.4M includes the calculations for the inlet channel design. A trapezoidal inlet three feet wide and 1.4 feet deep with 3H:1V side slopes will be sufficient to transmit the maximum design flow of 5.94 cfs. This cross section will allow for one foot of freeboard. The peak flow velocity of 4.1 fps requires protection by riprap with and average diameter of 2 inches.

The spillway outlet channel of Pond 012 was evaluated to determine its suitability to transmit the maximum design discharge (Appendix 3.4M). The existing open channel spillway on Pond 012B has a bottom width of 6 feet, side slopes of 2H:1V, and a channel slope of 50%. The design flow depth with the design discharge of 3.26 cfs is 0.1 feet. Since the existing channel is one foot deep, 0.9 feet of freeboard will be available during a 25 year 6 hour storm event during Phase I of reclamation. The maximum flow velocity of 5.9 fps requires an average riprap diameter of 4 inches. The existing outlet channel on Pond 012B has 9 inch riprap, and thus the outlet channel need not be modified for reclamation.

Pond 012 will be dewatered using the existing decant pipe system. The approximate flowline elevation of the dewatering device is 6093.5, which is above the maximum sediment storage elevation. Refer to Exhibit 3.4-9B for a typical section of the existing decant system. Prior to dewatering, the impounded water will be sampled and tested to insure that it meets NPDES discharge requirements.

Sediment removal will be performed when the sediment reaches an elevation of 6092.1, which corresponds to 60% of the maximum design sediment volume.

Pond 013 - As indicated in the calculations contained in Appendix 3.4I, the storm runoff volume for the 10-year 24-hour event is 101,200 cubic feet. This assumes a curve number of 75 for the 59.6 acres of undisturbed area contributing Pond 013. A curve number of 80 was used for the 24.1 acres of reclaimed area. The stage-area and stage-capacity data for Pond 013 were previously determined from the pond topography contained in Exhibit 3.4-13 (See Appendix 3.4I). From the stage-capacity curve, the allowable storage at the spillway elevation (6,255.0 ft) is approximately 396,000 cubic feet. Therefore, there is 294,000 cubic feet available for sediment storage, far more volume than is necessary.

Appendix 3.4I includes an evaluation of the spillway and the inlet and outlet channels for the 25-year 24-hour storm event (2.3 inches of precipitation (Miller, et. al., 1973)). These existing structures were deemed adequate for operational hydrology design flows. With a reduction of storm runoff design flows during reclamation, the existing structures are suitable for use during Phase I of reclamation without modifications.

According to R645-301-742.221.34 (DOGM, 1990), a non-clogging dewatering device must be installed in the pond. Because the pond does not require reconstruction, it will be dewatered using a pump system. The pond will be dewatered to elevation 6250.2, the maximum sediment storage elevation. The pump system will include an oil skimmer to prevent floating matter from being discharged from the pond during dewatering. Prior to dewatering, the impounded water will be sampled and tested to insure that it meets NPDES discharge requirements.

Sediment removal will be performed when the sediment reaches an elevation of 6245.5, which corresponds to 60% of the maximum design sediment volume. A sediment marker installed in the pond will be used to monitor sediment levels.

3.4-4(4) Reclamation Alternative Sediment Controls

Castle Gate Mine proposes to employ the following alternative methods in varying degrees to limit and control sediment erosion in those reclaimed areas whose storm runoff does not flow to sedimentation Ponds 011 or 012:

1. Filter fabric (silt) fences
2. Surface ripping
3. Mulch
4. Chemical (tackifier) added to mulch
5. Straw bales
6. Seeding
7. Reseeding areas that do not exhibit successful germination

Based on Simons, Li & Associates (1983, Table 8.1), these methods constitute some of the best available control technology for the purpose of mining reclamation.

The proposed alternative sediment control measures can be classified into three categories: filtering structures, mechanical treatment, and surface protection measures. Filtering structures inhibit runoff and sediment transport capacity by reducing flow velocity. They also physically trap sediment in the filter openings while allowing water to pass through. Mechanical treatment increases surface roughness thereby reducing overland flow velocity, which minimizes the sediment transport capacity. Detaining some of the would-be runoff also improves soil moisture for plant germination. Surface protection measures include mulching, mulch binders, netting, and seeding. These measures are the most effective controls since they minimize the amount of soil detached by raindrop impact, and thus limit soil loss at the source. Surface protection measures also increase the surface roughness and increase water infiltration into the ground.

Simons, Li & Associates (1983, Figure 7.3) indicates that synthetic filter fabric is more efficient than straw bales at trapping silt and, therefore, synthetic fabric fences and not straw bales will be utilized wherever possible. The sections of the reclamation stream channels protected by silt fences are indicated on Exhibit 3.4-3A. The fences will be installed parallel to the contours with the ends of the fences turned up perpendicular to the contours to contain the sediment. Silt fences will be installed in accordance with Figures 3.4-1 and 3.4-2. The filter fabric will be composed of a UV-resistant, perforated synthetic fabric with an integral supportive netting. A separate supportive backing could be used in lieu of the integral netting. To prevent sediment runoff from passing under the fence, the fabric will be secured by burying the bottom edge of it in a small trench along the length of the fence.

Calculations have been performed that verify that a single tier system of 36" high silt fences will be adequate to capture sediment during a 10-year 6-hour storm event without failing, assuming they are properly maintained. Length, spacing, and angle of the fence segments are contingent on the slope of the channel, the slope of the reclaimed surface immediately adjacent to the channel, and the relative expected sediment load along each specific reach of the canyon. For example, fence segments along the east side of CGRD-5 along the upper reach should be 50 feet in length and spaced approximately 55 feet on center, angled at about 45° from a line perpendicular to the channel. This general configuration will allow the fence segments to be parallel to the contours adjacent to the channel, and to provide a 10 foot projected overlap. See Figures 3.4-1 and 3.4-2 for a typical silt fence installation, and Appendix 3.4N for supporting calculations.

Mechanical treatment of slopes of less than 20% will be performed by ripping the soil to a depth of 18" to 24". Ripper shanks should be spaced about seven feet apart, and create parallel slots four to ten inches wide. Ripping will loosen the soil and allow root penetration and increase moisture storage. This will allow for quicker vegetation establishment, which will reduce erosion.

In regard to surface protection measures, a chemical additive will be used in combination with wood fiber mulch to help prevent the removal of the mulch by wind. The mulch itself can significantly reduce the amount of sediment yield from an area (Simons, Li & Associates, 1983, p. 4.30) The mulch also helps retain moisture to allow for seed germination. Based on a rainfall intensity factor of 0.61 inches per hour, the minimum mulch application rate is 0.9 tons per acre to prevent mulch removal by rainfall (Simon et al., 1983, Figure 4.14). The referenced figure assumes that no chemical binder will be used. The intensity factor corresponds to a 10-year 6-hour storm event. Mulch, with a tackifier, will be applied at the rate of 2,000 pounds per acre.

Permanent plant growth is the best method of controlling erosion from slopes, according to Simons, Li & Associates (1983, p. 4.44). Upon completion of the grading in accordance with the plan depicted in Exhibit 3.4-3, and ripping of the soil, the reclaimed area will be seeded with grasses and legumes. The species seed mix is addressed in Chapter 9. Seeding will be performed at the appropriate time of the year in consideration of available moisture for germination. Areas in which the seed does not germinate will be reseeded.

Appendix 3.4N presents calculations that quantify the sediment yield that could be expected annually and during a 10-year 6-hour storm event with and without various sediment control measures in place. These calculations were performed to compare the improvement of the sediment control measures listed above against background levels. The cumulative implementation of each sediment control measure substantially reduces the amount of sediment eroded from the reclaimed areas, to the point that the mulch theoretically inhibits soil loss more effectively than the undisturbed ground cover. Since the undisturbed areas contributing sediment to the stream channels through silt fences are often larger than the reclaimed areas, most of the sediment erosion will occur from the undisturbed areas. More than 90% of the sediment loss trapped by the silt fence along CGRD-5 was calculated to be from the undisturbed areas. Thus, the background sediment loss overshadows the sediment loss from the reclaimed areas once the wood fiber mulch is in place. In addition, the combination of the surface sediment controls on the reclaimed areas and the silt fences along the channels reduces the silt load from the reclaimed areas to the streams by approximately 80% from what it would be if the same reclaimed areas were undisturbed and in their natural state.

Whenever possible, a minimum of one method of sediment control will be in place during reclamation construction. Filter fabric (silt) fences will be installed to collect sediment runoff from areas which will not report to sedimentation Ponds 011 and 012 as soon as it is feasible to do so. Upon completion of the grading and soil ripping, the reclaimed area will be seeded and mulched using either hydromulching or straw tacked by a suitable netting.

The possibility exists that a 10-year 6-hour storm (or larger) will occur during the grading and removal of the sedimentation ponds. Although every reasonable effort will be made to have at least one sediment control measure in place, there may be a period of time when that is not feasible. However, the probability that a 10-year event will occur during the construction period of approximately six months is only 5.1% (Linsley and Frazini, 1979, Eq. 5-3). This probability is relatively small, and thus no special measures will be taken to address the possibility.

The alternative sediment controls constructed during Phase I reclamation will be inspected quarterly or after every major storm event. Observations made during these inspections, as well as corrective actions taken, will be recorded. Corrections to any weaknesses in the implementation of the sediment control plan will be remedied immediately to prevent future silt runoff into the Price River. Corrective action will be taken when trapped sediment builds up along a silt fence to half its height, when the sediment fence is listing more than 20 degrees from the vertical, when the straw bales become 50% saturated with silt, when a gully greater than six inches in depth is created due to lack of vegetation establishment, or when the mulch and seed have been transported by wind or overland flow. Corrective action will consist of repairing/replacing or adding filter fabric fences as necessary, replacing straw bales, regrading of the ground surface only as necessary to fill in six inch gullies caused by erosion, and reseeding and mulching to reestablish vegetation. Soil material trapped by sediment control structures that is not used in repairing erosion damage on the site will be removed and disposed of within the boundaries of the Castle Gate Mine permit area.

All alternative sediment control structures will remain in place for a minimum of two years after the last seeding, until the removal is authorized by DOGM, until vegetation over the reclaimed area is properly established in accordance with R645-301-763.100, and the water quality bond release standards of R645-301-880.320 are complied with.

3.4-5 Reclamation Timetable

No permanent reclamation is proposed during the renewal period of 1989 -1994. However, the following time frames can be used to estimate the length of time for reclamation.

1. Demolition	Week 1 - 36
2. Grading	Week 36 - 60
3. Installation of Alternative Control Measures (ASCM's)	Week 36 - 40
4. Resoiling	Week 60 - 72
5. Seed bed preparation	Week 73
6. Seeding & mulching	Week 74 After Oct. 1
7. Pond and ASCM maintenance	0 - 2 years after seeding
8. Removal of Ponds 011, 012, 013, and removal of ASC structures	All alternative sediment control structures will remain in place for a minimum of two years after the last seeding, until the removal is authorized by DOGM, until vegetation over the reclaimed area is properly established in accordance with R645-301-763.100, and the water quality bond release standards of R645-301-880.320 are complied with.

9. Vegetation and water monitoring	2 - 10 years after seeding
10. Reclamation Monitoring	Until Bond Release

3.4-6 Stream Buffer Zones

Castle Gate Mine has Valid Existing rights to perform underground mining and reclamation activities within the Stream Buffer Zone, defined as the area within 100′ of the Price River. Located within this Stream Buffer Zone is the ancillary road A-2 which was constructed prior to SMCRA. The location of the Stream Buffer Zone is shown on Exhibit 3.4-2. The Stream Buffer Zone will be marked with signs which state "Stream Buffer Zone".

3.4-7 Transportation Facilities

Beltlines - The coal is transported out of the underground mining complex on a 48" beltline. The beltline crosses US Highway 50 through a tunnel. The beltline continues across the Price River and the Denver and Rio Grande Railroad through a steel tube which prevents any spillage from falling into the river or onto the railroad. The beltline proceeds to a transfer house, breaker and finally to the raw coal storage stacker tube. After processing, the coal is again belted to the clean coal stacker tube where the coal is stored for shipment. The coal is loaded onto unit trains from the clean coal storage pile by a 72" beltline to the unit train loadout. A description of the unit train loadout is located in Section 3.8. The location of beltline facilities are shown on Exhibit 3.4-1.

Roads - Table 3.4-18 is a list of primary and ancillary roads used to facilitate access to areas of the Castle Gate Preparation Plant. The roads are shown on Exhibits 3.4-2A and typical cross sections are found on Figures 3.4-3 through 3.4-9.

The roads were constructed prior to SMCRA and were reconstructed to meet the design standards of R645-301.

In the event of a catastrophic event such as flood or earthquake, the primary roads will be repaired as soon as practical after damage has occurred if the road is necessary to support mining and reclamation activities.

The primary and ancillary roads shown on Exhibit 3.4-2A were constructed using non toxic and non-acid bearing materials in their surface. No embankments were constructed to support the road. The refuse haul road and truck dump roads (P-2 and P-3) were constructed on cut and fill slopes. Both the refuse haul road and truck dump roads are constructed on substantial rock. These rock road cuts exceed the 1.3 static factor of safety required in R645-301-534.130.

The side slopes of the roads are revegetated. The runoff from the roads is channelled in ditches or overland flows which controls or prevents erosion.

The culverts used in the road construction were designed to sustain the vertical soil pressure, passive resistance of the foundation and the weight of the vehicles using the road.

3.4.8 References

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TABLE 3.4-1

PREPARATION PLANT
OPERATIONAL HYDROLOGY WATERSHED CHARACTERISTICS

WATERSHED (CGWS-)	CURVE NUMBER	TIME OF CONCENTRATION (HR)	DRAINAGE AREA (Acres)	PEAK FLOW (CFS)[a]
U1	78	.106	7.05	0.95
U2	78	.066	2.38	0.36
U3A	75	.710	1027.1	42.5
U3B	75	.386	172.9	7.44[b]
U4	78	.085	6.78	0.96
U5	82	.098	7.03	1.78
U6	78	.181	52.11	5.83
U7	82	.108	5.96	1.48
U8	78	.137	20.59	2.57
D1(A&B)	90	.162	12.60	6.83
D2A	78	.144	41.10	5.02
D2B	82	.099	7.60	1.92
D2C	82	.098	14.83	3.75
D2D	85	.052	4.66	1.76
D2E	85	.213	6.49	2.00
D2F	85	.046	3.38	1.29
D2G	82	.081	2.26	0.59

TABLE 3.4-1 (Continued)

PREPARATION PLANT
OPERATIONAL HYDROLOGY WATERSHED CHARACTERISTICS

WATERSHED (CGWS-)	CURVE NUMBER	TIME OF CONCENTRATION (HR)	DRAINAGE AREA (Acres)	PEAK FLOW (CFS)
D3(A&B)	90	.142	14.48	7.98
D4(A&B)	90	.217	14.73	7.59
D5	85	.038	1.56	0.60

(a) Peak flows are based on a 10-year 6-hour storm event.

(b) See Table 3.4-20 for 100-year 6-hour storm event peak flows associated with Schoolhouse Canyon watersheds.

TABLE 3.4-2

PREPARATION PLANT
OPERATIONAL HYDROLOGY SUMMARY OF
DIVERSION DITCH GEOMETRIES

DIVERSION DITCH ID	MINIMUM BOTTOM WIDTH[a] (FT)	MINIMUM SIDE SLOPES (H:V)	MINIMUM DEPTH (FT)	MINIMUM CHANNEL SLOPE (%)	MAXIMUM CHANNEL SLOPE (%)	MAXIMUM FLOW DEPTH (FT)	MINIMUM FREEBOARD (FT)	MAXIMUM FLOW VELOCITY (FPS)	MINIMUM RIPRAP D50[a] (IN)
CGD-1	2.0	1:1	0.6	2	10	0.22	0.38	3.29	1.0
CGD-2	1.0	1.5:1	0.5	7	10	0.12	0.38	2.76	NONE
CGD-3	10.0	.8:1	1.0	6	10	0.71	0.29	7.88	10.0
CGD-4	1.0	1:1	0.7	2	6.5	0.33	0.37	3.36	1.0
CGD-8	2.0	1:1	2.4	1	10	2.04	0.36	9.06	14.0
CGD-9	1.5	1:1	0.6	1	9.2	0.22	0.38	3.26	1.0
CGD-10	2.0	1.5:1	0.9	3	5	0.56	0.34	5.53	3.0
CGD-11	3.0	1:1	0.5	12.5	12.5	0.18	0.32	4.48	2.0
CGD-12	0	1.5:1	1.6	1.3	2.6	1.29	0.31	4.15	NONE
CGD-13	0	1.5:1	1.6	1.4	3.0	1.25	0.35	4.32	NONE
CGD-14	1.0	1.5:1	1.0	1.0	1.0	0.65	0.35	2.63	NONE

TABLE 3.4-2 (Continued)

PREPARATION PLANT
OPERATIONAL HYDROLOGY SUMMARY OF
DIVERSION DITCH GEOMETRIES

DIVERSION DITCH ID	MINIMUM BOTTOM WIDTH[a] (FT)	MINIMUM SIDE SLOPES (H:V)	MINIMUM DEPTH (FT)	MINIMUM CHANNEL SLOPE (%)	MAXIMUM CHANNEL SLOPE (%)	MAXIMUM FLOW DEPTH (FT)	MINIMUM FREEBOARD (FT)	MAXIMUM FLOW VELOCITY (FPS)	MINIMUM RIPRAP D50[b] (IN)
CGD-15	1.0	1.5:1	1.0	1.0	3.0	0.66	0.34	3.94	NONE
CGD-16	0	1.5:1	0.9	7.0	10	0.54	0.36	4.72	1.5
CGD-17	0	1.5:1	1.0	6.7	10	0.68	0.32	6.41	2.0
GENERIC DITCH	0	3:1	1.0	6	11	0.31	0.69	4.28	1.5

(a) Minimum bottom width measured at minimum depth from top of channel.
(b) Minimum riprap requirements for ditches constructed in soil. If ditches are constructed on bedrock, riprap is not required. If ditch is well vegetated, riprap is not required for velocities < 4 ft/sec. Refer to Appendix 3.4D.

Note: See Table 3.4-21 for information on diversion ditches 5, 6, 7, 18 and 19.

TABLE 3.4-3

PREPARATION PLANT
OPERATIONAL HYDROLOGY SUMMARY OF DIVERSION BERM GEOMETRIES

BERM (CGB-)	MINIMUM TOP WIDTH (FT)	MINIMUM HEIGHT (FT)
1	0.5	2.0
2	1.0	2.0
3	2.0	3.0
4	1.0	2.0
5	1.0	3.0
6	3.0	4.0
7	0.5	1.0

TABLE 3.4-4

**PREPARATION PLANT
OPERATIONAL HYDROLOGY CULVERT DISCHARGE SUMMARY**

CULVERT (CGC-)	MINIMUM SIZE AND TYPE	INLET TYPE	AVAILABLE HEADWATER OVER TOP (FT)	INLET CONTROL CAPACITY (CFS)[a]	DRAINAGE BASINS	PEAK FLOW (CFS)
1	18" CMP	PROJECT	>1	5.7	U1	0.95
2	(2) 84" CMP	MITERED	>1	590	POND 011, U2, U3A, U3B, U4	56.5
3	24" CMP	DROP	2	12.5	U4	0.96
5	60" CMP	DROP	8	128	U5, POND 013	33.58
6	12" CMP	PROJECT	>1	2.1	D7	0.60
7	18" CMP	CONNECTS POND 012A TO POND 012B (SEE POND CALCULATIONS)				
8	60" X 120" BOX	HEADWALL	7	280	U6, U7, U8	9.88

(a) Capacity based on HW/D = 1.0.

Note: See Table 3.4-24 for information on Culvert CGC-4.

TABLE 3.4-5

PREPARATION PLANT
OPERATIONAL HYDROLOGY CULVERT SUMMARY

CULVERT (CGC-)	SIZE AND TYPE	OUTLET SLOPE[a] (%)	PEAK FLOW[b] (CFS)	PEAK VELOCITY[b] (FPS)	REQUIRED D50[b] (IN)
1	18" CMP	1	0.95	2.4	NONE
2	(2) 84" CMP	5	56.5	9.5	7
3	24" CMP	4	0.96	3.8	1
5	60" CMP	10	33.58	13.4	15
6	12" CMP	4	0.60	3.6	1
7	18" CMP	CONNECTS POND 012A TO POND 012B (SEE POND CALCULATIONS)			
8	60" X 120" BOX	1	9.88	2.9	NONE

(a) Field measurement.
(b) See Appendix 3.4F for details.

Note: See Table 3.4-25 for information on Culvert CGC-4.

TABLE 3.4-6

PREPARATION PLANT
POND 011 (Existing) STAGE-CAPACITY DATA

STAGE	ELEVATION (FT)	AREA (FT^2)	INCREMENTAL VOLUME (FT^3)	CUMULATIVE VOLUME (FT^3)
Bottom	81.5	708		0
			413	
	82.0	944		413
			2,244	
	84.0	1,300		2,657
			2,900	
	86.0	1,600		5,557
			3,904	
	88.0	2,304		9,461
			5,228	
	90.0	2,924		14,689
			6,504	
	92.0	3,580		21,193
			7,552	
	94.0	3,972		28,745
			9,216	
	96.0	5,244		37,961
			5,602	
Principle Overflow	97.0	5,960		43,563
			6,450	
Emergency Spillway	98.0	6,940		50,013
			7,454	
Top of Embankment	99.0	7,968		57,467

TABLE 3.4-7A

PREPARATION PLANT
POND 012A (Existing) STAGE-CAPACITY DATA

STAGE	ELEVATION (FT)	AREA (FT^2)	INCREMENTAL VOLUME (FT^3)	CUMULATIVE VOLUME (FT^3)
Bottom	92.0	5,997		0
			15,244	
	94.0	9,247		15,244
			22,351	
	96.0	13,104		37,595
			18,610	
Primary Spillway	97.3	15,527		56,205
			11,325	
	98.0	16,831		67,530
			5,132	
Emergency Spillway	98.3	17,381		72,662
			12,616	
	99.0	18,664		85,278
			19,581	
Top of Embankment	100	20,497		104,859

TABLE 3.4-7B

PREPARATION PLANT
POND 012B (Existing) STAGE-CAPACITY DATA

STAGE	ELEVATION (FT)	AREA (FT^2)	INCREMENTAL VOLUME (FT^3)	CUMULATIVE VOLUME (FT^3)
Bottom	84.0	3,325		0
			3,929.5	
	85.0	4,534		3,929.5
			5,170	
	86.0	5,806		9,099.5
			6,473	
	87.0	7,140		15,572.5
			7,828.5	
	88.0	8,517		23,401
			18,893	
	90.0	10,376		42,294
			10,829	
Primary Spillway	91.0	11,282		53,123
			6,932	
	91.6	11,826		60,055
			4,803	
	92.0	12,188		64,858
			10,040	
Top of Embankment	92.8	12,913		74,898

TABLE 3.4-8

PREPARATION PLANT
POND 013 (Existing) STAGE-CAPACITY DATA

ELEVATION (FT)	AREA (FT^2)	INCREMENTAL VOLUME (FT^3)	CUMULATIVE VOLUME (FT^3)
6232	3275.0		0
		8,859.8	
6234	5584.8		8859.8
		13,278.0	
6236	7693.2		22,137.8
		17,642.6	
6238	9949.4		39,780.4
		22,529.4	
6240	12,580.5		62,309.8
		27,960.9	
6242	15,380.4		90,270.7
		33,339.9	
6244	17,959.5		123,610.6
		38,217.8	
6246	20,258.3		161,828.4
		42,879.4	
6248	22,621.1		204,707.8
		47,699.8	
6250	25,078.7		252,407.6
		52,768.3	
6252	27,689.6		305,175.9
		58,825.2	
6254	31,135.6		364,001.1
		65,731.9	
6256	34,596.3		429.733.0
		70,833.7	
6258	36,237.4		500,566.7
		36,647.7	
6259	37,058.0		537,214.4

TABLE 3.4-9

PREPARATION PLANT
RECLAMATION MASS BALANCE SUMMARY

Project: CASTLE GATE MINE
CARBON COUNTY, UTAH

Site description: PREPARATION PLANT

Subsite: PERMIT
Description: Volume calculation within disturbed area boundary

Volume Information
Method: Grid
First surface: EG Second surface: FG

Cut: 127745.2007 cu yds Fill: 115839.9721 cu yds
Net Volume: 11905.2286 cu yds (Cut)

Maximum cut: 21.295846 feet
Location: 2178745.028204 - 511506.279713
Maximum fill: 23.553025 feet
Location: 2177960.461057 - 511515.189874

Tue Sep 01 14:26:55 1992

Volume calculation by GRID method with a node spacing of 25 feet.

SOURCE: Softdesk, Inc. (formerly DCA Software, Inc.)
Earthworks Grading module
Registration #ERHE 15426
Registered to EarthFax Engineering, Inc.

VOLUME OF TOPSOIL REQUIRED FOR REFUSE PILE

AREA: 30 ACRES
THICKNESS: 2.0 FEET

30 ACRES x 43,560 SF/ACRE x 2.0 FEET x 1 CY / 27 CF = 96,800 CY

TABLE 3.4-10

PREPARATION PLANT AREA
RECLAMATION WATERSHED CHARACTERISTICS

WATERSHED	CURVE NUMBER	AREA (Acres)	TIME OF CONCENTRATION (HR)	DISCHARGE (CFS)
CGRWS-U1	78	20.48	0.114	2.70[b]
CGRWS-U2	78	5.96	0.099	2.92[a]
CGRWS-U3	78	50.68	0.186	5.62[b]
CGRWS-U4	78	9.18	0.072	1.35[b]
CGRWS-U5	80	83.32	0.243	40.20[a]
CGRWS-U6	75	174.58	0.346	36.55[a],7.56[b]
CGRWS-U7	75	1054.58	0.694	179.78[a]
CGRWS-U8	75	4.04	0.083	1.54[a]
CGRWS-U9	75	29.38	0.102	1.92[b]
CGRWS-U10	75	3.95	0.048	1.60[a]
CGRWS-R1	80	7.05	0.060	4.24[a]
CGRWS-R2	80	21.89	0.100	12.59[a]
CGRWS-R3	80	11.58	0.301	5.18[a]
CGRWS-R4	80	1.56	0.031	0.35[b]

[a] Based on the 100-Year 6-Hour storm event.
[b] Based on the 10-Year 6-Hour storm event.

TABLE 3.4-11

PREPARATION PLANT AREA
RECLAMATION CHANNEL DISCHARGE SUMMARY

DIVERSION DITCH	CONTRIBUTORY WATERSHED	TOTAL DRAINAGE AREA (Acres)	DESIGN DISCHARGE (CFS)
CGRD-1	CGRWS-U1	20.48	2.70
CGRD-2	CGRWS-U3	50.68	5.62
CGRD-3B	CGRWS-U4,U5	92.5	41.55
CGRD-3C	CGRWS-U4,U5,R4	94.06	41.90
CGRD-5	CGRWS-U6,U7	1229.16	222.90
CGRD-6	CGRWS-U6,U7	1229.16	222.90
CGRD-10	CGRWS-U9.R3	40.96	3.43

Note: See Table 3.4-20 for information on diversion ditches 3a, 4, 7, 8 and 9.

TABLE 3.4-12

PREPARATION PLANT AREA
RECLAMATION CHANNEL SUMMARY

Reclamation Channel	Minimum Bottom Width (FT)[a]	Side Slopes (H:V)	Minimum Channel Depth (FT)	Maximum Bottom Slope (%)	Minimum Bottom Slope (%)	Maximum Flow Depth (FT)	Freeboard (FT)	Maximum Velocity (FT/S)	Minimum Riprap D_{50} (IN)
CGRD-1	3	3:1	1.2	27	11	0.2	1.0	5.52	3[c]
CGRD-2	3	3:1	1.4	30	4	0.4	1.0	7.32	5[c]
CGRD-3B(MS)[b]	3	3:1	2.0	10	5	1.0	1.0	9.06	7[c]
CGRD-3B(SS)[b]	3	3:1	2.0	16	10	< 1.0	> 1.0	-	12[d]
CGRD-3C	3	3:1	2.0	24	14	0.8	1.2	-	18[d]
CGRD-5(MS)	18	1.5:1	2.7	10	2	1.7	1.0	11.13	14[c]
CGRD-5(SS)	18	1.5:1	2.7	14	10	< 1.7	> 1.0	-	18[d]
CGRD-6	18	3:1	2.5	6	2	1.5	1.0	9.81	4[c]
CGRD-10[e]	3	1.2:1	1.2	10	1	0.2	1.0	3.08	3[c]
TYPICAL BERM/SWALE	8	2:1,5:1	1.0[f]	3	1	0.3	0.7	2.5	none

[a] Minimum bottom width measured at minimum depth from top of channel.
[b] MS = mild slope. SS = steep slope.
[c] Riprap D_{50} calculated by using the Searcy method developed for the U.S. D.O.T..
[d] Riprap D_{50} calculated by using the Simons et al./OSM steep slope design methodology.
[e] Temporary Reclamation channel (Phase I only).
[f] Berm top width = 1.0 feet.

Note: See Table 3.4-21 for information on reclamation diversions 3a, 4, 7, 8 and 9.

TABLE 3.4-13

PREPARATION PLANT AREA
RECLAMATION CULVERT DISCHARGE SUMMARY

CULVERT	CONTRIBUTORY WATERSHED	TOTAL DRAINAGE AREA (Acres)	DESIGN DISCHARGE (CFS)
CGRC-1	CGRWS-U2,U3	56.64	8.64
CGRC-2	CGRWS-U4,U5,R4	94.06	41.90
CGRC-3	CGRWS-U6,U7	1229.16	222.90

TABLE 3.4-14

PREPARATION PLANT AREA RECLAMATION CULVERT SUMMARY

CULVERT (CGRC-)	SIZE & TYPE	SLOPE (%)	PEAK FLOW (CFS)	PEAK VELOCITY (FPS)	ACTUAL D_{50}[d] (IN)
1[a]	60" X 120" Box	1	8.64	3.45	1
2[b]	60" Concrete	10	41.90	20.86	39
3[c]	2-84" CMP	5	222.90	17.38	30

(a) This is an existing culvert previously labeled CGC-8.
(b) This is an existing culvert previously labeled CGC-5.
(c) This is an existing culvert previously labeled CGC-2.
(d) Actual riprap size exceeds minimum requirements under reclamation conditions.

TABLE 3.4-15

PREPARATION PLANT AREA RECLAMATION CHANNELS RIPRAP AND FILTER BLANKET VOLUMES

CHANNEL	RIPRAP D_{50} (IN)	LENGTH (FT)	PERIMETER (FT)	RIPRAP THICKNESS (IN)	RIPRAP VOLUME (FT^3)	FILTER THICKNESS (IN)	FILTER VOLUME (FT^3)
CGRD-1	3	275	10.6	6	1,458	6	1,458
CGRD-2	5	200	11.9	10	1,983	6	1,190
CGRD-3B(SS)	7	200	15.6	14	3,640	7	1,820
CGRD-3B(MS)	12	250	15.6	15	4,875	7	2,275
CGRD-3C	18	300	15.6	23	8,970	11	4,290
CGRD-5(MS)	14	250	27.7	28	8,079[a]	14	4,040[b]
CGRD-5(SS)	18	1,050	27.7	23	27,873[a]	11	13,331[b]
CGRD-6	4	300	33.8	8	6,760	6	5,070
CGRD-10	Riprap in place						
TOTALS					64,638[e] (4,525 tons)[c]		33,474[e] (2,176 tons)[d]

[a] Assumes that 50% of the riprap currently lining the existing channel can be reused.
[b] Assumes that new filter material will be required along only 50% of the existing channel.
[c] Assumes a riprap in-place density of 140 pcf.
[d] Assumes a filter in-place density of 130 pcf.
[e] Total volume changed once diversion CGRD-4 was considered a School House Canyon diversion.

Notes: 1. See Table 3.4-22 for School House Canyon riprap and filter blanket volumes.
2. Riprap at the base of culverts is neglected for the volume calculations.

TABLE 3.4-16

PREPARATION PLANT
POND 011 (RECLAMATION) STAGE-CAPACITY DATA

STAGE	ELEVATION (FT)	AVERAGE AREA (FT^2)	INCREMENTAL VOLUME (FT^3)	CUMULATIVE VOLUME (FT^3)
Bottom	81.0	684		0
			894	
	82.0	1100		894
			2,700	
	84.0	1,600		3,594
			3,920	
	86.0	2,320		7,514
			5,332	
	88.0	3,012		12,864
			6,872	
	90.0	3,860		19,718
			8,432	
	92.0	4,572		28,150
			10,252	
	94.0	5,680		38,402
			12,512	
	96.0	6,832		50,914
			7,292	
Principle Overflow	97.0	7,752		58,206
			8,284	
Emergency Spillway	98.0	8,816		66,490
			9,408	
Top of Embankment	99.0	10,000		75,898

TABLE 3.4-17

PREPARATION PLANT
POND 012 (RECLAMATION) STAGE-CAPACITY DATA

STAGE	ELEVATION (FT)	AREA (FT^2)	INCREMENTAL VOLUME (FT^3)	CUMULATIVE VOLUME (FT^3)
Bottom	6090.0	3,696		0
			11,069	
	6092.0	7,373		11,069
			18,855	
	6094.0	11,482		29,924
			26,711	
	6096.0	15,229		56,635
			16,382	
	6097.0	17,534		73,017
			9,055	
Primary Spillway	6097.5	18,687		82,072
			9,632	
	6098.0	19,840		91,704
			20,993	
Top of Embankment	6099.0	22,145		112,697

TABLE 3.4-18

PREPARATION PLANT AREA ROAD DESCRIPTIONS

ROAD	TYPE	SURFACE	DESCRIPTION
P-1	Primary	Asphalt	Road P-1 is the main entrance road to Castle Gate Preparation Plant. The road begins at the south permit boundary line and runs north to the Preparation Plant. The road has a minimum width of 20 feet and an average grade of 4%. The road is used by mine personnel, coal trucks and delivery vehicles. Figure 3.4-3 is a typical section of the road. The road will be maintained by filling potholes with asphalt and washing with a water truck as needed.
P-2	Primary	Rock	Road P-2 is the refuse haul road which begins near the southeast corner of the office/warehouse building and runs south to the refuse pile. The road has a minimum width of 20 feet. The grade varies from 4% to 10% depending on the location along the route. The primary purpose of the road is to haul refuse from the Preparation Plant to the refuse pile. Maintenance will be grading the road as necessary to maintain drainage. Figure 3.4-4 is a typical section of the haul road.
P-3	Primary	Rock	Road P-3 is the haulage road used by coal trucks to access the truck dump. This road begins about 300 feet north of the truck scale and runs north to the truck dump area. The minimum width on this road is 15 feet with a grade that varies from 2% to 6%. The road will be maintained by grading as necessary to establish drainage and provide a driveable surface. Figure 3.4-5 is a typical section of the haul road.
A-1	Ancillary	Rock	Road A-1 is an ancillary road which is used to access the parking area for the bathhouse at the Preparation Plant. The road is located just south of the thickener. The road is a minimum of 15 feet wide and the grade varies from 2% to 6%. The road will be maintained by grading when necessary to provide drainage and a driveable road surface. Figure 3.4-6 is a typical section of the haul road.

TABLE 3.4-18 (Continued)

PREPARATION PLANT AREA ROAD DESCRIPTIONS

ROAD	TYPE	SURFACE	DESCRIPTION
A-2	Ancillary	Rock	Road A-2 begins just west of the water treatment plant and runs north parallel to the Price River along the western disturbed area boundary line. The road continues as a public road past the northern disturbed area boundary line. The road is used for access to the raw water pond, clean coal pile, and unit train loadout. The road has a minimum width of 18 feet and a gradient of 2% to 4%. The road will be maintained by grading when necessary to establish drainage and provide driveable road surface. Figure 3.4-7 is a typical section of the road.
A-3	Ancillary	Rock	Road A-3 begins near the southeast corner of the office/warehouse building and continues east past the substation to an area which is used for Preparation Plant parts storage. The minimum width of the road is 12 feet. The gradient varies from 2% to 6%. The road will be maintained by grading when necessary to establish drainage and provide a driveable road surface. See Figure 3.4-8 for typical cross section.
A-4	Ancillary	Rock	Road A-4 joins road A-2 just east of the thickener. The road is used to access the area which contains the thickener overflow pond and the raw coal pile. The minimum road width is 12 feet and the gradient varies from 2% to 6%. The road will be maintained by grading in order to establish drainage and provide a driveable surface. See Figure 3.4-9 for a typical section of the road.

TABLE 3.4-19

PREPARATION PLANT SLOPE PARAMETERS

SECTION	SOIL TYPE	MOIST UNIT WEIGHT (PCF)[a]	SATURATED UNIT WEIGHT (PCF)[a]	COHESION (PSF) [b],[c]	ANGLE OF INTERNAL FRICTION (°)[b],[c]	PORE PRESSURE PARAMETER	PORE PRESSURE CONSTANT
Pond 011 C - C′	Silty Sand (SM)	115	135	100	34	0	0
Pond 011 D - D′	Silty Sand (SM)	115	135	25[d],200[e]	34	0	0
Pond 012A G - G′	Silty Sand (SM)	115	135	25[d],200[e]	34	0	0
Pond 012B F - F′	Silty Sand (SM)	115	135	100	34	0	0
Pond 012B H - H′	Silty Sand (SM)	115	135	25[4],200[5]	34	0	0

[a] See Appendix 3.4K for unit weight calculations based on NAVFAC DM-7, 1971, and Hoek, 1981.
[b] Hoek, 1981. Table 1 - Typical Soil and Rock Properties. pg 23.
[c] NAVFAC DM-7. 1971. Table 9-1. Typical Properties of Compacted Materials.
[d] Saturated soil layer.
[e] Moist, unsaturated soil layer.

TABLE 3.4-20

SCHOOL HOUSE CANYON REFUSE AREA DIVERSION DISCHARGE SUMMARY

REFERENCE LETTER [a]	MINING PHASE	CONTRIBUTORY WATERSHED	TOTAL DRAINAGE AREA (AC)	DESIGN DISCHARGE[b]
	Current Operation	**Exhibit 3.4-2**	**Current Operation**	**Current Operation**
A	CGD-5	CGWS-U3B	172.9	33.9
B	CGD-6 (Upper)	CGWS-D2A, D2B, D2E	55.4	30.4[c]
C	CGD-6 (Lower)	CGWS-D2A, D2B, D2E	55.4	30.4
D	CGD-7 (Upper)	CGWS-D2C, D2E	21.0	18.6
E	CGD-7 (Lower)	CGWS-D2C, D2D, D2E	25.7	22.6
F	NA	NA	NA	NA
G	CGD-19	CGWS-D2A, D2B, D2E	55.4	30.4
H	NA	NA	NA	NA
	Final Operation	**Exhibit 3.4-2C**	**Final Operation**	**Final Operation**
A	CGD-5	CGWS-U3B	172.9	33.9
B	CGD-6 (Upper)	CGWS-D2B,D2C	24.1	16.7
C	CGD-6 (Lower)	CGWS-D2B, D2C, D2D, D2E	41.7	34.2[c]
D	CGD-7 (Upper)	CGWS-D2A, D2C	35.9	22.8[c]
E	CGD-7 (Lower)	CGWS-D2A, D2C, D2E, D2F	58.3	44.9[c]
F	CGD-18	CGWS-D2B	14.7	6.9[c]
G	CGD-19	CGWS-D2B, D2C, D2D, D2E	41.7	34.2[c]
H	NA	NA	NA	NA
	Final Reclamation	**Exhibit 3.4-3**	**Final Reclamation**	**Final Reclamation**
A	CGRD-4	CGRWS-U6	174.6	36.6[c]
B	CGRD-8	CGRWS-U5A	28.6	15.9
C	CGRD-9 (Upper)	CGRWS-U5A, U5C	40.9	23.0
D	CGRD-7	CGRWS-U5B	26.6	15.5
E	CGRD-3A	CGRWS-U5B, U5D	42.4	25.0
F	NA	NA	NA	NA
G	NA	NA	NA	NA
H	CGRD-9 (Lower)	CGRWS-U5A, U5C	40.9	23.0[c]

(a) Drainage diversion/channels with the same reference letter are identically located.
(b) All design flows are based on the 100-year 6-hour storm event.
(c) Refers to maximum design peak flow throughout all mine phases. (Ditch designed for this discharge value).
NA = Not applicable
Note: See Table 3.4-2 for generic ditch values.

TABLE 3.4-21

SCHOOL HOUSE CANYON REFUSE AREA
DIVERSION SUMMARY BASED ON MAXIMUM PEAK FLOW DESIGN

Refuse Area Channel	Bottom Width (FT)	Side Slopes (H:V)	Depth (FT)	Min. Channel Slope (%)	Max. Channel Slope (%)	Max. Flow Depth (FT)	Freebrd (FT)	Max. Flow Velocity[a] (FPS)	Min. D50 (IN)
CGD-5/CGRD-4	5.0	1:1	2.21	2	20	1.21	1.00	10.62	[b]
CGD-6 (Upper)/CGRD-8	0.0	1.5:1 (L) 10:1 (R)	1.5	1	1	1.23	0.27	3.51	None
CGD-6 (Lower)/CGRD-9 (Upper)	3.0	3:1	1.85	8	40	0.85	1.00	-	18
CGD-7 (Upper)/CGRD-7	0.0	10:1 (L) 1.5:1 (R)	1.5	1	1	1.10	0.40	3.26	None
CGD-7 (Lower)/CGRD-3A	5.0	3:1	1.71	13	40	0.71	1.00	-	21
CGRD-9 (Lower)	3.0	3:1	1.55	20	50	0.55	1.00	-	18
CGD-18	3.0	3:1	1.26	17	17	0.26	1.00	7.17	5
CGD-19	4.0	1.75:1	1.75	10	10	0.73	1.02	8.83	7

Maximum flow velocity calculated only for Searcy/U.S. D.O.T. design procedure.
No riprap required. Diversion is excavated into bedrock.

TABLE 3.4-22

SCHOOL HOUSE CANYON REFUSE AREA CHANNEL RIPRAP AND FILTER BLANKET VOLUMES

CHANNEL	RIPRAP D_{50} (IN)	LENGTH (FT)	RIPRAP THICKNESS (IN)	RIPRAP VOLUME (FT^3)	FILTER THICKNESS (IN)	FILTER VOLUME (FT^3)
CGD-5/CGRD-4	None	Diversion is excavated into bedrock				
CGD-6 (Upper)/ CGRD-8	None required. Design velocity is non-erosive (see Appendix 3.4J)					
CGD-6 (Lower)/ CGD-9 (Upper)	18	1000	23	28224	11	13524
CGD-7 (Upper)/ CGRD-7	None required. Design velocity is non-erosive (see Appendix 3.4J)					
CGD-7 (Lower)/ CGRD-3A	21	1300	26	44572	13	22594
CGRD-9 (Lower)	18	450	23	11059	11	5300
CGD-18	5	Riprap in-place.[a]				
CGD-19	10	Riprap in-place.[a]				
TOTALS				101,233 (7,087 tons)[b]		58,796 (3,822 tons)[c]

[a] CGD-18 and CGD-19 are operational diversions and will be removed at the beginning of the reclamation phase.
[b] Assumes a bulk density of 140 pcf.
[c] Assumes a bulk density of 130 pcf.

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UTAH DIVISION OIL, GAS AND MINING
PRICE FIELD OFFICE

TABLE 3.4-23

SCHOOL HOUSE CANYON REFUSE AREA
CULVERT DISCHARGE SUMMARY

REFERENCE LETTER [a]	MINING PHASE	CONTRIBUTORY WATERSHED	TOTAL DRAINAGE AREA (Acres)	DESIGN DISCHARGE[b]
	Current Operation	**Exhibit 3.4-2**	**Current Operation**	**Current Operation**
A	CGC-4	CGWS - D2A, D2B, D2E	55.4	30.40
	Final Operation	**Exhibit 3.4-2C**	**Final Operations**	**Final Operations**
A	CGC-4	CGWS - D2B, D2C, D2D, D2E	41.7	34.20[c]

(a) Culverts with the same reference letter are identically located.
(b) All design flows are based on the 100-year 6-hour storm event.
(c) Refers to maximum design peak flow throughout all mine phases. Culvert designed for this discharge value.

TABLE 3.4-24

**SCHOOL HOUSE CANYON REFUSE AREA
SUMMARY OF CULVERT CAPACITY**

CULVERT	SIZE AND TYPE	INLET TYPE	REQUIRED HEADWATER OVER TOP[a] (FT)	DRAINAGE BASINS (EXHIBIT 3.4-2C)	PEAK FLOW (CFS)
CGC-4	24" CMP with 18" HDPE Extension	Improved	1.2	CGWS-D2B, D2C, D2D, D2E	34.20

[a] Measured from the top of the rectangular inlet opening

TABLE 3.4-25

**SCHOOL HOUSE CANYON REFUSE AREA
CULVERT SUMMARY**

CULVERT	SIZE AND TYPE	OUTLET SLOPE (%)	PEAK FLOW[a] (CFS)	PEAK VELOCITY[a] (FPS)	REQUIRED D_{50} (IN)
CGC-4	24" CMP with 18" HDPE Extension[a]	60	34.20	53.4	24[b]

[a] See Appendix 3.4J for details.
[b] Several boulders ranging in size from 2 feet to 4 feet in diameter.

APPENDIX 3.4A

GOLDER ASSOCIATES REPORT, "DESIGN OF A COAL REFUSE DISPOSAL SYSTEM, PHASE II; DETAILED DESIGN, SCHOOL HOUSE CANYON REFUSE DISPOSAL FACILITY", JANUARY 1978

APPENDIX 3.4B

EXCERPTS CONCERNING REFUSE ENGINEERING CHARACTERISTICS TAKEN FROM GOLDER ASSOCIATES REPORT ON "DESIGN OF A COAL REFUSE DISPOSAL SYSTEM, PHASE I, SITE FEASIBILITY STUDY", SEPTEMBER 1977

APPENDIX 3.4C

HORROCKS AND CAROLLO ENGINEERS REPORT,
"SLOPE STABILITY ANALYSIS ON COAL REFUSE PILE AT CASTLE GATE
PREPARATION PLANT", MARCH 1983.

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DISTURBED AND UNDISTURBED AREA RUNOFF CALCULATIONS

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AS-BUILT CALCULATIONS FOR PONDS 012A AND 012B

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AS-BUILT CALCULATIONS FOR POND 013

SUPPLEMENT TO

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COMPUTER OUTPUT

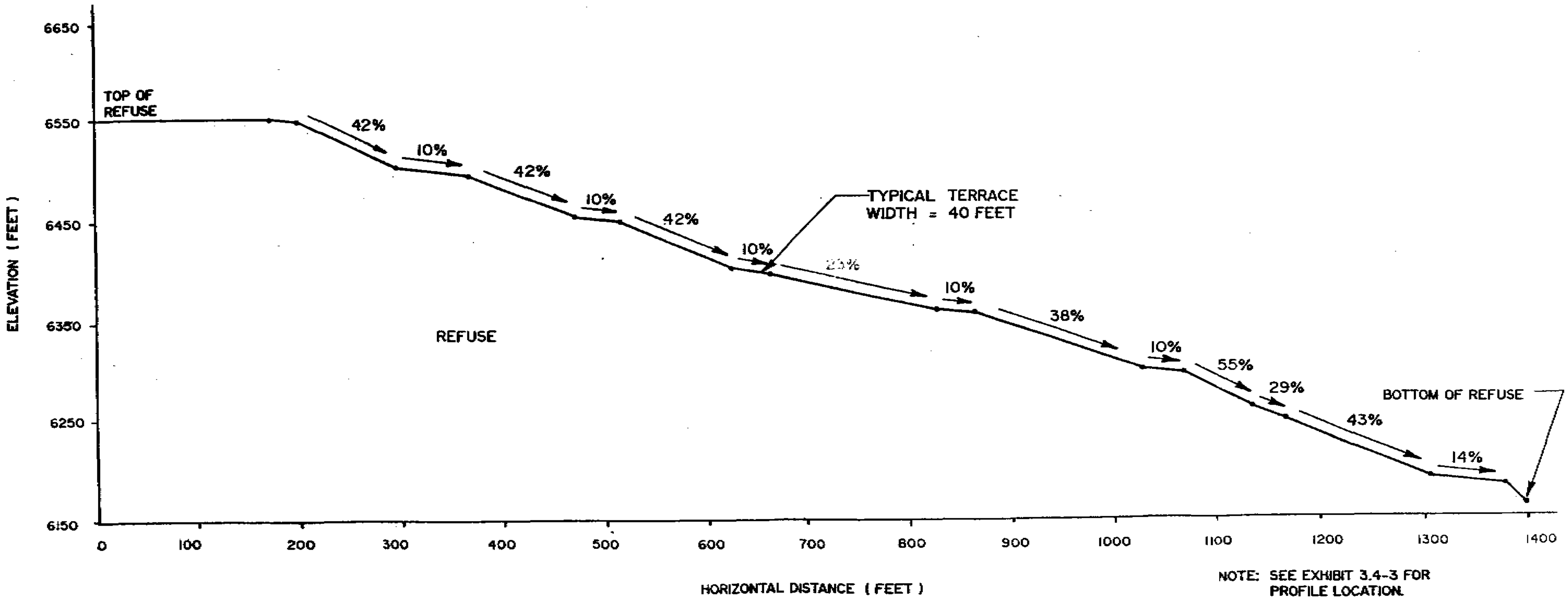
APPENDIX 3.4P-2

EMBANKMENT STABILITY ANALYSIS FOR POND 012A
COMPUTER OUTPUT

APPENDIX 3.4P-3

EMBANKMENT STABILITY ANALYSES FOR POND 012B
COMPUTER OUTPUT

REFUSE PILE PROFILE
FINAL RECLAMATION PHASE
CASTLE GATE COAL MINE
CARBON COUNTY, UTAH
A-A'



NOTE: SEE EXHIBIT 3.4-3 FOR PROFILE LOCATION.

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FIGURE 3.4-10

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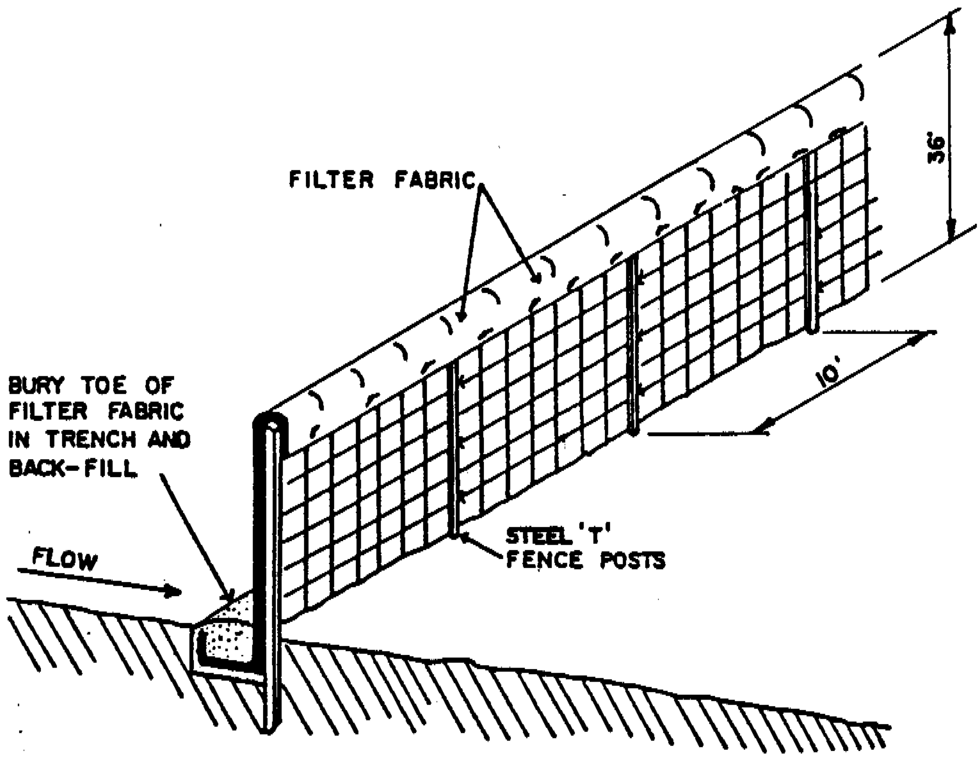

SOURCE: BARFIELD ET AL, 1981

FIGURE 3.4-1 TYPICAL SILT FENCE

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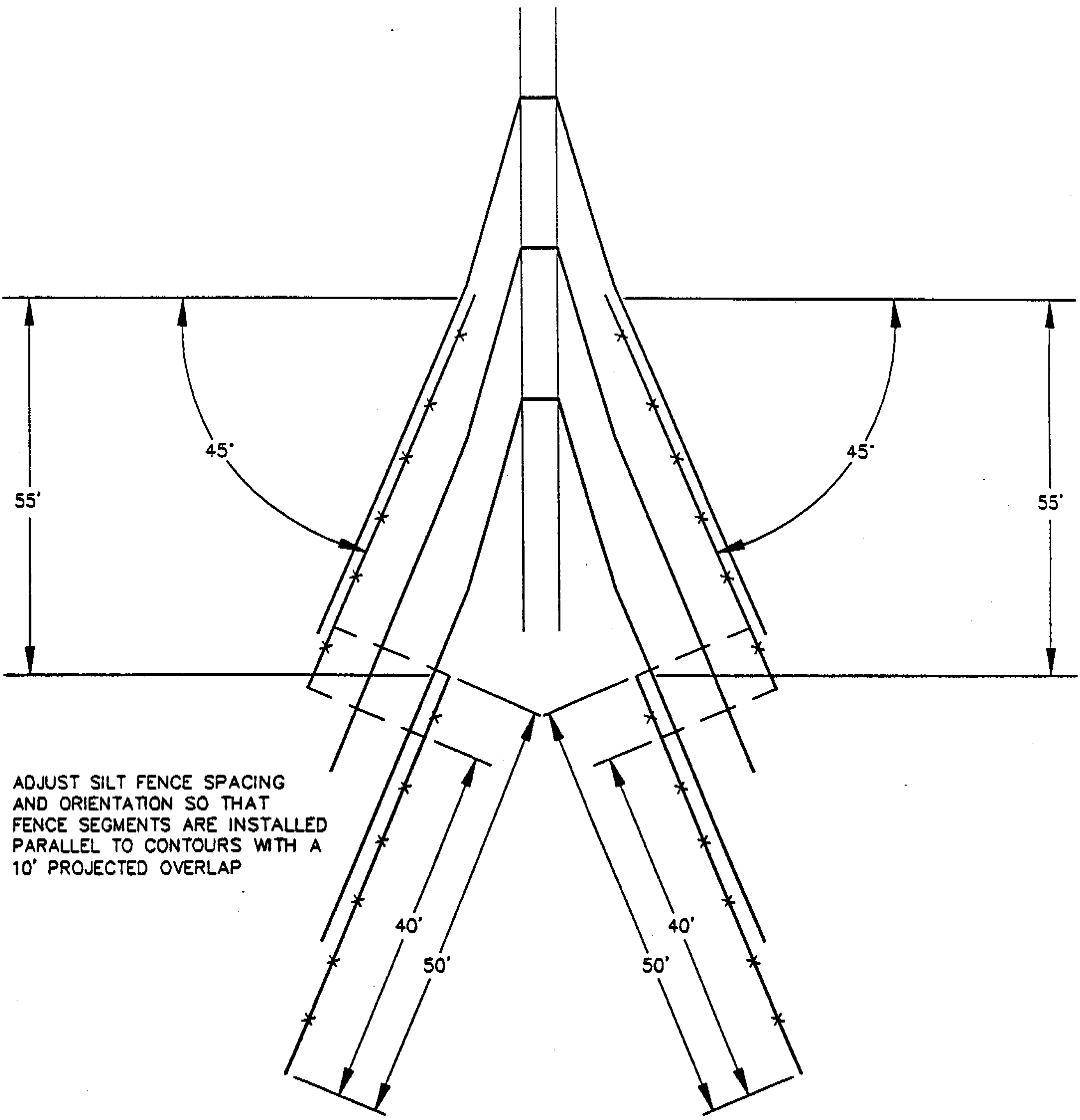
CASTLE GATE MINE
PREPARATION PLANT
RECLAMATION-PHASE I



NOT TO SCALE

FIGURE 3.4-2 TYPICAL SILT FENCE LAYOUT

FIGURE 3.4-3

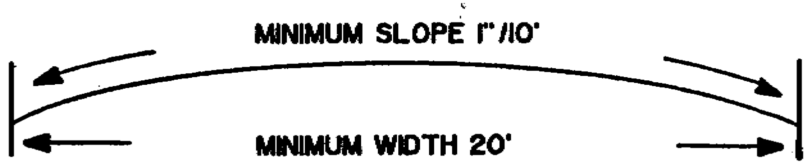
PRIMARY ROAD P-I
TYPICAL SECTION

SURFACE TREATMENT - ASPHALT

REGISTERED PROFESSIONAL ENGINEER
07973
RICHARD H.
ALLISON, JR.
5/1/92
STATE OF UTAH

I CERTIFY THAT THESE DRAWINGS WERE PREPARED UNDER MY SUPERVISION AND ARE TRUE AND CORRECT TO THE BEST OF MY KNOWLEDGE ________
RICHARD H. ALLISON JR.

SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES



SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-18 FOR DESCRIPTION AND MAINTENANCE OF ROAD

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FIGURE 3.4-4

PRIMARY ROAD P-2
TYPICAL SECTION

SURFACE TREATMENT - ROCK

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07973
RICHARD H. ALLISON, JR.
5/1/92
STATE OF UTAH

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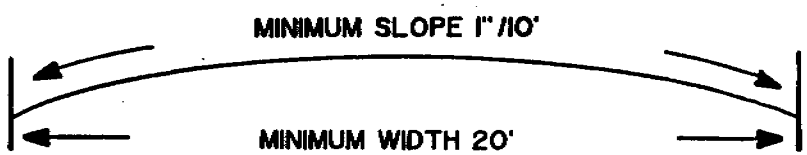

SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-18 FOR DESCRIPTION AND MAINTENANCE OF ROAD

EarthFax

FIGURE 3.4-5

PRIMARY ROAD P-3
TYPICAL SECTION

SURFACE TREATMENT - ROCK

REGISTERED PROFESSIONAL ENGINEER
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RICHARD H.
ALLISON, JR.
STATE OF UTAH

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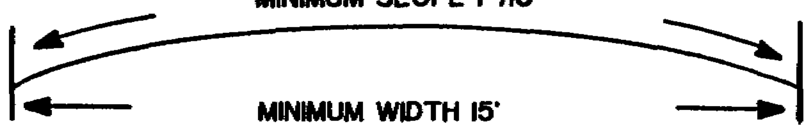

SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-18 FOR DESCRIPTION AND MAINTENANCE OF ROAD

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FIGURE 3.4-6

ANCILLARY ROAD A-I TYPICAL SECTION

SURFACE TREATMENT - ROCK

REGISTERED PROFESSIONAL ENGINEER
07973
RICHARD H.
ALLISON, JR.
5/1/92
STATE OF UTAH

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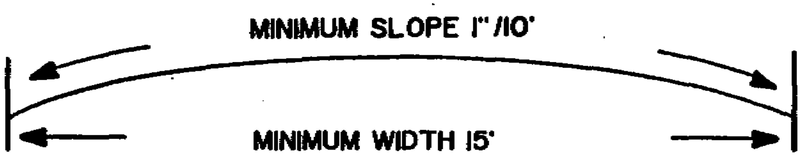
RICHARD H. ALLISON JR.

SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES



SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-I8 FOR DESCRIPTION AND MAINTENANCE OF ROAD

EarthFax

FIGURE 3.4-7

ANCILLARY ROAD A-2
TYPICAL SECTION

SURFACE TREATMENT - ROCK

REGISTERED PROFESSIONAL ENGINEER
07973
RICHARD H. ALLISON, JR.
5/1/92
STATE OF UTAH

. CERTIFY THAT THESE [illegible] WERE PREPARED UNDER MY SUPERVISION AND ARE TRUE AND CORRECT TO THE BEST OF MY KNOWLEDGE ______________
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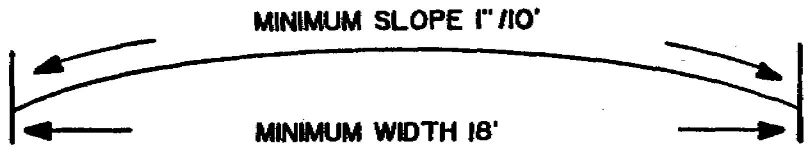
SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES



SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-18 FOR DESCRIPTION AND MAINTENANCE OF ROAD

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FIGURE 3.4-8

ANCILLARY ROAD A-3
TYPICAL SECTION

SURFACE TREATMENT - ROCK

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RICHARD H.
ALLISON, JR.
5/1/92
STATE OF UTAH

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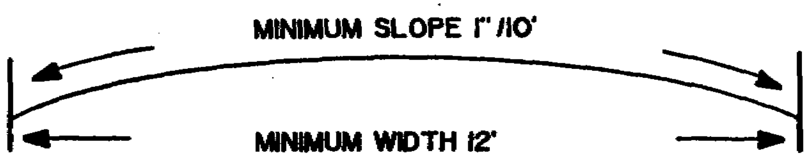
SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES



SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-18 FOR DESCRIPTION AND MAINTENANCE OF ROAD

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FIGURE 3.4-9

ANCILLARY ROAD A-4 TYPICAL SECTION

SURFACE TREATMENT - ROCK

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RICHARD H. ALLISON, JR.
3/1/92
STATE OF UTAH

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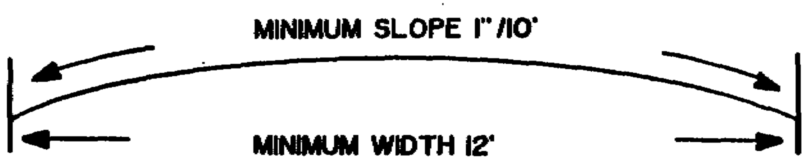
SEE EXHIBIT 3.4-2. FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES



SEE EXHIBIT 3.4-2 FOR DITCH AND BERM TREATMENT LOCATIONS AND TABLE 3.4-2 FOR DITCH GEOMETRIES

NOT TO SCALE

SEE TABLE 3.4-18 FOR DESCRIPTION AND MAINTENANCE OF ROAD

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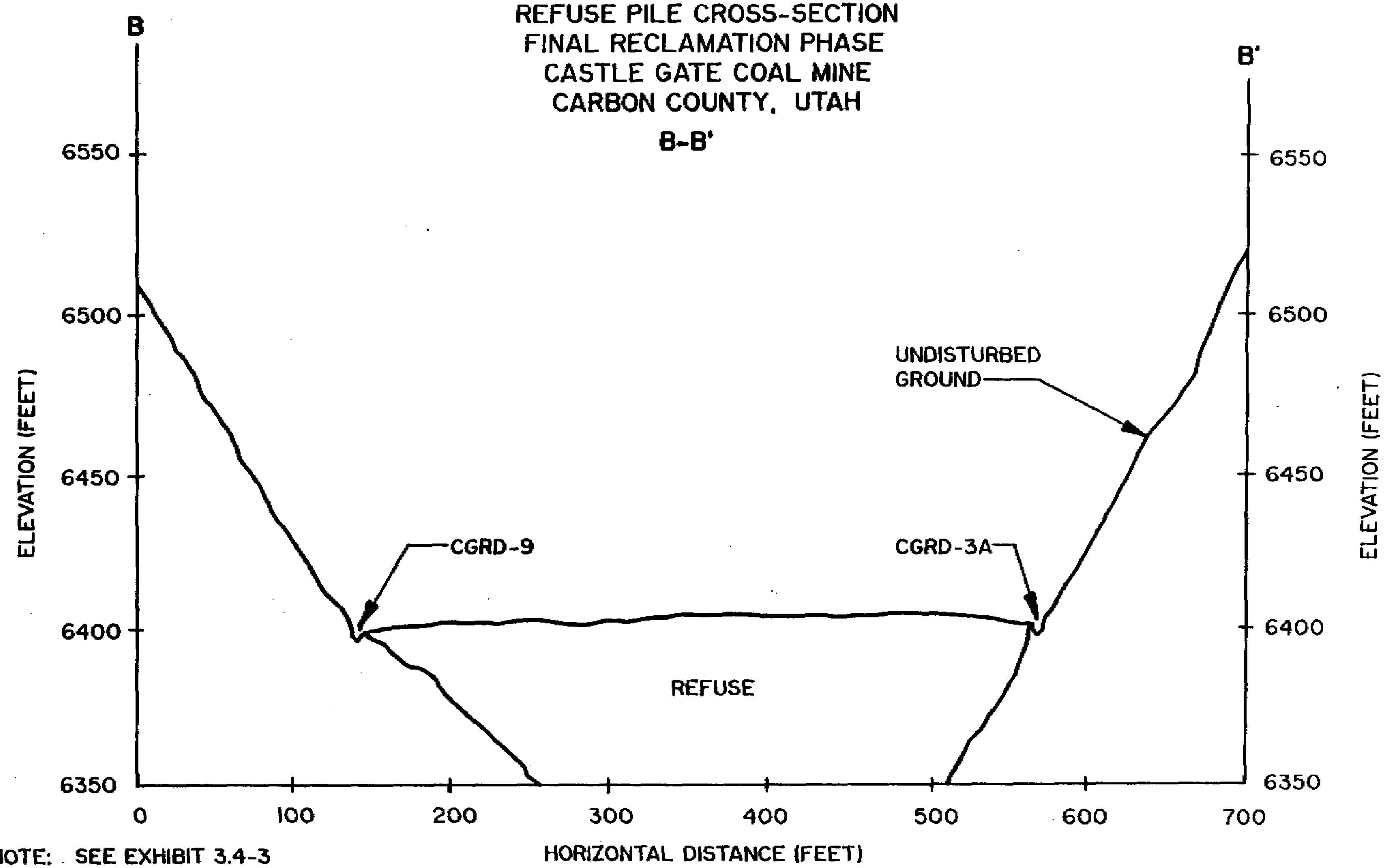
REFUSE PILE CROSS-SECTION
FINAL RECLAMATION PHASE
CASTLE GATE COAL MINE
CARBON COUNTY, UTAH

B-B'



NOTE: SEE EXHIBIT 3.4-3
FOR CROSS-SECTION
LOCATION

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FIGURE 3.4- 11

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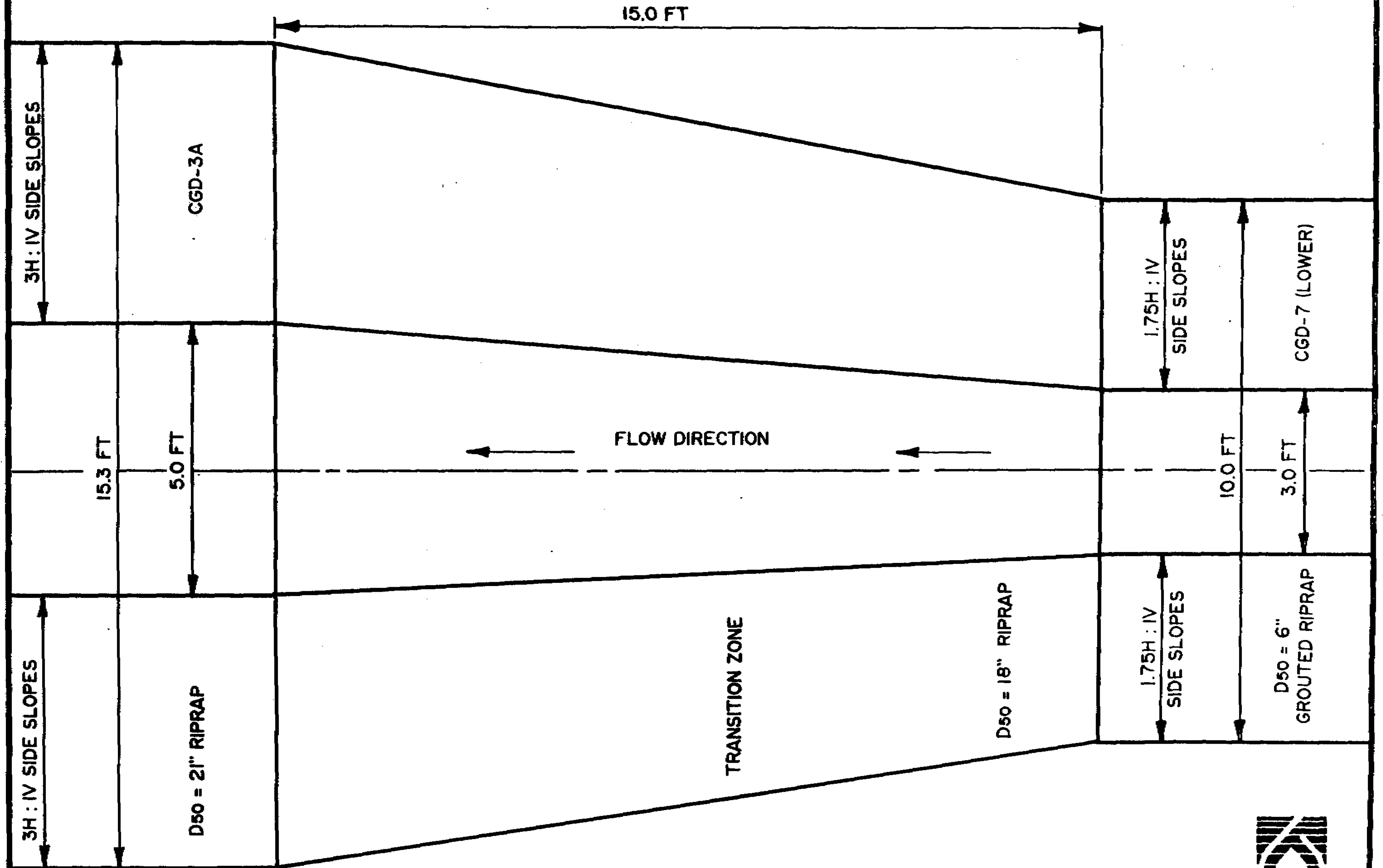

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FIGURE 3.4-12. DIVERSION TRANSITION BETWEEN GROUTED DITCH CGD-7 (LOWER) AND CGD-7 (LOWER) / CGRD-3A

EarthFax

APPENDIX 3.4A

GOLDER ASSOCIATES REPORT, "DESIGN OF A COAL REFUSE DISPOSAL SYSTEM, PHASE II; DETAILED DESIGN, SCHOOL HOUSE CANYON REFUSE DISPOSAL FACILITY", JANUARY 1978

Golder Associates

CONSULTING MINING AND GEOTECHNICAL ENGINEERS

REPORT TO
AMERICAN ELECTRIC POWER
SERVICE CORPORATION
ON THE

DESIGN OF A COAL REFUSE DISPOSAL SYSTEM
PHASE II: DETAILED DESIGN

SCHOOLHOUSE CANYON
REFUSE DISPOSAL FACILITY

CASTLEGATE UTAH

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January 1978 S77212

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CONSULTING MINING AND GEOTECHNICAL ENGINEERS

January 18, 1978
E/78/47

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

ATTN: MR. LEE MC CLOSKEY

SUBJECT: DESIGN OF COAL REFUSE DISPOSAL SYSTEM
PHASE II: DETAILED DESIGN
SCHOOLHOUSE CANYON REFUSE DUMP FACILITY

Gentlemen:

Please find enclosed Golder Associates final Phase II design report for the proposed Schoolhouse Canyon Refuse Dump Facility. As previously agreed, we are forwarding an original version plus a completed bound copy of the report herein. A further copy has been forwarded, as requested, to Lancaster, Ohio.

We trust that this report (along with the Specifications previously forwarded) is sufficiently comprehensive to adequately describe the proposed scheme, and to permit extraction of the required information for submission to Governmental Agencies, as appropriate. We would appreciate receiving copies of any such submissions for our files. For your reference, the agencies which are considered to have some regulatory involvement are summarized in the attached table. Specific approval to commence construction however is not believed necessary so long as the MESA District Manager has already been advised of the AEPSC intent to use Schoolhouse Canyon as a disposal facility.

Please feel free to contact either myself or Allen Gass should you have any questions or require any further information or assistance.

In conclusion, I should like to add that we have appreciated the opportunity afforded us, in being involved in the planning of your coal refuse disposal scheme. We wish you every success in commissioning the new mine, plant and disposal facilities as expeditiously as possible.

Sincerely yours,

GOLDER ASSOCIATES, INC.

G A Mathieson

Graham A. Mathieson
Project Engineer

GAM:mm
77212
Attachment: Table of Governmental Regulating Authorities

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ccd.chapter3/4

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1.0 INTRODUCTION

1.1 On completion of the Phase I Siting Feasibility Study, Golder Associates (GAI) was requested by American Electric Power Service Corporation (AEPSC) to proceed with Phase II: the detailed design of a coal refuse disposal facility to be located in Schoolhouse Canyon, Castle Gate, Utah. During this phase several visits were made to the site by GAI engineers involved in the project: Messrs. Gass, Mathieson, Bowen, Cross and Coddington. A presentation of preliminary plans was also made by Mr. Mathieson at the ofices of the State of Utah, Department of Natural Resources, Division of Oil, Gas and Mining (DOGM) in Salt Lake City.

1.2 During the course of Phase II work Technical Specifications were prepared and forwarded to AEPSC on the recommended refuse disposal equipment and on the proposed Refuse Haul Road System, Diversion Ditch and Settling Pond. Additional recommendations were made on improvements considered necessary on the existing Barn Canyon Drainage Channel.

1.3 This report has been prepared in fulfillment of Golder Associates' Phase II work commitment. The purpose here is to present plans of the proposed refuse pile development, and to provide additional design support for the drawings and specifications which have been previously forwarded.

1.4 The report is structured to firstly discuss in Section 3 the impact of the recently published (December 1977) Surface Mining Control and Reclamation (OSM) regulations. Considerations relating to refuse disposal operations, including dump design, equipment and costs, are then presented in Section 4. Section 5 develops the necessary hydrologic analysis in support of the Diversion Ditch and Settling Pond Designs. Finally, a discussion is given in Section 6 on the geotechnical considerations which have been incorporated into the design, and which will influence refuse disposal operations.

2.0 SUMMARY

2.1 Design of the Schoolhouse Canyon refuse dump and its associated structures has been based on accepted engineering practices and complies with applicable State and Federal Regulations cited. Some uncertainty presently exists, however, about the legality of refuse disposal within a canyon site in the light of recent (December, 1977) Surface Mining Control and Reclamation Regulations. Despite this uncertainty, the design was completed at the direction of AEPSC to form a comprehensive design package on which to seek government approval. The Division of Oil, Gas and Mining and other State of Utah regulating authorities have given conditional approval of the design concept, pending review of the final plans and specifications.

2.2 The equipment selection and dump design along with the construction and monitoring recommendations presented in this report are predicated to a great extent on an assumption that the combined plant refuse will be free-draining, and will generally permit normal placement and compaction operations leading to a stable refuse pile. This assumption was based on a study of samples fabricated from the existing preparation plant refuse and the expectation that the refuse will continue to be generally free of clayey materials. It is recommended that further laboratory testing and geotechnical analysis be undertaken on a representative

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refuse sample once the new preparation plant is operating. The results of this work, together with the operating experience accumulated to date, will be used as the basis for any required modification of placement procedures or dump slope configurations.

2.3 Detailed design should be initiated on the thickener underflow (plant filter cake under current plant design), underground injection scheme which was conceptually introduced in the GAI Phase I report. Successful implementation of such a scheme would reduce refuse moisture control problems in the dump, improve stability and lower overall disposal costs.

2.4 The dump and settling pond construction should be inspected by a qualified AEPSC geotechnical engineer or outside consultant. Full-time supervision is recommended during the Settling Pond Embankment Construction phase. After the operation commences, the refuse pile should also be inspected periodically to review all monitoring data and to recommend changes, if necessary, to placement procedures. Between inspections, any unusual conditions which develop should be communicated to the inspecting engineer by the refuse system operators.

2.5 The refuse pile as designed has an estimated capacity of 3 1/2 million tons, which corresponds to about 7 1/2 years' life at currently projected production rates. Depending on dump stability considerations, and on the comparative economic merit of switching disposal operations to alternate sites (Barn or Royal Canyons), this life could be extended by raising the elevation of the diversion ditch or by continued dumping adjacent to it.

2.6 Estimated Capital Costs for the Refuse Disposal Facility are summarized below:

Haul Road Construction	$ 93,000
Diversion Ditch Construction	$ 28,500
Settling Pond Construction	$140,000
Barn Canyon Channel Improvements - allow	$ 10,000
	$271,500
Add 20% Contingency	54,500
Sub-Total Construction	$326,000
Other Costs	
Disposal Equipment (Incl. sales tax)	$552,000
Golder Associates Fees	$ 97,000
Total Estimate - Pre-operating Expense for Refuse Disposal	$975,000

2.7 Estimated unit operating costs range between $0.51 and $0.61 per ton of refuse disposed between the years 1978 and 1982 respectively. If the total pre-operating expense was to be amortized over the 3.5 million ton dump capacity, the total unit cost including depreciation is expected to range between $0.79 and $0.90 per ton of refuse. This estimate ignores the effect of taxes on capital invested, an additional truck purchase in 1979, and the remaining equipment life available at the end of 7 1/2 years.

3.0 LEGAL CONSIDERATIONS

3.1 MESA/EPA/Other Regulations

During Phase I all applicable State and Federal regulations which might affect the design and construction of the refuse pile were studied and summarized in the Phase I report. The regulations reviewed at that time included those published by EPA, MESA, the State of Utah Department of Natural Resources and the Utah State Division of Health. This review was supplemented by telephone discussions with MESA and EPA representatives. The design of the Schoolhouse Canyon Refuse Disposal Facility (including the associated Diversion Channel and Settling Pond) was undertaken in conformance to these regulations.

3.2 Comments on the New OSM Regulations

The Surface Mining Control and Reclamation Act of 1977, Pub. L.95-87, led to the creation of a new Office of Surface Mining Control and Reclamation (OSM) within the Department of Interior, with the responsibility of publishing proposed rules in relation to the provisions of the act. These were promulgated on September 7, 1977, in the Federal Register. The proposed rules covered both surface mines and the surface impacts of underground mines.

In general the rules reflected and strengthened all of the existing EPA standards with respect to "protection of the hydrologic system", and also contained many additional requirements, including those for reclamation. The most important of these was in section 715.15 wherein it was stated that: "waste material must not be disposed of in valley or head-of-hollow fills". Clarification on this was sought by telephone on September 21, 1977, from one of the authors of the regulations, Mr. George Davis. He said that a scheme of the type planned for AEPSC would be unacceptable under the regulations unless the disposal area:

a. could be classified as other than a "valley" or "head-of-hollow" fill;
b. was of a minor nature; or
c. did not involve preparation plant refuse.

The Schoolhouse Canyon Refuse Disposal Scheme meets none of these conditions. The new rules were also discussed by telephone with EPA in Denver, and with the Division of Oil, Gas and Mining, Department of Natural Resources, State of Utah (DOGM).

AEPSC was advised of Golder Associates' concern as to the possible prohibition of the Schoolhouse Canyon Dump Facility. GAI was directed by AEPSC to complete the design as soon as possible so as to provide a basis for government approval. It was also agreed that preliminary comments should be sought from the DOGM who had issued the Braztah mining permit, and who would probably be responsible to administer the final SMCR regulations.

Accordingly, GAI proceeded with the design and on November 21, 1977, GAI and AEPSC presented preliminary plans to representatives of the State of Utah Divisions of Oil, Gas and Mining; Health; Solid Waste Management and Water Rights. In view of the favorable reaction expressed by those in attendance at the meeting, the design continued, assuming that the canyon disposal approach would be approved. It was considered desirable that the final design should incorporate, where possible, the proposed OSM final rules, as if they would become mandatory. Subsequent written communications received from the above listed Government Divisions have been forwarded with comments to AEPSC, as attachments to the Technical Specifications.

On December 13, 1977, final OSM rules were published in a revised format which segregated Surface from Underground Mining. Valley and head-of-hollow waste fills remain prohibited in the rules relating to surface mining, to which cross-reference is made in the rules dealing with underground mines.

A lengthy preamble, to the new final rules in the Federal Register, dealt with comments received from the public on the proposed rules. It is understood from this discussion that "wastes" were generally deleted from the final regulations because it was recognized that the disposal of waste materials (such as preparation plant refuse) is controlled separately under the MESA regulatory program. The initial SMCR regulatory program as it now stands regulates only such refuse wastes, where they are used: in backfilling or grading of mined areas; in impoundments; or in dam construction. The preamble states that "Complete control over placement of . . . coal processing wastes (etc). . . will not be addressed until the permanent regulatory program with the exception that they are not allowed in valley or head-of-hollow fills. . . This prohibition is necessary to keep such materials out of drainage channels. . . "

In a telephone conversation with Mr. Paul Reeves of OSM on January 9, 1978, GAI learned that in the initial regulatory program the prohibition of wastes in valley or head-of-hollow fills was intended only to preclude the mixing of such wastes with spoil and overburden from surface mines, and to regulate dams constructed of, or impounding, waste materials. He said that the disposal of waste materials will continue to be controlled under current State or Federal (eg, MESA) programs until the promulgation of a final OSM regulatory program. Finally, with respect to a canyon refuse disposal scheme, he said that disposal operations could commence if a State permit had been issued. The operation would, however, be subject to OSM inspection for compliance with the initial regulatory program. The company, if found to be in "non-imminent hazard" violation of these standards, would be given "a reasonable time of abatement" up to a maximum of 90 days.

Despite the above comments, strict adherence to the published OSM rules leaves some uncertainty as to the legality of the Schoolhouse Scheme. It is probable, however, that, as long as one can demonstrate adequate protection of the hydrologic system, then OSM should not be concerned, leaving the approval of the dump design to MESA. Since existing MESA design requirements have been met, hopefully there should not be any problems.

3.3 Applicable OSM Regulations (Dec. 13, 1977)

This section aims at briefly summarizing only those new OSM rules which have been interpreted as affecting the design, construction and operation of the refuse pile and its associated structures. The designs previously forwarded and presented herein are believed to be in general conformance with these OSM standards, to which all mining operations must comply by May 3,1978.

The following points, in the order in which they appear in the regulations, are considered to be the most important:

a. During reclamation operations, waste (i.e. refuse) materials must be covered by a minimum of four feet of "non-toxic" material.

b. Valley or head-of-hollow fills must be placed on adequately prepared sites, must utilize underdrains, must have slopes not exceeding 2:1, and must be constructed in lifts of four feet thickness or less. (Since "waste" materials are supposedly prohibited in such fills, the applicability of these rules is uncertain.)

c. Effluent discharge limitations will be determined on a case-by-case basis but, for precipitation events up to the 10-year, 24-hour storm, the effluent must not exceed 45 mg/l Total Suspended Solids, 7 mg/l Iron, 4 mg/l Manganese, and must have a pH between 6.0 and 9.0.

d. Surface water must be comprehensively monitored in accordance with an approved program.

e. Diversion structures may be required by the regulatory authority and, if these are of a permanent nature, they must be designed to safely pass the peak runoff from the 100-year recurrence interval recipitation event.

f. Sediment control measures are mandatory and must be designed to provide:

- at least a 24-hour detention time;
- at least 1 sq. ft. of surface area/50 gallons/day inflow from the 10-year, 24-hour precipitation event;
- additional sediment storage volume equal to 0.2 acre-feet for each acre of disturbed land within the upstream drainage area (the proposed rules, however, required the pond to be sized to have a capacity equal to the 10-year, 24-hour storm runoff plus the sediment allowance - the above-stated surface area provision wa substituted in the final rules);
- a spillway system designed to safely discharge runoff from the 100-year, 6-hour precipitation event if the pond embankment exceeds 20 feet height or if the pond has a storage volume of 20 acre-feet or more;
- for the removal of sediment accumulation reaching 80% of the design sediment storage allowance;
- an embankment static safety factor of 1.5 for the normal pond water level;
- a minimum embankment top width greater then (H+35)/5 where H is the upstream embankment height.
- appropriate seepage control barriers;
- construction supervision and certification by a registered professional engineer;
- ultimate removal of the pond and subsequent return of the ground surface to the approximate original contour, when mining operations cease.

g. Discharge structures from sedimentation ponds and diversion structures must be controlled, where necessary, using appropriate energy dissipators to minimize erosion.

h. To avoid contamination of natural surface waters, waste materials from coal preparation plants must be buried or otherwise treated no later than 90 days <u>after</u> the cessation of filling in the disposal area.

i. Haul roads must be constructed in a manner which minimizes the potential for additional contributions of suspended solids to natural stream flow. These must be removed and regraded when no longer required.

j. The overall sustained gradient on the haul roads must not exceed 10%. The roads must provide drainage ditches and other structures capable of passing the peak runoff from a 10-year, 24-hour precipitation event and must also be surfaced with a durable, non-acid-forming material.

k. Topsoil must be removed from the areas to be disturbed by surface operations and stockpiled for use in revegetation when such areas are no longer required for mining operations.

l. "A diverse, effective and permanent vegetative cover capable of self-regeneration and plant succession, and adequate to control soil erosion: must be established on all land disturbed by mining operations.

m. Operator must pay a reclamation fee of 15 cents for each ton of underground coal produced.

The proposed Schoolhouse Canyon Refuse Dump Facility Design and the recommendations made within this report are intended to satisfy the above rules, and all other applicable State and Federal regulations cited to date. In any event, a design basis is now complete, thus paving the way of further refinement, government submissions, permit applications, and discussions, leading to final approval of the scheme.

3.4 Land

As shown on Figure 6-1(a) later in this report, the proposed refuse dump is located almost entirely within a square area of State Coal Surface Rights. The haul road system, diversion channel and settling pond as proposed lie within Federal Fee Land boundaries. thus, for the Schoolhouse Canyon Facility, Land negotiations are not necessary. The effect of the refuse pile being located on state land may have some bearing on the channels necessary to obtain approval.

4.0 REFUSE DISPOSAL CONSIDERATIONS

4.1 General

This section outlines those considerations involved in the transport and placement of refuse, and in the design of the Schoolhouse Canyon dump. The criteria affecting the dump design are firstly discussed, dump development plans and strategy are then presented, followed by comments on disposal equipment. Finally an operating cost budget is given. Further discussion on the geotechnical aspects of this design, with particular reference to refuse drainage, is given in Section 6.

4.2 Refuse Dump Design

As discussed in the Phase I report, the only practical method of dump development in canyons was considered to be one which involves disposal of-refuse in level layers from the canyon mouth upwards. This was to entail a system of dump face ramps which would be progressively extended through time to grain elevation.

Following an AEPSC decision on October 31, 1977, to utilize a single in—canyon settling pond (see section 5.4.4), design of the Schoolhouse Canyon Refuse Dump Facility proceeded on the basis of a dump toe in the Canyon bottom at 6220 elevation.

4.2.1 Dump Face Ramp System

For the design of the ramp system referred to in Phase I, the following criteria were adopted:

a. Single truck roadway width (Minimum 20 feet), due to an anticipated low traffic density.

b. Ramp gradient maxima of 10% and 8% on straights and curves respectively, in response to a OSM requirement and truck manufacturer's recommendations.

c. Minimum centerline curve radius of 50 feet on switchbacks (truck turning radiums: 30 feet).

d. Maximum inter-ramp slope angle on the dump face of 2:1 (MESA limitation).

An initial design was developed using a ramp system which commenced at the dump toe. However, due to the above restrictions, the narrow canyon width and the large area required by switchbacks, this design resulted in an almost continuous series of switchbacks, a correspondingly flat overall dump face angle, and therefore a tonnage capacity smaller than that which would otherwise be possible. Further trial designs lead to a haul road system as presented in the specifications and as shown on Figure 6-1(a). The proposed system requires the construction of a main canyon haul road from the preparation plant to a point, on the north canyon wall, 200 feet upstream from the dump toe at elevation 6320. At this point sufficient canyon width is available to reduce the number of switchbacks, and therefore to maintain a reasonable overall dump slope angle. A temporary haul ramp is required to provide access for dumping of the initial 100 vertical feet of refuse. This is discussed in section 4.3.

The switchback configuration is the critical factor in the determination of the overall dump slope angle. Advantage has been taken of this to design the ramp system with additional truck passing width, while still maintaining a maximum inter-ramp dump face slope of 2:1. A typical switchback design which demonstrates this concept is shown in Figure 4-2(a). Figure 4-2(b) gives the design ramp cross-section. The ramp system initially designed to reach the 6560 elevation. However, the minimum design elevation for the diversion ditch of 6550 has been used as a limit for the maximum pile capacity. Figure 6-2(A) of section 6 shows a longitudinal section through the proposed refuse dump. The overall dump face angle from toe to crest is 19.

4.2.2 Dump Capacity

Subsequent to the design of the dump face ramp system, volumes were determined at 20-ft. vertical intervals on the basis of the contour overlay given in Figure 4-2(c). The dump tonnage capacity curve of Figure 4-2(d) was obtained using a tonnage factor of 118 pounds/cubic foot for compacted refuse, and the production schedule of Table 4-2 (taken from the Phase I report).

The proposed dump has a capacity in the order of $3.5x10^6$ tons when filled to the 6550 elevation. This corresponds to a life of approximately 7 1/2 years. Depending on the stability of the dump and on the economic merit of switching disposal operations to alternate sites, this life could be extended by relocating the diversion ditch or by continued dumping adjacent to it. As discussed in the Phase I report, however, hauling costs at this time would probably favor another site such as Barn or Royal Canyon. Further analysis of these possibilities should be undertaken in late 1978, such that early moves could be initiated to overcome existing constraints (land, powerlines) and to design and construct other facilities (hydrologic structure, highway underpass to Royal Canyon, etc.).

4.3 Dump Development Strategy

4.3.1 Yearly Dumping Plans

Figures 4-3(a) to 4-3(d) depict dump configurations corresponding to the end of production years 1 (1978), 3 (1980), 5 (1982), and 7 (1984).

As shown on the 1978 plan, the proposed dump involves the construction of a Temporary Haul Ramp from the end of the main haul road at 6320 elevation, down in to the canyon floor to reach the 6220 dump toe. During the first year this ramp would be progressively covered. In 1979 and thereafter, successive lifts would be placed and compacted to integrate the ramp system into the dumpface as shown in the figures.

TABLE 4-2

REFUSE PRODUCTION SCHEDULE

Year	Est. R.O.M. Production	Clean Coal	Total Plant Refuse	Breaker Refuse	Coarse Refuse	Fine Refuse	Filter Refuse	Total Refuse	Cum. Total Refuse
1978	1,661	1,411	250	63	84	52	51	278	278
1979	2,079	1,765	314	78	106	66	64	349	627
1980	2,521	2,140	381	95	128	80	78	423	1,050
1981	2,890	2,454	436	109	147	91	89	484	1,534
1982	3,297	2,800	447	124	168	104	101	552	2,086
1983	3,439	2,920	519	130	175	108	106	576	2,662
1984	3,534	3,000	534	133	180	112	109	593	3,255
1990					These annual tonnages				6,813
1995									9,778
2000					remain the same until 2007.				12,743
2007									16,894

NOTES: 1. All figures in thousands of dry tons, except Total and Cumulative Total Refuse Figures, which are in thousands of wet tons.

2. Derivation of this data was discussed at length in Section 3 of Golder Associate Phase I Report, "Site Suitability Study".

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4.3.2 Operational Factors

Refuse will be hauled and end-dumped on the top of the dump in piles spaced such that the bulldozer could spread the material in lifts of not more than two feet compacted thickness. In addition to bulldozing refuse, the tractor would rip any frozen dump surfaces, remove snow from the dump and generally prepare the surface prior to spreading subsequent lifts. As shown on the figures, the bulldozer would progressively extend the dump perimeter drainage ditches in the colluvium walls adjacent to the dump. These would be connected to the haul ramp drainage ditch at switchbacks to minimize the amount of dump face erosion. At the end of each working day, the bulldozer would also grade the dump surface to ensure good drainage in these perimeter dump drains.

To satisfy legal and geotechnical requirements, all topsoil, vegetation and other organic material must be removed prior to forming the refuse pile. Some of this unsuitable foundation material will be removed during the construction of the haul road system. However, it is recommended that, as the dump develops, all such material (along with some other "make-up" colluvial soils in the valley floor) is loosened by the bulldozer, excavated by the 988 front-end loader, and placed at the front of each lift at the dump outface, to facilitate reclamation and to reduce erosion. This material will also be compacted to form a "skin" or dump surface facade of about four to five feet thickness, comprising reasonable vegetation - supporting soil. Experiments should be conducted to determine soil additive needs and to identify those plant species which would satisfy the long-term SMCR and State requirements for revegetation.

4.4 Refuse Disposal Equipment

4.4.1 Comparison of Hauling Unit

Several types of refuse hauling units were considered during Phase II. The following were regarded as viable alternatives, and are compared in table 4-4(a):

a. End-dump off-highway trucks - 35 ton class such as the Cat 769B and Wabco 35C units.

b. Rubber-tired scrapers such as the Cat 631D, 37.5 ton unit.

c. A specifically designed coal refuse hauler - the MRS 1-110S/RH110, 50-ton unit.

Despite its very low ground pressure, good maneuverability and capability of spreading material in thin lifts, the MRS unit is currently operating at only one mine, (AEPSC, Southern Apalachian Coal Company). Although it has apparently performed well for the past 18 months, it has not been adequately field-proven and therefore is not recommended for initial purchase. Moreover, to ensure hauling dependability, two 50-ton MRS units would be initially required and as such would then provide excess hauling capacity, and would have a considerably higher capital cost compared to end-dump trucks.

The apparent absence of clay minerals in the refuse suggests that it should be free-draining once placed in the dump, and as such, the end-dump truck ground pressures would be tolerable. Refuse draining characteristics, along with possible methods to expedite it, are discussed in Section 6. Consideration was given to a 4-ton, 4-wheel drive truck of the International series to provide better insurance against bogging down. However, these units are very expensive and typically experience high operating costs. It was recommended in the Equipment Specifications that serious consideration should be given initially to the possibility of renting two trucks. This would provide, prior to a commitment to purchase, a probationary period to evaluate the suitability of the trucks to actual operating conditions.

TABLE 4-4(a)

COMPARISON OF HAULING UNITS

Factor	END DUMP TRUCKS	RUBBER TIRED SCRAPERS	MRS REFUSE HAULER
1. Typical Model	Cat 769B, Wabco 35C	Cat 631D	MRS 1-110S/RH110
2. Drive train	2 wheel	2 wheel	4 wheel
3. Rated Payload	35 tons	37.5 tons	50 tons
4. Struck Capacity	22 cy	21 cy	32 cy
5. Field Proven	Yes	Yes	No - 1 unit only
6. a. Tire Footprint Pressure	75-80 psi	52-62 psi	45-57 psi
b. Tire Flotation on Soft Ground	Poor	Good	Very Good
7. a. GVW: FWHP ratio	303-318 lb/HP	36.4 lb/HP	411 lb/HP
b. NVW: Payload ratio	0.88:1	1.16:1	0.78:1
c. Performance on adverse grades	Good	Reasonable	Poor
d. Traction	Reasonable	Good	Very Good
8. Heated Body (freezing in winter)	Yes	No	No
9. Fines leakage	Can be overcome with sideboards, tailgates.	Yes, from scraper bowl	Yes, probable
10. Spreading Capability	Poor - Auxiliary dozer required	Some - Auxiliary dozer required	Has spreader blade and dozer blade attachment - possible elimination of auxiliary dozer.
11. Capital Cost/unit 2	$147,000-$165,000	$249,500	$260,500
12. Total New Tire Cost 3	$8,601-$10,836	$13,563	$20,808
13. Owning and Operating cost/hour 4	$45.18	$64.14	$62.62
14. Other Comment	Lower unit costs as haul increases.	Scrapper bowl can be dropped for added braking safety.	Unit designed for refuse hauling and may prove a good selection long term.

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Data sources:

1. From calculations made on the basis of equipment and tire manufacturer's data.
2. From quotations received from equipment distributors, FPB Helper, Utah, with certain options, excluding sales tax.
3. From Jensen Tire Co., Salt Lake City.
4. From Owning and Operating Cost Estimates made by GAI during Phases I and II.

End-dump trucks were considered a better choice than scrapers because of:

a. their exhaust-heated body,
b. the fact that high moisture content fines should be more easily retained within the truck than within a scraper bowl,
c. their lower capital cost,
d. their probable lower unit operating costs beyond year 1 compared to scrapers, and
e. the their suitability for longer hauls to alternate sites in the future.

4.4.2 <u>Truck Requirements</u>

As a basis for required truck productivities, an estimated Daily and Hourly Refuse Production Schedule has been derived fom data presented in the Phase I report and modified to reflect a five-day/week rather than six-day/week breaker operating schedule. This schedule is included here as Table 4-4(b).

Gradeability charts for the Cat 769B and Wabco 35C trucks are given in Appendix a. Neither truck is considered significantly better mechanically, operationally or economically than the other. However, for estimating purposes, the Cat 769 gradeability chart was used in conjunction with Preparation Plant and Breaker haul profiles in analyses of hauling productivities. These analyses are also given in Appendix A, for the first five years of operation.

Due to an estimated low heap angle for the wet combined plant refuse, difficulties may be experienced in maintaining the 35-ton rated truck payload, when hauling upgrade. These problems, however, were considered to be resolvable through sequential truck body modifications

TABL[illegible]-4(b)

ESTIMATED DAILY AND HOURL[illegible]USE PRODUCTION SCHEDULE

YEAR	BREAKER REFUSE PRODUCTION Wet tons/hr 1	BREAKER REFUSE PRODUCTION Wet tons/day 1	PREP. PLANT REFUSE PRODUCTION Wet tons/hr 2	PREP. PLANT REFUSE PRODUCTION Wet tons/day 3	Hrs. of Daily Pre. Plant Operation Required to Meet Production	Total Refuse Production Wet tons/day
1978	12.4	260	150	963	7	1,223
1979	15.3	321	150	1,215	8	1,536
1980	18.6	391	150	1,473	10	1,864
1981	21.8	458	150	1,684	11	2,142
1982	24.3	510	150	1,921	13	2,431
1983	25.5	536	150	2,003	14	2,539
1984	26.1	548	150	2,065	14	2,613

Notes:
1. Based on 50 weeks/year, 5 days/week, 3 shifts/day, 7 hours/shift, i.e., 5,250 operating hours/year.
2. Based on 147 dry tons/hour from flow sheet x moisture content (1.133) x plant running factor (90%).
3. Based on 220 operating days/year.

as discussed in the Equipment Specifications cover letter. For the purpose of truck productivity analysis, therefore, the 35-ton rated payload was accepted.

A summary of estimated yearly truck productivities taken from the Appendix A results is given in Table 4-4(c). This data was combined with the hourly production schedule data of Table 4-4(b), to estimate the yearly truck requirements which are given in Table 4-4(d). Initially two trucks should meet the production demand, while beyond 1978 it is estimated that three units will be required.

On day shift in 1978 it is anticipated that two trucks would haul from the 100-ton breaker, and 300-ton preparation plant refuse bins. they would start and end the shift with these bins empty. On average:

- the plan would require:
  963 tons divided by 100 tons/truck hr. = 9.63 truck hours.
- the breaker would require:
  260/3 tons divided by 80 tons/truck hr. = 1.08 truck hours
- therefore, overall truck utilization on day shift =

  $$\frac{\text{truck operating hours}}{\text{truck available hours}} = \frac{9.63 + 1.08}{2 \text{ trucks} \times 7.5 \text{ hrs.}} = 71\%$$

On afternoon and night shifts only one truck would haul breaker refuse as needed. On average on both shifts:

- the breaker would require:
  260 x 2/3 tons divided by 80 tons/hr. = 2.17 truck hours
- therefore, overall truck utilization on afternoon and night shifts =

  $$\frac{\text{truck operating hours}}{\text{truck available hours}} = \frac{2.17}{7.5 \text{ hrs.} \times 2 \text{ trucks} \times 2 \text{ shifts}} = 7\%$$

This schedule was assumed to estimate manpower needs and operating costs. However, in the light of actual operating experience, a better approach during 1978 might be to allow the breaker refuse to overspill the bin for subsequent load-out with the 988 loader

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immediately prior to, or on, the following day shift. This would probably be cheaper, safer and eliminate the possible need for early purchase of a dump light-plant. A further alternative would be to increase the breaker refuse bin capacity.

4.4.3 Support Equipment

4.4.3.1 Crawler Tractor

The estimated average daily refuse production is 2000 tons for the first few years of operation. This is equivalent to about 1,480 LCY. Assuming that all dozer work would be done on day shift, with an effective operating time of seven hours, the required Dozer Productivity = 212 LCY/hour.

Considering the use of Caterpillar equipment and using an "average" operator, a 45 min/hour job efficiency, a straight blade and level dozing with a 100 ft. push, estimated productivities** are: 207 LCY/hour and 325 LCY/hour for D6S and D7S dozers respectively. A D6 would be barely adequate to meet production with no allowance for down time. The purchase of a crawler tractor of at least 200 FWHP was recommended in the Equipment Specifications cover letter, after consideration of these productivity estimates, and of the following activities which will also be performed by the machine:

**Caterpillar Performance Handbook, Edition 8, Page 4-20, Caterpillar Tractor Company.

TABLE 4-4(c)
YEARLY TRUCK PRODUCTIVITY SUMMARY

Year	Round Trip Haul Distance (feet)	Total Cycle Time (minutes)	Average Speed (feet/min.)	Productivity per Unit (Tons/ operating hour)
A. REFUSE FROM BREAKER				
1978	8,680	19.7	590	80
1979	9,480	20.7	604	76
1980	10,180	21.8	606	72
1981	11,080	23.3	607	68
1982	11,580	24.0	609	66
B. REFUSE FROM PREPARATION PLANT				
1978	5,670	15.7	530	100
1979	6,470	16.7	553	94
1980	7,170	17.8	560	88
1981	8,070	19.2	568	82
1982	8,570	20.0	571	79

TABLE 4-4(d)
ESTIMATED AVERAGE TRUCK REQUIREMENTS

Year	Calculated Number of Trucks Required on Average		Recommended Fleet
	Breaker	Plant	
1978	0.16	1.50	2
1979	0.20	1.60	3
1980	0.26	1.70	3
1981	0.32	1.83	3
1982	0.37	1.90	3

a. Road construction
b. Vegetation removal and topsoil/colluvium dozing for dump reclamation
c. Dump perimeter drainage ditch development and maintenance
d. Possible assistance to bogged-down trucks
e. Road drainage ditch and diversion ditch maintenance
f. Dump surface snow removal and ripping
g. Possible towing of suitable compaction

The question of the need for a Low Ground Pressure (LGP) dozer with side tracks was also considered. However, it is believed that the dump will normally support the truck tire footprint pressure of 75-80 psi, and therefore should also support approximately 8 1/2 psi exerted by a D7 dozer with standard tracks.

4.4.3.2 Motor Patrol

The estimated daily refuse haulage is:

260 tons	breaker	Refuse @ 0,82 mile	=	214 ton-mile
963 tons	plant	Refuse @ 0.54 mile	=	517 ton-mile
		Total/day	=	731 ton-mile

A Cat 12G motor grader or equivalent should be capable of about 1000 ton-mile/hour and therefore the average daily usage on the refuse haul road system should be less than one hour. It was recommended in the equipment Specifications that AEPSC purchase a good used machine with minimum 115 FWHP and a 12-foot moldboard. The grader would also be used in the general plant area maintaining parking lots, ditches and the raw coal haul road.

4.4.3.3 Other Support Equipment

Water loss from trucks is at this time unpredictable. However, since it might be sufficient to control dust, it was recommended that AEPSC should not acquire a water truck until the need for such a unit was definitely established.

Although not cited during this study, dumplighting may be required under some Federal or State regulation. Operationally, dump lighting should not be necessary, however, until afternoon shift operations increase in 1980.

It has been assumed that the Cat 988 front-end loader currently in use at the Hardscrabble Canyon Preparation Plant, would be available for Schoolhouse Canyon refuse disposal operations. This loader will be required for cleanup around the refuse bins and in loading out topsoil/colluvium material for reclamation purposes as discussed above.

Depending on the dump moisture conditions, and the compaction achieved with the dozer and haulage trucks, it may be desirable to reduce the lift thickness (the dozer will have the necessary additional spreading capacity to do this), or perhaps purchase a suitable sheepsfoot or vibratory compactor.

4.5 Manpower and Operating Cost Budget

Estimated hourly owning and operating costs for the 35-ton end-dump trucks, 200 HP dozer, and 135 HP Motor Patrol are given in Tables 4-5(a), 4-5(b) and 4-5(c) respectively. For estimating purposes the Cat 769B truck has been used to develop these costs. Although depreciation has been included to determine estimated hourly ownership costs, only actual estimated cash operating costs have been used to derive the five-year disposal operations budget.

Tables 4-5(d), 4-5(e) and 4-5(f) contain estimates of total yearly operating costs for the trucks, dozer and grader respectively.

Truck productivities were taken from the analysis given in Appendix A. For labor costs, 250 days/year and $100/man day were used. It has also been assumed that, on afternoon and night shifts, the equipment operators would be utilized around the plant for other productive work when not hauling refuse. Alternatively, this refuse in the first few years could be disposed of by overtime operators commencing before the regular day shift.

A manpower summary and overall operating cost budget is given in Table 4-5(g).

HOURLY OWNING AND OPERATING COST ESTIMATE

Machine Designation	35T END DUMP
DEPRECIATION VALUE	
1. Delivered Price (including attachments)	$164,503
2. Less Tire Replacement Costs:	
Front 18.00 x 25 -32 P.R. (2)	
Drive 18.00 x 25 -32 P.R. (4)	
Rear ______	8,602
3. Delivered Price Less Tires	155,901
4. Less Resale Value or Trade In	33,000
5. NET VALUE FOR DEPRECIATION	122,901
OWNING COSTS	
6. Depreciation: $\frac{\text{Net Depreciation Value}}{\text{Depreciation Hours}}$	
VALUE 122901 / HOURS 20000	6.15
7. Interest, Insurance, Taxes	
Annual Rates: Int 10% Ins 1% Taxes 1%	
Annual Use in Hours 1.814	
$\frac{\text{Factor X Delivered Price (Item 1)}}{1000}$	
$\frac{.04 \times 164,503}{1000}$	6.58
TOTAL HOURLY OWNING COST	12.73
OPERATING COST	
8. Operating Labor Including Fringes	12.50
9. Repair Labor	3.00
10. Repair Parts	6.00
11. Fuel	3.61
12. Lubricants	.90
13. Expendable Parts	-0-
14. Tires 2,500 Hr.	3.00
15. Outside Repairs	-0-
16. Shop Costs	3.00
17. Special Items	-0-
HOURLY TRUCK OPERATING COST (EXCL. DRIVER)	19.95
TOTAL HOURLY OWNING AND OPERATING COST	$ 45.18

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HOURLY OWNING AND OPERATING COST ESTIMATE

Machine Designation	200 HP DUMP
DEPRECIATION VALUE	
1. Delivered Price (including attachments)	$132,000
2. Less Tire Replacement Costs:	
Front ____	
Drive ____	
Rear ____	
3. Delivered Price Less Tires	132,000
4. Less Resale Value or Trade In	31,000
5. NET VALUE FOR DEPRECIATION	101,000
OWNING COSTS	
6. Depreciation: $\frac{\text{Net Depreciation Value}}{\text{Depreciation Hours}}$	
$\frac{\text{VALUE } 101000}{\text{HOURS } 15000}$	6.73
7. Interest, Insurance, Taxes	
Annual Rates: Int 10% Ins 1% Taxes 1%	
Annual Use in Hours 950*	
$\frac{\text{Factor X Delivered Price (Item 1)}}{1000}$	
$\frac{0.75 \times 132{,}000}{1000}$	9.98
TOTAL HOURLY OWNING COST	16.63
*REFLECTS ONLY USE IN DOZING REFUSE	
OPERATING COST	
8. Operating Labor Including Fringes	12.50
9. Repair Labor	3.08
10. Repair Parts	6.16
11. Fuel	3.23
12. Lubricants	.81
13. Expendable Parts	.50
14. Tires 2,500 Hr.	-0-
15. Outside Repairs	-0-
16. Shop Costs	3.08
17. Special Items	-0-
HOURLY TRUCK OPERATING COST (EXCL. DRIVER)	16.86
TOTAL HOURLY OWNING AND OPERATING COST	$ 45.99

TABLE 4-5(c)
4-18

HOURLY OWNING AND OPERATING COST ESTIMATE

Machine Designation	135 HP MOTOR PATR 12' BLADE
DEPRECIATION VALUE	
1. Delivered Price (including attachments)	$82,250 NEW $ 60,000 USED
2. Less Tire Replacement Costs:	
Front 13.00 x 24 (10 PR)	
Drive 13.00 x 24 (10 PR)	
Rear 13.00 x 24 (10 PR)	1,500
3. Delivered Price Less Tires	58,500
4. Less resale Value or Trade In	10,000
5. NET VALUE FOR DEPRECIATION	$ 48,500
OWNING COSTS	
6. Depreciation: $\frac{\text{Net Depreciation Value}}{\text{Depreciation Hours}}$	
$\frac{\text{VALUE } 48500}{\text{HOURS } 17000}$	2.85
7. Interest, Insurance, Taxes	
Annual Rates: Int 10% Ins 1% Taxes 1%	
Annual Use in Hours 321*	
$\frac{\text{Factor X Delivered Price (Item 1)}}{1000}$	
$\frac{.15 \times 60{,}000}{1000}$	9.00
TOTAL HOURLY OWNING COST	11.85
*DOES NOT REFLECT USE FOR ANY OTHER PURPOSE THAN HAUL ROAD MAINTENANCE	
OPERATING COST	
8. Operating Labor Including Fringes	12.50
9. Repair Labor	1.25
10. Repair Parts	2.75
11. Fuel	2.06
12. Lubricants	.52
13. Expendable Parts	.50
14. Tires 2,500 Hr.	.75
15. Outside Repairs	-0-
16. Shop Costs	1.25
17. Special Items	-0-
HOURLY TRUCK OPERATING COST (EXCL. DRIVER)	9.08

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TOTAL HOURLY OWNING AND OPERATING COST $ 33.43

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TABLE 4-5(d)
YEARLY TRUCK OPERATING COST ESTIMATE

	Breaker Refuse Wet Tons	Truck Productivity WT/Hr.	Plant Refuse Wet tons	Truck Productivity WT/Hr.	Total Required Truck Hours	Operating Cost Excluding Driver	SCHEDULED DRIVERS Day Shift	Aft'n. Shift	Midnight Shift	Truck Driver cost ($)	Total Operating Cost ($)	Cost/ Ton Refuse
1978	65,000	80	213,000	100	2,942	58,693	3	1/8	1/8	56,250	114,943	0.40
1979	80,000	76	269,000	94	3,914	78,084	2	1/4	1/4	62,500	140,584	0.40
1980	98,000	72	325,000	88	5,054	100,827	2	1	1/4	81,250	182,077	0.43
1981	112,000	68	372,000	82	6,184	123,371	3	1	1/4	106,250	229,621	0.47
1982	128,000	66	424,000	79	7,306	145,755	3	2	1/3	133,333	279,088	0.51

Table [illegible]-5(e)

YEARLY DOZER OPERATING COST ESTIMATE

	Total Refuse Wet Tons	Dozer Productivity WT/Hr.	Required Dozer Hours	Operating Cost Excluding Operator	Sched. Oper. Day Shift	Labor Cost ($)	Total Operating ($)	Cost/ Ton Refuse ($)
1978	278,000	440	632	10,656	1/2	12,500	23,156	0.08
1979	349,000	440	793	13,370	1/2	12,500	25,870	0.07
1980	423,000	440	961	16,202	3/4	18,750	34,952	0.08
1981	484,000	440	1,100	18,546	1	25,000	43,546	0.09
1982	552,000	440	1,254	21,142	1	25,000	46,142	0.08

T E 4-5(f)

YEARLY MOTOR PA. OPERATING COST ESTIMATE

	Ton-Mi. Haulage	Patrol Productivity TM/Hr.	Required Patrol Hours	Operating Cost Excluding Operator	Sched. Oper. Day Shift	Labor Cost ($)	Total Operating ($)	Cost/ Ton Refuse ($)
1978	167,794	1,000	168	1,525	1/8	3,125	4,650	0.02
1979	236,632	1,000	237	2,152	1/8	3,125	5,277	0.02
1980	315,141	1,000	315	2,860	1/4	6,250	9,110	0.02
1981	401,799	1,000	402	3,650	1/4	6,250	9,900	0.02
1982	484,462	1,000	484	4,395	1/4	6,250	10,645	0.02

TABLE 4-5(q)
MANPOWER SUMMARY AND OVERALL OPERATING COST BUDGET

	Total Refuse Production Wet Tons	Number of Operator			Oper. Labor Cost ($)	Eqpt. Oper. Cost ($)	Total Cost/Ton Refuse ($)
		Day Shift Men	Afternoon Shift Men	Night Shift Men			
1978	278,000	2-5/8	1/8	1/8	71,875	70,874	0.51
1979	349,000	2-5/8	1/4	1/4	78,125	93,606	0.49
1980	423,000	3	1	1/4	106,250	119,889	0.53
1981	484,000	4-1/4	1	1/4	137,500	145,567	0.58
1982	552,000	4-1/4	2	1/3	164,583	171,292	0.61
TOTAL	2,068,000				558,333	601,228	0.56

SCHOOLHOUSE CANYON REFUSE DUMP FACILITY
Typical Haul Ramp Cross Section

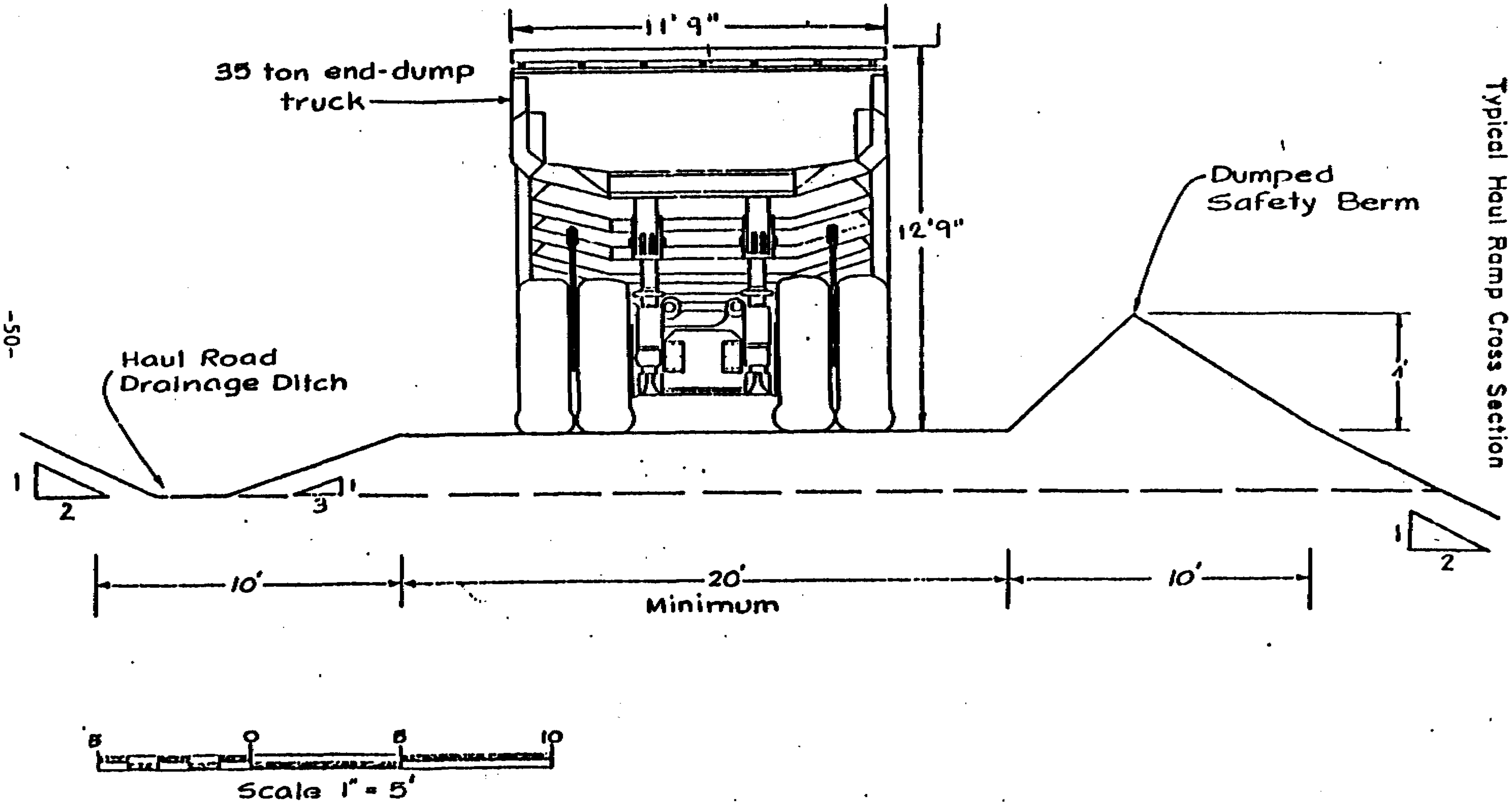

Figure 4-2 (b)

5.0 HYDROLOGIC CONSIDERATIONS

Hydrologic analysis in Phase II centered around two major structures: 1) a diversion channel to intercept and divert runoff above the refuse pile to Barn Canyon; and 2) a settling pond to receive runoff from the disturbed area below the diversion channel. The following discussion on hydrologic considerations deals with design criteria and briefly discusses earlier studies of alternative sediment control schemes.

5.1 Watershed Areas

The Schoolhouse Canyon watershed has a total area of 260 acres and has been divided into four segments for hydrologic analyses. These are shown in Figure 5-1(a) and are described below:

a. Undisturbed Area to Diversion Channel: The watershed area above the diversion structure from which runoff will be diverted to Barn Canyon (193 acres).

b. Undisturbed Area to Pond: The watershed area which is undisturbed by the refuse pile and associated construction works and from which runoff will flow to the settling pond (40 acres).

c. Disturbed Area to Pond: The watershed area which is disturbed by the refuse pile and associated construction works and from which runoff will flow to the settling pond (23 acres).

d. Haul Road Drainage: The watershed area from which runoff will be intercepted by a drainage ditch along the haul road

(4 acres). This will not contribute to the settling pond within Schoolhouse Canyon, but rather will be treated in the settling ponds adjacent to the plant.

The disturbed area includes the refuse pile plus associated areas disturbed by the diversion channel, the haul road on the west side of the canyon, and a drill site access road on the east side of the canyon near the settling pond. The actual area of the refuse pile will increase each year as shown in Figure 5-1(b). However, for hydrologic design purposes the maximum area to be disturbed by the refuse pile and associated activities has been used.

A small additional area (3 acres) is shown in Figure 5-1(a), which falls outside of the Schoolhouse Canyon watershed but which will contribute to runoff intercepted by the haul road ditch.

5.2 Precipitation Data

The available precipitation data from a gaging station at Price, Utah (Utah State University, 1971) is given in Table 5-2(a). No gaging stations are located nearer to the Castle Gate area. However, regional precipitation data for 6-hour and 24-hour events is available and is tabulated in Table 5-2(b) (NOAA Rainfall Frequency Atlas for Utah, 1974). Generally the precipitation at the Price Station is about 94% of that in the Castle Gate area; thus, the Price data was considered applicable to the Schoohouse Canyon watershed when increased by about 6%.

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5.3 Diversion Channel

5.3.1 General

During Phase I of this study, a diversion channel to Ketchum Canyon on the Schoolhouse Canyon south wall had been considered. Early in Phase II, however, it became apparent that a better alternative would be the construction of a diversion channel on the north wall ot Barn Canyon. The latter was expected to encounter more intact rock close to the natural surface, offer less long-term instability, and was considerably shorter than the other scheme. A dozer trail was blazed along the north wall and the design proceeded on the basis of a north wall diversion channel.

It was not considered necessary to extend the north wall diversion channel to intercept runoff from the south wall at this time because the settling pond capacity as designed meets with current federal regulations. However, this might be considered in the future to avoid treatment of some of the runoff from the undisturbed land on the south wall (40 acres). Once the pile reaches planned capacity, another alternative might be to intercept some of this south wall flow in a lined channel which could be inexpensively constructed on the dump surface.

5.3.2 Design Flow Rate

* The peak flow rate to be carried by the diversion channel has

*Ref: TRICO International, Inc., 1976 Report for Master Drainage Study for American Electric Power Service Corporation Coal Mining Facilities Near Castle Gate, Utah.

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been determined using the Rational Method, which is commonly used in small watersheds (less than five square miles) where no stream-gaging data exists. Its use is described by many authors including gray (1970).* In equational form:

$$Q_p = ciA$$

Where Q_p = peak flow rate (cfs)
c = runoff coefficient based on watershed characteristics
i = rainfall intensity (in/hr) of a storm whose duration is equal to the time of concentration of the watershed
A = area of the watershed (acres)

The time of concentration is given empirically by:

$$t_c = 0.0078L^{0.77}\ S^{-0.385}$$

where t_c = time of concentration (min)
L = maximum length of travel of water (ft)
S = slope, equal to H/L where H is the elevation difference between the most remote point in the watershed and the outlet (ft)

For that portion of the Schoolhouse Canyon watershed above the diversion point, the following parameters have been determined:

H = 1610 ft
L = 5000 ft
S = 0.32
t_c = 8.5 min
c = 0.65 /0i = 3.12 in/hr (Intensity of 8.5 min, 100 year storm)
A = 193 acres
Q_p = 391 cfs

TRICO International, Inc. (1976)*** determined peak flow rates in

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*Ref: Donald M. Grey, 1970, "Handbook on the Principles of Hydrology."

**Ref: TRICO International, Inc., 1976 Report for Master Drainage Study for American Electric Power Service Corporation Coal Mining Facilities Near Castle Gate, Utah.

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Schoolhouse Canyon for 6-hour and 12-hour storms using the Soil Conservation Service. "TR-20 Project Formulation" computer program, and found that maximum flow resulted from the 100-year/24-hour storm. Peak flow rates were calculated at a point in the upper reach of Schoolhouse Canyon and at the outlet. The areas and corresponding maximum peak flows at the two points from TRICO's calculations were as follows:

Upper Reach: Area = 96 acres, Q_p = 200 cfs
Outlet: Area = 250 acres, Q_p = 500 cfs

Assuming that Q_p is proportional to the area of the watershed contributing to runoff, a peak flow rate of approximately 400 cfs is indicated for the area (193 acres) above the diversion point. This figure supports the 391 cfs calculated above by the Rational Method and thus, 400 cfs was considered a reasonable peak flow rate in the diversion channel for a 100-year storm.

5.3.3 Channel Dimensions

The dimensions of the diversion channel were determined by the peak flow rate, the permissible side slopes on the channel banks, the channel gradient, and the size limitations presented by the excavation equipment. The design flow rate of the diversion channel was set at 400 cfs; channel banks can be excavated at a 1/2:1 slope on the uphill and a [illegible] downhill side of the channel, and the channel botto[illegible] is 15 feet which is roughly [illegible] feet wider than a [illegible]

The Manning equation describing flow in an open channel is:

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

where Q_p = flow rate (cfs)
n = Manning coefficient
A = cross-sectional area of channel (ft^2)
R_h = hydraulic radius (ft) equal to area divided by wetted perimeter
S = channel gradient

A Manning coefficient of N = 0.050 was considered reasonable for a rough channel excavated in rock. The depth of flow was found from the equation above using the peak flow rate, Manning coefficient, bottom width of the channel, bank slopes, and the channel gradients.

Approximate utilization of the dozer trail blazed for the diversion channel necessitated a channel gradient varing from 4 percent near the diversion structure to 1 percent near the outlet in Barn Canyon. Typical design cross sections for the channel are given in Figure 5-3 for three channel slopes.

Flow may be either subcritical or supercritical depending upon the flow velocity and the channel dimensions. Subcritical flow is generally most desirable in open channels. An indication of the type of flow is given by the Froude number:

$$F = \frac{V}{\frac{gA}{b}}$$

where F = Froude number
g = acceleration due to gravity (32 ft/sec^2)
A = channel area (ft^2)
b = channel width at water surface (ft)
V = velocity (ft/sec) for $F > 1$ flow is supercritical and for $F < 1$ flow is subcritical. Froude numbers are also given in Figure 5-3 for each channel cross section.

Due to the relatively steep gradient of 4% along the upper reach of the channel, peak flow will probably be supercritical. This could result in standing waves particularly at the curves and other irregularities in the channel. A two-foot freeboard has been added over the entire length of the diversion channel to reduce the risk of any overtopping due to such waves. As the channel slope decreases, the water velocity will also decrease, resulting in a lower erosion capacity. The flatter gradient of 1% through the ridge cut, as the channel enters Barn Canyon, should reduce the tendency for erosion on the outer bank of the channel through the curve. Smaller gradients also reduce the potential for supercritical flows and the resulting standing wave problems.

5.3.4 Outlet

The proposed diversion channel will discharge its flow into Barn Canyon at the point where the channel daylights with the natural slope. The water will flow down a small gully and enter the main channel in Barn Canyon. At present, no improvements are considered necessary in this gully or at is confluence with the main Barn Canyon floor. If excessive erosion should occur, some channel protection may be required. Such problems will become more apparent after the diversion ditch is operating, at which time they can be handled appropriately.

Improvements to the existing Barn Canyon channel near the preparation plant are considered necessary to adequately contain the combined storm runoff from Barn and Schoolhouse (above the diversion channel) Canyons. The suggested improvements include:

TABLE 5-2(a)

ESTIMATED RETURN PERIODS FOR SHORT DURATION PRECIPITATION (INCHES) - PRICE, UTAH**

Return Period (yrs)	Duration 5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	.08	.13	.17	.23	.29	.37	.44	.62	.78	.95
2	.12	.18	.23	.32	.40	.49	.58	.80	1.00	1.20
5	.16	.25	.32	.44	.56	.68	.79	1.07	1.32	1.58
10	.20	.31	.39	.54	.68	.81	.94	1.25	1.53	1.82
25	.24	.37	.47	.65	.82	.98	1.13	1.50	1.83	2.18
50	.28	.43	.54	.75	.95	1.12	1.29	1.71	2.08	2.47
100	.31	.49	.62	.85	1.08	1.27	1.45	1.91	2.32	2.74

TABLE 5-2(b)

PRECIPITATION FOR CASTLE GATE AREA**

** Storm	Precip (in)	Storm	Precip (in)
2 yr-6 hr	.92	2 yr-24 hr	1.30
5 yr-6 hr	1.20	5 yr-24 hr	1.65
10 yr-6 hr	1.32	10 yr-24 hr	1.90
25 yr-6 hr	1.65	25 yr-24 hr	2.30
50 yr-6 hr	1.85	50 yr-24 hr	2.70
100 yr-6 hr	2.05	100 yr-24 hr	2.90

*Ref: Utah State University, 1971, Department of Soils and Biometeorology Bulletin No. 1.

**Ref: National Oceanic and Atmospheric Administration, 1974, NOAA Atlas 2, Vol. VI, Rainfall Frequency Maps of Utah.

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a. Construction of rip rapped berm along the top of the existing

b. Installation of an additional pipe arch under the road at the channel outlet to the Price River.

c. Erosion protection on the bank of the Price River opposite the pipe arch outlets.

These improvements were discussed in a letter to AEPSC dated December 8, 1977.

5.4 Settling Pond

5.4.1 Design Volume

The design volume of the settling pond was based on the runoff resulting from a 10-year/24-hour storm as required by current federal regulations. This runoff will occur on both disturbed and undisturbed areas below the diversion (see figure 5-1(a)). An additional volume allowance of 0.2 acre-feet per acre of disturbed land has been made for sediment which will be required by the new federal OSM regulations. No sediment allowance has been made for the undisturbed land contributing runoff to the pond based upon our understanding of the definition of "disturbed land" in the OSM regulations.

The 24-hour storm runoff has been estimated using the Soil Conservation Service Curve Number Method described by Mockus (1972)** Precipitation from the 10-year/24-hour storm (1.90 inches) and a curve

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*Mockus, Victor, 1972, Hydrology, Section 4 of the National Engineering Handbook, Soil Conservation Service U.S. Departmetn of Agriculture.

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number of 93, based upon the soil and vegetation conditions of the watershed, have been used in Figure 5-4(a) to find runoff. The runoff from the design storm (1.23 inches) was multiplied by the area of the watershed (63 acres) to yield the total volume of runoff to the settling pond, equal to 6.5 acre-feet. An additional 4.6 acre-feet has been allowed for sediment storage (.2 acre-feet/acre x 23 acres) and thus a total required pond value at the final dump configuration of 11.1 acre-feet was determined.

According to the 1" to 50' topography, a settling pond capacity of 10.7 acre-feet is indicated up to the 6,205 spillway elevation (see Table 5-4 below). It is anticipated that clearing and grubbing operations will result in a slight expansion of the pond capacity to achieve the 11.1 acre-feet requirement. However, it should be noted that this requirement is for the final dump configuration when the disturbed area will reach a maximum. At the end of the first year, for instance, the combined runoff/sediment requirement has been estimated at 8.6 acre-feet, which is adequately met by the proposed pond. Moreover, the embankment crest as designed is five feet above the spillway elevation, and therefore additional short-term storage capacity is provided. Settling pond values required for different 24-hour storms are illustrated in Figure 5-4(b). It is apparent that the proposed pond should easily contain runoff from a very large storm or from several smaller storms occurring in close sequence when the pond is free of sediment load.

TABLE 5-4

SETTLING POND CAPACITY

Elevation	Area (Acres)	Incremental Volume (Acre-Feet)	Cumulative Volume (Acre-Feet)
6175	0		
6180	0.027	0.068	0.068
6185	0.167	0.485	0.553
6190	0.311	1.195	1.748
6195	0.480	1.978	3.726
6200	0.693	2.933	6.658
6205	0.916	4.023	10.681
6210	1.194	5.275	15.956

An additional requirement of the latest OSM regulations (December 13, 1977) is that "the sedimentation pond must provide at least a 24-hour detention time and a surface area of at least one square foot for each 50 gallons per day of inflow for runoff entering the pond that results from a 10-year/24-hour precipitation event." Although this rule was not considered in the design, the following calculations indicates that the requirement is satisfied.

- Pond Inflow for 10-year/24-hour storm

  $= 1.23 \text{ inches} \times 63 \text{ acres} \times \frac{1 \text{ ft}}{12 \text{ ins}} \times 43{,}560 \frac{\text{fs } 2}{\text{acre}} \times 7.48 \frac{\text{gallons}}{\text{ft } 3}$

  = 2,100,000 gallons

- Pond Surface area at discharge

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$= 0.916 \text{ acres} \times 43,560 \frac{ft2}{acre}$

$= 39.800 \text{ ft } 2$

- Surface Area/50 gallons/day of inflow

$= 0.95 \text{ ft } 2 = 1$

Appropriate dewatering schedules should permit the 24-hour detention time requirement to be easily achieved. In the end however, it is the effluent standard which must be met for all storms less than the 10-year/24-hour event, despite the pond design guidelines and rules discussed above. These effluent limitations which are given in Section 3.0, should be achievable with the proposed pond, and through controlled dewatering practices.

5.4.2 Spillway

The settling pond spillway has been designed as an emergency structure to prevent overtopping of the pond embankment. Both embankment abutments were considered as alternative spillway locations. However, the south abutment was chosen because it provided a greater spillway length, hence a flatter gradient, and because of its better overall rock quality. Both of these factors were considered important from the standpoints of flow hydraulics and vehicular access to the pond area via the channel floor.

Utah regulations (Utah Division of Water Rights) stipulate a spillway capacity of 50 cfs per square mile of drainage area or about 20 cfs for the entire Schoolhouse Canyon watershed. Failure of the diversion channel, however, could result in a maximum of 500 cfs (400

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cfs from the area above diversion and 100 cfs from the area below) of flow to the settling pond. Actual flows down the spillway will be smaller than these unless the pond is full (due to the dampening effect of the pond on the peak flow).

The design capacity of the spillway, assuming a 5-foot water depth and a 15-foot channel width, is 520 cfs which is sufficient to pass peak flows resulting from the unlikely situation of a full settling pond and a failure of the diversion channel occurring simultaneously.

Water flowing down the spillway wil enter the existing 60-inch culvert in Schoolhouse Canyon. A trash rack will be placed over the inlet to prevent debris from plugging the inlet or entering the culvert. This matter along with other aspects of the Settling Pond Construction were discussed in the Technical Specifications completed in December, 1977.

5.4.3. Pond Maintenance and Dewatering

The new federal OSM regulations will require that sediment is removed from the sedimentation pond when its volume accumulates to 80% of the design allowance for sediment. On the basis of current information it is not possible to reliably estimate the rate at which sediment will accumulate within the pond. Sediment removal, however, should not be necessary for several years, and could then be accomplished simply through the use of small front-end loader and trucks hauling out via the spillway floor during the summer months.

It has been estimated from permeability testing that seepage into the pond walls, beneath and through the pond embankment may range from 5,000 to 50,000 gallons/day. Irrespective of the initial seepage, the pond can be expected to experience early siltation and in consequence reduced seepage losses. At best, natural dewatering through infiltration and evaporation is not expected to amount to more than a few feet of water level drop/month, and therefore some other means of dewatering clarified water is considered necessary. Decant systems were initially contemplated but because of the relatively small pond size, coupled with problems in control of discharge during operation, they were rejected. Pumping was considered a more flexible approach.

A suitable 100-500 gpm centrifugal pump capable of handling dirty water should be purchased or rented. This could be installed on a simple float with the suction of the pump about a foot below the water surface to allow for skimming of clear surface water, while preventing cavitation. Alternatively the pump could be mounted on a small trailer which could be lowered to the water surface down a dozer cut rail extended from the spillway. Pump discharge would consist of rubber hose, thence to plastic, steel, or aluminum pipe to the lip of the spillway. A 3 to 5 inch line would be suitable depending on the pump size used.

In the long term, once the need for pumping has been established with actual storm runoff and sedimentation rates, possible flocculating additives, etc., a more permanent installation, perhaps involving an electrically powered pump to avoid gasoline supply and reduce servicing requirements, might be contemplated.

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5.4.4 Alternative Sediment Control Schemes

Early in Phase II it became evident from the proposed OSM regulations, that a relatively large allowance for sediment would have to be made in the settling pond total capacity. Therefore the feasibility of a single in-canyon settling pond, as contemplated during Phase I, was in question and alternative schemes for handling sediment control of runoff emanating from the refuse pile area were examined. Three conceptual alternatives were studied economically, and were discussed with AEP personnel on site. Briefly, these were:

Scheme A

Construction of a small settling pond within Schoolhouse Canyon with overflow channelled to supplemental storage capacity in an expanded plant thickener pond.

Scheme B

Construction of a single large settling pond within Schoolhouse Canyon.

Scheme C

Direct entry of the disturbed area runoff into the existing 60-inch culvert system. The culvert discharge would then be intercepted at a point between the D&RG railroad tracks and the Price River, thence channelled to the old existing settling ponds to the south and treated there prior to final river discharge.

Although Scheme C was conceptually very attractive, the problems associated with its proximity to the Price River, obtaining clearances

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from the railroad and possible construction difficulties were considered by AEPSC to offer strong potential for delaying the project. This approach was therefore rejected from further consideration.

Scheme A offered economic advantage over Scheme B primarily because of its smaller embankment volume and because in Scheme B the refuse pile had to be pushed further up-canyon, thus involving additional haul road construction and marginally higher refuse disposal costs. Despite the capital and operating costs savings attributable to Scheme A, AEPSC preferred the Scheme B approach because it would be self-contained, and would not involve the use of valuable real estate or facilities in the preparation plant area. Thus, on October 31, Phase II proceeded on the basis of a single settling pond within Schoolhouse Canyon and in consequence a refuse pile starting elevation of 6,220 in the canyon floor.

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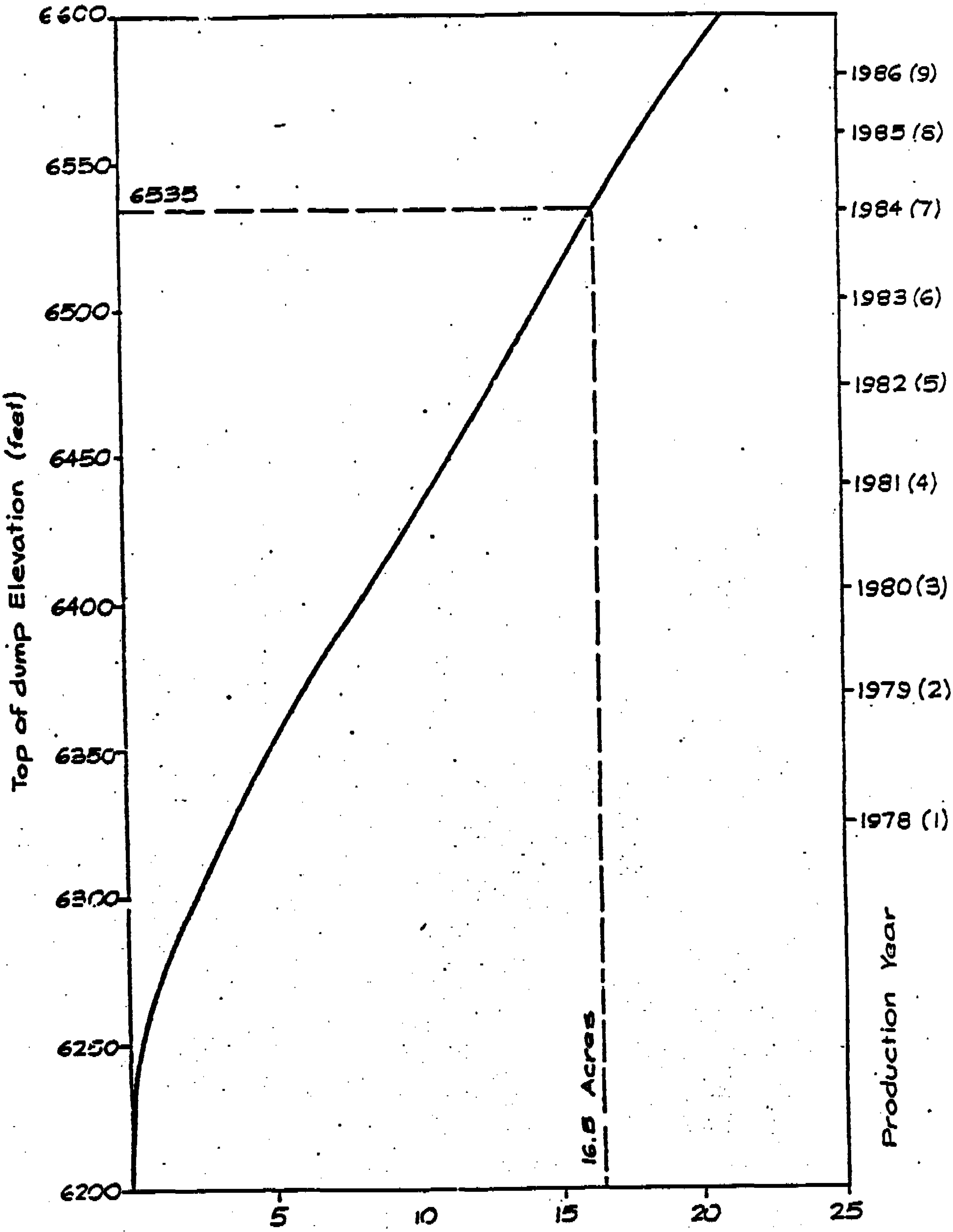
INCREASE IN REFUSE PILE
AREA WITH TIME

Figure 5-1 (b)



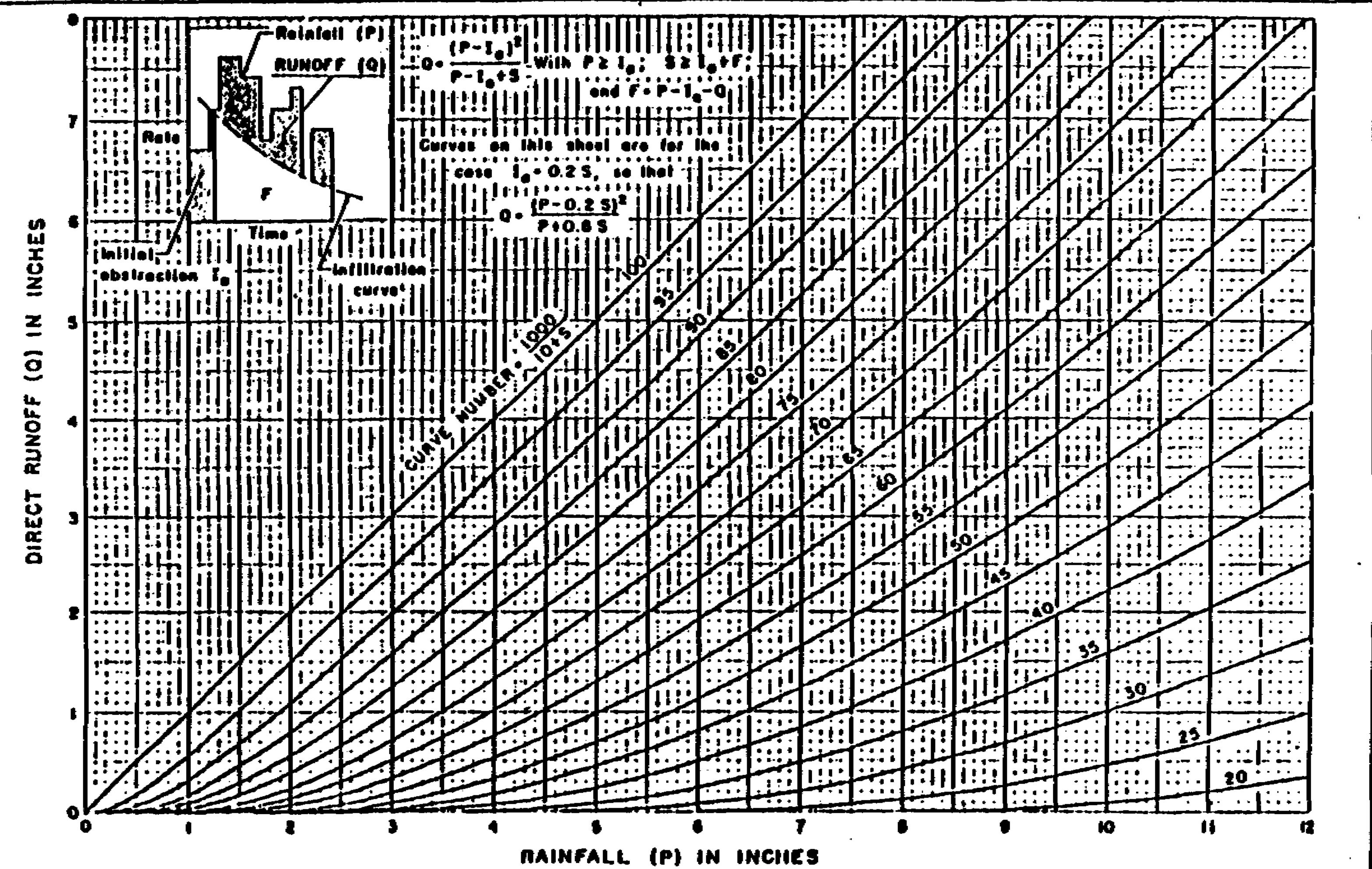
Project No. 577212 Reviewed Date 12-77

Project No. 772/2 Reviewed JK Date 18-6-77

SCS CURVE NUMBER METHOD FOR ESTIMATING DIRECT RUNOFF FROM RAINFALL

HYDROLOGY: SOLUTION OF RUNOFF EQUATION $Q = \frac{(P-0.2S)^2}{P+0.8S}$

P = 0 to 12 inches
Q = 0 to 8 inches

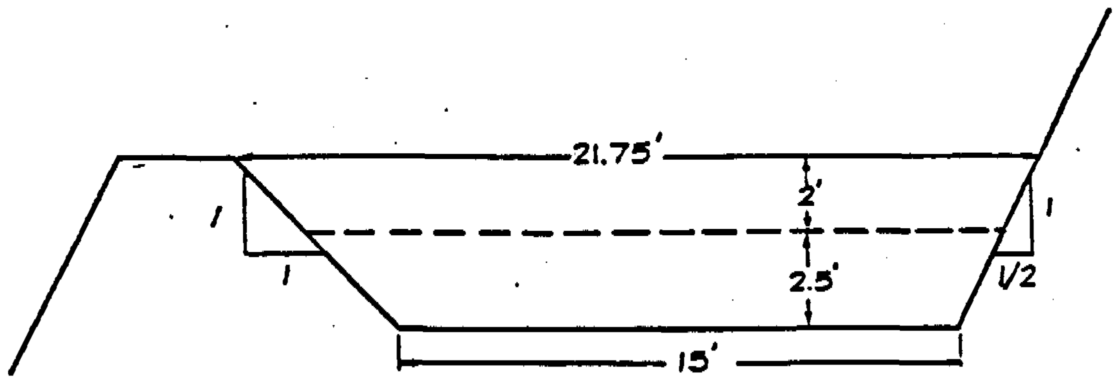


Reference: Soil Conservation Service, 1972

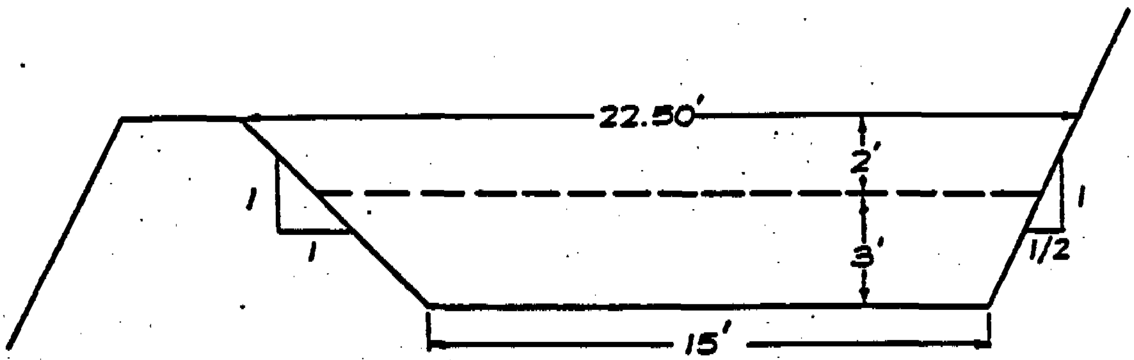
Figure 5-4(a)

TYPICAL CHANNEL CROSS SECTIONS

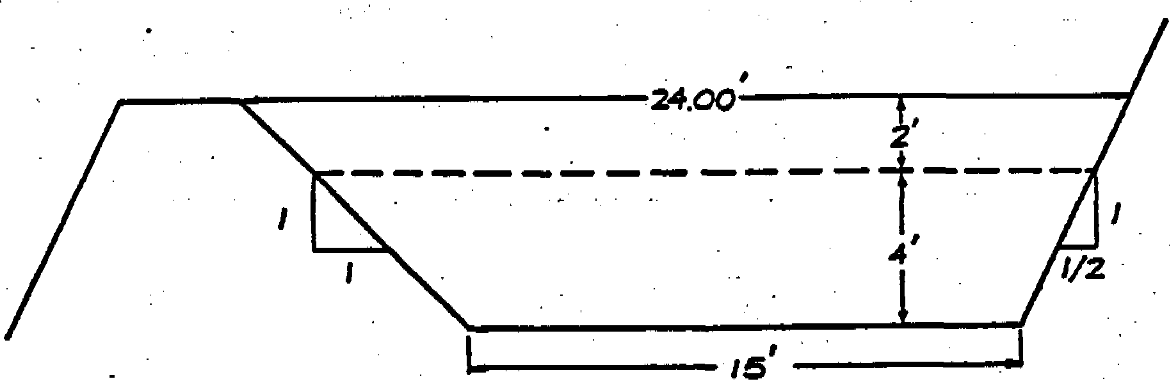
Figure 5-3



4% CHANNEL GRADIENT FROUDE NUMBER = 1.10



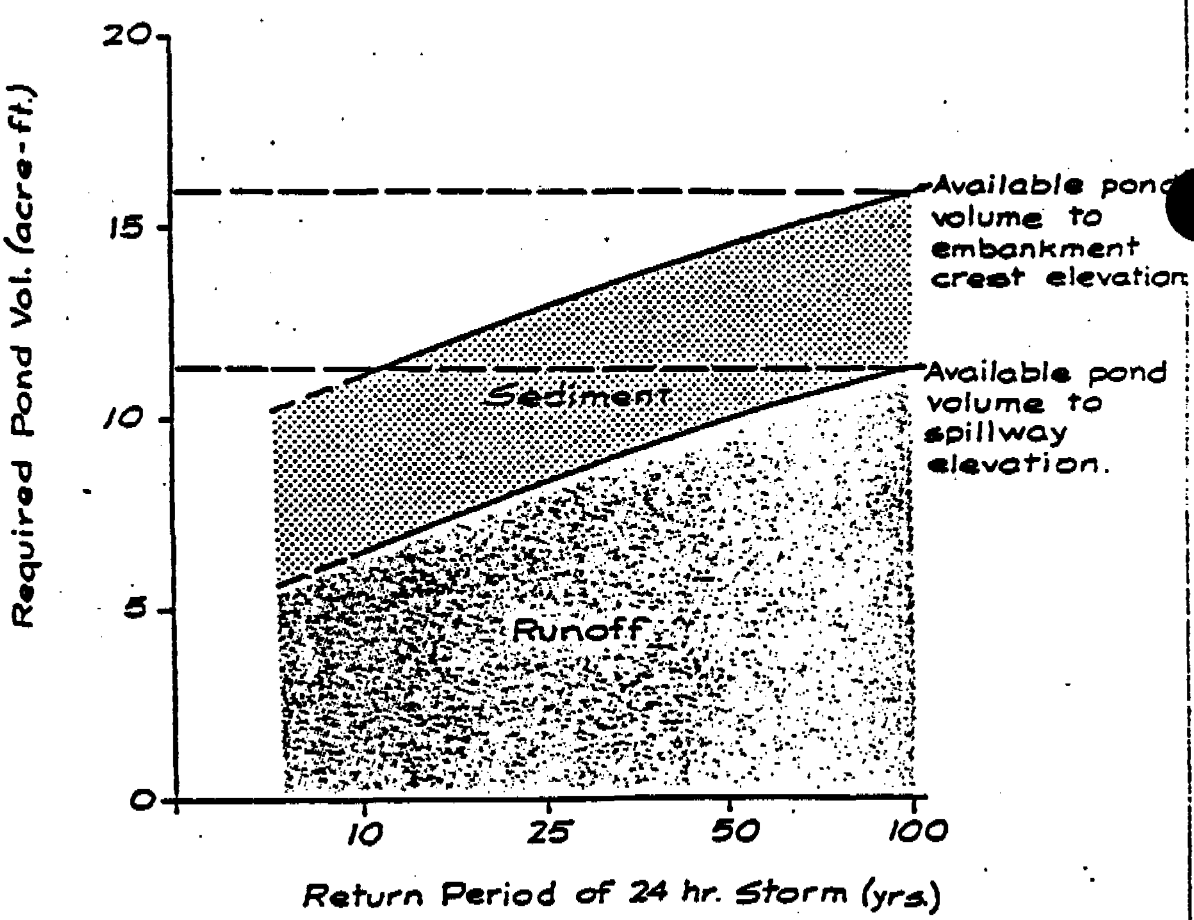
2% CHANNEL GRADIENT FROUDE NUMBER = .79



1% CHANNEL GRADIENT FROUDE NUMBER = .57

REQUIRED POND VOLUMES (RUNOFF AND SEDIMENT) FOR 24 HOUR STORM OF VARIOUS RETURN PERIODS

Figure 5-4(b)



6.0 GEOTECHNICAL CONSIDERATIONS

6.1 General

Following the decision to proceed with Phase II and prior to preparing the final design for a refuse disposal system in Schoolhouse Canyon, it was necessary to carry out investigations of subsurface conditions for the various components of the system. In addition, it was necessary to evaluate materials found in the area for construction of the settling pond embankment. The investigations comprised bulldozer cuts, test pits and borings. Based on the results of this work, geotechnical design criteria were established for the diversion of the upper Schoolhouse Canyon runoff to Barn Canyon, for the haul road from the plant to the refuse dump, and for the settling pond embankment/spillway system. An assessment of the refuse dump stability was also made.

Geologically, the lower portion of Schoolhouse Canyon is cut into the Blackhawk Unit of the Mesaverde Group of the Upper Cretaceous. The beds of the Blackhawk Unit consist of interbedded sandstones, siltstones, shales and coal, with strata thicknesses generally less than 10 feet. The head of Schoolhouse Canyon is founded in the Castle Gate Unit, which is a cliff-forming sandstone. Overlying the slopes and floor of the canyon is a mixture of colluvial and alluvial soils derived by weathering of the Blackhawk and Castle Gate Units. These soils consist generally of cobbles and boulders interspersed in a matrix of sand and gravel with some silt and a trace of clay. The colluvial slopes are generally in a marginally stable condition and slough under the influence of gravity, wind and water. Recently there has been some disposal of construction debris and miscellaneous fill in Schoolhouse Canyon in the approximate areas shown on Figure 6-1(a).

6.2 INVESTIGATIONS

6.2.1 Diversion to Barn Canyon

An exploratory dozer trail was cut near the alignment of the proposed diversion ditch to Barn Canyon; the proposed ditch alignment is shown on Figure 6-1(a). The trail was cut using a Komatsu D-155A with a two-shank ripper. Along much of the trail it was possible to cut into the hillside using the blade with only minor amount of ripping. However, cutting through the divide between Schoolhouse and Barn Canyons, ripping was required in sandstone, and it was found that ripping became ineffective below a depth of 10 to 15 feet.

The cut made by the dozer was about 15 feet wide with cut slopes ranging from 1:1 to nearly vertical with some overhangs. The upstream end of the trail was cut predominantly through colluvium for a distance of about 200 feet. For the next 600 feet, the cut slope was predominantly rock with only a few feet of colluvium cover. The rock consisted of beds of siltstone, shale and sandstone with a maximum bed thickness in sandstone of about 10 feet. The trail then passed near the base of a sandstone outcrop where the siltstone and shale beds had thinned. This outcroup had some overhanging ledges and blocks over its length of about 100 feet. From there on into the Barn Canyon drainage, the cut was predominantly in siltstone and shale with thin sandstone beds.

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6.2.2 Refuse dump Foundations

Four test pits were excavated to determine the subsurface conditions beneath the toe of the proposed refuse dump. These test pits 4, 5, 6 and 10 were excavated using a Massey-Ferguson MF40 or John Deere JD600 backhoe. Depending upon the setup position, the maximum reach on these backhoes was between about 10 to 14 feet. The locations of the test pits are shown on Figure 6-1(a).

The four pits were excavated in areas that contained some fill. These fills contained coal, construction debris, soil and a sand/straw mixture in a loose to compact relative density. Below this fill a colluvial/alluvial soil was encountered in test pits 6 and 10. This colluvial/alluvial soil comprised a brown coarse to fine gravel and coarse to fine sand, trace silt, occasional to numerous cobbles and boulders with a compact to dense relative density. More of the colluvial/alluvial soil was encountered in the test pits excavated in the settling pond area. None of the test pits excavated encountered bedrock or groundwater. Logs of the test pits are given in Appendix C.

6.2.3 Haul Road

An exploratory dozer trail, similar to the one cut for the diversion ditch, was cut near the alignment of the proposed haul road. Only minimal amounts of ripping were required and most of the cut was easily made using only the blade of the Komatsu D-0155A.

The cut was excavated along the slope between Barn and Schoolhouse Canyons and then in Schoolhouse Canyon. Along the slope between the two canyons the trail was cut in predominantly colluvium with some siltstone, shale and thin sandstone beds. As the trail made the turn into Schoolhouse Canyon, a greater portion of sandstone was encountered with a reduction in thickness of colluvial cover. From this point the trail approximately followed the dip of the beds to the floor of the canyon. The actual haul road alignment will, however, enter the canyon floor considerably upstream. This alignment will cross several outcropping sandstone beds which are estimated to be about 10 feet thick.

6.2.4 Settling Pond Foundations

Six test pits were excavated and five boreholes were drilled in the settling pond area to provide conditions. The test pits were excavated as discussed in section 6.2.2. They encountered colluvial/alluvial soil except for approximately two feet of fill at the surface in Test Pit 7. This fill consisted of the colluvial/alluvial soil mixed with a small amount of coal refuse.

The five boreholes were rotary drilled and cased through the colluvium in the area of the proposed settling pond embankment. One borehole was drilled in each abutment and three were located beneath the embankment in the valley floor, as shown on Figure 6-1(b). The abutment boreholes (B1-C and B-3) stated in the fresh rock exposed in drilling pads cut by a dozer. They were cored and logged continuously to a depth of 125 feet. Pressure packer tests were run throughout their length. Boreholes B-2, B-4 and B-5, located beneath the embankment, penetrated the colluvium/alluvium and were cored at least 15 feet into sound rock. Falling-head permeability tests were conducted in the overburden and pressure permeability tests were run in the rock. The Test Pits and Records of Boreholes are given in Appendices C and D respectively. Locations of the boreholes and test pits are shown on Figure 6-1(a).

The thickness of fill or colluvium/alluvium encountered in the test pits and boreholes (except the abutment boreholes) ranged up to about 40 feet.

The bedrock cored in the boreholes demonstrated little evidence of deep weathering; however, at the rock surface there are some open fractures. The rock encountered consisted of interbedded sandstone, siltsones, shales, organic shales and coal. Of these, the sandstones and siltstones are more competent and generally thicker bedded. Once sandstone/siltstone stratum, encountered in Borehole B-3, was approximately 37 feet thick.

Falling-head permeability tests in the overburden were run by filling the casing to the top and measuring the rate of fall of water in the casing. The pressure packer tests were run by sealing off a section of the borehole and injecting water at approximately 10, 20 and 30 psi. Sections of approximately 5 or 10 feet were sealed off in these tests by use of a single packer above the bottom of the advancing borehole or by a double pneumatic packer system. Results of the tests are shown on the Records of Boreholes in Appendix D. The permeabilities of the different materials measured generally range from 10

$132_{-6}$to 10_{-4}cm/sec in the rock abutments and from $10_{-3}$to 10 $_{-2}$cm/sec in the colluvium in the valley floor.

6.2.5 Settling Pond Embankment Materials

Three possibilities were considered as potential sources for embankment borrow. The most obvious source is from excavations for the haul road and diversion ditch.

A second possible source is the General Coal Borrow Pit on the west side of U.S. Highway 6/50 across from Barn Canyon, which has been used as a source of general fill for the preparation plant. The material presently being used as a plant road base fill came from Corn Borrow Pit. This is a third potential borrow source.

These three materials were sampled from stock piles or test pits. Rock larger than about three inches in diameter were removed by hand in the sampling process. The particle gradations and compaction characteristics were determined in the laboratory and the results are presented in Appendix B. The results indicate that materials from the three potential sources are very similar in their engineering characteristics as discussed in Section 6.4.

6.3 REFUSE DUMP

6.3.1 General

This section presents geotechnical considerations relating to the design and placement of the refuse dump. Of these perhaps the most important are the engineering properties of the refuse material itself as they affect its placement and the subsequent stability of the dump.

Because the preparation plant was not yet in operation, the properties of the refuse could not be determined directly. Rather, as discussed below, it has been necessary to infer the character of the refuse from projections by the preparation plant designers, and from a review of the published properties of waste from their mines. It is considered that the probable ranges in engineering properties of the refuse have been determined within reasonable limits and the dump was designed accordingly. It will be necessary during the first year of operation to determine the characteristics of the refuse material as it is finally produced and to revise both the design and placement procedures discussd below. In the meantime, the intent has been during Phase II to develop sufficiently flexible guidelines for placement to accommodate a wide range of material properties and placement conditions.

6.3.2 Engineering Properties of the Refuse

The engineering properties of refuse were studied and the results presented in detail in the Phase I Report. From information supplied by Dravo Corporation, the total refuse output is expected to be composed of the following:

Material	Size	Proportion of Total (% by weight)	Moisture Content (% by weight)
Breaker Refuse	12" to 4"	25	5
Coarse Refuse	4" to 2"	35	5
Fine Refuse	1/2" to No. 28 Mesh	20	8
Filter Cake	Finer than No. 28 Mesh	20	33
	TOTALS	100	11

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Using refuse material obtained from the present preparation plant in Hardscrabble Canyon, a sample having the above composition was fabricated. At overall moisture contents of 11 percent, there was considerable free water which appeared to drain readily from the sample. Grading curves for the fabricated sample and its fines content are given on Figure 6-3(a). Also shown in this Figure are compaction characteristics of the sample, both with and without the fines (filter cake). These results support the observation of the free-draining nature of the fabricated sample, because a reasonable degree of compaction is achieved over a wide range of moisture contents. The fines were determined to be non-plastic.

To the extent that the fabricated sample is representative of the material which will be delivered by the preparation plant, it is not anticipated that there will be serious problems of placement. Given its free-draining characteristics, it is likely that there will be appreciable loss of moisture along the conveyor system, in the storage bins and in the trucks before the material finally reaches the dump. These possibilities, together with the indicated compaction characteristics, support the conclusion of relatively trouble-free placement.

The observed free-draining characteristics of the fabricated sample are related to the low percentage of fines and the fact that the fines (simulated filter cake) are also non-plastic. Because of the lack of clayey materials associated with the coals at Castle Gate, it is not anticipated that the preparation plant will actually produce a filter-cake of significant plasticity. In the event that this anticipation is not proven to be correct, difficulties such as those discussed below in Section 6.3.2 could be experienced.

6.3.3 Experience at Other Mines

Recognizing the potential problems associated with combined refuse drainage and placement, information was sought from MESA personnel who were familiar with a variety of combined refuse operations across the U.S. A visit was also made to such a plant operating at Centralia, Washington.

Difficulties which are being experienced by these operations are summarized below:

a. Plant filtering efficiency and refuse draining characteristics depend in part on the plasticity of the refuse and in part on the plant flow sheet, equipment and operating practices.

b. Plant moisture control is always a problem.

c. Combined refuse operations suffer additional disposal difficulties during periods of heavy rain, snow and frost.

d. Homogeneous mixing of the filter cake with the other refuse streams seldom occurs. Due to plant process sequencing, several truck-loads of unmixed filter cake per shift are often placed, which typically produces soft, wet zones within the dump.

e. Hauling mobility over these soft refuse zones is commonly impaired, causing equipment to bog down.

f. Refuse in storage bins over shutdown periods often drys and cakes and water has to be added to facilitate handling.

g. Combined refuse dump slopes typically appear stable at angles ranging from 11 or less, to 20, depending in part on the ability to control moisture.

h. Difficulties are also often experienced in retaining refuse within hauling units.

Many of the problems outlined above typically occur where refuse clay content (plasticity) is high. It is clear that proper moisture control in refuse placement will be critical to the success of the dump design proposed, and may become extremely difficult to achieve when, and if, mining occurs in high clay content zones.

6.3.4 Refuse Pile Design - General

The refuse dump configuration, design criteria and slope protection requirements are discussed in detail in Section 4.0. In general, these requirements were developed in response to operational, legal and surface runoff constraints rather than geotechnical constraints. Provided the refuse can be adequately drained and compacted, the dump design presented in Section 4.0 is considered suitable and should be stable.

6.3.5 Removal of Unsuitable Foundation Material

As shown on Figure 6-1(a) and discussed in Section 6.1, there are areas of existing poor quality fill within the proposed dump limits. These unsuitable materials should be excavated down to firm natural ground. The majority of this fill will be removed during the construction of the haul road system. Any other unsuitable material encountered in the valley floor as the waste pile is developed upwards should also be removed. In excavating the material, due care should also be exercised so as not to produce any low-lying areas which could trap water draining away from the recently placed refuse.

6.3.6 Dump Stability

Figure 6-3(b) gives a longitudinal section along the center of the proposed final dump configuration. The limit of filling after one year of operation is also indicated. The approximate geometry of the proposed dump is as follows:

	End of Year 1	End of Year 7
Maximum Thickness	85 ft.	200 ft.
Maximum Height (toe to crest)	125 ft.	330 ft.
Maximum Slope Angle	26.5 (2:1)	26.5 (2:1)
Average Slope Angle (toe to crest)	26.5 (2:1)	19.0 (2.9:1)

The following table shows the predicted factor of safety of the dump for assumed strength parameters given in Figure 6-3(c) and for different slope drainage characteristics.

	FACTOR OF SAFETY	
	After ear 1	Year 7 Configuration
Fully Drained Slope		
Maximum inferred strength	1.5	Greater than 1.5
Minimum inferred strength	1.25	Greater than 1.5
Partially Drained Slope		
Maximum inferred strength	Less than 1.0	1.0
Minimum inferred strength	Less than 1.0	Less than 1.0

The analysis producing the above results was based on the data and simplified procedures presented in the Phase I Report. A theoretically rigorous stability analysis was considered inappropriate at this time, since the errors induced by the uncertainty in refuse strength parameters probably exceed the relatively minor errors induced by the use of more simplified analyses. During the first year of operation, observations of refuse placement and compaction behavior in conjunction with laboratory tests on representative samples will enable the stability of the dump to be more accurately evaluated. Based on these evaluations, any necessary modifications to procedures and overall dump configuration could be initiated.

As shown in the above table, the dump in a fully drained condition has an adequate factor of safety over the range of anticipated refuse strength parameters. However, even a modest build-up of water within the embankment has a severe effect on stability. Thus, it is essential to maintain proper drainage of excess water contained in the fresh refuse during placement, and through good control of surface runoff water.

6.3.7 Control of Drainage

As indicted above, the control of water and drainage within the dump is critical to ensure stability. However, until the nature and behavior of the refuse is actually established, it is impossible to predict what control measures, if any, will be required. Therefore, rather than specify such measures at this time it is recommended that placement begin with the assumption that the refuse will be free-draining and readily compactable with only minimal control measures being taken. Then, over the course of the early months of operation, the properties of the refuse should be established and the need for additional drainage control evaluated.

In particular, it is assumed that the alluvial and colluvial material in the canyon will be sufficiently pervious to act as an underdrain for the refuse. To promote drainage from the refuse to the base of the dump, it is recommended that the coarse breaker refuse segregated in each lift along the longitudinal axis of the Canyon, to provide a central core of pervious material to which the refuse can drain. This minimal measure could be accomplished without additional cost to the disposal operation and would provide a degree of positive control over the buildup of water within the dump mass. As placement of the dump proceeds, water levels within the mass of the dump should be monitored as discussed below and the adequacy of the above procedure evaluated. If additional drainage control measures are indicated, then the internal drain system may have to be increased to include lateral feeders to the central drain and possibly additional control measures at the toe of the dump. Utilization of additional coarse colluvial material from the upstream canyon floor may be required in this case to supplement the supply of breaker refuse.

6.3.8 Construction Considerations

The general dump construction requirements are discussed in detail in Section 4.0. The following factors relate to the geotechnical aspects of the refuse dump development, refuse placement, drainage and compaction:

a. Although MESA requires that the left thickness not exceed two feet, it may be advantageous to reduce this to facilitate drainage and improve compaction. This should be established by trial and error early in the operation.

b. New lifts should be placed only over refuse that has had time to drain and has been properly compacted to provide a stable base for the new lift. The production schedule indicates that beyond year 1, each lift should have some 10 to 15 days to drain prior to placement of the next lift. Areas which remain wet and soft should be allowed more time to dry and/or be scarified and recompacted, if necessary.

c. The dump surface should always be graded to facilitate drainage away from recently placed fill toward surface drainage courses. It may be advantageous to bulldoze shallow ditches at each lift elevation to improve surface drainage.

d. Care should be taken not to fill over any frozen refuse which has not been properly drained and compacted.

e. Truck-loads containing predominantly filter cake should be spread out in a thin lift, and allowed sufficient time to dry, particularly during adverse weather.

f. It may often be necessary to place the refuse, allow time for drying, and then to compact the lift.

6.3.9 <u>Slope Monitoring</u>

Refuse dumps have been susceptible to some catastrophic failures in the recent past. Many of these disasters were considered by geotechnical engineers to be unnecessary, because it was felt that there had been significant unheeded warning of imminent failure. It is therefore considered prudent to install, maintain and observe a system for monitoring potential slope movements and groundwater levels in the Schoolhouse Canyon Refuse Dump.

Two relatively simple monitoring systems are considered appropriate for the Schoolhouse Canyon Dump. These are surface monuments in conjunction with line stakes, and standpipe piezometers. Figure 6-3(d) shows a conceptual plan view of the suggested monitoring program. This program could be supplemented with more sophisticated systems should signs of instability be noted.

Progressive installation of a system of surface monuments in conjunction with line stakes should provide both a qualitative and quantitative evaluation of surface expressions of slope movement. This displacement monitoring system would be developed as follows:

a. Installation of six instrument stations set in rock (three either side of the dump) for survey triangulation of dump face monuments.

b. Installation of a row of one-inch diameter pipe or rods (five feet long driven three or four feet into the dump) placed at 25-foot centers approximately every 100 feet horizontally up the face of the dump. These rods could be coated with irridescent paint and would be placed initially in a straight line, at as close to the same elevation as possible, and would be roughly perpendicular to the centerline of the dump face.

The first row of rods should be placed within 50 feet of the toe of the dump, when the dump had reached this corresponding elevation. These rod lines could be observed by dump operation personnel to note curvature or offsets in the lines indicating movement and potential instability.

c. On each of the above mentioned rod lines a concrete survey monument would be placed approximately 100 feet on each side of the dump centerline. These monuments would permit displacement measurement periodically by triangulation from the six survey stations.

Since it has been determined that groundwater can have a critical effect on stability of the dump, a series of standpipe piezometers should also be installed. These standpipes could be built into the dump as it increased in height. They could be constructed of two-inch PVC pipe with the lower ten-foot section slotted. Sections could be added as the dump was raised, taking due care to avoid refuse falling into, or damaging, the pipe. At each location three standpipes should be installed at different elevations. The bottom of the lowest standpipe should be within five feet of the natural ground surface. The second should be founded at about 1/3 of the ultimate dump height at that location and the third at 2/3 of that height. A general layout illustrating this proposed groundwater monitoring scheme is shown on Figure 6-2(b). Water levels should be taken in all of the standpipe piezometers whenever the dump face monuments are surveyed.

The results of the survey and piezometer data should be continuously plotted and periodically analyzed by a qualified geotechnical engineer familiar with the material properties, placement techniques and stability of the refuse pile. Based on this data analysis, the overall stability of the dump could be evaluated. Should the data indicate excessive movement and/or excess pore pressure, it would be necessary to alter the construction procedure. Modifications might include flatter slopes, installation of underdrains, decreased rate of dumping, and/or other procedures.

6.3.10 Final Comment

Based on the information available, it is believed that the proposed scheme should be operationally feasible and stable provided it is properly implemented. However, in the unlikely event that severe refuse handling, placement and compaction problems are encountered, the following might be considered to permit continued operation:

a. Temporary flattening of dump face slope angle.

b. Development of underdrains as discussed in section 6.3.7.

c. Simultaneous dump development in another canyon to increase operational flexibility.

d. Artificial refuse stablization measures.

e. Underground disposal of thickener underflow fines as discussed in the Phase I report. This approach is strongly recommended economically and geotechnically, irrespective of the outcome of combined refuse disposal operations.**

6.4 Settling Pond Embankment

6.4.1 Location and Configuration

The proposed Settling Pond Embankment was located near the mouth of Schoolhouse Canyon with the axis approximately on the N 511,000 coordinate line. This axis was designed to have a slight curvature which is convex in the upstream direction so that the embankment will tend to "spread" against the abutments when the pond is full of water. It was designed on a circular arc of 450.0 foot radius with the center at N 510,560.0 and E 2,178,730.0. The embankment has a proposed crest width of 20 feet with a 0.5 foot camber at the center. Both the upstream and down stream slopes were set at 3 horizontal to 1 vertical. The embankment layout along with that of the associated spillway is shown in Figure 6-1(b).

6.4.2 Control of Seepage

The Settling Pond has been designed to retain water only long enough to settle out undesirable suspended solids. When the water has sufficiently clarified, it will be pumped into the culvert system beneath the plant and railroad for discharge into the Price River. A portion of the water is expected to seep into the valley floor and into

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**A recent paper by Jankovsky, "Disposal of Coal Refuse Slurry Underground" (Mining Congress Journal, September 1977), may be of interest.

the embankment. This water will be filtered by the ground before it enters the Price River water regime. Due to this filtering action, seepage below the embankment was considered allowable as long as the stability of the embankment was not affected.

Two seepage conditions which might affect stability need to be considered. First, seepage through the dam must be kept deep within the embankment to improve stability and prevent breakout above the downstream toe. Second, seepage below the dam and at the abutments must be controlled to prevent piping failure. The use of a blanket drain and relief wells at the toe and good seals at the abutments is intended to provide the necessary control. Details of the embankment drainage control system are presented in Figure 6-1(b).

It has been specified in the design documents that the abutments be prepared by excavating to sound rock from above the crest, while maintaining a slight batter against which the embankment can be compacted. Overhanging or loose rock is to be removed by jackhammers or light blasting. Hand scaling and cleaning by means of compressed air may be required in order to properly prepare the abutment surface for placement of the embankment.

6.4.3 Spillway

The spillway shown in the Settling Pond specifications has been designed to provide adequate flow capacity and therefore to avoid over-topping of the embankment or spillway. The spillway requires that the slopes be established either in sound rock or be well rip-rapped. The spillway ditch side slopes should be cut at 1/2:1 in sound rock and cut in 1:1 in compacted fill where fill and rip-rap are required. the channel profile was designed to provide a stilling basin before water enters the culvert system, and to allow access for maintenance and pumping of the pond.

6.4.4 Embankment Fill Materials

It was concluded that a homogeneous earth embankment using well-graded silty materials such as the colluvium found in the area would provide the most economical section. As noted in Section 6.2.5, three possible borrow sources are: (1) excavated material from haul road and diversion ditch; (2) material from General Coal Borrow Pit; and (3) road Base material from the Corn Borrow Pit. These materials are similar, and, therefore, due to the proximity and availability of the colluvium in Schoolhouse Canyon it has been suggested that the colluvium should be utilized to the maximum extent possible. The gradation specification for the blanket drain was based on the use of colluvium from Schoolhouse Canyon. However, this blanket drain gradation should be suitable with the other two materials mentioned.

6.4.5 Embankment Stability

It is expected that the settling pond embankment stability will be governed largely by the condition of the contact area between the fill and the abutments. Working room is limited and unless care is taken in abutment preparation and thorough fill compaction in these zones, uncontrolled seepage could occur, leading to piping failure.

Slope stability of the proposed embankment was also assessed in light of the strength properties obtained from the consolidated-drained triaxial tests given in Appendix B. Given the flat slopes recommended for the embankment, it is considered that the embankment will be stable under all conditions of operation.

6.5 Haul Road And Diversion Ditch

The haul road and diversion ditch have generally been designed to be cut into rock. Inspection of the dozer trails and nearby cuts suggested that very steep slopes can be stable for significant periods of time. With the relatively flat-lying for sedimentary rocks found in Schoolhouse Canyon, slopes carefully cut at 1/2:1 in sound rock should be stable throughout the life of the facility. Overlying colluvial soil and highly-fractured rock slopes, however, should be cut to 1 1/2:1 to maintain stability.

To maintain a reasonably straight alignment and minimize the volume of rock excavation, portions of the diversion ditch are likely to be founded in soil. Where this occurs, it will be necessary to over-excavate and re-compact the soil to develop the stability required. These low, well-compacted cuts should stand at 1:1 with only routine maintenance required.

The natural colluvial slopes in the canyon are at approximately their maximum stable configuration. The addition of sidecast material to these slopes will probably not permit large thicknesses of fill at the top of the slopes and will most likely result in the development of a thin layer of sidecast material extending to the toe of the slope at its angle of repose.

Respectfully Submitted,

GOLDER ASSOCIATES, INC.

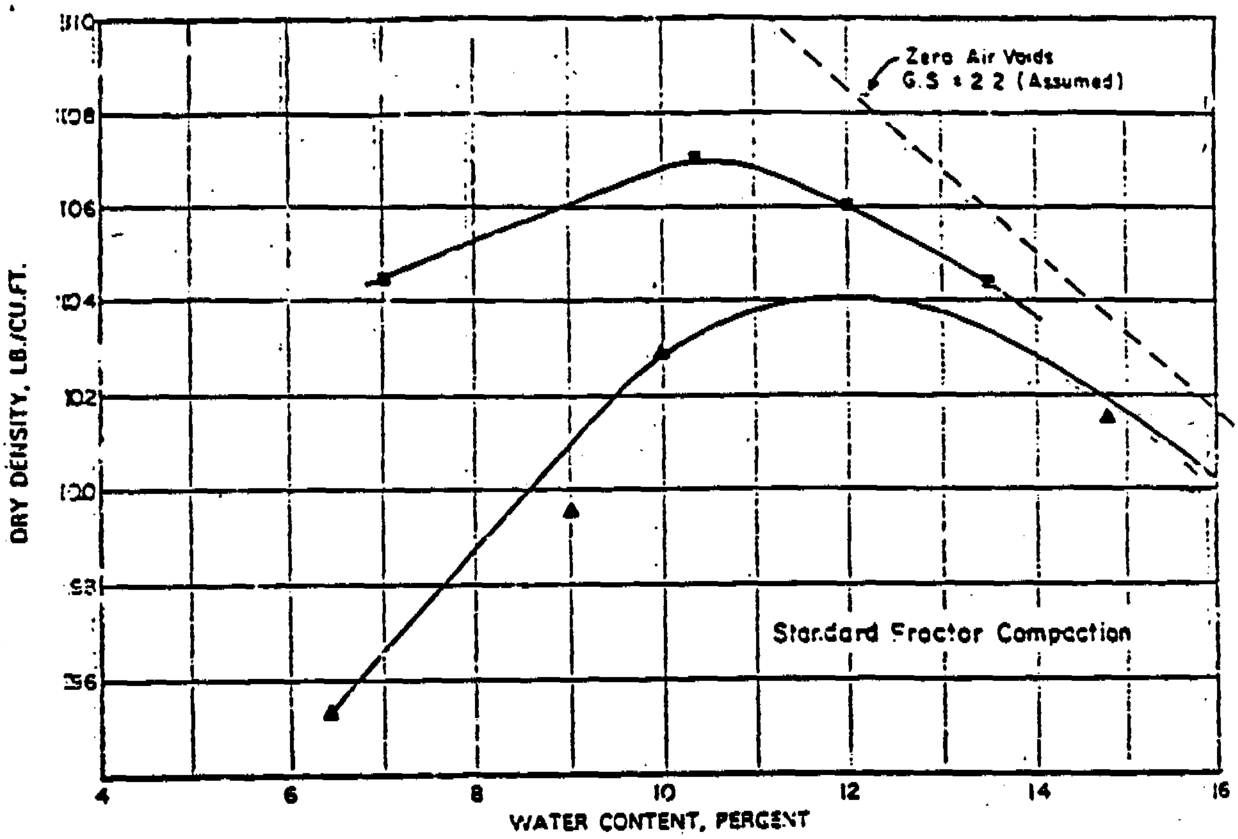
A. A. Gass
Principal-in-Charge

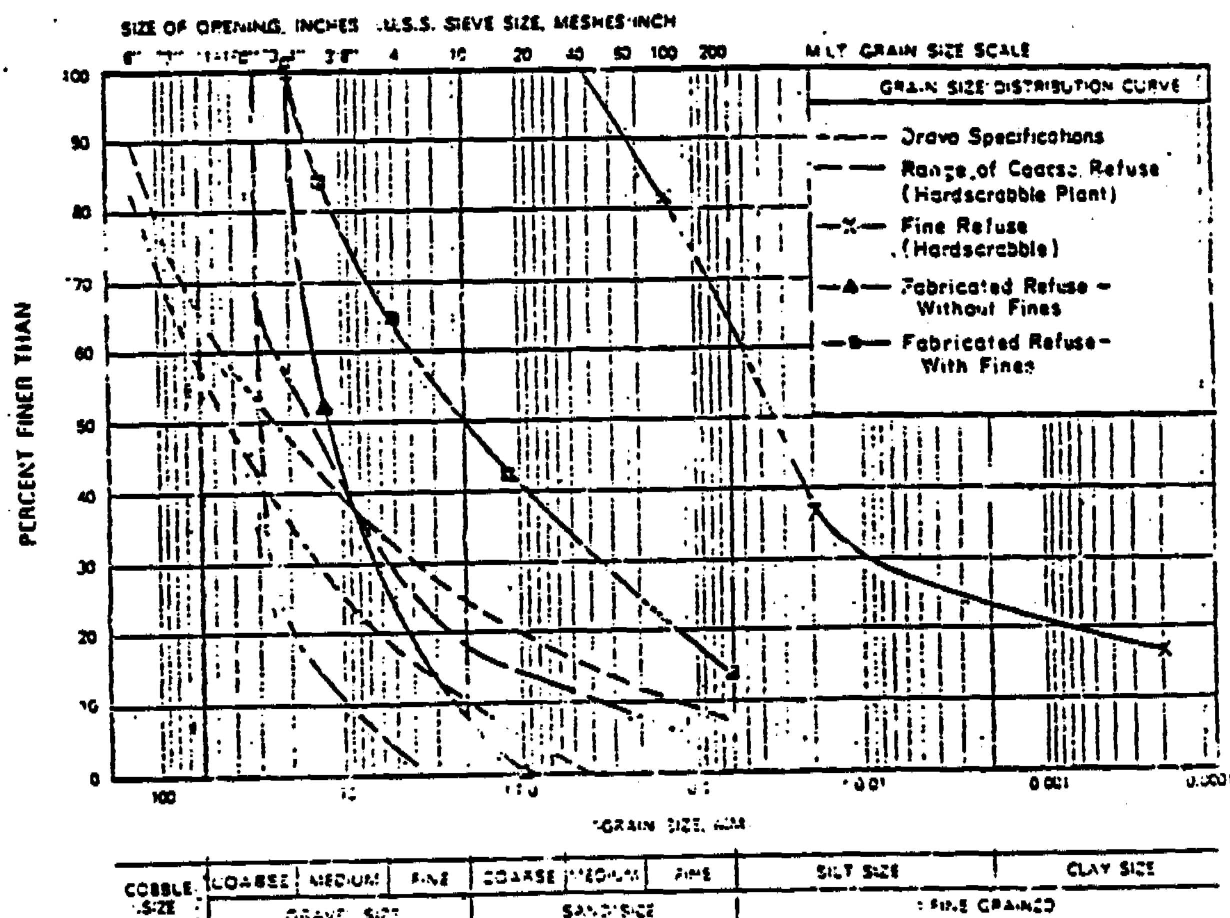
G. A. Mathieson
Project Engineer

AAG/GAM:hd
S77212

LABORATORY COMPACTION TEST RESULTS

FIGURE 6-3 (a)





Project No. 77212

Golder Associates 3.4ALxxxv

Date Jan 1977

By

APPENDIX 3.4B

EXCERPTS CONCERNING REFUSE ENGINEERING CHARACTERISTICS TAKEN FROM GOLDER ASSOCIATES REPORT ON "DESIGN OF A COAL REFUSE DISPOSAL SYSTEM, PHASE I, SITE FEASIBILITY STUDY", SEPTEMBER 1977

APPENDIX 3.4B

Excerpts concerning refuse engineering characteristics taken from Golder Associates Report on "Design of a Coal Refuse Disposal System, Phase I, Site-Feasibility Study", September 1977.

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The canyon bottom increases from the canyon head to its mouth. Near the mouth of some of the larger canyons, the overburden thickness could exceed 50 to 60 feet, although 20-30 feet is probably more representative for most of the canyon length.

The true groundwater table in the area is believed to correspond roughly with the major streams and rivers. Thus the refuse piles will be constructed well above the natural water table. Due to the near horizontal bedding of the bedrock formations, some local perched water table conditions may exist during spring snowmelt and after heavy rainfall. These conditions may result in some seeps appearing on the canyon walls.

5.3 Engineering Characteristics of Refuse Material

In order to assess the stability of the proposed refuse pile and evaluate the engineering behavior of the refuse, it is necessary to determine the engineering characteristics of the refuse material. The important properties which might affect the results of this feasibility study include gradation, moisture content, unit weight, compaction characteristics, weathering characteristics, permeability, and strength. It was not possible to obtain a representative sample of the proposed refuse. Thus the discussion presented in this section and the stability analysis presented in Section 5.4 is preliminary and may have to be revised pending more information and testing.

A sample of the proposed refuse material was fabricated based on information obtained from Dravo Corporation (plant designers) and utilizing material obtained from a test trench in the current AEP refuse pile in Hardscrabble Canyon. Since the existing plant in Hardscrabble is different from the new plant, the sample was not representative of the refuse material from the new plant. However, it was assumed for this study that the mineral composition is similar with the primary differences being gradation and moisture content. A total of four bag samples were obtained from the test trench at depths up to 7 1/2 feet. The samples were very similar as indicated by gradation tests results shown in Appendix B. The samples were sieved and mixed together in the proportions necessary to fabricate two samples representing the following anticipated gradation:

Refuse with Filter Cake:

Source	Size	Percentage	Water Content
Breaker Refuse	12" to 4"	25%	5%
Coarse Refuse	4" to 1/2 "	35%	5%
Fine Refuse	1/2" to No. 28 Mesh	20%	8.5%
Filter Cake	No. 28 Mesh to 0	22%	33%

Refuse Without Filter Cake:

Source	Size	Percentage	Water Content
Breaker Refuse	12" to 4"	31%	5%
Coarse Refuse	4" to 1/2 "	43%	5%
Fine Refuse	1/2" to No. 28 Mesh	26%	8.5%

This information was obtained verbally form Mr. Ed Seolnick of Dravo Co. and from Dravo Drawing "Material Flowsheet - Coal Preparation Plant, Castle Gate, Utah".

A limited laboratory testing program was performed on the two fabricated refuse samples. These tests included sieve tests, hydrometer tests, Atterberg Limits, specific gravity tests, weathering tests, and compaction tests. Results are presented in detail in Appendix B and can be summarized as follows:

a. Gradation (See Figure B-24, Appendix B):
   With Filter Cake - gravel with cobbles and silty sand.
   Without Filter Cake - gravel with cobbles and about 10% coarse sand.

b. Moisture (based on Dravo data):
   With Filter Cake - 14%
   Without Filter Cake 5%

c. Plasticity (of fines): Non-plastic.

d. Specific Gravity (overall): estimated about 2.2

e. Compaction (see Figures B-2.5 and B-2.6, Appendix B):
   With Filter Cake - 107 pcf @ 10.5%.
   Without Filter Cake - 104 pcf @ 12%.

f. Weathering: Randomly chosen rock fragments exhibited wide range of sensitivity to weathering. Some fragments showed no signs of degradation even after 4 wetting and drying cycles. Other samples decomposed rapidly. However, none of the samples exhibited any plasticity but appeared to weather to silt.

Due to the preliminary nature of this study, and the lack of a reliable representative sample, no strength tests were performed. Rather the strength behavior of the proposed refuse material was estimated based on its anticipated composition and on published strength data on similar materials. The literature reviewed and the pertinent information abstracted is summarized in Appendix B. In general, the strength behavior of coal refuse is not well understood. There appears to be no reliable correlation between strength and other refuse characteristics. In addition, the data reported in the literature exhibits a wide range of strength values. However, it is considered that the strength behavior of the proposed refuse, assuming placement and compaction in two-foot lifts with adequate drainage of excess moisture, can be approximated as follows:

a. The refuse is a cohesionless material with a probable friction angle between 32 to 38 degrees at low stresses.

b. With increasing confining stress, the friction angle decreases. Thus the strength envelope becomes curved at high stresses and exhibits apparent cohesion. This behavior is not unique to coal refuse and is a property of most coarse-grained materials.

c. Based on published information relating the decrease in the friction angle to the confining pressure, in conjunction with the strength data on coal refuse, Figure 5.1 was constructed. This figure represents the most reasonable estimate of the probable maximum and minimum strength envelope for the refuse. The procedure and assumptions used to develop these curves are discussed in Appendix B.

d. Consideration was given to the effect on the strength of separating the filter cake material. It is believed that the same degree of compaction, both types of refuse probably have very similar strength properties. In fact, the refuse with the filter cake may even be superior since it would be more uniformly graded. The most significant difference results from a higher moisture content of refuse containing the filter cake. However, as discussed below, even with the high

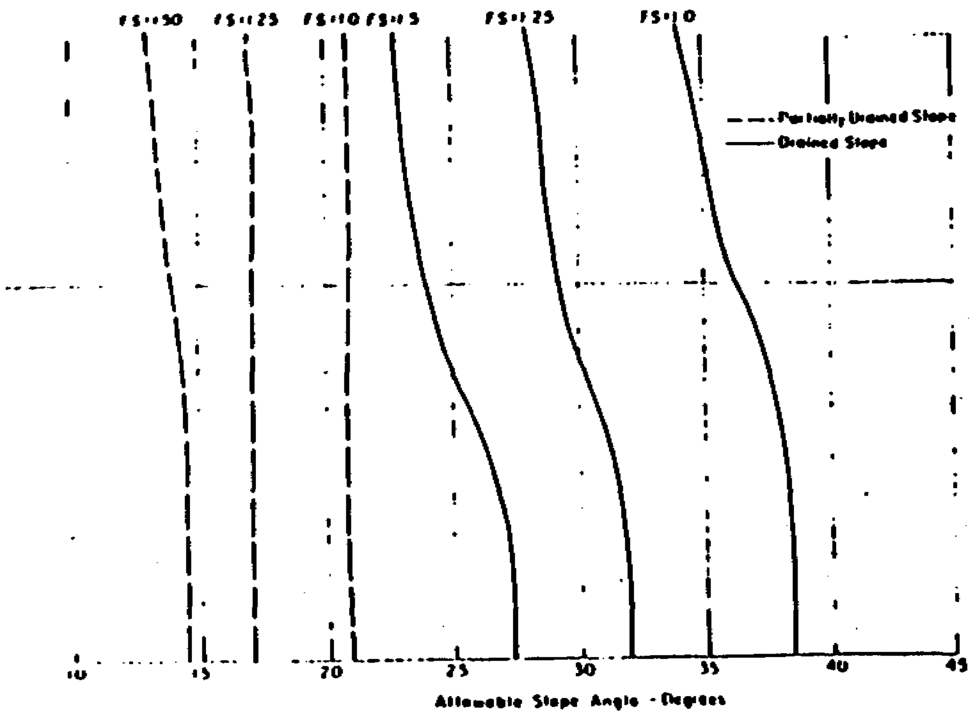


Figure 5.1A MAXIMUM INFERRED STRENGTH VALUES

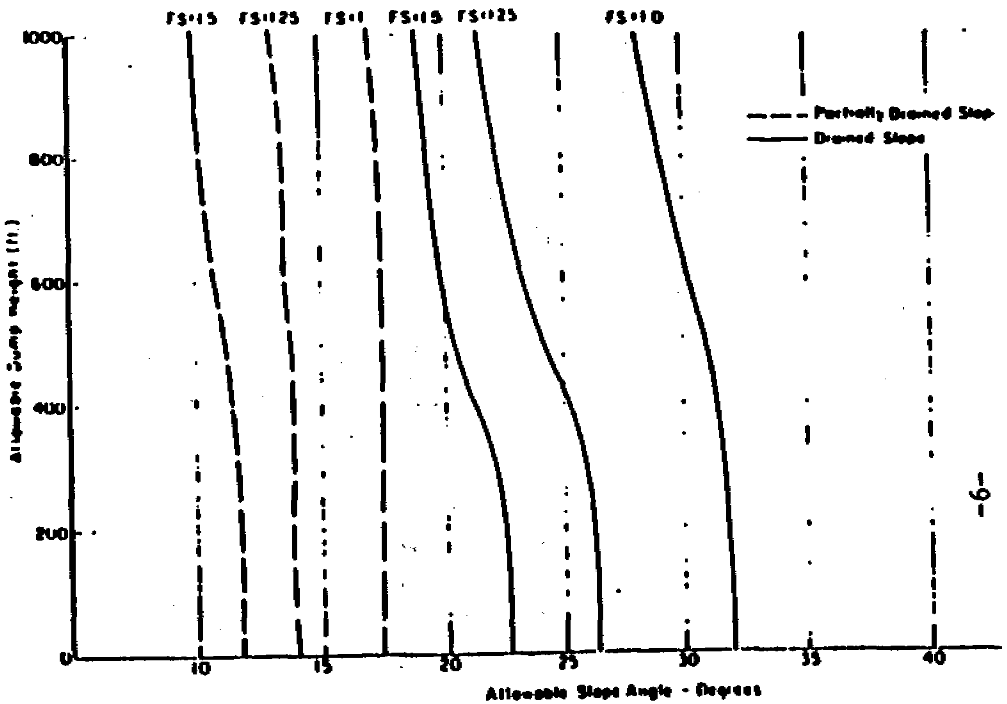
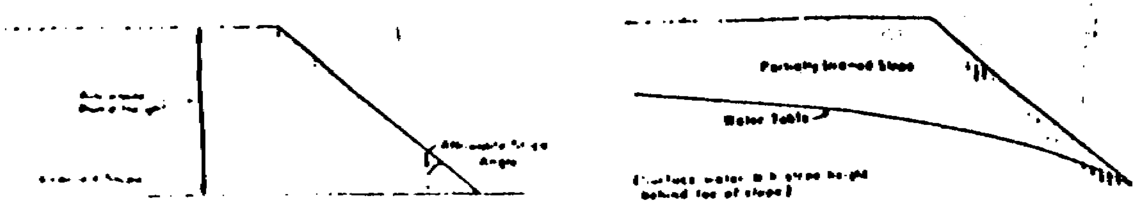


Figure 5.1B MINIMUM INFERRED STRENGTH VALUES



-6-

moisture content, it is believed that the refuse including the filter cake can be suitably placed and compacted. Thus, based on the limited available information, it is not feasible to delineate a meaningful strength difference between the refuse material with or without the filter cake material. This preliminary conclusion may later be revised pending more information and testing.

The in-place density and final moisture content of the refuse pile will have a significant effect on its strength behavior. Assuming the refuse is placed in two-foot lifts and lightly compacted with dozers or scrapers, the final density and moisture content will be determined by the compaction characteristics of the refuse, the initial moisture content, the permeability of the refuse, surface drainage conditions, weathering, and rate of fill placement. Refuse containing the filter cake, which has an anticipated initial moisture content of some 14%, is potentially a much more difficult material to place and properly compact. However, the results of the compaction tests indicate that the refuse can be effectively compacted over a wide range of moisture conditions. Even at 14% moisture, which is practically at saturation, the material was compacted to about 96% of maximum density.

The anticipated rate of fill placement will probably be less than one lift per day (assuming a two-lift and a total of about 1,600 tons placed per day). In addition, the initial permeability of uncompacted fill is expected to be quite high and should allow rapid drainage of excess water (permeability after compaction and weathering will probably be much lower). Considerable drainage of excess water may also occur in the refuse storage bin and in the trucks or scrapers en route to the refuse pile. This high permeability to conjunction with the slow filling rate and generally arid climate could result in considerable drying of each lift. Therefore, based on the information available, the refuse material even with the filter cake can probably be adequately compacted. Adverse conditions such as heavy rain, snow melt or extended frost may occasionally make proper placement of the refuse, especially with the filter cake, difficult. However, through proper construction procedures, it is believed that these problems could be overcome. These procedures might include grading the fill for optimum surface drainage, ripping and recompacting frozen layers, using thinner lifts, a greater compacting effort, and/or other appropriate procedures.

Consideration was also given to potential weathering effects. The results of the weathering tests indicated that some of the refuse material is very susceptible to weathering and decomposition. However, as reported by Thomson and Rodin (1972), after an initial quick physical degradation, probaly due to the compaction equipment, very little, if any, further breakdown appears to occur below a depth of a few feet. This was evident in the test trench in the existing refuse pile at Hardscrabble Canyon. Although some breakdown and weathering had occurred here, especially at the top of each two-foot lift, there was no evidence of excessive decomposition, or of increased degradation with fill depth and age. In addition, weathering products appear to comprise a non-plastic silt rather than a plastic clay found in some of the bentonitic shales in Utah. In conclusion, it does not appear that weathering will have a significantly adverse impact on the refuse pile.

The final permeability of the refuse is also a concern. Due to compaction, mechanical breakdown and weathering, the fill will tend to form a zone of lower permeability at the top of each fill lift. This could result in local perched water conditions after heavy rainfalls or during spring snow melt. However, lack of information precludes conclusive comment in this regard.

5.4 Stability Analysis

The allowable refuse pile slope angle and corresponding height are important constraints on canyon disposal schemes. These constraints may significantly affect the total refuse volume capacity of a canyon, the geometry of the refuse pile, and the cost of refuse disposal. The impact of these constraints are discussed in detail in Section 7.0.

As discussed in Section 4.2, MESA does not regulate dump height nor the dump factor of safety, provided the refuse is placed in two-foot lifts and has no slopes exceeding 2 horizontal to 1 vertical. Should any slope exceed 2:1, then MESA requires a minimum factor of safety of 1.5 for the overall refuse pile. In general, for the disposal schemes considered in this study, the ideal slope would have an overall slope angle of about 2:1 but would locally exceed 2:1 between haul road ramps. Thus, one of the primary purposes of the stability analysis in this phase of the study was to assess the feasibility of locally exceeding 2:1 slopes while maintaining an overall factor of safety in excess of 1.5. Naturally, regardless of MESA requirements, the refuse pile must be properly designed and have an acceptable factor of safety (although it need not be as high as 1.5 if the MESA 2:1 slope requirements are satisfied).

For this sitting feasibility phase (Phase I), a rigorous stability analysis of all the refuse schemes would be inappropriate and unwarranted. Rather, the stability analysis performed was a non-rigorous evaluation which could be equally applied to all canyons. During Phase II design a rigorous stability analysis may have to be performed on specific disposal schemes. Details of the non-rigorous analysis conducted to date are discussed in Appendix B.

The results of the preliminary slope stability analysis are shown on Figure 5.1. These plots show the relationship between the allowable average slope angle and the allowable refuse height for different factors of safety and different assumed strength characteristics. Figure 5.1A is based on a fully drained slope while Figure 5.1B is based on a partially drained slope. From these curves it is obvious that proper drainage of the refuse pile is very important. Even a slight build-up of seepage pressures could have a very adverse effect on the stability of the refuse pile. Also, in order to exceed 2:1 on inter-ramp slopes, it may be necessary to flatten the overall slope to less than 2:1. Based on the maximum probable refuse strength values and a drained slope, refuse piles in excess of about 300 feet would have to be flattened to less than 2:1 overall, in order to meet the MESA factor of safety requirement. Based on the minimum probable strength values, the overall slope would have to be flattened to less than about 22 degrees to justify steep inter-ramp slopes.

5.5 Other Geotechnical Considerations

Some preliminary consideration was given to placing the refuse in the flats in the gentle sloping areas south of Kenilworth and west of Helper as shown on Figure 3.2.

APPENDIX 3.4C

HORROCKS AND CAROLLO ENGINEERS REPORT,
"SLOPE STABILITY ANALYSIS ON COAL REFUSE PILE AT CASTLE GATE
PREPARATION PLANT", MARCH 1983

APPENDIX 3.4C

SLOPE STABILITY ANALYSIS

ON

COAL REFUSE PILE

AT CASTLE GATE PREPARATION PLANT

PRICE RIVER COAL COMPANY

Horrocks & Carollo Engineers

APPENDIX 3.4C

SLOPE STABILITY ANALYSIS
ON
COAL REFUSE PILE
AT CASTLE GATE PREPARATION PLANT
PRICE RIVER COAL COMPANY

MARCH 1983

REGISTERED PROFESSIONAL ENGINEER
No. 3535
HAROLD LEE
WIMMER
STATE OF UTAH

Prepared by
Harold Lee Wimmer, P.E.
Utah P.E. No. 3535

HORROCKS & CAROLLO ENGINEERS
One West Main
American Fork, Utah 84003
801-756-7628

SECTION I

At the request of Price River Coal Company, I reviewed the report prepared by Golder Associates dated January 18, 1978, regarding the design of the coal refuse disposal system, including the detailed design of the Schoolhouse Canyon Refuse Dump facility. In particular, I have reviewed the geometric considerations for the dump site, the material considerations, and comments relating to construction contained in said report.

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SECTION II

CONSTRUCTION RECORDS

On January 28, 1983, I visited the site in conjunction with Rob Wiley, Environmental Engineer, and Frank Pero, of the Price River Coal Company, and reviewed in detail the provisions taken at the site during construction in accordance with the previously mentioned "Golder Report". I also reviewed with Mr. Pero (who was present during the construction), the construction records including construction pictures which enabled me to determine that the dump site was constructed in basic accordance with the plans to its present state.

In particular, large sandstone rocks from the diversion channel construction were bladed to the bottom of the existing canyon to provide for the draining of seepage waters from the refuse material.

SECTION III

MATERIAL TESTING

While at the site, using a Troxler 3411B nuclear density gauge, I determined the in-place density of the refuse material. I also obtained moisture density samples and samples of the refuse material, which I returned to the lab for additional testing. The results of these in-place determinations (attached in the Appendix) indicate that the average in-place density of the material varied from 84% to 110% of the laboratory obtained T-99 standard proctor.

When the coal refuse is thoroughly mixed and remolded the T-99 Proctor valve increases significantly due to additional breakdown of the "bedrock" characteristics of the material (see "Composite Coal Refuse Pile" T-99 Standard Proctor in Appendix). I submitted a sample of the refuse material to Chen and Associates, a consulting soil and foundation engineering firm, to determine the relationship of the loading to the shear stress, and to determine the internal cohesion. These results are included in the Appendix. The material gradation results are also included in the Appendix. The gradation results indicate that the material is free draining, nonplastic, and falls within the gradation bands contained in the "Goldner Report".

SECTION IV

OCCURRENCE OF GROUND WATER AND PORE PRESSURE BUILDUP

The results of the gradation analysis indicate that the material is free draining. This was further observed at the site through reviewing the existing material in place and by analyzing the records kept on the ground water observation pipes in the refuse pile. The data (summarized) for the ground water observation records is contained in the Appendix.

Basically the records confirm that the material is free draining and no pore pressure build up is occurring. The maximum recorded depth of water (6') occurred during the wet portion of an above normal precipitation year.

SECTION V

FACTOR OF SAFETY

A compure model was constructed to analyze the stability of the refuse pile, and the following conditions were assumed.

1. Ground Water at six feet (the highest level recorded to date).
2. In-place densities of 90 pounds per cubic foot.
3. Geometric configuration to conform to the proposed site when completed.

A computer simulation was then applied to this situation to determine various failure planes. The "Method of Slices" is the basis for the modified Bishop method computer program. Various failure planes were investigated to determine a minimum factor of safety. The results of these computer runs and a copy of the computer listing is attached in the Appendix. The results of these computer simulations indicate that the minimum static factor of safety is 4.6, and the minimum factor of safety with a .1 g earthquake loading is 2.6.

SECTION VI

CONCLUSIONS

In conclusion the coal refuse disposal pile as now existing is:

1. Free draining.
2. The maximum water depth measured by monitoring has been six feet, and this occurred during an abnormal wet period of time. The monitoring wells show several inches of water or less during most of the year.
3. No movement of the refuse pile has been detected.
4. There is no water pore pressure buildup in the refuse pile.
5. The computer simulation on failure planes indicates that the factor of safety is at least 2.6 with a .1 g earthquake loading.

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APPENDIX

CASTLE GATE COAL COMPANY
SCHOOLHOUSE CANYON REFUSE DUMP
GROUNDWATER PIEZOMETER DATA SUMMARY

Date	OBSERVATION STATION NUMBER				
	#10	#11	#12	#13	#14
10-21-80	T	D	T	T	T
11-04-80	T	T	T	1"	T
12-02-80	T	T	T	2"	T
1-06-81	T	D	T	1"	T
2-02-81	T	T	T	1"	T
3-03-81	D	D	T	T	T
4-08-81	1"	T	1"	2"	T
5-06-81	T	D	T	1"	T
6-02-81	D	D	D	D	D
7-07-81	D	D	D	D	D
8-13-81	D	D	T	T	D
8-08-81	D	D	T	T	D
9-08-81	D	D	T	T	D
10-08-81	T	T	1"	2"	T
11-09-81	D	D	D	T	D
12-10-81	T	T	D	T	T
1-13-82	D	D	T	2"	T
2-11-82	D	D	T	8"	D
2-25-82	D	D	T	1'	T
3-03-82	D	D	2"	3'	D
3-12-82	D	D	18"	5'	D
3-18-82	D	D	2'	6'	T

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PRICE RIVER COAL COMPANY

INTER-OFFICE MEMORANDUM

R. L. Wiley

DATE: 1-11-83

FROM: F. L. Pero

C.C.:

SUBJECT: Refuse Pile Construction

Construction of this facility was begun in 1978 and completed in 1979. During this time close communication with the State Engineer's Office was maintained and the site was visited several times by representatives of that office.

The primary concerns of the regulators were the competency of the pond embankment and drainage of the pile itself. In an effort to allay these fears, the pond embankment was constructed with engineered backfill and tight contruction specifications were maintained.

The rock underdrain was constructed using material excavated from the diversion structure. The diversion was cut entirely in rock and runs parallel to the canyon floor for most of its length. The blasted rock was dozed into a blanket at least 4 ft. thick and is uniformly mixed rock ranging in size up to about 4 ft. There are larger pieces, but these occur only randomly. No less than 60% of the material is in the 2 ft. minus range, 25% is 2 ft. to 3 ft. range, 10% is 3 ft. to 4 ft. and no more than 5% is larger than about 5 ft. diameter. Also, a crushed rock underdrain was installed between the toe of the pond embankment and the trash rack inlet on the pond overflow ditch. This was designed to collect any ground water which might collect either at the abutments or beneath the pond embankment.

As mentioned before, very tight controls were exercised during the construction of this faility. This consisted partly of very comprehensive soil and compaction testing. Nuclear density tests were performed on every 6" compacted lift throughout the embankment height, with no less than 3 tests taken at random locations on every lift. Laboratory series tests were conducted several times during the construction to ensure that the correct proctor information was being used to determine in-place density. Copies of all test results were furnished to the State Engineer's Office.

Frank L. Pero

Frank L. Pero

FP:jp

MATERIALS AND TESTS DIVISION

MOISTURE-DENSITY RELATIONS

PROJECT NAME Price River Coal Company DATE 2-14-83

PROJECT NO. - SAMPLE NO. 1 (COAL) Refuse Pile

METHOD OF COMPACTION T-99 PROCTOR

TEST NO.	1	2	3	4	5	6	7	8
CYL. & WET EARTH IN GRAMS	7859	7944	7979	7949	7929			
CYLINDER WT. IN GRAMS	5527	5527	5527	5527	5527			
WET EARTH IN GRAMS	2332	2417	2452	2422	2402			
WET DENSITY IN LBS/CU. FT.	68.5	71.0	72.1	71.2	70.6			
DISH NUMBER	108.3	101.7	107.4	181.2	180.8			
DISH & WET SOIL WT. IN GRAMS	396.0	430.6	466.1	690.2	657.4			
DISH & DRY SOIL WT. IN GRAMS	375.5	411.9	442.8	643.8	600.8			
WATER WT. IN GRAMS	20.5	18.7	23.3	46.4	56.6			
DISH & DRY SOIL WT. IN GRAMS	375.5	411.9	442.8	643.8	600.8			
DISH WT. IN GRAMS	108.0	101.1	107.2	180.6	180.0			
DRY SOIL WT. IN GRAMS	267.5	310.8	335.6	463.2	420.8			
MOISTURE IN % OF DRY WT.	4.8	6.0	6.9	10.0	13.5			
DRY DENSITY IN LBS./CU. FT.	65.4	67.0	67.4	64.7	62.2			

$$\text{DRY DENSITY} = \frac{\text{WET DENSITY}}{100 + \%\ \text{MOISTURE}} \times 100$$

TESTED BY: Mark W. Foster

MATERIALS ENGINEER

MATERIALS AND TESTS DIVISION

MOISTURE-DENSITY RELATIONS

PROJECT NAME Price River Coal Company DATE 2-14-83

PROJECT NO. ______ SAMPLE NO. 1 (COAL) Refuse Pile

METHOD OF COMPACTION T-99 PROCTOR

TEST NO.	1	2	3	4	5	6	7	8
CYL & WET EARTH IN GRAMS	7859	7944	7979	7949	7929			
CYLINDER WT. IN GRAMS	5527	5527	5527	5527	5527			
WET EARTH IN GRAMS	2332	2417	2452	2422	2402			
WET DENSITY IN LBS/CU. FT.	68.5	71.0	72.1	71.2	70.6			
DISH NUMBER	108.3	101.7	107.4	181.2	180.8			
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WATER WT. IN GRAMS	20.5	18.7	23.3	46.4	56.6			
DISH & DRY SOIL WT. IN GRAMS	375.5	411.9	442.8	643.8	600.8			
DISH WT. IN GRAMS	108.0	101.1	107.2	180.6	180.0			
DRY SOIL WT. IN GRAMS	267.5	310.8	335.6	463.2	420.8			
MOISTURE IN % OF DRY WT.	4.8	6.0	6.9	10.0	13.5			
DRY DENSITY IN LBS./CU. FT.	65.4	67.0	67.4	64.7	62.2			

$$\text{DRY DENSITY} = \frac{\text{WET DENSITY}}{100 + \text{\% MOISTURE}} \times 100$$

TESTED BY: Mark W. Foster

MATERIALS ENGINEER

HORROCKS ENGINEERS

Project Name: Price River Coal — Date: 2-13-85

Project No.: — Station or Pit Location:

Sample No.: 1 (COAL) [illegible] — Requested by: [illegible] Nimmer

AS RECEIVED GRADATION

Screen Size	Weight (g)	Percent Retained	Percent Passing	SPECS.
3"				
1½"	181.7	8.9	91.1	
1"	78.5	3.9	87.2	
3/4"	121.7	6.0	81.2	
1/2"	229.2	11.2	70.0	
3/8"	165.9	8.1	61.9	
#4	345.5	16.9	45.0	
Wet Wt. #4				
Wt. -#4				
Total Wt. Dry				

WASHED GRADATION AFTER CRUSHING (2500 GM. DRY SAMPLE)

Screen Size	Weight Retained	Percent Retained	Percent Passing	Total % Passing	SPECS.
#8					
#10					
#16	331.6	16.3	28.7		
#20					
#30	100.7	4.9	23.8		
#40					
#50	101.8	5.0	18.8		
#100	86.5	4.2	14.6		
#200	38.9	1.9	12.7		
-#200	258.2	12.7	0.0		
Total Wt.	2040.2	100.0			

MOISTURE DETERMINATION

	-#4
Container & Wet Soil Weight (gm.)	110.86
Container & Dry Soil Weight (gm.)	102.81
H_2O Loss	8.05
% Moisture	10.00

A.A.S.H.O Classification

A-2-6 (1

Wt. before washing 2042.9

Wt. after washing 1784.7

-#200 from Wash: 243.4

COPIES TO:

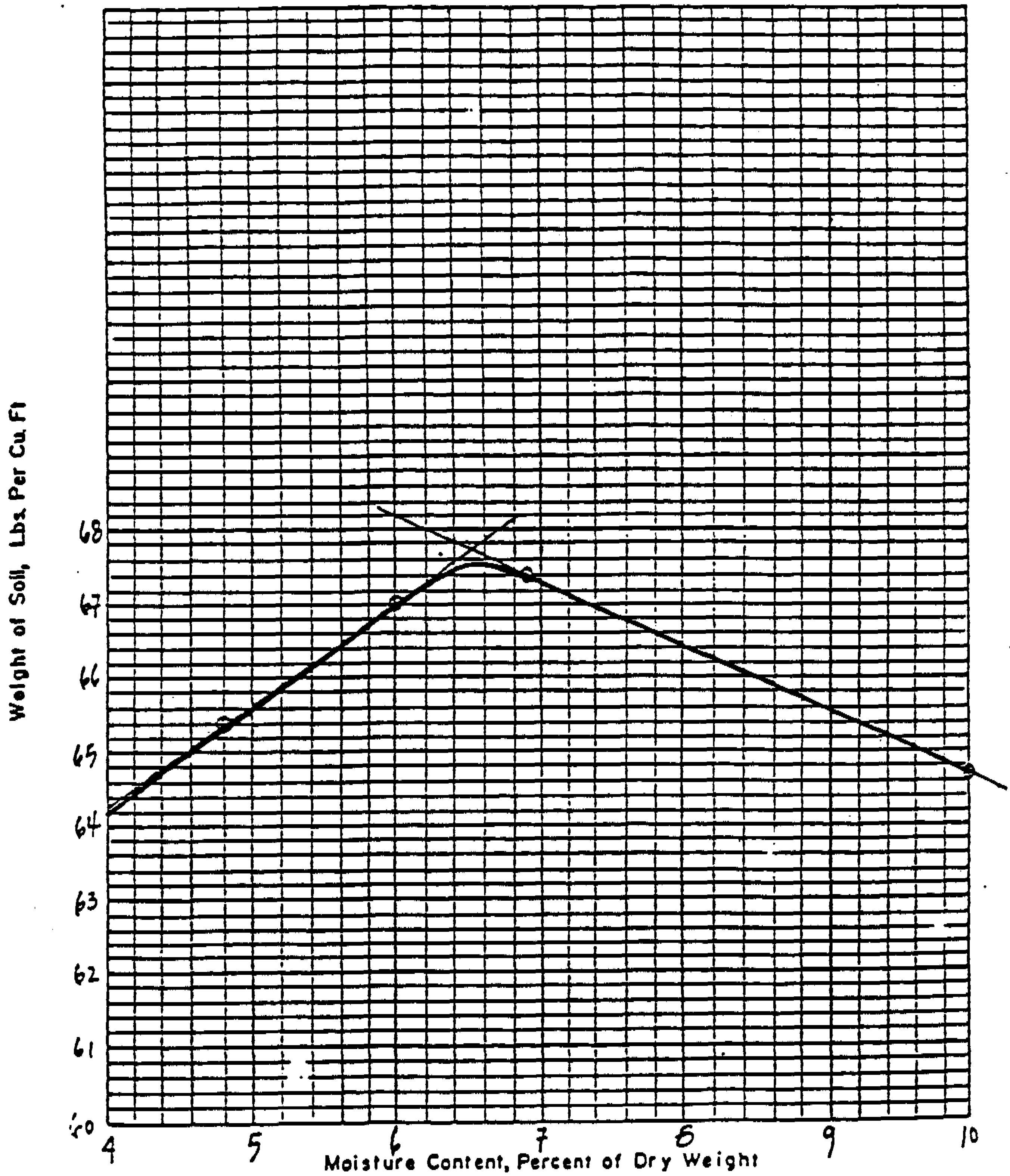
ENGINEERS

MOISTURE-DENSITY GRAPH

Project Name Price River Coal - Refuse Pile Date 2-13-83

Project No. ____ Sample No 1 (Coal) Refuse pile

Method of Compaction ____ P.C.F. 67.5 Optimum Moisture 6.5



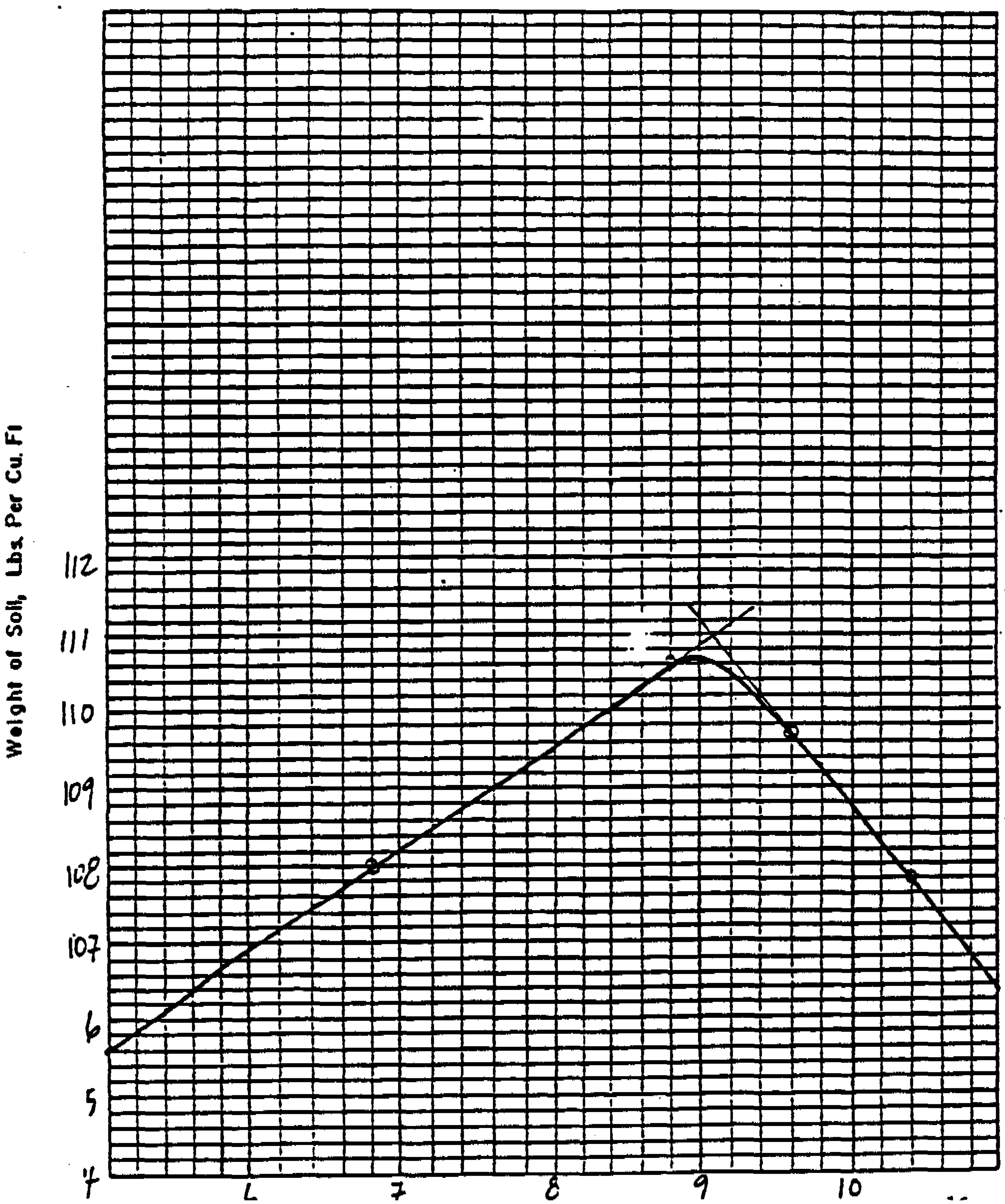
HORROCKS ENGINEERS

MOISTURE—DENSITY GRAPH

Project Name PRICE RIVER COAL COMPANY Date 2-21-83

Project No. ______ Sample No 1 & 2 (COMPOSITE COAL)

Method of Compaction T-99 PROCTOR P.C.F. 110.6 Optimum Moisture 9.0%



MATERIALS AND TESTS DIVISION

MOISTURE-DENSITY RELATIONS

PROJECT NAME PRICE RIVER COAL COMPANY DATE 2-21-83

PROJECT NO. ______ SAMPLE NO. 1 & 2 (COMPOSITE COAL)
Refuse Pile

METHOD OF COMPACTION T-99 PROCTOR

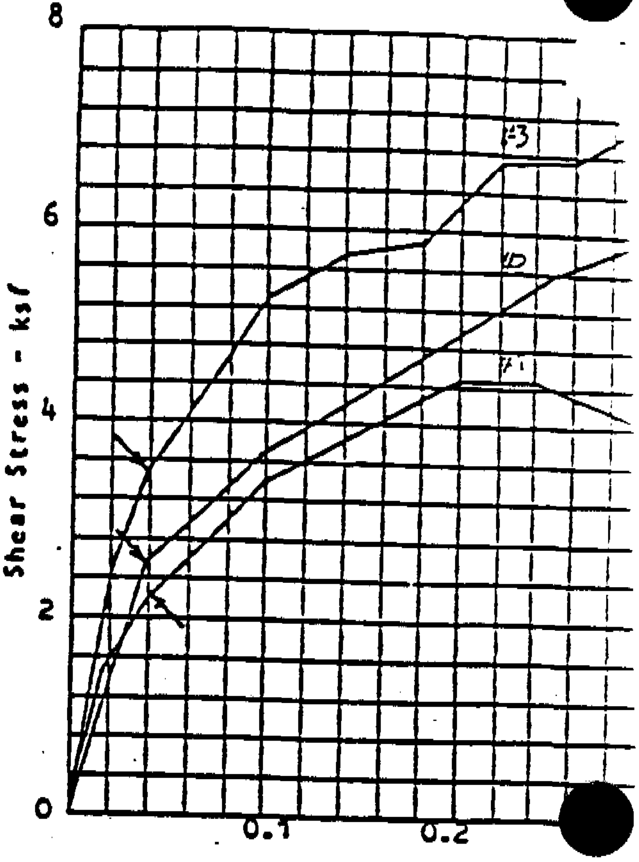
TEST NO.	1	2	3	4	5	6	7	8
CYL & WET EARTH IN GRAMS	9446	9616	9612	9572				
CYLINDER WT. IN GRAMS	5522	5522	5522	5522				
WET EARTH IN GRAMS	3924	4094	4090	4050				
WET DENSITY IN LBS/CU. FT.	115.3	120.3	120.2	119.0				
DISH NUMBER	101.7	137.0	5-G	4-G				
DISH & WET SOIL WT. IN GRAMS	444.9	404.3	337.4	321.4				
DISH & DRY SOIL WT. IN GRAMS	422.9	382.7	320.0	304.3				
WATER WT. IN GRAMS	22.0	21.6	17.4	17.1				
DISH & DRY SOIL WT. IN GRAMS	422.9	382.7	320.0	304.3				
DISH WT. IN GRAMS	100.1	131.7	138.4	140.0				
DRY SOIL WT. IN GRAMS	322.8	245.6	181.6	164.3				
MOISTURE IN % OF DRY WT.	6.8	8.8	9.6	10.4				
DRY DENSITY IN LBS/CU. FT.	108.0	110.6	109.7	107.8				

$$\text{DRY DENSITY} = \frac{\text{WET DENSITY}}{100 + \%\ \text{MOISTURE}} \times 100$$

TESTED BY: Mark W. Foster

MATERIALS ENGINEER

TEST NUMBER	1	2	3	4
LOCATION	SAMPLE #2			
HEIGHT-INCH	1.00	1.00	1.00	
DIAMETER-INCH	1.93	1.93	1.93	
WATER CONTENT - %	12.7	13.4	13.3	
DRY DENSITY - pcf	98.3	93.4	95.1	
CONSOL. LOAD - ksf	---	---	---	
NORMAL LOAD - ksf	1.5	3.0	4.5	
SHEAR STRESS - ksf	2.3	2.6	3.5	



TYPE OF SPECIMEN: Remolded

SOIL DESCRIPTION: Coal Refuge

Non-plastic, -200=23%

TYPE OF TEST: Quick

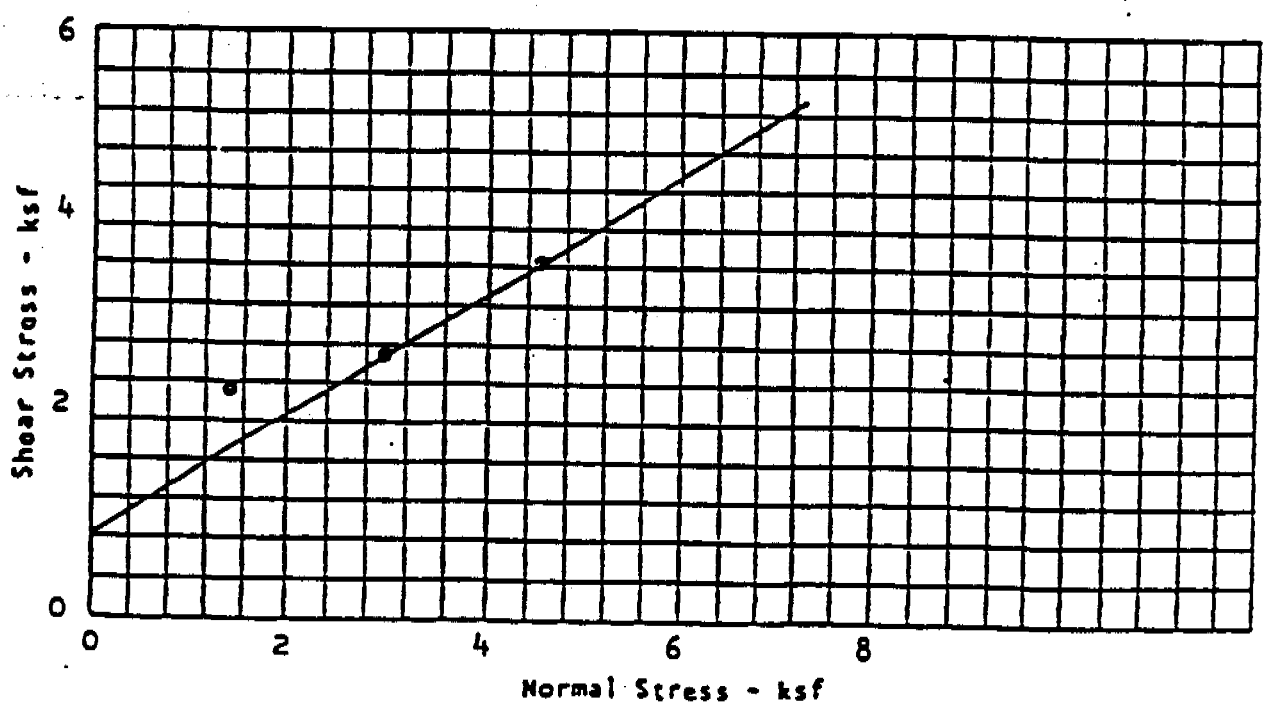
Strain rate 0.04 inches per min.

0.04 inches per min.

TAN Ø: 0.60

Ø: 31°

COHESION - ksf: 0.8



One West Main
P.O. Box 377
American Fork, Utah 84003
Telephone (801) 756-7628

LIQUID LIMIT AND PLASTICITY INDEX

PROJECT NAME Price River Coal Company DATE 2-13-83
PROJECT No. 8124-81

LIQUID LIMIT SAMPLE No. 1 (COAL)

CAN 5 WET 43.85 DRY 39.00
TAPS 13 DRY 39.00 CAN 22.72
H_2O 4.85 DRY WT 16.28 % H_2O 29.79 A

LL A 27.5

CAN ____ WET ____ DRY ____
TAPS ____ DRY ____ CAN ____ LL B ____
H_2O ____ DRY WT ____ % H_2O ____ B LL C ____

3) 27.5 L.L.

CAN ____ WET ____ DRY ____
TAPS ____ DRY ____ CAN ____ 17.3 P.L.
H_2O ____ DRY WT ____ % H_2O ____ C 44.8 P.I.

PLASTIC LIMIT

CAN 10 WET 36.77 DRY 34.64
DRY 34.64 CAN 22.34
H_2O 2.13 DRY WT 12.3 PL 17.32 (% H_2O)

LIQUID LIMIT SAMPLE No. 2 (COAL)

CAN 12 WET 51.84 DRY 46.53
TAPS 26 DRY 46.53 CAN 22.63
H_2O 5.31 DRY WT 23.90 % H_2O 22.20 A

CAN XX WET 45.12 DRY 40.90 LL A 21.37
TAPS 24 DRY 40.90 CAN 22.33 LL B 22.61
H_2O 4.22 DRY WT 18.57 % H_2O 22.72 B LL C 22.76

CAN C WET 31.77 DRY 28.42 3) 21.9 L.L.
TAPS 14 DRY 28.42 CAN 14.72 14.9 P.L.
H_2O 3.35 DRY WT 13.70 % H_2O 24.45 C 36.8 P.I.

PLASTIC LIMIT

CAN B WET 23.96 DRY 22.77
DRY 22.77 CAN 14.77
H_2O 1.19 DRY WT 8.00 PL 14.88 (% H_2O)

LIQUID LIMIT SAMPLE No. 3 (SILTY SAND)

CAN 2 WET 51.71 DRY 48.42
TAPS 13 DRY 48.42 CAN 22.17
H_2O 3.29 DRY WT 26.25 % H_2O 12.53 A

CAN ____ WET ____ DRY ____ LL A 11.6
TAPS ____ DRY ____ CAN ____ LL B ____
H_2O ____ DRY WT ____ % H_2O ____ B LL C ____

CAN ____ WET ____ DRY ____ 3) 11.6 L.L.
TAPS ____ DRY ____ CAN ____ 18.4 P.L.
H_2O ____ DRY WT ____ % H_2O ____ C 30.0 P.I.

PLASTIC LIMIT

CAN X WET 41.15 DRY 39.54
DRY 39.54 CAN 30.80
H_2O 1.61 DRY WT 8.74 PL 18.42 (% H_2O)

NO. OF TAPS

16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36

TESTED BY: Mark W. Foster

MATERIALS AND TESTS DIVISION

MOISTURE-DENSITY RELATIONS

PROJECT NAME Price River Coal Company DATE 2-14-83

PROJECT NO. ______ ______ SAMPLE NO. 2 (COAL) Refuse Pile

METHOD OF COMPACTION T-99 PROCTOR

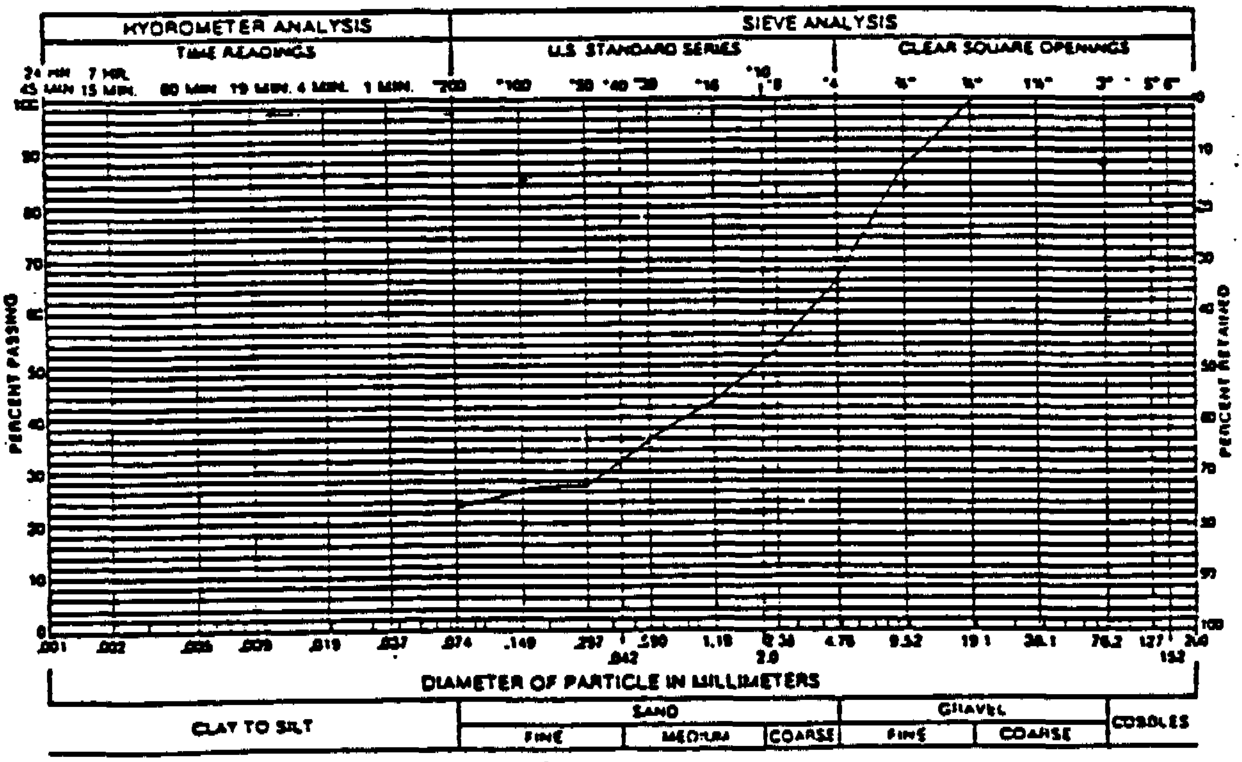
TEST NO.	1	2	3	4	5	6	7	8
CYL. & WET EARTH IN GRAMS	8164	8204	8207	8209	8164			
CYLINDER WT. IN GRAMS	5527	5527	5527	5527	5527			
WET EARTH IN GRAMS	2637	2677	2680	2682	2637			
WET DENSITY IN LBS/CU. FT.	77.5	78.7	78.8	78.8	77.5			

DISH NUMBER	B	C	A	137.0	D			
DISH & WET SOIL WT. IN GRAMS	304.8	273.9	255.0	500.0	336.2			
DISH & DRY SOIL WT. IN GRAMS	292.5	261.3	243.1	475.7	319.3			
WATER WT. IN GRAMS	12.3	12.6	11.9	24.3	16.9			
DISH & DRY SOIL WT. IN GRAMS	292.5	261.3	243.1	475.7	319.3			
DISH WT. IN GRAMS	45.7	44.5	46.9	137.1	134.8			
DRY SOIL WT. IN GRAMS	246.8	216.8	196.2	338.6	184.5			
MOISTURE IN % OF DRY WT.	5.0	5.8	6.1	7.2	9.2			
DRY DENSITY IN LBS./CU. FT.	73.8	74.4	74.2	73.5	71.0			

$$\text{DRY DENSITY} = \frac{\text{WET DENSITY}}{100 + \%\ \text{MOISTURE}}$$

Mark W Foster

chen and associates, inc.



GRAVEL 34 % SAND 43 % SILT AND CLAY 23 %

LIQUID LIMIT % PLASTICITY INDEX %

SAMPLE OF Coal Refuge FROM Sample #2

HORROCKS ENGINEERS

Project Name: Price River Coal — Date: 2-13-83
Project No.: — Station or Pit Location:
Sample No: 2 (COAL) Refuse Pile — Requisted by: Lee Wimmer

AS RECEIVED GRADATION

Screen Size	Weight (g)	Percent Retained	Percent Passing	SPECS.
3"				
1 1/2"	134.8	6.7	93.3	
1"	172.6	8.6	84.7	
3/4"	64.0	3.2	81.5	
1/2"	161.6	8.0	73.5	
3/8"	78.7	3.9	69.6	
#4	241.5	12.0	57.6	
Wt. 4				
Dry Wt. - # 4				
Total Wt. Dry				

WASHED GRADATION AFTER CRUSHING (2500 GM. DRY SAMPLE)

Screen Size	Weight Retained	Percent Retained	Percent Passing	Total % Passing	SPECS.
#8					
#10					
#16	469.4	23.4	34.2		
#20					
#30	179.0	8.9	25.3		
#40					
#50	120.5	6.0	19.3		
[illegible]	116.0	5.8	13.5		
#200	36.3	1.8	11.7		
-#200	235.9	11.7	0.0		

Wt. before washing 2016.1
Wt. after washing 1779.9

MOISTURE DETERMINATION

	-# 4
Container & Wet Soil Weight (gm.)	103.11
Container & Dry Soil Weight (gm.)	96.05
H_2O Loss	5.26
% Moisture	7.49

A.A.S.H.O Classification

A-2-6 (4)

HORROCKS ENGINEERS

? Name Price River Coal Date 2-13-83

Project No. Station or Pit Location

Sample No 2 (Coal) Refuse Pile Requisted by Lee Wimmer

AS RECEIVED GRADATION

Screen Size	Weight (g)	Percent Retained	Percent Passing	SPECS.
3"				
1½"	134.8	6.7	93.3	
1"	172.6	8.6	84.7	
3/4"	64.0	3.2	81.5	
1/2"	161.6	8.0	73.5	
3/8"	78.7	3.9	69.6	
#4	241.5	12.0	57.6	
Wt 4				
Dry Wt -#4				
Total Wt Dry				

WASHED GRADATION AFTER CRUSHING
(2500 GM DRY SAMPLE)

Screen Size	Weight Retained	Percent Retained	Percent Passing	Total % Passing	SPECS.
#8					
#10					
#16	469.4	23.4	34.2		
#20					
#30	179.0	8.9	25.3		
#40					
50	120.5	6.0	19.3		
0	116.0	5.8	13.5		
#200	36.3	1.8	11.7		
-#200	235.9	11.7	0.0		
Total Wt	2010.3	166.0			

MOISTURE DETERMINATION

	-#4
Container & Wet Soil Weight (gm.)	103.11
Container & Dry Soil Weight (gm.)	98.05
H_2O Loss	5.06
% Moisture	7.49

A.A.S.H.O Classification

A-2-6 (4)

Wt. before washing 2016.1

Wt. after washing 1777.9

-23- [illegible] COPIES TO:

HORROCKS ENGINEERS

3/16/83

SLOPE STABILITY ANALYSIS

for

PRICE RIVER COAL - SLOPE STABILITY

DATA FILE: "PRSLOP"

PROJECT NUMBER: 8301-20e

by: BMP

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1280.0	6322.0	100.0	98.39

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	16.5	2.9	800	1.9	5.0	31	1285.1
2	44.8	4.0	800	1.9	13.7	31	1287.0
3	66.9	5.1	800	1.9	20.5	31	1288.8
4	82.8	6.2	800	1.9	25.4	31	1290.7
5	92.5	7.3	800	1.9	28.4	31	1292.6
6	95.9	8.4	800	1.9	29.4	31	1294.5
7	93.0	9.5	800	1.9	28.5	31	1296.4
8	83.8	10.6	800	1.9	25.7	31	1298.3
9	68.0	11.7	800	1.9	20.9	31	1300.2
10	45.8	12.8	800	1.9	14.0	31	1302.1
11	16.9	13.9	800	1.9	5.2	31	1304.0

ITERATION	INITIAL	CALCULATED
1	1.0000	98.5289
2	98.5289	98.3824
3	98.3824	98.3901
4	98.3901	98.3901

FACTOR OF SAFETY= 98.39 AT X= 1280 Y= 6322 R= 100
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1300.0	6320.0	100.0	10.99

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	494.3	-8.9	800	5.9	151.6	31	1284.5
2	1342.5	-5.5	800	5.9	411.7	31	1290.4
3	2006.8	-2.2	800	5.9	615.4	31	1296.2
4	2489.4	1.2	800	5.9	763.4	31	1302.1
5	2790.5	4.6	800	5.9	855.7	31	1308.0
6	2908.5	7.9	800	5.9	891.9	31	1313.8
7	2839.8	11.4	800	5.9	870.9	31	1319.7
8	2578.6	14.8	800	5.9	790.8	31	1325.5
9	2116.7	18.3	800	5.9	649.1	31	1331.4
10	1442.8	21.9	800	5.9	442.5	31	1337.3
11	541.7	25.6	800	5.9	166.1	31	1343.1

ITERATION	INITIAL	CALCULATED
1	1.0000	10.2658
2	10.2658	10.9826
3	10.9826	10.9888
4	10.9888	10.9889

FACTOR OF SAFETY= 10.99 AT X= 1300 Y= 6320 R= 100
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1415.0	6340.0	100.0	22.90

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	152.8	-3.2	800	4.0	46.8	31	1409.4
2	415.6	-1.0	800	4.0	127.4	31	1413.3
3	621.6	1.3	800	4.0	190.6	31	1417.3
4	770.8	3.6	800	4.0	236.4	31	1421.3
5	862.9	5.9	800	4.0	264.6	31	1425.3
6	897.3	8.2	800	4.0	275.2	31	1429.3
7	873.2	10.5	800	4.0	267.8	31	1433.2
8	789.4	12.8	800	4.0	242.1	31	1437.2
9	644.3	15.2	800	4.0	197.6	31	1441.2
10	436.1	17.6	800	4.0	133.7	31	1445.2
11	162.3	20.0	800	4.0	49.8	31	1449.2

ITERATION	INITIAL	CALCULATED
1	1.0000	21.1985
2	21.1985	22.8931
3	22.8931	22.9003
4	22.9003	22.9003

FACTOR OF SAFETY= 22.90 AT X= 1415 Y= 6340 R= 100
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1286.0	6372.0	150.0	28.42

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	133.4	-.1	800	4.4	40.9	31	1285.8
2	362.9	1.6	800	4.4	111.3	31	1290.2
3	542.7	3.3	800	4.4	166.4	31	1294.6
4	672.6	4.9	800	4.4	206.3	31	1298.9
5	752.2	6.6	800	4.4	230.7	31	1303.3
6	781.1	8.3	800	4.4	239.5	31	1307.6
7	758.7	10.0	800	4.4	232.7	31	1312.0
8	684.4	11.7	800	4.4	209.9	31	1316.4
9	557.1	13.4	800	4.4	170.8	31	1320.7
10	375.8	15.1	800	4.4	115.3	31	1325.1
11	139.4	16.8	800	4.4	42.7	31	1329.4

ITERATION	INITIAL	CALCULATED
1	1.0000	26.2362
2	26.2362	28.4098
3	28.4098	28.4172
4	28.4172	28.4172

FACTOR OF SAFETY= 28.42 AT X= 1286 Y= 6372 R= 150
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	Ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1367.0	6350.0	150.0	2.60

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	Ø	X
1	9915.2	-28.1	800	17.4	3040.7	31	1296.5
2	26369.4	-20.8	800	17.4	8086.6	31	1313.9
3	38905.4	-13.8	800	17.4	11931.0	31	1331.4
4	47950.4	-7.0	800	17.4	14704.8	31	1348.8
5	53726.9	-.3	800	17.4	16476.3	31	1366.3
6	56307.5	6.4	800	17.4	17267.6	31	1383.7
7	55630.7	13.2	800	17.4	17060.1	31	1401.2
8	51488.2	20.2	800	17.4	15789.7	31	1418.6
9	43472.9	27.5	800	17.4	13331.7	31	1436.1
10	30857.9	35.3	800	17.4	9463.1	31	1453.5
11	12294.6	44.0	800	17.4	3770.3	31	1470.9

ITERATION	INITIAL	CALCULATED
1	1.0000	2.5673
2	2.5673	2.5976
3	2.5976	2.5982

FACTOR OF SAFETY= 2.60 AT X= 1367 Y= 6350 R= 150
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1372.0	6382.0	150.0	12.59

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	478.8	-4.6	800	6.7	146.8	31	1359.9
2	1302.2	-2.1	800	6.7	399.4	31	1366.6
3	1947.8	.5	800	6.7	597.3	31	1373.3
4	2415.7	3.0	800	6.7	740.8	31	1379.9
5	2705.4	5.6	800	6.7	829.7	31	1386.6
6	2815.0	8.1	800	6.7	863.3	31	1393.2
7	2741.6	10.7	800	6.7	840.7	31	1399.9
8	2480.9	13.3	800	6.7	760.0	31	1406.6
9	2027.6	16.0	800	6.7	621.8	31	1413.2
10	1374.4	18.6	800	6.7	421.5	31	1419.9
11	512.6	21.3	800	6.7	157.2	31	1426.6

ITERATION	INITIAL	CALCULATED
1	1.0000	11.7019
2	11.7019	12.5822
3	12.5822	12.5888
4	12.5888	12.5889

FACTOR OF SAFETY= 12.59 AT X= 1372 Y= 6382 R= 150
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	Ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1330.0	6412.0	200.0	3.78

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	Ø	X
1	4446.4	-14.4	800	15.2	1363.6	31	1280.3
2	12036.6	-9.9	800	15.2	3691.2	31	1295.5
3	17957.9	-5.5	800	15.2	5507.1	31	1310.8
4	22262.1	-1.1	800	15.2	6827.1	31	1326.0
5	24970.0	3.2	800	15.2	7657.7	31	1341.2
6	26077.4	7.6	800	15.2	7997.1	31	1356.5
7	25546.2	12.0	800	15.2	7834.2	31	1371.7
8	23309.8	16.6	800	15.2	7148.3	31	1387.0
9	19261.6	21.2	800	15.2	5906.9	31	1402.2
10	13244.6	25.9	800	15.2	4061.7	31	1417.4
11	5030.3	30.9	800	15.2	1542.6	31	1432.7

ITERATION	INITIAL	CALCULATED
1	1.0000	3.6088
2	3.6088	3.7789
3	3.7789	3.7827

FACTOR OF SAFETY= 3.78 AT X= 1330 Y= 6412 R= 200
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1337.0	6428.0	200.0	14.32

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	447.9	-2.1	800	7.2	137.4	31	1329.7
2	1218.6	-.0	800	7.2	373.7	31	1336.9
3	1822.6	2.0	800	7.2	558.9	31	1344.1
4	2259.7	4.1	800	7.2	693.0	31	1351.3
5	2528.9	6.2	800	7.2	775.5	31	1358.5
6	2628.5	8.2	800	7.2	806.1	31	1365.6
7	2556.1	10.3	800	7.2	783.9	31	1372.8
8	2308.0	12.4	800	7.2	708.0	31	1380.0
9	1882.6	14.5	800	7.2	577.3	31	1387.2
10	1272.6	16.7	800	7.2	390.3	31	1394.4
11	473.0	18.8	800	7.2	145.1	31	1401.5

ITERATION	INITIAL	CALCULATED
1	1.0000	13.2787
2	13.2787	14.3157
3	14.3157	14.3226
4	14.3226	14.3227

FACTOR OF SAFETY= 14.32 AT X= 1337 Y= 6428 R= 200
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1272.0	6472.0	250.0	55.12

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	62.6	3.7	800	4.0	19.2	31	1288.3
2	170.2	4.7	800	4.0	52.2	31	1292.3
3	254.3	5.6	800	4.0	78.0	31	1296.3
4	314.7	6.5	800	4.0	96.5	31	1300.4
5	351.5	7.4	800	4.0	107.8	31	1304.4
6	364.3	8.4	800	4.0	111.7	31	1308.4
7	353.2	9.3	800	4.0	108.3	31	1312.4
8	317.8	10.2	800	4.0	97.4	31	1316.4
9	257.9	11.2	800	4.0	79.1	31	1320.4
10	173.5	12.1	800	4.0	53.2	31	1324.4
11	64.1	13.1	800	4.0	19.7	31	1328.5

ITERATION	INITIAL	CALCULATED
1	1.0000	50.7365
2	50.7365	55.1092
3	55.1092	55.1169
4	55.1169	55.1169

FACTOR OF SAFETY= 55.12 AT X= 1272 Y= 6472 R= 250
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	Ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1304.0	6470.0	250.0	7.91

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	Ø	X
1	1307.1	-4.5	800	11.0	400.8	31	1284.2
2	3555.0	-2.0	800	11.0	1090.2	31	1295.3
3	5317.3	.5	800	11.0	1630.6	31	1306.3
4	6594.8	3.1	800	11.0	2022.4	31	1317.3
5	7385.4	5.6	800	11.0	2264.9	31	1328.4
6	7684.3	8.1	800	11.0	2356.5	31	1339.4
7	7483.5	10.7	800	11.0	2294.9	31	1350.5
8	6771.7	13.3	800	11.0	2076.6	31	1361.5
9	5533.8	15.9	800	11.0	1697.0	31	1372.5
10	3750.8	18.6	800	11.0	1150.3	31	1383.6
11	1398.6	21.3	800	11.0	428.9	31	1394.6

ITERATION	INITIAL	CALCULATED
1	1.0000	7.3828
2	7.3828	7.9035
3	7.9035	7.9097
4	7.9097	7.9098

FACTOR OF SAFETY= 7.91 AT X= 1304 Y= 6470 R= 250
EARTHQUAKE= .10

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	Ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1280.0	6322.0	100.0	165.20

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	Ø	X
1	16.5	2.9	800	1.9	5.0	31	1285.1
2	44.8	4.0	800	1.9	13.7	31	1287.0
3	66.9	5.1	800	1.9	20.5	31	1288.8
4	82.8	6.2	800	1.9	25.4	31	1290.7
5	92.5	7.3	800	1.9	28.4	31	1292.6
6	95.9	8.4	800	1.9	29.4	31	1294.5
7	93.0	9.5	800	1.9	28.5	31	1296.4
8	83.8	10.6	800	1.9	25.7	31	1298.3
9	68.0	11.7	800	1.9	20.9	31	1300.2
10	45.8	12.8	800	1.9	14.0	31	1302.1
11	16.9	13.9	800	1.9	5.2	31	1304.0

ITERATION	INITIAL	CALCULATED
1	1.0000	151.9435
2	151.9435	165.1985
3	165.1985	165.1982
4	165.1982	165.1983

FACTOR OF SAFETY= 165.20 AT X= 1280 Y= 6322 R= 100
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1300.0	6320.0	100.0	18.65

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	494.3	-8.9	800	5.9	151.6	31	1284.5
2	1342.5	-5.5	800	5.9	411.7	31	1290.4
3	2006.8	-2.2	800	5.9	615.4	31	1296.2
4	2489.4	1.2	800	5.9	763.4	31	1302.1
5	2790.5	4.6	800	5.9	855.7	31	1308.0
6	2908.5	7.9	800	5.9	891.9	31	1313.8
7	2839.8	11.4	800	5.9	870.9	31	1319.7
8	2578.6	14.8	800	5.9	790.8	31	1325.5
9	2116.7	18.3	800	5.9	649.1	31	1331.4
10	1442.8	21.9	800	5.9	442.5	31	1337.3
11	541.7	25.6	800	5.9	166.1	31	1343.1

ITERATION	INITIAL	CALCULATED
1	1.0000	17.3680
2	17.3680	18.6469
3	18.6469	18.6536
4	18.6536	18.6536

FACTOR OF SAFETY= 18.65 AT X= 1300 Y= 6320 R= 100
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1415.0	6340.0	100.0	38.62

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	152.8	-3.2	800	4.0	46.8	31	1409.4
2	415.6	-1.0	800	4.0	127.4	31	1413.3
3	621.6	1.3	800	4.0	190.6	31	1417.3
4	770.8	3.6	800	4.0	236.4	31	1421.3
5	862.9	5.9	800	4.0	264.6	31	1425.3
6	897.3	8.2	800	4.0	275.2	31	1429.3
7	873.2	10.5	800	4.0	267.8	31	1433.2
8	789.4	12.8	800	4.0	242.1	31	1437.2
9	644.3	15.2	800	4.0	197.6	31	1441.2
10	436.1	17.6	800	4.0	133.7	31	1445.2
11	162.3	20.0	800	4.0	49.8	31	1449.2

ITERATION	INITIAL	CALCULATED
1	1.0000	35.6891
2	35.6891	38.6083
3	38.6083	38.6157
4	38.6157	38.6157

FACTOR OF SAFETY= 38.62 AT X= 1415 Y= 6340 R= 100
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1286.0	6372.0	150.0	47.81

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF. WEIGHT	ø	X
1	133.4	-.1	800	4.4	40.9	31	1285.8
2	362.9	1.6	800	4.4	111.3	31	1290.2
3	542.7	3.3	800	4.4	166.4	31	1294.6
4	672.6	4.9	800	4.4	206.3	31	1298.9
5	752.2	6.6	800	4.4	230.7	31	1303.3
6	781.1	8.3	800	4.4	239.5	31	1307.6
7	758.7	10.0	800	4.4	232.7	31	1312.0
8	684.4	11.7	800	4.4	209.9	31	1316.4
9	557.1	13.4	800	4.4	170.8	31	1320.7
10	375.8	15.1	800	4.4	115.3	31	1325.1
11	139.4	16.8	800	4.4	42.7	31	1329.4

ITERATION	INITIAL	CALCULATED
1	1.0000	44.0878
2	44.0878	47.8061
3	47.8061	47.8136
4	47.8136	47.8136

FACTOR OF SAFETY= 47.81 AT X= 1286 Y= 6372 R= 150
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1367.0	6350.0	150.0	4.60

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	9915.2	-28.1	800	17.4	3040.7	31	1296.5
2	26369.4	-20.8	800	17.4	8086.6	31	1313.9
3	38905.4	-13.8	800	17.4	11931.0	31	1331.4
4	47950.4	-7.0	800	17.4	14704.8	31	1348.8
5	53726.9	-.3	800	17.4	16476.3	31	1366.3
6	56307.5	6.4	800	17.4	17267.6	31	1383.7
7	55630.7	13.2	800	17.4	17060.1	31	1401.2
8	51488.2	20.2	800	17.4	15789.7	31	1418.6
9	43472.9	27.5	800	17.4	13331.7	31	1436.1
10	30857.9	35.3	800	17.4	9463.1	31	1453.5
11	12294.6	44.0	800	17.4	3770.3	31	1470.9

ITERATION	INITIAL	CALCULATED
1	1.0000	4.4918
2	4.4918	4.5940
3	4.5940	4.5957

FACTOR OF SAFETY= 4.60 AT X= 1367 Y= 6350 R= 150
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1330.0	6412.0	200.0	6.50

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	4446.4	-14.4	800	15.2	1363.6	31	1280.3
2	12036.6	-9.9	800	15.2	3691.2	31	1295.5
3	17957.9	-5.5	800	15.2	5507.1	31	1310.8
4	22262.1	-1.1	800	15.2	6827.1	31	1326.0
5	24970.8	3.2	800	15.2	7657.7	31	1341.2
6	26077.4	7.6	800	15.2	7997.1	31	1356.5
7	25546.2	12.0	800	15.2	7834.2	31	1371.7
8	23309.8	16.6	800	15.2	7148.3	31	1387.0
9	19261.6	21.2	800	15.2	5906.9	31	1402.2
10	13244.6	25.9	800	15.2	4061.7	31	1417.4
11	5030.3	30.9	800	15.2	1542.6	31	1432.7

ITERATION	INITIAL	CALCULATED
1	1.0000	6.1468
2	6.1468	6.4977
3	6.4977	6.5026

FACTOR OF SAFETY= 6.50 AT X= 1330 Y= 6412 R= 200
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1337.0	6428.0	200.0	24.16

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	447.9	-2.1	800	7.2	137.4	31	1329.7
2	1218.6	-.0	800	7.2	373.7	31	1336.9
3	1822.6	2.0	800	7.2	558.9	31	1344.1
4	2259.7	4.1	800	7.2	693.0	31	1351.3
5	2528.9	6.2	800	7.2	775.5	31	1358.5
6	2628.5	8.2	800	7.2	806.1	31	1365.6
7	2556.1	10.3	800	7.2	783.9	31	1372.8
8	2308.8	12.4	800	7.2	708.8	31	1380.0
9	1882.6	14.5	800	7.2	577.3	31	1387.2
10	1272.6	16.7	800	7.2	390.3	31	1394.4
11	473.0	18.8	800	7.2	145.1	31	1401.5

ITERATION	INITIAL	CALCULATED
1	1.0000	22.3388
2	22.3388	24.1488
3	24.1488	24.1561
4	24.1561	24.1561

FACTOR OF SAFETY= 24.16 AT X= 1337 Y= 6428 R= 200
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1272.0	6472.0	250.0	92.55

SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	62.6	3.7	800	4.0	19.2	31	1288.3
2	170.2	4.7	800	4.0	52.2	31	1292.3
3	254.3	5.6	800	4.0	78.0	31	1296.3
4	314.7	6.5	800	4.0	96.5	31	1300.4
5	351.5	7.4	800	4.0	107.8	31	1304.4
6	364.3	8.4	800	4.0	111.7	31	1308.4
7	353.2	9.3	800	4.0	108.3	31	1312.4
8	317.8	10.2	800	4.0	97.4	31	1316.4
9	257.9	11.2	800	4.0	79.1	31	1320.4
10	173.5	12.1	800	4.0	53.2	31	1324.4
11	64.1	13.1	800	4.0	19.7	31	1328.5

ITERATION	INITIAL	CALCULATED
1	1.0000	85.1348
2	85.1348	92.5376
3	92.5376	92.5454
4	92.5454	92.5454

FACTOR OF SAFETY= 92.55 AT X= 1272 Y= 6472 R= 250
EARTHQUAKE= 0.00

WATER UNIT WEIGHT= 62.40

POINT	X-ORD	Y-ORD
1	0.00	6220.00
2	1270.00	6220.00
3	1520.00	6340.00
4	1570.00	6340.00
5	1640.00	6380.00
6	1720.00	6380.00
7	1790.00	6420.00
8	1860.00	6420.00
9	1970.00	6480.00
10	2040.00	6480.00
11	2150.00	6540.00
12	2210.00	6540.00
13	2225.00	6550.00
14	3050.00	6550.00
15	3100.00	6560.00
16	2150.00	6350.00
17	3100.00	6350.00

LINE	LEFT	RIGHT	SOIL
1	13	14	1
2	1	2	2
3	2	16	2
4	16	17	2

SOIL	UNIT WEIGHT	COHESION	ø	SATURATED
1	90	800	31	NO
2	90	800	31	YES

CIRCLE	X-ORD	Y-ORD	RADIUS	FACTOR OF SAFETY
	1304.0	6470.0	250.0	13.39

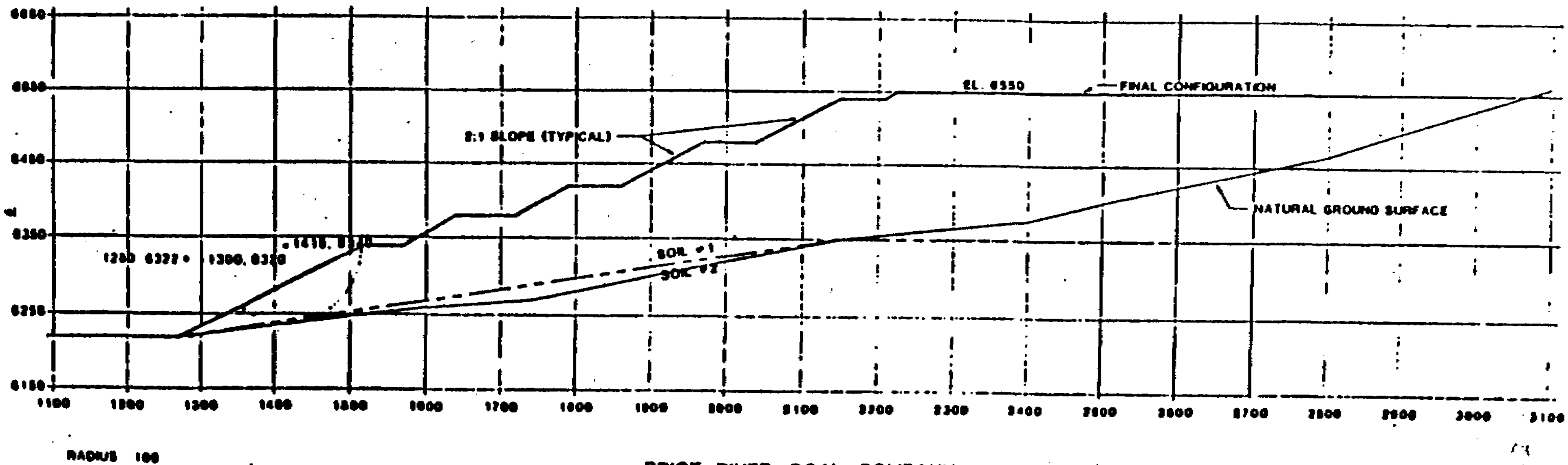
SLICE	WEIGHT	INCLINATION	COHESION	WIDTH	EFF WEIGHT	ø	X
1	1307.1	-4.5	800	11.0	400.8	31	1284.2
2	3555.0	-2.0	800	11.0	1090.2	31	1295.3
3	5317.3	.5	800	11.0	1630.6	31	1306.3
4	6594.8	3.1	800	11.0	2022.4	31	1317.3
5	7385.4	5.6	800	11.0	2264.9	31	1328.4
6	7684.3	8.1	800	11.0	2356.5	31	1339.4
7	7483.5	10.7	800	11.0	2294.9	31	1350.5
8	6771.7	13.3	800	11.0	2076.6	31	1361.5
9	5533.8	15.9	800	11.0	1697.0	31	1372.5
10	3750.8	18.6	800	11.0	1150.3	31	1383.6
11	1390.6	21.3	800	11.0	428.9	31	1394.6

ITERATION	INITIAL	CALCULATED
1	1.0000	12.4399
2	12.4399	13.3835
3	13.3835	13.3903
4	13.3903	13.3903

FACTOR OF SAFETY= 13.39 AT X= 1304 Y= 6470 R= 250
EARTHQUAKE= 0.00

SCHOOLHOUSE CANYON REFUSE FACILITY

LONGITUDINAL SECTION THROUGH DUMP



3.4C-xliii

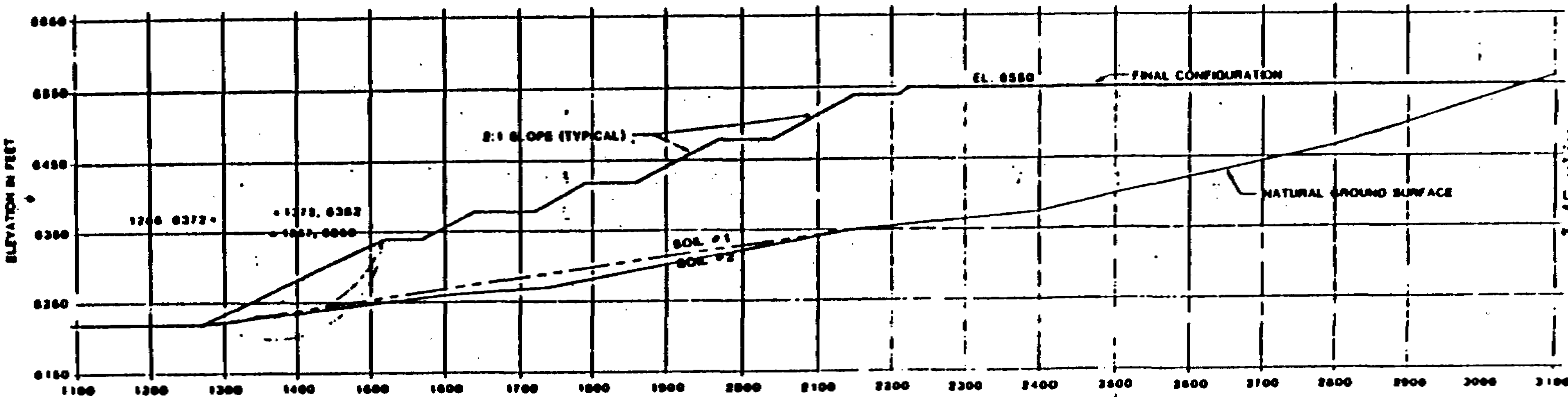
PRICE RIVER COAL COMPANY

SCALE 1"=100'

HORROCKS ENGINEERS

SCHOOLHOUSE CANYON REFUSE FACILITY

LONGITUDINAL SECTION THROUGH DUMP



RADIUS 150'

PRICE RIVER COAL COMPANY

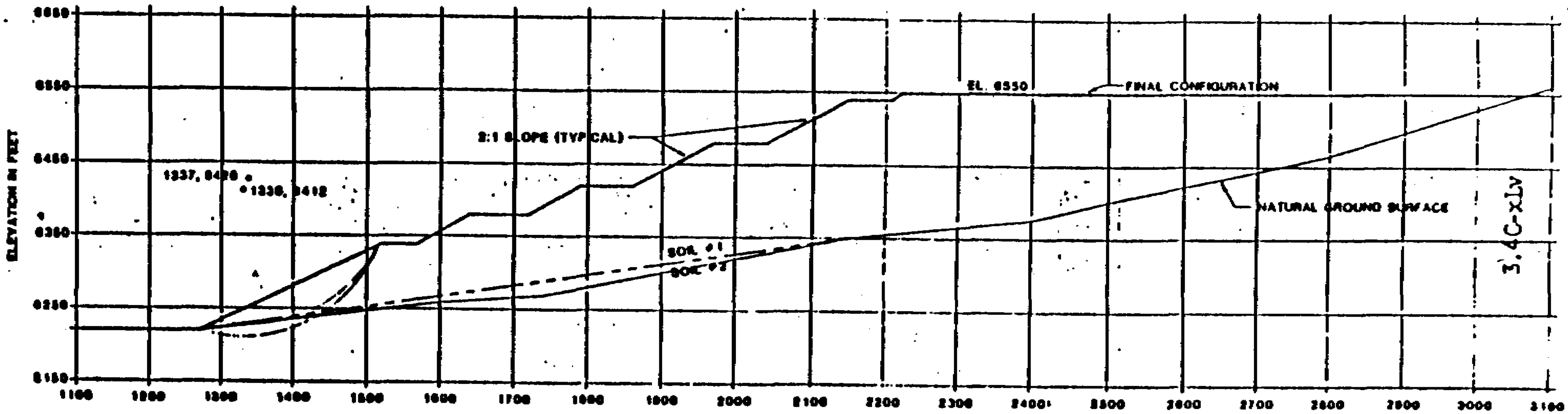
SCALE 1"=100'

HORROCKS ENGINEERS

3.4C-xliv

SCHOOLHOUSE CANYON REFUSE FACILITY

LONGITUDINAL SECTION THROUGH DUMP



RADIUS : 200'

PRICE RIVER COAL COMPANY

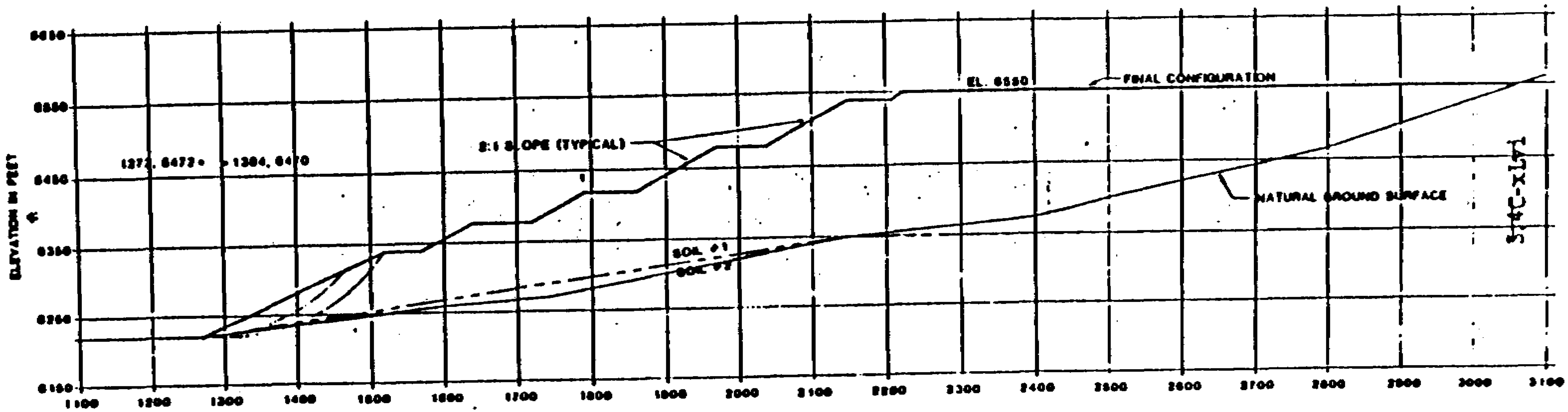
SCALE 1":100'

HORROCKS ENGINEERS

3.4C-xlv

SCHOOLHOUSE CANYON REFUSE FACILITY

LONGITUDINAL SECTION THROUGH DUMP



RADIUS 250'

PRICE RIVER COAL COMPANY

SCALE 1"100'

HORROCKS ENGINEERS

DATA FILE 'PRSLOP'

```
5000  DATA PRICE RIVER COAL - SLOPE STABILITY ! FILE "PRSLOP"
5010  DATA 0,62.4,.1
5040  DATA 17,0,6220,1270,6220,1520,6340,1570,6340,1640,6380,1720,6380,1790,6420
5050  DATA 1860,6420,1970,6480,2040,6480,2150,6540,2210,6540,2225,6550,3050,6550
5060  DATA 3100,6560,2150,6350,3100,6350
5070  DATA 4,13,14,1,1,2,2,2,16,2,16,17,2
5080  DATA 2,90,800,31,1,90,800,31,0
```

"SLOPE"

A slope stability program utilizing the simplified or "modified Bishop" method.

The program was written by John P. Cross, P.E., Processing Manager of STS Consultants, Northbrook, Illinois. This program was printed in the October 1982 issue of "CIVIL ENGINEERING."

This version was copied from "CIVIL ENGINEERING" and edited for the Hewlett-Packard 9845 desk-top computer by Horrocks Engineers in March 1983. The format for the input and the output was changed from the original version, however, the program itself was not changed.

HORROCKS ENGINEERS
ONE WEST MAIN STREET
AMERICAN FORK, UTAH 83003
TELEPHONE (801)756-7628

```
42     OPTION BASE 1
44     OVERLAP
46     PRINTER IS 16
48     PRINT "SLOPE STABILITY ANALYSIS"
50     DIM P(50,2),L(50,3),S2(5,4),A(50),F(50,7),Z(50,4),H$[80],Sbits$(0:1)[3]
52     INTEGER Logo(2)
54     Sbits$(1)=" NO"
56     Sbits$(0)="YES"
58     S9=10
60     J6=0
62     OUTPUT 5;"R"
64     ENTER 5;M,D,Time$
66     Date$=VAL$(M)&"/"&VAL$(D)&"/83"
68     PRINTER IS 0
70     PRINT "",LIN(4),TAB(80-LEN(Date$));Date$,LIN(0);
72     GOSUB Logo   ! PRINT HORROCKS ENGINEERS' LOGO
74     PRINT LIN(5),TAB(25),"SLOPE STABILITY ANALYSIS",LIN(2),TAB(38),"for",LIN(2
);
```

*** INPUT OF PROGRAM VARIABLES ***

```
78     INPUT "ENTER THE DATA FILE NAME",File$
79     INPUT "ENTER THE PROJECT NUMBER",Pn$
82     INPUT "ENTER THE USER'S INITIALS",User$
136    LINK File$,5000
190    READ H$
191    PRINT TAB(40-LEN(H$)/2);H$
195    PRINT LIN(30),TAB(30),"DATA FILE: "&CHR$(34)&File$&CHR$(34),LIN(1)
200    PRINT TAB(25),"PROJECT NUMBER: "&Pn$,LIN(1)
201    PRINT TAB(37),"by: "&User$
203    PRINTER IS 16
210    READ S0
211    IF S0=0 THEN 270
230    READ S6
250    READ S7
270    READ W0
290    READ E1
310    READ P1
311    PRINTER IS 16
315    PRINT "POINT      X-ORD        Y-ORD"
320    FOR I=1 TO P1
331      PRINT SPA(2),I;
```

```
332     IMAGE 3X,2(7D.3D,2X)
350     READ P(I,1),P(I,2)
371     PRINT USING 332;P(I,1),P(I,2)
380   NEXT I
400   READ L1
401   PRINT LIN(1),"LINE    FROM      TO    SOIL BENEATH"
402   IMAGE 3X,2(4D,3X),2X,2D
410   FOR I=1 TO L1
421     PRINT I;
440     READ L(I,1),L(I,2),L(I,3)
480     PRINT USING 402;L(I,1),L(I,2),L(I,3)
490   NEXT I
510   READ S1
511   PRINT LIN(1),"SOIL  UNIT WEIGHT  COHESION    "&CHR$(218)&"   SATURATION"
512   IMAGE 3X,4D.DD,2X,9D,3X,3D,3X,3A
520   FOR I=1 TO S1
531     PRINT I;
550     READ S2(I,1),S2(I,2),S2(I,3),S2(I,4)
610     PRINT USING 512;S2(I,1),S2(I,2),S2(I,3),Sbit$(S2(I,4))
620   NEXT I
```

```
*** CIRCLE DEFINITION  ***

640   F9=0
641   PRINTER IS 16
650   PRINT "CIRCLE DEFINITION"
660   INPUT "ENTER THE X-ORD, Y-ORD, AND RADIUS OF THE FAIL SURFACE  FORMAT X,Y,
R ",X,Y,R
```

```
*** CHECK TO SEE IF CIRCLE EXCEEDS TOP LINE END POINTS ***

730   U1=P1
740   FOR I=2 TO P1
750     IF (P(I,1)<P(I-1,1)) AND (U1=P1) THEN 770
760     GOTO 780
770     U1=I-1
780   NEXT I
790   J1=R*R-(P(1,2)-Y)^2
800   J2=R*R-(P(U1,2)-Y)^2
810   IF J1<=0 THEN 830
820   IF (J1>0) AND (P(1,1)>X-SQR(J1)) THEN 860
830   IF J2<=0 THEN 850
840   IF (J2>0) AND (P(U1,1)<X+SQR(J2)) THEN 860
850   GOTO 880
860   DISP "CIRCLE EXCEEDS TOP LINE END POINTS";
870   GOTO 4380
```

```
*** DEFINE INTERSECTION OF CIRCLE WITH LINES ***

890   FOR I=1 TO L1
900     X1=P(L(I,1),1)
910     Y1=P(L(I,1),2)
920     X2=P(L(I,2),1)
930     Y2=P(L(I,2),2)
940     IF X2=X1 THEN 960
950     GOTO 970
960     S=9.99E10
970     IF X2<>X1 THEN 990
980     GOTO 1000
990     S=(Y2-Y1)/(X2-X1)
1000    IF ABS(S)<1.0E-5 THEN 1150
```

```
1010    C1=X1-Y1/S
1020    C2=1/S^2+1
1030    C3=2*C1/S-2*X/S-2*Y
1040    C4=C1^2-2*X*C1+X^2+Y^2-R^2
1050    C5=C3^2-4*C2*C4
1060    IF C5<0 THEN 1080
1070    GOTO 1090
1080    Z(I,1)=0
1090    IF C5<0 THEN 1630
1100    Q1=(-C3+SQR(C5))/(2*C2)
1110    Q2=(-C3-SQR(C5))/(2*C2)
1120    Q3=Q1/S+C1
1130    Q4=Q2/S+C1
1140    GOTO 1240
1150    C5=R^2-(Y-Y1)^2
1160    IF C5<0 THEN 1180
1170    GOTO 1190
1180    Z(I,1)=0
1190    IF C5<0 THEN 1630
1200    Q3=X+SQR(C5)
1210    Q4=X-SQR(C5)
1220    Q1=Y1
1230    Q2=Y1
1240    J1=0
1250    J2=0
1260    IF (ABS(S)<=9.99E9) AND (Q3>=X1) AND (Q3<=X2) THEN 1280
1270    GOTO 1290
1280    J1=1
1290    IF (ABS(S)<=9.99E9) AND (Q4>=X1) AND (Q4<=X2) THEN 1310
1300    GOTO 1320
1310    J2=1
1320    IF (S<-9.99E9) AND (Q1>=Y2) AND (Q1<=Y1) THEN 1340
1330    GOTO 1350
1340    J1=1
1350    IF (S<-9.99E9) AND (Q2>=Y2) AND (Q2<=Y1) THEN 1370
1360    GOTO 1380
1370    J2=1
1380    IF (S>9.99E9) AND (Q1>=Y1) AND (Q1<=Y2) THEN 1400
1390    GOTO 1410
1400    J1=1
1410    IF (S>9.99E9) AND (Q2>=Y1) AND (Q2<=Y2) THEN 1430
1420    GOTO 1440
1430    J2=1
1440    Z(I,1)=J1+J2
1450    IF J1=1 THEN 1470
1460    GOTO 1480
1470    Z(I,2)=Q3
1480    IF J1=1 THEN 1500
1490    GOTO 1510
1500    Z(I,3)=Q1
1510    IF (J1=0) AND (J2=1) THEN 1530
1520    GOTO 1540
1530    Z(I,2)=Q4
1540    IF (J1=0) AND (J2=1) THEN 1560
1550    GOTO 1570
1560    Z(I,3)=Q2
1570    IF (J1=1) AND (J2=1) THEN 1590
1580    GOTO 1600
1590    Z(I,4)=Q4
1600    IF (J1=1) AND (J2=1) THEN 1620
1610    GOTO 1630
1620    Z(I,3)=Q2
1630  NEXT I
1640  X4=0
1650  X5=9.99E20
1660  I1=1
1670  FOR I=1 TO L1
```

```
1680    IF Z(I,1)>=1 THEN 1700
1690    GOTO 1710
1700    A(I1)=Z(I,2)
1710    IF Z(I,1)>=1 THEN 1730
1720    GOTO 1740
1730    I1=I1+1
1740    IF Z(I,1)=2 THEN 176
1750    GOTO 1770
1760    A(I1)=Z(I,4)
1770    IF Z(I,1)=2 THEN 1790
1780    GOTO 1800
1790    I1=I1+1
1800  NEXT I
1810  IF I1=1 THEN 1830
1820  GOTO 1840
1830  PRINT "CIRCLE DOES NOT INTERSECT SLOPE"
1840  IF I1=1 THEN 4380
```

*** SET UP SLICE ARRAY ***

```
1860  FOR I=1 TO I1-1
1870    IF A(I)>X4 THEN 1890
1880    GOTO 1900
1890    X4=A(I)
1900    IF A(I)<X5 THEN 1920
1910    GOTO 1930
1920    X5=A(I)
1930  NEXT I
1940  FOR I=1 TO P1
1950    IF (P(I,1)<X4) AND (P(I,1)>X5) THEN 1970
1960    GOTO 1980
1970    A(I1)=P(I,1)
1980    IF (P(I,1)<X4) AND (P(I,1)>X5) THEN 2000
1990    GOTO 2010
2000    I1=I1+1
2010  NEXT I
2020  I1=I1-1
2030  FOR I=1 TO I1
2040    FOR J=1 TO I1-1
2050      IF A(J+1)>A(J) THEN 2090
2060      J1=A(J+1)
2070      A(J+1)=A(J)
2080      A(J)=J1
2090    NEXT J
2100  NEXT I
2110  U1=0
2120  FOR I=1 TO I1-1
2130    IF A(I)<A(I+1) THEN 2150
2140    GOTO 2160
2150    U1=U1+1
2160    IF A(I)<A(I+1) THEN 2180
2170    GOTO 2190
2180    A(U1)=A(I)
2190  NEXT I
2200  U1=U1+1
2210  A(U1)=A(I1)
2220  I1=U1
```

*** DEFINE SLICE BOUNDARIES ***

```
2240  Q1=A(I1)-A(1)
2250  Q2=Q1/S9
2260  U1=I1
```

```
2270  FOR I=1 TO U1-1
2280    Q3=A(I+1)-A(I)
2290    Q4=INT(Q3/Q2)+1
2300    C1=Q3/Q4
2310    C2=A(I)
2320    FOR J=1 TO Q4
2330      IF J<Q4 THEN 2350
2340      GOTO 2360
2350      I1=I1+1
2360      IF J<Q4 THEN 2380
2370      GOTO 2390
2380      A(I1)=C2+C1
2390      IF J<Q4 THEN 2410
2400      GOTO 2420
2410      C2=C2+C1
2420    NEXT J
2430  NEXT I
2440  FOR I=1 TO I1
2450    FOR J=1 TO I1-1
2460      IF A(J+1)>A(J) THEN 2500
2470      J1=A(J+1)
2480      A(J+1)=A(J)
2490      A(J)=J1
2500    NEXT J
2510  NEXT I
```

*** DEFINE SOIL PARAMETERS FOR EACH SLICE ***

```
2530  F1=I1-1
2540  FOR I=1 TO F1
2550    F(I,4)=A(I+1)-A(I)
2560    X6=F(I,4)
2570    F(I,7)=(A(I+1)+A(I))/2
2580    X3=F(I,7)
2590    Y1=Y-SQR(R^2-(A(I)-X)^2)
2600    Y2=Y-SQR(R^2-(A(I+1)-X)^2)
2610    A5=ATN(ABS(Y2-Y1)/F(I,4))
2620    IF Y2<Y1 THEN 2640
2630    GOTO 2650
2640    A5=-A5
2650    F(I,2)=A5
2660    IF A5=0 THEN 2680
2670    GOTO 2690
2680    F(I,2)=1.0E-5
2690    Y3=Y-SQR(R^2-(X3-X)^2)
2700    I4=0
2710    FOR J=1 TO L1
2720      L5=L(J,1)
2730      L6=L(J,2)
2740      IF (P(L5,2)<=Y3) AND (P(L6,2)<=Y3) THEN 2840
2750      IF (P(L5,1)<X3) AND (P(L6,1)<X3) THEN 2840
2760      IF (P(L5,1)>X3) AND (P(L6,1)>X3) THEN 2840
2770      Y6=P(L5,2)+(P(L5,2)-P(L6,2))/(P(L5,1)-P(L6,1))*(X3-P(L5,1))
2780      IF Y6<=Y3 THEN 2840
2790      I4=I4+1
2800      Z(I4,1)=Y6
2810      Z(I4,2)=L(J,3)
2820      W=0
2830      E=0
2840    NEXT J
2850    IF I4=1 THEN 2970
2860    FOR J=1 TO I4
2870      FOR J1=1 TO I4-1
2880        IF Z(J1,1)>=Z(J1+1,1) THEN 2950
2890        L5=Z(J1,1)
```

```
2900          L6=Z(J1,2)
2910          Z(J1,1)=Z(J1+1,1)
2920          Z(J1,2)=Z(J1+1,2)
2930          Z(J1+1,1)=L5
2940          Z(J1+1,2)=L6
2950        NEXT J1
2960      NEXT J
2970      I4=I4+1
2980      Z(I4,1)=Y3
2990      FOR J1=1 TO I4-1
3000        IF (I=1) AND (J1=1) AND (X3>=S6) THEN 3020
3010        GOTO 3030
3020        I6=S0-Y1
3030        IF (I=F1) AND (J1=1) AND (X3>=S6) AND (X3<=S7) THEN 3050
3040        GOTO 3060
3050        J6=S0-Y2
3060        W=W+(Z(J1,1)-Z(J1+1,1))*X6*S2(Z(J1,2),1)
3070        IF (Z(J1,1)<S0) AND (X3>=S6) AND (X3<=S7) THEN 3090
3080        GOTO 3100
3090        W=W+(S0-Z(J1,1))*X6*W0
3100        IF S2(Z(J1,2),4)>.95 THEN 3120
3110        GOTO 3130
3120        E4=S2(Z(J1,2),1)
3130        IF S2(Z(J1,2),4)<.95 THEN 3150
3140        GOTO 3160
3150        E4=S2(Z(J1,2),1)-W0
3160        E=E+(Z(J1,1)-Z(J1+1,1))*X6*E4
3170      NEXT J1
3180      F(I,1)=W
3190      F(I,5)=E
3200      F(I,3)=S2(Z(I4-1,2),2)
3210      F(I,6)=2*PI*(S2(Z(I4-1,2),3)/360)
3220    NEXT I
3221    NORMAL
3230    IF F9=0 THEN 3360
3240    PRINT USING 3250;CHR$(210)
3250    IMAGE "SLICE   WEIGHT   INCLINATION   COHESION   WIDTH    EFF WEIGHT    "A
"         X"
3280    O=360/(2*PI)
3290    FOR I=1 TO F1
3300      PRINT USING 3320;I,F(I,1),F(I,2)*O,F(I,3),F(I,4),F(I,5),F(I,6)*O,F(I,7)
3320      IMAGE 3D,10D.D,7D.D,12D,9D.D,11D.D,7D,7D.D
3340    NEXT I
3350    PRINT
3360    D=0
3361    PRINTER IS 0
3370    FOR I=1 TO F1
3380      D=D+F(I,1)*SIN(ABS(F(I,2)))*(F(I,2)/ABS(F(I,2)))
3390      D=D+E1*F(I,1)*COS(ABS(F(I,2)))
3400    NEXT I
3410    IF I6>0 THEN 3430
3420    GOTO 3440
3430    I7=W0*I6*I6*(R-I6/3)/(2*R)
3440    IF I6>0 THEN 3460
3450    GOTO 3470
3460    D=D-SGN(D)*I7
3470    IF (I6>0) AND (F9=1) THEN 3490
3480    GOTO 3510
3490    PRINT USING 3500;I7
3500    IMAGE "DRIVING FORCE COUNTER BALANCE OF",10D.2D
3510    IF J6>0 THEN 3530
3520    GOTO 3540
3530    I7=W0*J6*J6*(R-J6/3)/(2*R)
3540    IF J6>0 THEN 3560
3550    GOTO 3570
3560    D=D+SGN(D)*I7
3570    IF (J6>0) AND (F9=1) THEN 3590
```

```
3580  GOTO 3610
3590  PRINT USING 3600;I7
3600  IMAGE "DRIVING FORCE INCREASE OF",10D.2D


*** ITERATIVE SOLUTION FOR FACTOR OF SAFETY ***

3620  F0=1
3630  R4=0
3640  I6=0
3650  FOR I=1 TO F1
3660    R1=F(I,3)*F(I,4)+F(I,5)*TAN(F(I,6))
3670    R2=1/COS(ABS(F(I,2)))
3680    R3=1+TAN(F(I,6))*TAN(F(I,2))/F0
3690    R4=R4+R1*(R2/R3)
3700  NEXT I
3710  F2=R4/D
3720  I6=I6+1
3730  IF F9=1 THEN 3750
3740  GOTO 3820
3750  IF I6=1 THEN 3770
3760  GOTO 3800
3770  PRINT
3780  PRINT USING 3790
3790  IMAGE "ITERATION",11X,"INITIAL",10X,"CALCULATED"
3800  PRINT USING 3810;I6,F0,F2
3810  IMAGE 3X,3D,13X,3D.4D,12X,3D.4D
3820  IF I6>10 THEN 3840
3830  GOTO 3850
3840  PRINT "WILL NOT CLOSE"
3850  IF I6>10 THEN 3970
3860  IF ABS(ABS(F0)-ABS(F2))<.005 THEN 3900
3870  F0=ABS(F2)
3880  R4=0
3890  GOTO 3650
3900 !
3901  IF NOT F9 THEN
3902    PRINTER IS 16
3903  ELSE
3904    PRINTER IS 0
3905  END IF
3910  PRINT
3920  PRINT USING 3930;F2,X,Y,R
3930  IMAGE "FACTOR OF SAFETY= ",5D.2D,"  AT X= ",4D," Y= ",4D, " R= ",4D
3940  PRINT USING 3950;E1
3950  IMAGE "EARTHQUAKE= ",2D.2D
3951  IF F9 THEN 4380
3960  PRINT
3961  A$=""
3970  INPUT "DO YOU WISH A FORMAL PRINTOUT (Y/N)",A$
3990  IF UPC$(A$[1,1])="N" THEN 4320
3991  PRINTER IS 0
4030  IMAGE @"WATER UNIT WEIGHT=",3D.2D
4040  PRINT USING 4030;W0
4041  IF S0 THEN
4050    PRINT
4060    IMAGE "SUBMERGENCE AT "3D.2D,"  FROM  ",3D.1D,"  TO  ",3D.1D
4070    PRINT USING 4060;S0,S6,S7
4071  END IF
4080  PRINT
4090  PRINT " POINT    X-ORD      Y-ORD"
4100  IMAGE 4D,7D.2D,7D.2D
4110  FOR I=1 TO P1
4120    PRINT USING 4100;I,P(I,1),P(I,2)
4130  NEXT I
4140  PRINT
```

3.4C-Liv

```
[illegible]
4160  IMAGE 4(5D)
4170  FOR I=1 TO L1
4180    PRINT USING 4160;I,L(I,1),L(I,2),L(I,3)
4190  NEXT I
4200  PRINT
4210  PRINT "SOIL        UNIT WEIGHT        COHESION      "&CHR$(210)&"      SATURA
ED"
4220  IMAGE 3D,15D,17D,9D,7X,3A
4230  FOR I=1 TO S1
4240    PRINT USING 4220;I,S2(I,1),S2(I,2),S2(I,3),Sbit$(S2(I,4))
4250  NEXT I
4260  PRINT
4270  PRINT "CIRCLE   X-ORD    Y-ORD   RADIUS  FACTOR OF SAFETY"
4280  IMAGE 12D.D,7D.D,7D.D,8D.2D
4290  PRINT USING 4280;X,Y,R,F2
4300  PRINT
4310  PRINT
4311  A$=""
4320  INPUT "DO YOU WISH A DIAGNOSTIC RUN (Y/N)",A$
4340  IF UPC$(A$[1,1])="N" THEN 4370
4360  F9=1
4370  IF UPC$(A$[1,1])<>"N" THEN 720
4371  A$=""
4380  INPUT "DO YOU WANT TO CONTINUE (Y/N)",A$
4400  IF UPC$(A$[1,1])<>"N" THEN 630
4401  DISP " FINISHED "
4410  STOP
4520 Logo:PLOTTER IS 13,"GRAPHICS"
4530  GRAPHICS
4540  SCALE 0,559,0,454
4550  LORG 2
4560  FOR I=0 TO 5
4570    Logo(1)=-2175
4580    Logo(2)=-4352
4590    R=454-I
4600    GLOAD Logo(*),0,R
4610  NEXT I
4620  FOR I=6 TO 14
4630    Logo(1)=-2115
4640    Logo(2)=-4352
4650    R=454-I
4660    GLOAD Logo(*),0,R
4670  NEXT I
4680  FOR I=15 TO 21
4690    Logo(1)=-2175
4700    Logo(2)=-4352
4710    R=454-I
4720    GLOAD Logo(*),0,R
4730  NEXT I
4740  CSIZE 15/4.54,9/15
4750  MOVE 27,450
4760  LABEL "HORROCKS"
4770  MOVE 27,437
4780  CSIZE 15/4.54,8/15
4790  LABEL "ENGINEERS"
4800  DUMP GRAPHICS 430,454
4810  GCLEAR
4820  EXIT GRAPHICS
4830  RETURN
```

Slope stability program

JOHN P. CROSS, P.E., M. ASCE
Data Processing Manager,
Project Engineer
STS Consultants
Northbrook, Illinois

FOR NATURAL or man-made slopes, the index of stability with respect to a sudden failure is known as the safety factor of the slope. The safety factor may be defined as the ratio of the potential resisting forces to the drive forces tending to cause movement. A slope on the verge of failure would have a safety factor of 1.0. The analysis of slope stability is, therefore, the analytical procedure of determining the most critical, i.e., the lowest, factor of safety of given or proposed slope.

Manual methods of slope stability analysis were developed prior to the advent of the electronic computer. These approaches resulted in high analysis costs and conservative slope configurations. Repetitive calculations lended themselves to computerized methods and numerous programs exist that have been written for large computer systems to perform slope stability analysis according to a number of theoretical methods.

The simplified or modified Bishop method is reasonably accurate for most purposes where the slope under analysis can be assumed to fail along a circular failure surface. The factor of safety is defined as the ratio of the resisting moments to driving moments around the center of the failure arc. Initially, a cross-section of the slope is drawn detailing soil strata and piezometric surfaces. A center point is then chosen from which an arc is taken through the cross-section. This arc represents the failure surface under evaluation. This failure zone is broken down into a series of slices which can be individually evaluated for their weight and strength characteristics. An illustration of a slope cross-section being defined by a series of slices is shown in Figure 1.

The forces acting on each slice are illustrated in Figure 1, where ΔX is the width of the slice, W is the weight of the slice, T is the force acting along the failure surface at the bottom of the slice, $\bar{N}$ is the effective force acting normally to the base of the slice and θ is the inclination of the failure surface or slice base. The factor of safety is defined as:

$$F = \frac{\Sigma \dfrac{(C \Delta X + \bar{N} \tan \Phi) \sec \theta}{1 + \dfrac{\tan \Phi \tan \theta}{F}}}{\Sigma W \sin \theta}$$

Where C is the cohesion, Φ is the friction angle and the summation occurs over each slice of the failure zone. As the factor of safety, F, occurs on both sides of the equation. An interactive solution where F is initially estimated and then back substituted until the calculated F and estimated F close within a specified tolerance.

The equation can be modified to handle two additional conditions by adding additional factors to the term defining the driving force. These two conditions are standing pools, i.e., submergence of a portion of the slope, and earthquake loading. For submergence, the weight of water acting above the slice is added to the weight of the slice itself. The total driving force is increased or decreased by the weight of water above or below the exit of the failure surface from the slope. The second condition of earthquake loading can be handled by increasing the driving force calculated for each slice by $EW\cos\theta$, where E is the earthquake loading factor. Similarly the resisting force is decreased by a decrease in the normal force due to the earthquake loading.

Following the calculation of the safety factor for this arc, the center or radius of the arc is modified to generate a new failure surface. The previously mentioned procedure is again followed with a new factor of safety being deter-

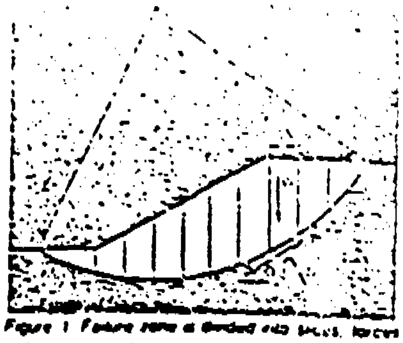


Figure 1 Failure zone is divided into slices, forces acting upon a slice are indicated

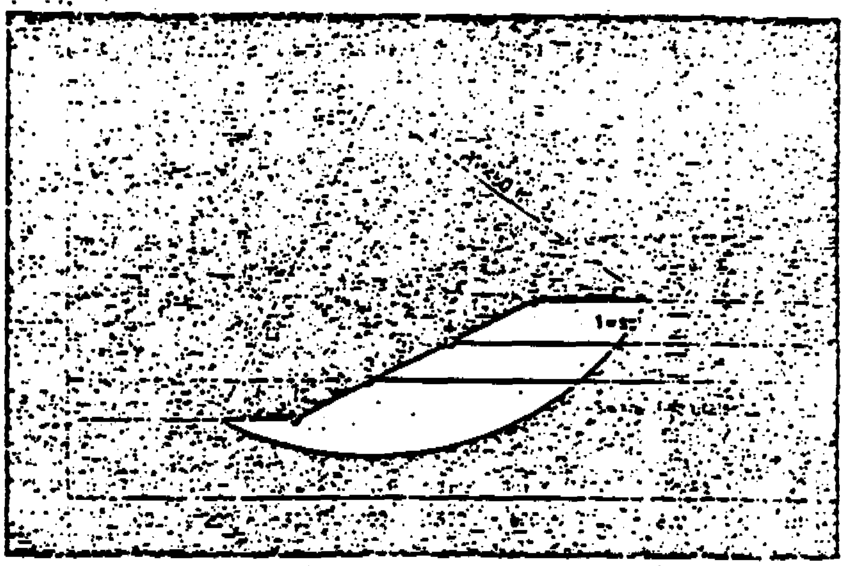


Figure 2. shows a typical cross-section and the input parameters required to define the cross-section for the program

Table for Figure 2.

Point	X	Y	Line	Left	Right	Soil	Soil	γ	Cohesion	Phi	Saturated
1	0	100	1	1	2	3	1	127	2000	20	No
2	300	100	2	2	3	3	2	130	1000	33	No
3	400	150	3	3	4	2	3	130	1000	33	Yes
4	500	200	4	4	5	1					
5	800	250	5	5	6	1					
6	1000	250	6	4	7	2					
7	1000	200	7	3	8	3					
8	1000	150									

0360-0556/82/0010-0071/$01.00

mined. This entire sequence is repeated until the failure surface for the minimum factor of safety is determined.

The program included in this article follows the same general procedure as previously defined. The program can be broken down into nine segments. Lines 100-620 are input routines for the entry of data defining the cross-section, lines 630-710 define the circle that will generate the failure surface, lines 720-860 perform a verification that the failure arc falls fully within the cross-section and lines 880-1540 define the intersection points between the line segments and the failure arc. The slice array is set up between line 1850 and 2220, with slice boundaries defined in lines 2230-2510. Lines 2520-3600 include the definition of the soil parameters for each slice and the actual iterative solution for the factor of safety occurs between lines 3610 and 3950. The remainder of the program is the formal output of the results.

The program includes a diagnostic print-out where all the slice parameters can be displayed for any given failure surface. As currently configured the program can handle models including

APPENDIX 3.4D

DISTURBED AND UNDISTURBED AREA RUNOFF CALCULATIONS

REGISTERED PROFESSIONAL ENGINEER
NO. 11350
WILLIAM S. HENDRICKSON
STATE OF UTAH

William S. Hendrickson
2/15/94

Prep Plant Area

Runoff Calculations - OPERATION PHASE

Assumptions:

- Design Storm = 10-year, 6 hour
  precip = 1.4 inches
  SCS type b distribution
- Curve numbers based on data contained in chapter 9 and site visits (professional judgement)

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Castle Gate Prep Plant
Watershed Areas

Watershed	Area (Acres)	Area (mi^2)
CGWS - U1	7.05	0.0110
U2	2.38	0.004
U3a	1027.09	1.605
U3b	172.86	0.270
U4	6.78	0.0106
U5	SEE APPENDIX	3.4J
U6	52.11	0.0814
U7	5.96	0.0093
U8	20.59	0.0322
CGWS - D1 (a+b)	12.60	0.0197
-D2G	2.26	0.0035
-D3 (A+B)	14.48	0.0226
-D4 (A+B)	14.73	0.0230
-D5	1.56	0.0024

SEE APPENDIX 3.4J FOR CALCULATIONS FOR SCHOOL HOUSE CANYON (REFUSE PILE)

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Castle Gate Prep Plant

Calculate Mean Watershed Slope:

$$\% \text{ Slope} = \left[.25(EM-EN)(LC_{25}+LC_{50}+LC_{75})/A \right] * 100$$

EM = Max. Elevation (ft.)
EN = Min. Elevation (ft.)
LC_{25} = Contour length at 25% of EM−EN (ft.)
A = Area of watershed (ft^2)

Watershed	EM	EN	LC_{25}	LC_{50}	LC_{75}	A(ft^2)	% Slope
CGWS-U1	7050	6150	500	200	50	307,100	54.9
U2	6440	6160	350	240	140	103,670	49.3
U3a	8455	6240	13000	16600	13250	4.47×10^7	53.0
U3b	8170	6550	3500	3750	2400	7.5×10^6	51.9
U4	6525	6180	600	600	520	2.95×10^5	50.2
U5	SEE	APPENDIX	3.4 J				
U6	7625	6136	1400	1100	1300	2.27×10^6	62.3
U7	6635	6136	400	200	100	2.60×10^5	33.6
U8	7200	6100	760	760	420	8.97×10^5	59.6
CGWS-D1(a+b)	6230	6144	1160	630	120	5.49×10^5	7.48
D2a							
D2b							
D2c							
D2d	SEE APPENDIX 3.4 J						
D2e							
D2f		6190	500	400	200	1.47×10^5	31.8
D2g	6650	6190	150	160	120	9.84×10^4	50.2
D3(a+b)	6220	6110	920	2250	600	6.31×10^5	16.4
D4(a+b)	6504.5	6100	400	850	400	6.42×10^5	26.0
D5	6345	6136	620	240	150	6.80×10^4	77.2

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Castle Gate Prep Plant

Calculate Time of Concentration:

$T_c = 1.67\ L$ L = Lag time

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900\,Y^{0.5}} \text{ hrs.}$$

Y = watershed slope %
l = hydraulic length (ft.)
$S = \frac{1000}{CN} - 10$
CN = Curve Number (Determined by field obs. + professional judgement)

Watershed	l (ft.)	CN	S	Y (%)	L (hr)	T_c (hr)
CGWS-U1	1520	78	2.82	54.9	0.064	0.106
U2	780	78	2.82	49.3	0.039	0.066
U3a	14,300	75	3.33	53.0	0.426	0.710
U3b	6,600	75	3.33	51.9	0.232	0.386
U4	1,080	78	2.82	50.2	0.051	0.085
U5	SEE APPENDIX 3.4J					
U6	3200	78	2.82	62.3	0.109	0.181
U7	1340	82	2.20	33.6	0.065	0.108
U8	2200	78	2.82	59.5	0.082	0.137
CGWS-D1 (a+b)	1240	90	1.11	7.48	0.097	0.162
D2a	SEE APPENDIX 3.4J					
D2b	SEE APPENDIX 3.4J					
D2c	SEE APPENDIX 3.4J					
D2d	SEE APPENDIX 3.4J					
D2e	SEE APPENDIX 3.4J					
D2f	SEE APPENDIX 3.4J					
D2g	1200	82	2.20	50.2	0.049	0.081
D3 (a+b)	1720	90	1.11	16.4	0.085	0.142
D4 (a+b)	3920	90	1.11	26.0	0.130	0.217
D5	700	85	1.76	72.2	0.023	0.038

UC-190-19 5

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: CASTLE GATE MINE

Comment: FLOW FROM FACE OF REFUSE WITHIN CGWS-D2F

Solve For Depth

Given Input Data:

Bottom Width.....	25.00 ft
Left Side Slope..	3.00:1 (H:V)
Right Side Slope.	3.00:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.3300 ft/ft
Discharge........	0.72 cfs

Computed Results:

Depth............	0.02 ft
Velocity.........	1.80 fps
Flow Area........	0.40 sf
Flow Top Width...	25.10 ft
Wetted Perimeter.	25.10 ft
Critical Depth...	0.03 ft
Critical Slope...	0.0425 ft/ft
Froude Number....	2.51 (flow is Supercritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: CASTLE GATE MINE

Comment: FLOW FROM FACE OF REFUSE WITHIN CGWS-D2F

Solve For Depth

Given Input Data:

Bottom Width.....	35.00 ft
Left Side Slope..	3.00:1 (H:V)
Right Side Slope.	3.00:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.4500 ft/ft
Discharge........	0.72 cfs

Computed Results:

Depth............	0.01 ft
Velocity.........	1.73 fps
Flow Area........	0.42 sf
Flow Top Width...	35.07 ft
Wetted Perimeter.	35.08 ft
Critical Depth...	0.02 ft
Critical Slope...	0.0458 ft/ft
Froude Number....	2.80 (flow is Supercritical)

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-U3A

```
------------------------------------------------------------
   STORM :                          WATERSHED :
  Dist.=SCS Type 'b' - 6 Hr        Area =   1027.10   acres
  Depth =  1.40   inches             CN = 75.00
  Duration =  6.00  hrs            Time conc.=  0.710  hrs
------------------------------------------------------------
```

OUTPUT SUMMARY

```
------------------------------------------------------------
    Runoff depth      0.13224    inches
    Initial abstr     0.66667    inches
    Peak flow =       42.46     cfs    (  0.04100   iph )
       at time     3.692    hrs
------------------------------------------------------------
```

INPUT FOR: CGWS-U3B

```
------------------------------------------------------------
   STORM :                          WATERSHED :
  Dist.=SCS Type 'b' - 6 Hr        Area =    172.90   acres
  Depth =  1.40   inches             CN = 75.00
  Duration =  6.00  hrs            Time conc.=  0.386  hrs
------------------------------------------------------------
```

OUTPUT SUMMARY

```
------------------------------------------------------------
    Runoff depth      0.13224    inches
    Initial abstr     0.66667    inches
    Peak flow =         7.44    cfs    (  0.04267   iph )
       at time     3.603    hrs
------------------------------------------------------------
```

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-U4

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 6.78 acres
Depth = 1.40 inches	CN = 78.00
Duration = 6.00 hrs	Time conc.= 0.085 hrs

OUTPUT SUMMARY

Runoff depth	0.19110	inches	
Initial abstr	0.56410	inches	
Peak flow =	0.96	cfs	(0.14100 iph)
at time	2.527	hrs	

~~INPUT FOR: CGWS-U5~~

STORM :	WATERSHED :
~~Dist.=SCS Type 'b' - 6 Hr~~	~~Area = 7.03 acres~~
~~Depth = 1.40 inches~~	~~CN = 82.00~~
~~Duration = 6.00 hrs~~	~~Time conc.= 0.098 hrs~~

~~OUTPUT SUMMARY~~

~~Runoff depth~~	~~0.29260~~	~~inches~~	
~~Initial abstr~~	~~0.43902~~	~~inches~~	
~~Peak flow =~~	~~1.78~~	~~cfs~~	~~(0.25110 iph)~~
~~at time~~	~~2.522~~	~~hrs~~	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-U6

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 52.11 acres
Depth = 1.40 inches	CN = 78.00
Duration = 6.00 hrs	Time conc.= 0.181 hrs

OUTPUT SUMMARY

Runoff depth	0.19110	inches	
Initial abstr	0.56410	inches	
Peak flow =	5.83	cfs	(0.11093 iph)
at time	2.582	hrs	

INPUT FOR: CGWS-U7

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 5.96 acres
Depth = 1.40 inches	CN = 82.00
Duration = 6.00 hrs	Time conc.= 0.108 hrs

OUTPUT SUMMARY

Runoff depth	0.29260	inches	
Initial abstr	0.43902	inches	
Peak flow =	1.48	cfs	(0.24603 iph)
at time	2.534	hrs	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-U8

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 20.59 acres
Depth = 1.40 inches	CN = 78.00
Duration = 6.00 hrs	Time conc.= 0.137 hrs

OUTPUT SUMMARY

Runoff depth	0.19110	inches	
Initial abstr	0.56410	inches	
Peak flow =	2.57	cfs	(0.12398 iph)
at time	2.557	hrs	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-D1(A&B)

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 12.60 acres
Depth = 1.40 inches	CN = 90.00
Duration = 6.00 hrs	Time conc.= 0.162 hrs

OUTPUT SUMMARY

Runoff depth	0.60604	inches	
Initial abstr	0.22222	inches	
Peak flow =	6.83	cfs	(0.53720 iph)
at time	2.527	hrs	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-D2F

STORM :		WATERSHED :	
Dist.=SCS Type 'b' - 6 Hr		Area =	3.38 acres
Depth =	1.40 inches	CN =	85.00
Duration =	6.00 hrs	Time conc.=	0.046 hrs

OUTPUT SUMMARY

Runoff depth	0.38991	inches	
Initial abstr	0.35294	inches	
Peak flow =	1.29	cfs	(0.37768 iph)
at time	2.509	hrs	

INPUT FOR: CGWS-D2G

STORM :		WATERSHED :	
Dist.=SCS Type 'b' - 6 Hr		Area =	2.26 acres
Depth =	1.40 inches	CN =	82.00
Duration =	6.00 hrs	Time conc.=	0.081 hrs

OUTPUT SUMMARY

Runoff depth	0.29260	inches	
Initial abstr	0.43902	inches	
Peak flow =	0.59	cfs	(0.25838 iph)
at time	2.516	hrs	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-D3(A&B)

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 14.48 acres
Depth = 1.40 inches	CN = 90.00
Duration = 6.00 hrs	Time conc.= 0.142 hrs

OUTPUT SUMMARY

Runoff depth	0.60604	inches	
Initial abstr	0.22222	inches	
Peak flow =	7.98	cfs	(0.54665 iph)
at time	2.518	hrs	

INPUT FOR: CGWS-D4(A&B)

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 14.73 acres
Depth = 1.40 inches	CN = 90.00
Duration = 6.00 hrs	Time conc.= 0.217 hrs

OUTPUT SUMMARY

Runoff depth	0.60604	inches	
Initial abstr	0.22222	inches	
Peak flow =	7.59	cfs	(0.51109 iph)
at time	2.546	hrs	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-D5

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 1.56 acres
Depth = 1.40 inches	CN = 85.00
Duration = 6.00 hrs	Time conc.= 0.038 hrs

OUTPUT SUMMARY

Runoff depth	0.38991	inches	
Initial abstr	0.35294	inches	
Peak flow =	0.60	cfs	(0.38201 iph)
at time	2.503	hrs	

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CASTLE GATE SCHOOL HOUSE REFUSE AREA (GENERIC DITCH)

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 1.67 acres
Depth = 2.10 inches	CN = 90.00
Duration = 6.00 hrs	Time conc.= 0.027 hrs

OUTPUT SUMMARY

Runoff depth	1.17972	inches	
Initial abstr	0.22222	inches	
Peak flow =	1.90	cfs	(1.12712 iph)
at time	2.502	hrs	

APPENDIX 3.4E

OPERATION PHASE
DIVERSION DITCH CALCULATIONS

REGISTERED PROFESSIONAL ENGINEER
NO. 11350
WILLIAM S. HENDRICKSON
STATE OF UTAH

CASTLEGATE AREA — Ditch Calculations

Methodology

Use Manning's equation to determine:

① Ditch capacity and freeboard based on the minimum channel slope

② Maximum velocity based on the maximum channel slope

Assumptions:

① Manning's n = 0.035 for ditches w/ gravel or rock cut and minimal vegetation.

② Manning's n = 0.040 for ditches w/ significant riprap and/or vegetation

③ Manning's n = 0.030 for ditches w/ smooth earthen sides and bottom (no obstructions).

Data Base

① Runoff calculations from companion appendix based on SCS curve number technology

② Minimum ditch sections + riprap sizes calculated.

③ Ditch slopes measured from topo map and/or field verified.

Ditch Summary
Prep Plant Area

Ditch	Min. BW (ft.)	Min. SS (H:V)	Min. Depth (ft.)	Min. Slope (%)	Max Slope (%)	Max flow depth (ft.)	Min. freeboard (ft.)	Max. flow Velocity (ft/s)	Min. D_{50} (in.)[(a)]
CGD-1	2.0	1:1	0.6	2	10	0.22	0.38	3.29	1.0
2	1.0	1.5:1	0.5	7	10	0.12	0.38	2.76	none
3	10.0	.8:1	1.0	6	10	0.71	0.29	7.88	10.0
4	1.0	1:1	0.7	2	6.5	0.33	0.37	3.36	1.0
~~5~~	~~5.0~~	~~1:1~~	~~0.8~~	~~2~~	~~3~~	~~0.47~~	~~0.33~~	~~3.29~~	~~none~~
6(UPPER), 6(LOWER)	SEE APPENDIX 3.4J								
8	2.0	1:1	2.4	1	10	2.04	0.36	9.06	14.0
9	1.5	1:1	0.6	1	9.2	0.22	0.38	3.26	1.0
10	2.0	1.5:1	0.9	3	5	0.56	0.34	5.53	3.0
11	3.0	1:1	0.5	12.5	12.5	0.18	0.32	4.48	2.0
12	0	1.5:1	1.6	1.3	2.6	1.29	0.31	4.15	none
13	0	1.5:1	1.6	1.4	3.0	1.25	0.35	4.32	none
14	1.0	1.5:1	1.0	1.0	1.0	0.65	0.35	2.63	none
15	1.0	1.5:1	1.0	1.0	3.0	0.66	0.34	3.94	none
16	0	1.5:1	0.9	7.0	10.0	0.54	0.36	4.72	1.5
17	0	1.5:1	1.0	6.7	10	0.68	0.32	6.41	2.0
7(UPPER), 7(MIDDLE), 7(LOWER)	SEE APPENDIX 3.4J								
GENERIC ROAD SIDE DITCH	0	3:1	1.0	6.0	11.0	0.40	0.60	5.01	1.5

Note (a) Minimum riprap requirements for ditches constructed in soil. If ditches are constructed on bedrock, riprap is not required. If ditch is well vegetated, riprap is not required for velocities < 4 ft/sec. Refer to Appendix 3.4D.

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PROJECT UC-150 PAGE 2 OF

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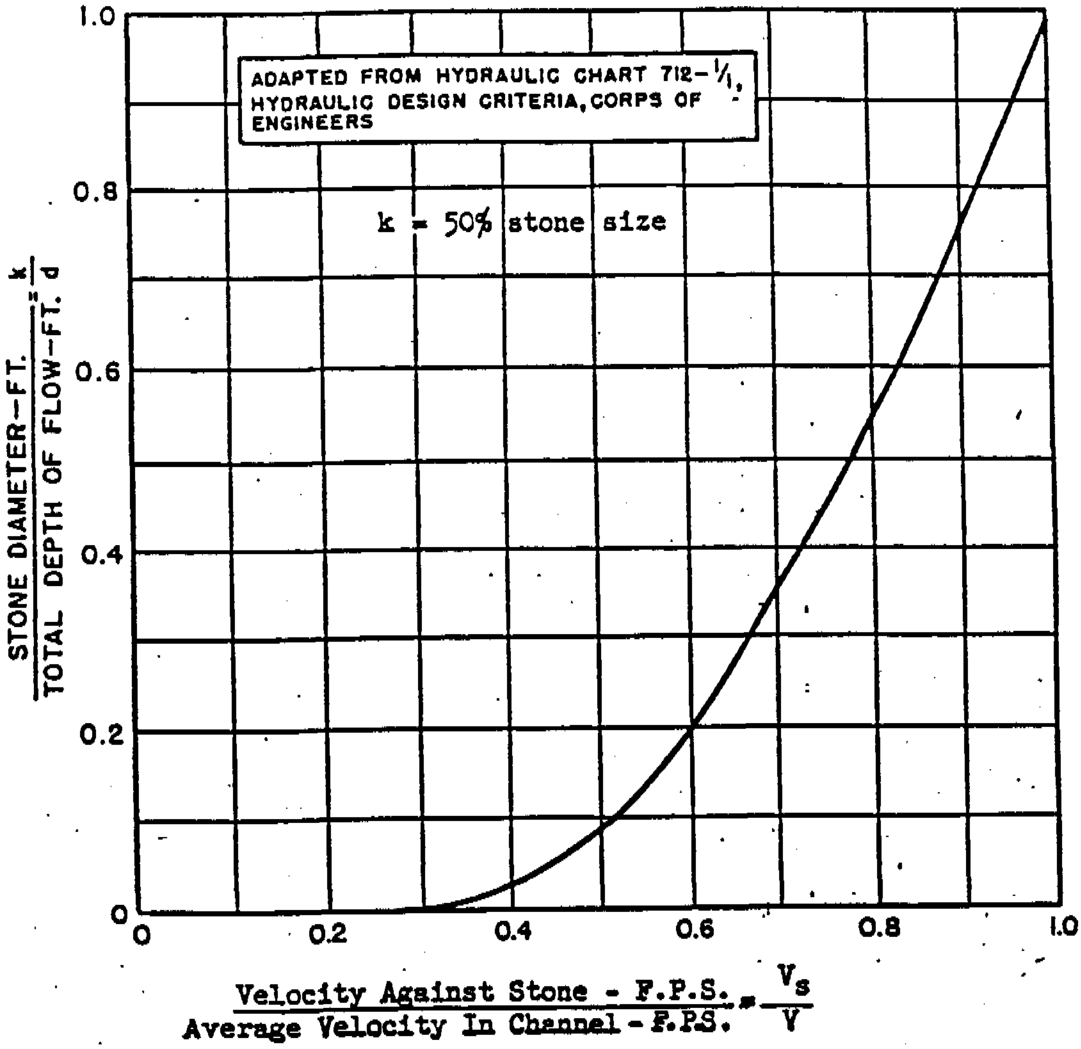
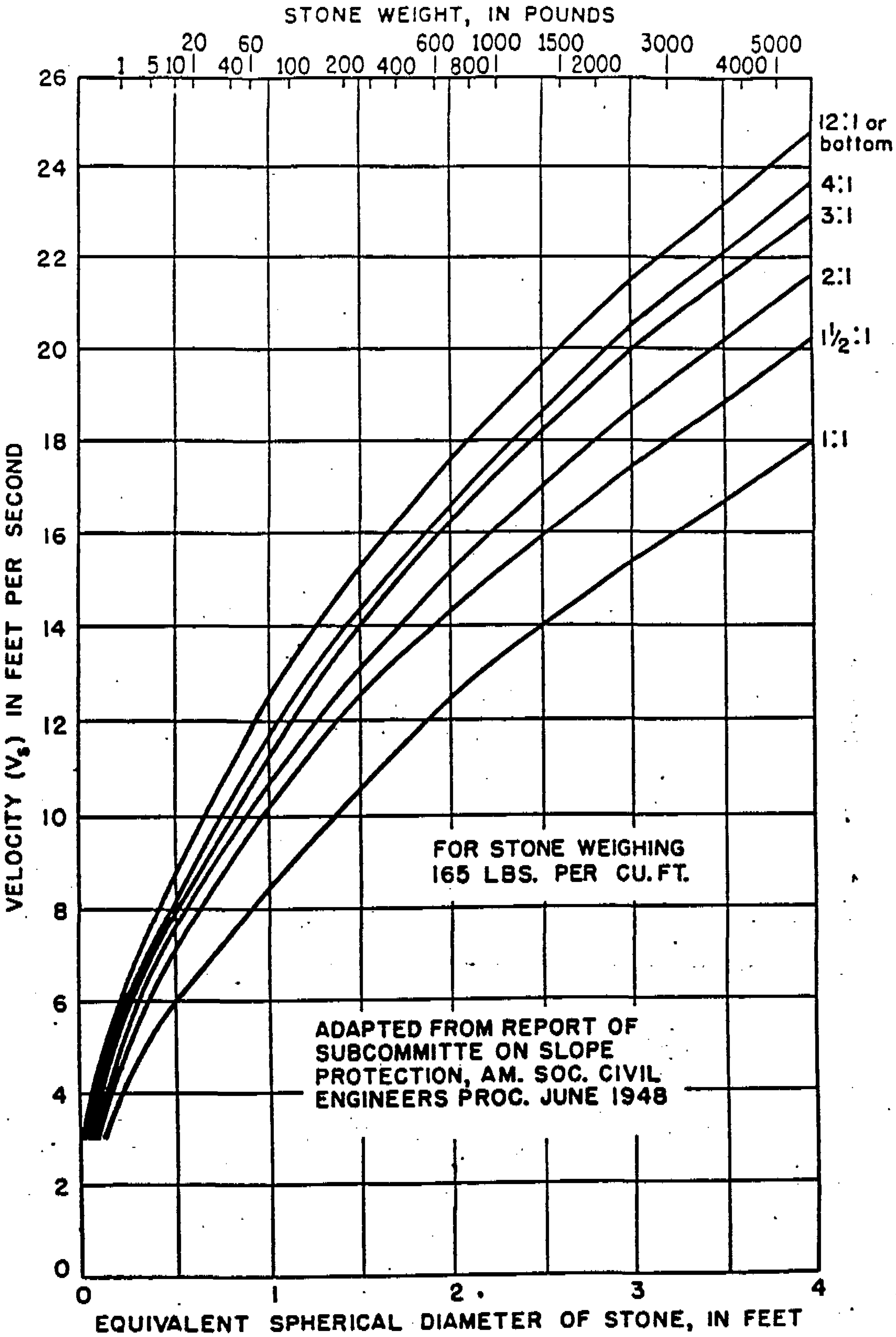


VELOCITY AGAINST STONE ON CHANNEL BOTTOM

FIGURE 1



SIZE OF STONE THAT WILL RESIST DISPLACEMENT FOR VARIOUS VELOCITIES AND SIDE SLOPES

FIGURE 2

Table 3.4 Permissible Velocities for Vegetated Channels.*

	Permissible Velocity, fps					
	Erosion Resistant Soils (% Slope)			Easily Eroded Soils (% Slope)		
Cover	0-5	5-10	Over 10	0-5	5-10	Over 10
Bermuda grass	8	7	6	6	5	4
Buffalo grass Kentucky bluegrass Smooth brome Blue grama Tall fescue	7	6	5	5	4	3
Lespedeza sericea Weeping lovegrass Kudzu Alfalfa Crabgrass	3.5	NR†	NR	2.5	NR	NR
Grass mixture	5	4	NR	4	3	NR
Annuals for temporary protection	3.5	NR	NR	2.5	NR	NR

* After Ree (1949).
† Not recommended.

Figure 3

Ref: Barfield, et. al. (1981)

Note: For Ditches that are well vegetated, riprap will not be required when flow velocities ≤ 4 ft/sec.

Diversion Ditch Calculations

CGD-1

Drainage Area = CGWS-U1
Peak Q = 0.95 cfs

Min. BW = 2.0'
Min. TW = 3.2'
Min. Depth = 0.6'
Side Slopes - 1H : 1V

Max slope = 10%
Min slope = 2%
$n = .035$

Min. Slope = 2%

Max flow depth = 0.22'

Min. channel depth = .22 + .30 = .52'
say 0.6'

Max. Slope = 10%

Max. Velocity = 3.29 ft/sec depth = .14'

try to size Riprap: assume $\frac{k}{d} = \frac{.10}{.14} = .71$

$V_s = .88 (3.29) = 2.9$ ft/s (fig. 1)

The need for riprap is marginal. Either $D_{50} \sim 1$ inch or a vegetated channel would be adequate

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-1 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	2.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0200 ft/ft — Min. slope
Discharge........	0.95 cfs

Computed Results:

Depth............	0.22 ft — Max flow depth
Velocity.........	1.95 fps
Flow Area........	0.49 sf
Flow Top Width...	2.44 ft
Wetted Perimeter.	2.62 ft
Critical Depth...	0.19 ft
Critical Slope...	0.0350 ft/ft
Froude Number....	0.77 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-1 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	2.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.1000 ft/ft — Max Slope
Discharge........	0.95 cfs

Computed Results:

Depth............	0.14 ft
Velocity.........	3.29 fps — Max Velocity
Flow Area........	0.29 sf
Flow Top Width...	2.27 ft
Wetted Perimeter.	2.38 ft
Critical Depth...	0.19 ft
Critical Slope...	0.0350 ft/ft
Froude Number....	1.62 (flow is Supercritical)

CGD-2

D.A. = 2.38 Acres (CGWS-U2)
Peak Q = 0.36 cfs

Min. BW = 1.0'
Min. TW = 2.5'
Min. Depth = 0.50'
Min SS = 1.5H : 1V

Max Slope = 10%
Min Slope = 7%
n = .035

Min. Slope = 7%

Max. flow depth = 0.12 ft

Min. channel depth = .12' + .3' = .42' ~ 0.50'

Max. Slope = 10%

Max. Velocity = 2.76 ft/s depth = .11'

With this low Velocity and shallow flow depth, no riprap is required. Ref. Barfield et al, (1981) and attached Figures 1+2.

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-2 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0700 ft/ft — Min Slope
Discharge........	0.36 cfs

Computed Results:

Depth............	0.12 ft — Max flow depth
Velocity.........	2.45 fps
Flow Area........	0.15 sf
Flow Top Width...	1.37 ft
Wetted Perimeter.	1.45 ft
Critical Depth...	0.15 ft
Critical Slope...	0.0387 ft/ft
Froude Number....	1.32 (flow is Supercritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-2 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.1000 ft/ft — Max Slope
Discharge........	0.36 cfs

Computed Results:

Depth............	0.11 ft
Velocity.........	2.76 fps — Max Velocity
Flow Area........	0.13 sf
Flow Top Width...	1.34 ft
Wetted Perimeter.	1.40 ft
Critical Depth...	0.15 ft
Critical Slope...	0.0387 ft/ft
Froude Number....	1.55 (flow is Supercritical)

CGD-3

D.A. = CGWS-3A and 3B (1199.95 Acres)
Peak Q = 42.46 + 7.44 = 49.90 cfs

Min. BW = 10'
Min. TW = 11.6'
Min. Depth = 1.0'
Min. SS = 0.8H : 1V

Min Slope = 6%
Max Slope = 10%
$n = .04$

Min. Slope = 6%

Max. Flow depth = 0.71'

Min. channel depth = 0.71 + 0.30 = 1.01' ~ 1.0'

Max. Slope = 10%

Max. Velocity = 7.88 ft/s

depth = 0.61 ft

Riprap: Assume $\frac{K}{d} \geq 1$ Side slopes 1:1

D_{50} (fig 2) = 0.85' = 10 inches

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-3 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	10.00 ft
Left Side Slope..	0.75:1 (H:V)
Right Side Slope.	0.75:1 (H:V)
Manning's n......	0.040
Channel Slope....	0.0600 ft/ft — Min slope
Discharge........	49.90 cfs

Computed Results:

Depth............	0.71 ft — Max flow depth
Velocity.........	6.70 fps
Flow Area........	7.44 sf
Flow Top Width...	11.06 ft
Wetted Perimeter.	11.77 ft
Critical Depth...	0.90 ft
Critical Slope...	0.0273 ft/ft
Froude Number....	1.44 (flow is Supercritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-3 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	10.00 ft
Left Side Slope..	0.75:1 (H:V)
Right Side Slope.	0.75:1 (H:V)
Manning's n......	0.040
Channel Slope....	0.1000 ft/ft — Max Slope
Discharge........	49.90 cfs

Computed Results:

Depth............	0.61 ft
Velocity.........	7.88 fps — Max Velocity
Flow Area........	6.33 sf
Flow Top Width...	10.91 ft
Wetted Perimeter.	11.51 ft
Critical Depth...	0.90 ft
Critical Slope...	0.0273 ft/ft
Froude Number....	1.82 (flow is Supercritical)

$\boxed{\text{CGD-4}}$

D.A. = CGWS-U4 (area = 6.78 Ac)
Peak Q = 0.96 cfs

Min. BW = 1.0'
Min. TW = 2.4'
Min. depth = 0.70'
Min. SS = 1H:1V

Min Slope = 2%
Max Slope = 6.5%
η = .035

Min Slope = 2%

Max flow depth = 0.33'

Min. channel depth =
0.33 + .30 = .63'
say 0.70'

Max Slope = 6.5%

Max. velocity = 3.36 ft/s

depth = .23'

Riprap : assume k = 2" = .15'

$\frac{k}{d} = \frac{.15}{.23} = .65$

$V_s = .85(3.36) = 2.86$ ft/s (fig 1)

The need for riprap is marginal.
Use D_{50} = 1" OR vegetated channel

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-4 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0200 ft/ft — Min slope
Discharge........	0.96 cfs

Computed Results:

Depth............	0.33 ft — Max depth
Velocity.........	2.22 fps
Flow Area........	0.43 sf
Flow Top Width...	1.65 ft
Wetted Perimeter.	1.92 ft
Critical Depth...	0.28 ft
Critical Slope...	0.0351 ft/ft
Froude Number....	0.76 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-4 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0650 ft/ft — Max slope
Discharge........	0.96 cfs

Computed Results:

Depth............	0.23 ft
Velocity.........	3.36 fps — Max Velocity
Flow Area........	0.29 sf
Flow Top Width...	1.46 ft
Wetted Perimeter.	1.66 ft
Critical Depth...	0.28 ft
Critical Slope...	0.0351 ft/ft
Froude Number....	1.34 (flow is Supercritical)

3.4.5

CGD - 5

D.A. = CGWS - U3b
Peak Q = 7.44 cfs

min. BW = 5.0'
Min. TW = 6.6'
min. depth = 0.80'
min. SS = 1H:1V

min Slope = 2%
Max Slope = 3%
n = .040

DELETED p14

Min. Slope = 2%

Max. flow depth = 0.47'

Min. channel depth = .47+.30 = .77'
say 0.80'

Max. Slope = 3%

Max. Velocity = 3.29 ft/s

depth = .42 ft

Riprap: Assume k=1" = .1'

$\frac{k}{d} = \frac{.1}{.42} = .024$

Vs = .63(3.29) = 2.1 ft/s (fig. 1)

From fig 2 ⇒ No Riprap Required.

3.4E

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-5 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	5.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.040
Channel Slope....	0.0200 ft/ft — Min. Slope
Discharge........	7.44 cfs

Computed Results:

Depth............	0.47 ft — Max depth
Velocity.........	2.89 fps
Flow Area........	2.58 sf
Flow Top Width...	5.94 ft
Wetted Perimeter.	6.33 ft
Critical Depth...	0.40 ft
Critical Slope...	0.0350 ft/ft
Froude Number....	0.77 (flow is Subcritical)

DELETED PA4

Open Channel Flow Module, Version 3.2 (c) 1990
Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708

34E

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-5 max. slope

Solve For Depth

Given Input Data:

Bottom Width....	5.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.040
Channel Slope....	0.0300 ft/ft — Max slope
Discharge........	7.44 cfs

Computed Results:

Depth............	0.42 ft
Velocity.........	3.29 fps — Max velocity
Flow Area........	2.26 sf
Flow Top Width...	5.83 ft
Wetted Perimeter.	6.18 ft
Critical Depth...	0.40 ft
Critical Slope...	0.0350 ft/ft
Froude Number....	0.93 (flow is Subcritical)

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Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708

CGD-8

D.A. = CGWS-US Peak Q = 1.78 cfs.
Also, CGD-8 must pass the peak discharge from Pond 013. Peak discharge = 31.8 cfs

Min BW = 2.0'
Min TW = 6.8'
Min depth = 2.4'
Min SS = 1H:1V

Min slope = 1%
Max Slope = 10%
n = .040

Min. Slope = 1%

Max flow depth = 2.04'

Min. channel depth = 2.04 + .3' = 2.34' ~ 2.40'

Max. Slope = 10%

Max Velocity = 9.06 ft/s depth = 1.12 ft

Riprap: assume $K/d \geq 1$ $V_s = V$ = 9.06 ft/s

Side Slopes = 1:1 from fig. 2

D_{50} = 1.15' = 13.8" = 14 inches

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-8 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	2.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.040
Channel Slope....	0.0100 ft/ft — min. slope
Discharge........	31.80 cfs

Computed Results:

Depth............	2.04 ft — Max. flow depth
Velocity.........	3.86 fps
Flow Area........	8.23 sf
Flow Top Width...	6.08 ft
Wetted Perimeter.	7.77 ft
Critical Depth...	1.53 ft
Critical Slope...	0.0307 ft/ft
Froude Number....	0.58 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-8 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	2.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.040
Channel Slope....	0.1000 ft/ft — Max. slope
Discharge........	31.80 cfs

Computed Results:

Depth............	1.12 ft
Velocity.........	9.06 fps — Max. velocity
Flow Area........	3.51 sf
Flow Top Width...	4.25 ft
Wetted Perimeter.	5.18 ft
Critical Depth...	1.53 ft
Critical Slope...	0.0307 ft/ft
Froude Number....	1.76 (flow is Supercritical)

$\boxed{\text{CGD-9}}$

D.A. = CGWS-D7
Peak Q = 0.60 cfs

Min. BW = 1.5'
Min. TW = 2.2'
Min. depth = 0.6'
Min. SS = 1H:1V

Min. Slope = 1%
Max. Slope = 9.2%
n = .030

Min. Slope = 1%

Max. flow depth = 0.22 ft

Min. channel depth = .22 + .30 = .52
say 0.6'

Max. slope = 9.2%

Max. velocity = 3.26 ft/s   depth = 0.11 ft

Riprap: Assume k = 1" $\frac{k}{d} = \frac{.08}{.11} = 0.73$

Vs = .90 (3.26) = 2.9 ft/sec   (fig 1).

From Fig 2 ⇒ The need for riprap is marginal. Use D_{50} = 1" or vegetated channel.

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-9 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.50 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0100 ft/ft – Min. slope
Discharge........	0.60 cfs

Computed Results:

Depth............	0.22 ft – Max flow depth
Velocity.........	1.57 fps
Flow Area........	0.38 sf
Flow Top Width...	1.94 ft
Wetted Perimeter.	2.13 ft
Critical Depth...	0.16 ft
Critical Slope...	0.0272 ft/ft
Froude Number....	0.63 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-9 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.50 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0920 ft/ft — Max. slope
Discharge........	0.60 cfs

Computed Results:

Depth............	0.11 ft
Velocity.........	3.26 fps — Max Velocity
Flow Area........	0.18 sf
Flow Top Width...	1.73 ft
Wetted Perimeter.	1.82 ft
Critical Depth...	0.16 ft
Critical Slope...	0.0272 ft/ft
Froude Number....	1.76 (flow is Supercritical)

CGD-10

D.A. = CGWS-U6 + U7
Peak Q = 5.83 + 1.48 = 7.31 cfs

Min. BW = 2.0'
Min. TW = 4.7'
Min. depth = 0.90'
Min. SS = 1.5H:1V

Min. Slope = 3%
Max. Slope = 5%
$n = .030$

Min. Slope = 3%

Max. flow depth = 0.56'

Min. channel depth = 0.56 + 0.30 = 0.86'
say 0.90'

Max. Slope = 5%

Max. velocity = 5.53 ft/s depth = 0.48'

Riprap: assume k = .25 $\frac{k}{d} = \frac{.25}{.48} = 0.52$

$V_s = .78(5.53) = 4.31$ ft/sec (fig 1.)

From Fig 2 ⇒ $D_{50} = .25' = \underline{\underline{3.0''}}$ (1.5:1 Side Slope)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-10 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	2.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0300 ft/ft — Min Slope
Discharge........	7.31 cfs

Computed Results:

Depth............	0.56 ft — Min. depth
Velocity.........	4.62 fps
Flow Area........	1.58 sf
Flow Top Width...	3.67 ft
Wetted Perimeter.	4.01 ft
Critical Depth...	0.63 ft
Critical Slope...	0.0190 ft/ft
Froude Number....	1.24 (flow is Supercritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-10 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	2.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0500 ft/ft — Max Slope
Discharge........	7.31 cfs

Computed Results:

Depth............	0.48 ft
Velocity.........	5.53 fps — Max Velocity
Flow Area........	1.32 sf
Flow Top Width...	3.45 ft
Wetted Perimeter.	3.75 ft
Critical Depth...	0.63 ft
Critical Slope...	0.0190 ft/ft
Froude Number....	1.58 (flow is Supercritical)

$\boxed{\text{CGD-11}}$ D.A. = CGWS - U8
Peak Q = 2.57 cfs

Min. BW = 3.0'
Min. TW = 4.0'
Min. depth = 0.50'
Min. SS = 1H:1V

Min. Slope = 12.5%
Max Slope = 12.5%
n = .035

Minimum slope = Maximum slope = 12.5%

Max flow depth = .18'

Min. channel depth = .18 + .30 = .48' ~ <u>0.50'</u>

Max. velocity = 4.48 ft/sec

Riprap: Assume $k = .15'$ $\frac{K}{d} = \frac{.15}{.18} = .83$

$V_s = .93(V) = 4.2$ ft/s (fig. 1)

(Fig 2) $D_{50} \sim .15' = \underline{2"}$

(use 3:1 slope for bottom slope since flow depth is so small + side slopes do not require much protection).

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-11 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	3.00 ft
Left Side Slope..	1.00:1 (H:V)
Right Side Slope.	1.00:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.1250 ft/ft — Ave. slope
Discharge........	2.57 cfs

Computed Results:

Depth............	0.18 ft — Max flow depth
Velocity.........	4.48 fps — Max velocity
Flow Area........	0.57 sf
Flow Top Width...	3.36 ft
Wetted Perimeter.	3.51 ft
Critical Depth...	0.27 ft
Critical Slope...	0.0307 ft/ft
Froude Number....	1.91 (flow is Supercritical)

$\boxed{\text{CGD-12}}$

D.A. = CGWS-3 (a+b)
Peak Q = 7.98 cfs

Min. BW = 0
Min TW = 4.8'
Min. depth = 1.6'
Min. SS = 1.5H : 1V

Min. Slope = 1.3%
Max. Slope = 2.6%
n = .035

Min. Slope = 1.3%

Max flow depth = 1.29 ft

Min. channel depth
= 1.29 + .3 = 1.59' ~ 1.6'

Max. Slope = 2.6%

Max Velocity = 4.2 ft/s

depth = 1.13'

Riprap: Assume k = 2" = .15' $\frac{k}{d}$ = .13

V_s = .55(4.2 ft/s) = 2.3 ft/s

Since V_s is so low, Riprap is not required (see fig. 2)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-12 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0130 ft/ft — min slope
Discharge........	7.98 cfs

Computed Results:

Depth............	1.29 ft — Max flow depth
Velocity.........	3.20 fps
Flow Area........	2.50 sf
Flow Top Width...	3.87 ft
Wetted Perimeter.	4.65 ft
Critical Depth...	1.12 ft
Critical Slope...	0.0277 ft/ft
Froude Number....	0.70 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-12 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0260 ft/ft — Max. slope
Discharge........	7.98 cfs

Computed Results:

Depth............	1.13 ft
Velocity.........	4.15 fps — Max velocity
Flow Area........	1.92 sf
Flow Top Width...	3.40 ft
Wetted Perimeter.	4.08 ft
Critical Depth...	1.12 ft
Critical Slope...	0.0277 ft/ft
Froude Number....	0.97 (flow is Subcritical)

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CGD-13

D.A. = CGWS-D4(a+b)
Peak Q = 7.59 cfs

Min. BW = 0
Min. TW = 4.8'
Min. depth = 1.60'
Min. SS = 1.5H:1V

min. slope = 1.4%
max. slope = 3.0%
η = .035

Min. Slope = 1.4%

Max. flow depth = 1.25'

Min. channel depth = 1.25' + .30' = 1.55' ~ 1.60'

Max. Slope = 3.0%

Max velocity = 4.32 ft/s depth = 1.08'

Riprap: Assume k = 2" = .15' $\frac{k}{d} = \frac{.15}{1.08} = 0.14$

$V_s = 0.55(4.32) = 2.4$ ft/s (fig 1)

Since V_s is so small, no riprap is required (fig. 2)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-13 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0140 ft/ft — Min. slope
Discharge........	7.59 cfs

Computed Results:

Depth............	1.25 ft — Max. depth
Velocity.........	3.25 fps
Flow Area........	2.34 sf
Flow Top Width...	3.75 ft
Wetted Perimeter.	4.50 ft
Critical Depth...	1.10 ft
Critical Slope...	0.0279 ft/ft
Froude Number....	0.72 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-13 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0300 ft/ft – Max. Slope
Discharge........	7.59 cfs

Computed Results:

Depth............	1.08 ft
Velocity.........	4.32 fps – Max Velocity
Flow Area........	1.76 sf
Flow Top Width...	3.25 ft
Wetted Perimeter.	3.90 ft
Critical Depth...	1.10 ft
Critical Slope...	0.0279 ft/ft
Froude Number....	1.03 (flow is Supercritical)

$\boxed{\text{CGD-14}}$ D.A. = CGWS - D1A
Peak Q = 3.41 cfs (1/2 of Peak Q from CGWS- D1a+b)

Min. BW = 1.0'
Min. TW = 4.0'
Min. depth = 1.0'
Min. SS = 1.5H:1V

Min. slope = 1%
Max. slope = 1%
n = .03

Max slope = Min slope = 1%

Max flow depth = .65'

Min. channel depth = .65+.30 = .95 ~ 1.0'

Max velocity = 2.63 ft/s
no riprap required

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-14 ave. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0100 ft/ft — Min / Max slope
Discharge........	3.41 cfs

Computed Results:

Depth............	0.65 ft — Max depth
Velocity.........	2.63 fps — Max velocity
Flow Area........	1.30 sf
Flow Top Width...	2.96 ft
Wetted Perimeter.	3.36 ft
Critical Depth...	0.54 ft
Critical Slope...	0.0213 ft/ft
Froude Number....	0.70 (flow is Subcritical)

$\boxed{\text{CGD-15}}$ D.A. = CGWS-D1B
Peak Q = 3.42 cfs (1/2 of Peak Q from CGWS-D1a+b)

Min BW = 1.0'
Min. TW = 4.0'
Min depth = 1.0'
Min SS = 1.5H:1V

Min. Slope = 1%
Max. Slope = 3%
n = .030

Min. Slope = 1%

Max. flow depth = 0.66'

Min. channel depth = .66 + .30 = .96' ~ 1.0'

Max. Slope = 3%

Max velocity = 3.94 ft/s

depth = .50'

Riprap: Assume k = 1" = .1'

$\frac{k}{d} = \frac{.1}{.5} = .20$

$V_s = .6(3.94) = 2.4$ ft/s (fig 1)

Since V_s is small, no riprap is required (fig. 2)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-15 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0100 ft/ft — Min slope
Discharge........	3.42 cfs

Computed Results:

Depth............	0.66 ft — Max depth
Velocity.........	2.63 fps
Flow Area........	1.30 sf
Flow Top Width...	2.97 ft
Wetted Perimeter.	3.36 ft
Critical Depth...	0.54 ft
Critical Slope...	0.0213 ft/ft
Froude Number....	0.70 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-15 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	1.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0300 ft/ft — Max slope
Discharge........	3.42 cfs

Computed Results:

Depth............	0.50 ft
Velocity.........	3.94 fps — Max velocity
Flow Area........	0.87 sf
Flow Top Width...	2.49 ft
Wetted Perimeter.	2.79 ft
Critical Depth...	0.54 ft
Critical Slope...	0.0213 ft/ft
Froude Number....	1.18 (flow is Supercritical)

CGD-16

D.A. = CGWS-US
Peak Q = 1.78 cfs

Min. BW = 0'
Min. TW = 2.7'
Min. depth = .9'
Min. SS = 1.5H:1V

Min. Slope = 7%
Max. Slope = 10%
n = .035

Min. Slope = 7%

Max. flow depth = .54'

Min. channel depth = .54 + .3 = .84' say .9'

Max. Slope = 10%

Max. velocity = 4.72 ft/s

depth = .5'

Riprap: Assume K = 2" = .15'

$\frac{K}{d} = \frac{.15}{.5} = .30$

$V_s = .67(4.72) = 3.2$ ft/s (fig. 1)

$D_{50} = .125' =$ 1.5 inches (fig 2, 1.5:1 slopes)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-16 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.0700 ft/ft — Min Slope
Discharge........	1.78 cfs

Computed Results:

Depth............	0.54 ft — Max depth
Velocity.........	4.13 fps
Flow Area........	0.43 sf
Flow Top Width...	1.61 ft
Wetted Perimeter.	1.93 ft
Critical Depth...	0.61 ft
Critical Slope...	0.0338 ft/ft
Froude Number....	1.41 (flow is Supercritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-16 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.035
Channel Slope....	0.1000 ft/ft — Max. slope
Discharge........	1.78 cfs

Computed Results:

Depth............	0.50 ft
Velocity.........	4.72 fps — Max Velocity
Flow Area........	0.38 sf
Flow Top Width...	1.50 ft
Wetted Perimeter.	1.81 ft
Critical Depth...	0.61 ft
Critical Slope...	0.0338 ft/ft
Froude Number....	1.66 (flow is Supercritical)

$\boxed{\text{CGD-17}}$

D.A. = CGWS-D4A
Peak Q = 3.8 cfs (assume 1/2 Q discharge from CGWS-D4a+b)

Min. BW = 0'
Min. TW = 3.0'
Min. depth = 1.0'
Min. SS = 1.5H:1V

Min. Slope = 6.7%
Max. Slope = 10%
n = .030

Min. Slope = 6.7%

Max flow depth = 0.68 ft

Min. channel depth = .68+.3 = .98' ~ 1.0'

Max. Slope = 10%

Max velocity = 6.41 ft/s   depth = .63'

Riprap: Assume k = 2" = .15' $\frac{k}{d} = \frac{.15}{.63} = .24$

V_s = .63(6.41) = 4.0 ft/s   (fig. 1)

D_{50} = .15' = 2.0 inches   (fig. 2, 1.5:1 Slopes)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-17 min. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0670 ft/ft — min. slope
Discharge........	3.80 cfs

Computed Results:

Depth............	0.68 ft — Max. depth
Velocity.........	5.51 fps
Flow Area........	0.69 sf
Flow Top Width...	2.03 ft
Wetted Perimeter.	2.44 ft
Critical Depth...	0.83 ft
Critical Slope...	0.0225 ft/ft
Froude Number....	1.67 (flow is Supercritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Area

Comment: CGD-17 max. slope

Solve For Depth

Given Input Data:

Bottom Width.....	0.00 ft
Left Side Slope..	1.50:1 (H:V)
Right Side Slope.	1.50:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.1000 ft/ft — Max. slope
Discharge........	3.80 cfs

Computed Results:

Depth............	0.63 ft
Velocity.........	6.41 fps — Max. velocity
Flow Area........	0.59 sf
Flow Top Width...	1.89 ft
Wetted Perimeter.	2.27 ft
Critical Depth...	0.83 ft
Critical Slope...	0.0225 ft/ft
Froude Number....	2.01 (flow is Supercritical)

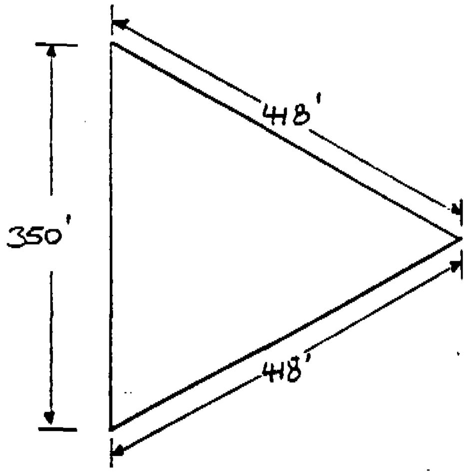
CASTLE GATE SCHOOL HOUSE REFUSE AREA.

GENERIC ROAD SIDE DITCH.

AREA = 1.67 Ac.
CN = 90
WSS = 46 %

$$L = \frac{(420)^{0.8}\ (1.1+1)^{(0.7)}}{1900\ (46)^{0.5}} = 0.016 \text{ HR.}$$

$$T_c = (1.67)(0.016) = 0.027 \text{ HR}$$



REFUSE AREA : GENERIC ROAD SIDE DITCH

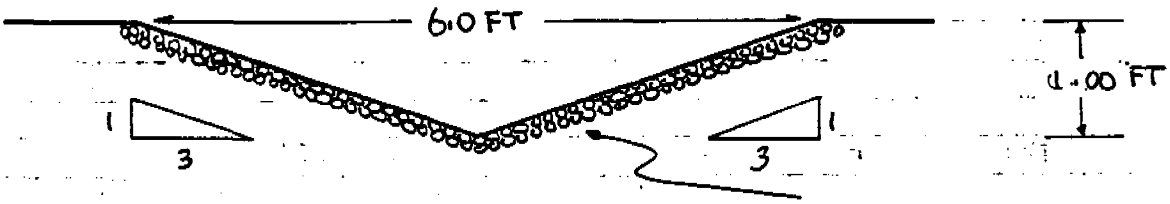
D = 0.40 FT (BASED ON MIN. CHANNEL SLOPE, V = 3.99 FPS)
V = 5.01 FPS (BASED ON MAX. CHANNEL SLOPE, D = 0.36 FT.)

ASSUME K = 0.1' ∴ $\frac{K}{D} = \frac{0.1'}{0.36'}$ ∴ $\frac{K}{D} = 0.28$

SIDE SLOPES ARE 3H : 1V FOR THIS CALCULATION.

∴ $\frac{V_s}{V} = 0.7$ ∴ $V_s = (0.7)(5.01)$ $V_s = 3.5$ FPS

∴ $\boxed{d_{50} \text{ REQUIRED} = 0.10' = 1.5 \text{ INCH RIPRAP.}}$



d_{50} REQUIRED = 1.5 INCH RIPRAP

WATER FLOW DEPTH = 0.40 FT.
FREEBOARD = 0.60 FT.
TOTAL DEPTH = 1.00 FT.
SLOPE ⇒ MAX = 0.110 FT/FT
MIN = 0.060 FT/FT
$n = 0.030$.

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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: REFUSE AREA

Comment: GENERIC ROAD SIDE DITCH

Solve For Depth

Given Input Data:

Left Side Slope..	3.00:1 (H:V)
Right Side Slope.	3.00:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.1100 ft/ft ← MAX.
Discharge........	1.90 cfs

Computed Results:

Depth............	0.36 ft
Velocity.........	5.01 fps
Flow Area........	0.38 sf
Flow Top Width...	2.13 ft
Wetted Perimeter.	2.25 ft
Critical Depth...	0.48 ft
Critical Slope...	0.0227 ft/ft
Froude Number....	2.10 (flow is Supercritical)

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Triangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: REFUSE AREA

Comment: GENERIC ROAD SIDE DITCH

Solve For Depth

Given Input Data:

Left Side Slope..	3.00:1 (H:V)
Right Side Slope.	3.00:1 (H:V)
Manning's n......	0.030
Channel Slope....	0.0600 ft/ft ← MIN.
Discharge........	1.90 cfs

Computed Results:

Depth............	0.40 ft
Velocity.........	3.99 fps
Flow Area........	0.48 sf
Flow Top Width...	2.39 ft
Wetted Perimeter.	2.52 ft
Critical Depth...	0.48 ft
Critical Slope...	0.0227 ft/ft
Froude Number....	1.58 (flow is Supercritical)

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CASTLE GATE MINE
OPERATIONAL HYDROLOGY
SUPPLEMENT TO APPENDIX 3.4E

REFUSE PILE - CGWS-D2F

DOGM CONCERN: OVERLAND FLOW ON THE WEST SIDE OF THE MAIN FACE OF THE REFUSE PILE MAY BE SUFFICIENTLY SIGNIFICANT TO CAUSE SURFICIAL EROSION OF THE REFUSE.

SOLUTION: CALCULATE A PEAK FLOW FROM THE WEST SIDE OF THE REFUSE PILE FACE AND ROUTE IT ACROSS THE NARROW WIDTH OF THE WATERSHED JUST ABOVE POND 013. COMPARE THE CALCULATED FLOW VELOCITY TO A MAXIMUM PERMISSABLE NON-ERODING VELOCITY FOR A VEGETATED GROUND SURFACE. (TAKE MEASUREMENTS FROM EXHIBIT 3.4-2)

AREA (PORTION OF CGWS-D2F)

$$1.0\ in^2 \times \left(200\ \frac{FT}{IN}\right)^2 = 40{,}000\ FT^2 \times \frac{ACRE}{43{,}560\ FT^2} = 0.92\ ACRES$$

CN = 85 (TABLE 3.4-1)

$T_c = 1.67\, T_L$

$$T_L = \frac{L^{0.8}(S+1)^{0.7}}{1900\,Y^{0.5}}$$ (BARFIELD, ET AL, 1981)

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

$$= \frac{1000}{85} - 10$$

$= 1.76$

$L = 400\ FT$

$WS = Y = 0.25\,(EM - EN)(LC_{25} + LC_{50} + LC_{75})\,/A$ (CHANG, 1989)

EN = 6190 FT
EM = 6340 FT
EM-EN = 150 FT

$0.25(EM-EN)+EN = 0.25(150)+6200 = 6238$
$0.50(EM-EN)+EN = 0.50(150)+6200 = 6275$
$0.75(EM-EN)+EN = 0.75(150)+6200 = 6313$

$L_{25} = 60$ FT

$LC_{50} = 80$ FT

$LC_{75} = 140$ FT

$WS=Y = 0.25(150)(60+80+140)/40,000$

$= 26\%$

$$T_L = \frac{(400)^{0.8}(1.76+1)^{0.7}}{1900(26)^{0.5}}$$

$= 0.025$

$T_c = 1.67(0.025)$
$= 0.042$ HRS

DESIGN STORM : 100-YEAR 6-HOUR

P = 2.0 INCHES (CHAPTER 7, TABLE 7-7)

STORM DISTRIBUTION : SCS TYPE 'B'

(AN SCS TYPE 'B' STORM DISTRIBUTION IS APPLICABLE TO 6 HOUR DESIGN STORMS, ACCORDING TO SCS ENGINEERING HANDBOOK, CH. 4, 1956)

RUN 'SCS HYDRO' TO CALCULATE PEAK FLOW.

PEAK FLOW = 0.72 CFS

AT THE BASE OF THE REFUSE PILE, THE FLOW TRAVELS IN THE FORM OF SHEETFLOW ACROSS A 25 FOOT WIDE SECTION OF THE FACE, MEASURED FROM THE WATERSHED BOUNDARY WITH CAWS-D2D ON ONE SIDE AND THE EDGE OF THE FACE OF THE PILE ON THE OTHER SIDE.

RUN FLOWMASTER I (HAESTED METHODS, 1991) TO DETERMINE DEPTH OF FLOW AND FLOW VELOCITY.

1. ASSUME: TRAPAZOIDAL CHANNEL
BOTTOM WIDTH = 25 FT
SS = 3:1
SLOPE = 10/30 = 0.33 FT/FT
n = 0.030 (REFUSE IS GRANULAR FLOW DEPTH WILL BE SHALLOW, FACE IS VEGETATED)
Q = 0.72 CFS

RESULTS: FLOW DEPTH = 0.02 FT (1/4")
VELOCITY = 1.8 FPS

2. CHECK STEEPER SECTION HIGHER UP THE FACE BETWEEN ELEVATION 6200 TO 6250

TRAPAZOIDAL CHANNEL
BOTTOM WIDTH = 35 FT
SS = 3:1
SLOPE = 50/110 = 0.45
n = 0.030
Q = 0.72 CFS

RESULTS: FLOW DEPTH = 0.01 (1/8")
VELOCITY = 1.7 FPS

SEE TABLE 3.2 IN BARFIELD, ET AL (1981) FOR LIMITING VELOCITIES ON VARIOUS SOILS.

THE REFUSE PILE SOIL IS GRANULAR, WITH SOME SILT SIZED PARTICLES

∴ FINE GRAVEL, CLEAR WATER MEDIA ⇒ V = 2.50 FPS

THUS, VELOCITIES BELOW 2.5 FPS ARE NON-ERODIBLE.

CONCLUSION:
SINCE THE CALCULATED FLOW VELOCITY IS LESS THAN THE LIMITING VELOCITY, OVERLAND FLOW EROSION SHOULD NOT BE A PROBLEM. ADDITIONAL DIVERSIONS TO COLLECT FLOW FROM THE FACE OF THE REFUSE PILE IN ITS CURRENT CONFIGURATION, ARE NOT NECESSARY.

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-D2F - FACE OF REFUSE PILE ONLY

```
-----------------------------------------------------------
    STORM :                            WATERSHED :
   Dist.=SCS Type 'b' - 6 Hr          Area =      0.92   acres
   Depth =  2.00   inches               CN = 85.00
   Duration =  6.00  hrs              Time conc.=  0.042  hrs
-----------------------------------------------------------
```

OUTPUT SUMMARY

```
-----------------------------------------------------------
    Runoff depth      0.79513    inches
    Initial abstr     0.35294    inches
    Peak flow =        0.72     cfs   (  0.77889  iph )
       at time     2.503    hrs
-----------------------------------------------------------
```

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Castle Gate Prep Plant

Peak Discharge Calculations

* Precip = 1.4 inches for 10-year 6-hour Storm
* Use SCS type b distribution

Watershed	CN	Area (Ac)	Tc (hr.)	Peak Q (cfs)
CGWS-U1	78	7.05	0.106	0.95
U2	78	2.38	0.066	0.36
U3a	75	1027.09	0.710	42.46
U3b	75	172.86	0.386	7.44
U4	78	6.78	0.085	0.96
U5	SEE APPENDIX 3.4J			
U6	78	52.11	0.181	5.83
U7	82	5.96	0.108	1.48
U8	78	20.59	0.137	2.57
CGWS-D1 (a+b)	90	12.60	0.162	6.83
D2a	SEE APPENDIX 3.4J			
D2b				
D2c				
D2d				
D2e				
D2f				
D2g	82	2.26	0.081	0.59
D3 (a+b)	90	14.48	0.142	7.98
D4 (a+b)	90	14.73	0.217	7.59
D5	85	1.56	0.038	0.60

EARTHFAX ENGINEERING, INC.
HYDROGRAPH GENERATION PROGRAM OUTPUT
BASED ON SCS CURVE NUMBER METHODOLOGY

INPUT FOR: CGWS-U1

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 7.05 acres
Depth = 1.40 inches	CN = 78.00
Duration = 6.00 hrs	Time conc.= 0.106 hrs

OUTPUT SUMMARY

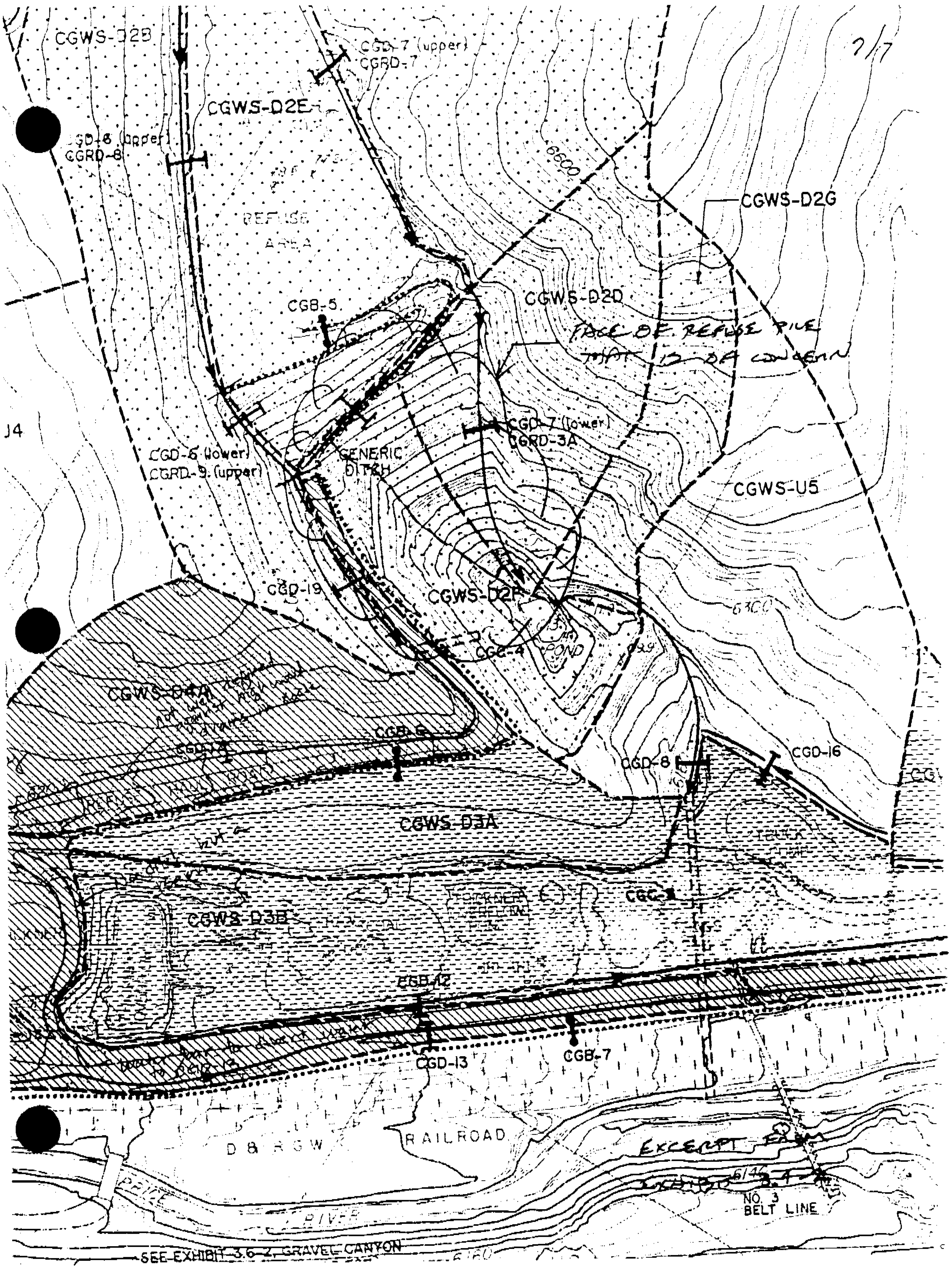
Runoff depth	0.19110	inches	
Initial abstr	0.56410	inches	
Peak flow =	0.95	cfs	(0.13341 iph)
at time	2.530	hrs	

INPUT FOR: CGWS-U2

STORM :	WATERSHED :
Dist.=SCS Type 'b' - 6 Hr	Area = 2.38 acres
Depth = 1.40 inches	CN = 78.00
Duration = 6.00 hrs	Time conc.= 0.066 hrs

OUTPUT SUMMARY

Runoff depth	0.19110	inches	
Initial abstr	0.56410	inches	
Peak flow =	0.36	cfs	(0.14856 iph)
at time	2.517	hrs	



7/7
CGWS-D2B
CGD-7 (upper)
CGRD-7
CGWS-D2E
CGD-8 (upper)
CGRD-8
6600
REFUSE
AREA
CGWS-D2G
CGB-5
CGWS-D2D
FACE OF REFUSE PILE
THAT IS OF CONCERN
CGD-7 (lower)
CGRD-3A
J4
CGD-6 (lower)
CGRD-9 (upper)
GENERIC
DITCH
CGWS-U5
CGD-19
CGWS-D2F
CGC-4
POND
6300
CGWS-D4A
CGB-6
CGD-8
CGD-16
CGWS-D3A
CGC-3
CGWS-D3B
CGB-12
CGD-13
CGB-7
D & R G W
RAILROAD
EXCERPT FROM
EXHIBIT 3.4
6146
NO. 3
BELT LINE
RIVER
SEE EXHIBIT 3.6-2, GRAVEL CANYON
6160

APPENDIX 3.4F

DIVERSION CULVERT CALCULATIONS

REGISTERED PROFESSIONAL ENGINEER
NO. 11350
WILLIAM S. HENDRICKSON
STATE OF

William S. Hendrickson
2/2/94

Prep Plant Area – Culvert Capacity

Culvert ID	Size	Inlet Type	Available Headwater over Culvert Top	Inlet Control Capacity[(a)] (cfs)	Contributing Watersheds	Peak Flow (cfs)[(b)]	
CGC-1	18" CMP	Projecting	> 1'	5.7	U1	0.95	OK
CGC-2	2-84" CMP	Mitered	> 1'	2×295= 590	Pond 011, U2, U3a, U3b, U4	56.5	OK
CGC-3	24" CMP	48" drop inlet (Headwall)	2'	12.5	U4	0.96	OK
CGC-4	SEE APPENDIX 3.4J						
CGC-5	60" CMP	48" × 64" Drop Inlet	8'	128	U5, Pond 013	33.58	OK
CGC-6	12" CMP	Projecting	> 1'	2.1	D7	0.60	OK
CGC-7	18" CMP	Connects Pond 012A to Pond 012B (See Pond Calcs)					
CGC-8	60"×120" Box	Headwall	7'	280[(c)]	U6, U7, U8	9.88	OK

Notes: (a) See nomograph #1 on next page. Capacity based on HW/D = 1.0.
(b) See runoff + Sed. pond Calc's. Multiple watersheds added numerically. (ie, assumes all peak flows are temporarily concurrent)
(c) See nomograph #2 – attached.

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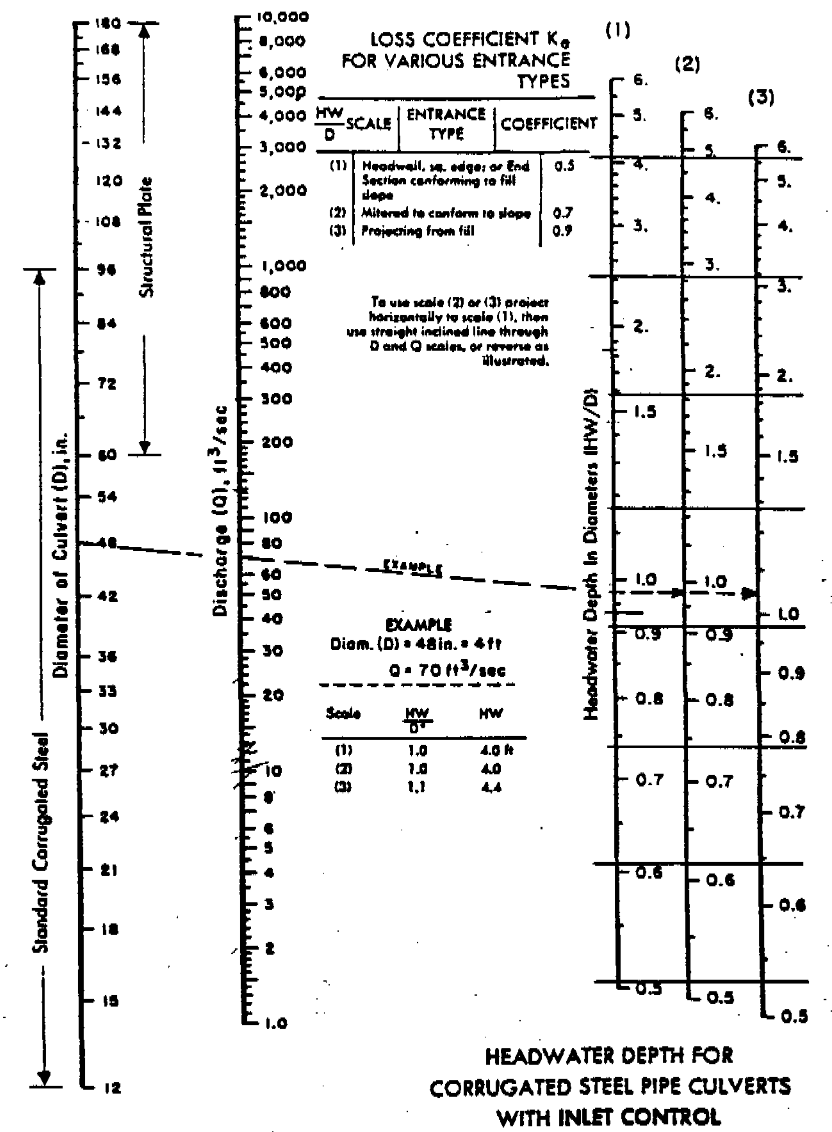
COMPUTED [illegible] DATE 7/6/90
CHECKED DATE



HEADWATER DEPTH FOR
CORRUGATED STEEL PIPE CULVERTS
WITH INLET CONTROL

FHWA HEC 5

Figure 4-28 *Inlet control* nomograph for corrugated steel *pipe* culverts. The manufacturers recommended keeping *HW/D* to a maximum of 1.5 and preferably to no more than 1.0.

Nomograph #1 for corrugated steel pipe culverts.

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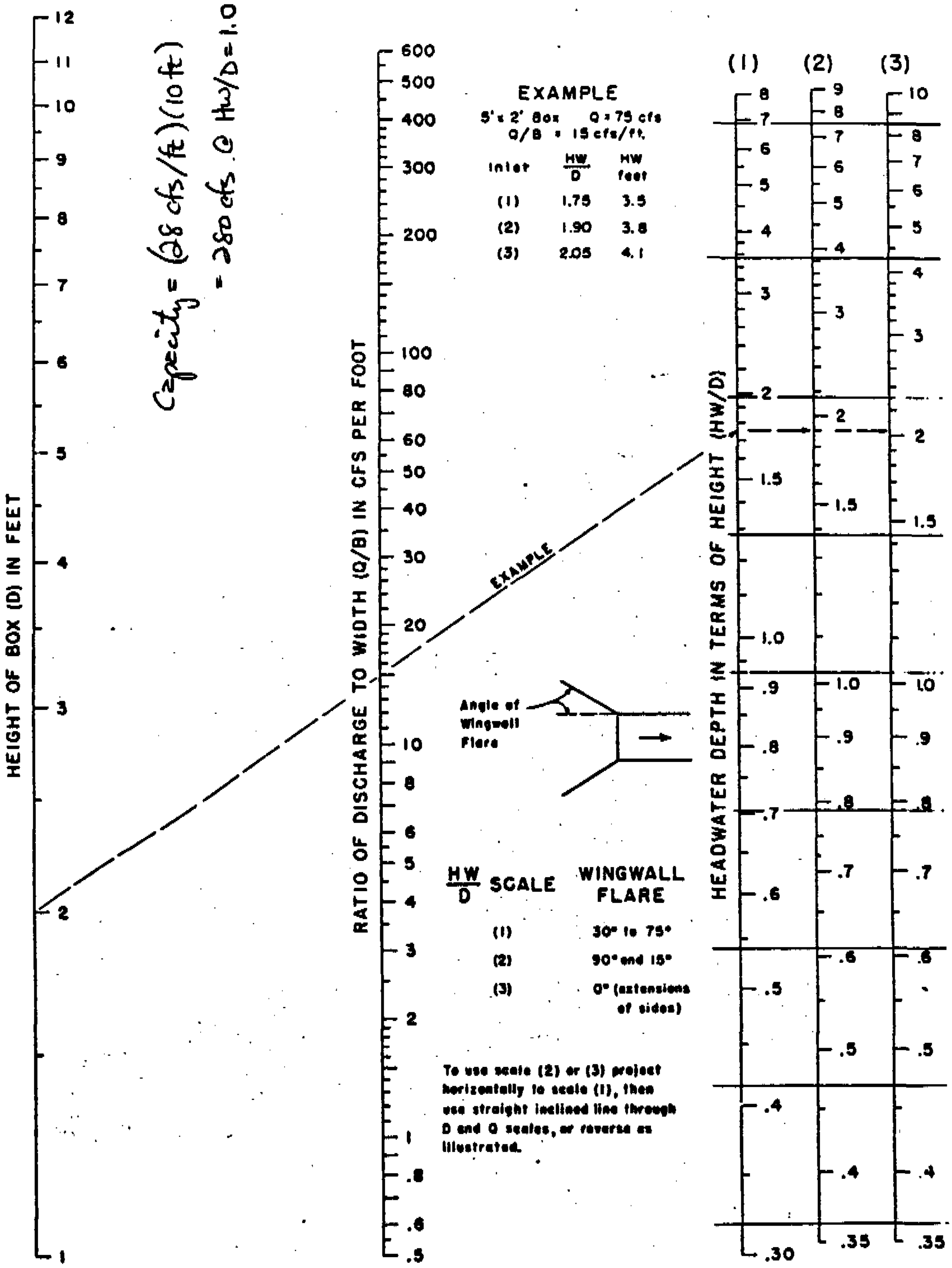
CHART 1

Capacity = (28 cfs/ft)(10 ft)
= 280 cfs @ HW/D = 1.0



HEADWATER DEPTH FOR BOX CULVERTS WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

Nomograph #2 for Box Culverts.

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Prep Plant Area Culverts
Min. Riprap Requirements

Culvert ID	Size	Slope (a) (%)	Peak (b) flow (cfs)	Velocity (c) (ft/s)	Minimum (d) Riprap D_{50} (in)
CGC-1	18" CMP	1	0.95	2.4	none
-2	2-84" CMP	5	56.5	9.5	7
-3	24" CMP	4	0.96	3.8	1
-4	SEE APPENDIX 3.4J				
-5	60" CMP	10	33.58	13.4	15
-6	12" CMP	4	0.60	3.6	1
-7	18" CMP	Connects Pond 012A to Pond 012B. See Pond Calcs			
-8	60"x120" Box	1	9.88	2.9	none

Notes: (a) Field measurement
(b) See runoff and sed. pond calcs.
(c) See attached calcs. Assume n=.024 for CMP + Concrete, and .010 for HDPE.
(d) See nomograph in this calc. Assume $K/d > 1 \therefore V_s = V$.

Table 3.4 Permissible Velocities for Vegetated Channels.*

Cover	Erosion Resistant Soils (% Slope) 0-5	5-10	Over 10	Easily Eroded Soils (% Slope) 0-5	5-10	Over 10
Bermuda grass	8	7	6	6	5	4
Buffalo grass Kentucky bluegrass Smooth brome Blue grama Tall fescue	7	6	5	5	4	3
Lespedeza sericea Weeping lovegrass Kudzu Alfalfa Crabgrass	3.5	NR†	NR	2.5	NR	NR
Grass mixture	5	4	NR	4	3	NR
Annuals for temporary protection	3.5	NR	NR	2.5	NR	NR

Permissible Velocity, fps

* After Ree (1949).
† Not recommended.

Figure 3

Ref: Barfield, et. al. (1981)

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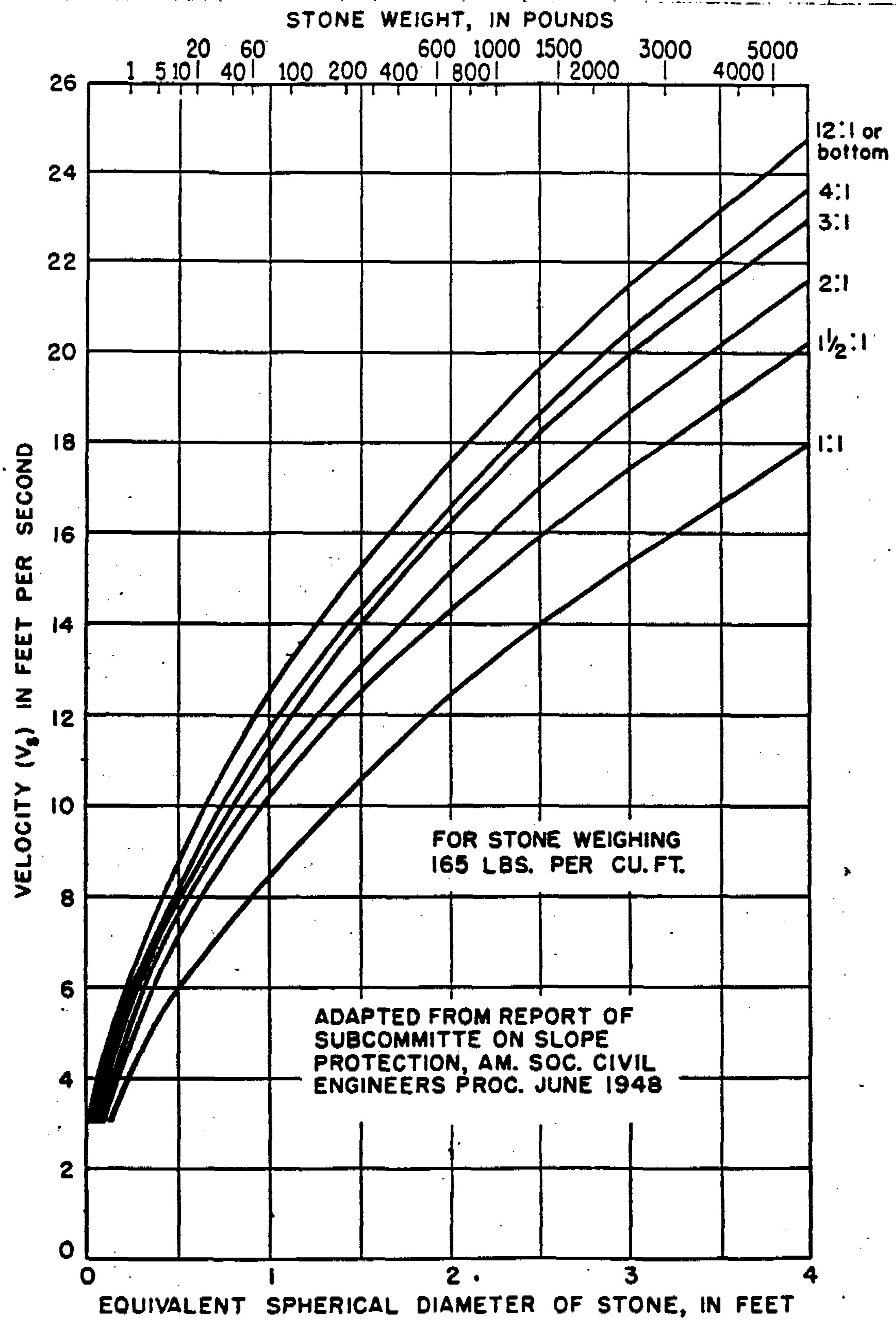

FIG. 2-SIZE OF STONE THAT WILL RESIST DISPLACEMENT FOR VARIOUS VELOCITIES AND SIDE SLOPES

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Prep Plant Culverts

Comment: CGC-1

Solve For Actual Depth

Given Input Data:

Diameter..........	1.50 ft
Slope.............	0.0100 ft/ft
Manning's n.......	0.024
Discharge.........	0.95 cfs

Computed Results:

Depth.............	0.41 ft
Velocity..........	2.39 fps
Flow Area.........	0.40 sf
Critical Depth....	0.36 ft
Critical Slope....	0.0168 ft/ft
Percent Full......	27.64 %
Full Capacity.....	5.69 cfs
QMAX @.94D........	6.12 cfs
Froude Number.....	0.77 (flow is Subcritical)

V= 2.39 ft/s No Riprap Required.

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Prep Plant Culverts

Comment: CGC-2

Solve For Actual Depth

Given Input Data:

Diameter..........	7.00 ft
Slope.............	0.0500 ft/ft
Manning's n.......	0.024
Discharge.........	28.25 cfs

Since there are two 84" culverts, velocity was determined for one culvert at 1/2 the peak discharge.

Computed Results:

Depth.............	0.91 ft
Velocity..........	9.54 fps
Flow Area.........	2.96 sf
Critical Depth....	1.34 ft
Critical Slope....	0.0103 ft/ft
Percent Full......	13.06 %
Full Capacity.....	773.76 cfs
QMAX @.94D........	832.33 cfs
Froude Number.....	2.12 (flow is Supercritical)

assume 12:1 bottom slope.

$D_{50} = 0.6$ ft = 7 inches

Open Channel Flow Module, Version 3.2 (c) 1990
Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Prep Plant Culverts

Comment: CGC-3

Solve For Actual Depth

Given Input Data:

Diameter..........	2.00 ft
Slope.............	0.0400 ft/ft
Manning's n.......	0.024
Discharge.........	0.96 cfs

Computed Results:

Depth.............	0.27 ft
Velocity..........	3.78 fps
Flow Area.........	0.25 sf
Critical Depth....	0.34 ft
Critical Slope....	0.0160 ft/ft
Percent Full......	13.51 %
Full Capacity.....	24.51 cfs
QMAX @.94D........	26.36 cfs
Froude Number.....	1.55 (flow is Supercritical)

assume 12:1 bottom Slope.

D_{50} = 0.1 ft = 1 inch

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Prep Plant Culverts

Comment: CGC-5

Solve For Actual Depth

Given Input Data:

Diameter..........	5.00 ft
Slope.............	0.1000 ft/ft
Manning's n.......	0.024
Discharge.........	33.58 cfs

Computed Results:

Depth.............	0.93 ft
Velocity..........	13.37 fps
Flow Area.........	2.51 sf
Critical Depth....	1.61 ft
Critical Slope....	0.0112 ft/ft
Percent Full......	18.56 %
Full Capacity.....	446.11 cfs
QMAX @.94D........	479.89 cfs
Froude Number.....	2.93 (flow is Supercritical)

Assume 4:1 slope for riprap sizing.

$D_{50} = 1.25' =$ 15 inches

Circular Channel Analysis & Design
Solved with Manning's Equation

Open Channel - Uniform flow

Worksheet Name: Prep Plant Culverts

Comment: CGC-6

Solve For Actual Depth

Given Input Data:

Diameter..........	1.00 ft
Slope.............	0.0400 ft/ft
Manning's n.......	0.024
Discharge.........	0.60 cfs

Computed Results:

Depth.............	0.27 ft
Velocity..........	3.57 fps
Flow Area.........	0.17 sf
Critical Depth....	0.32 ft
Critical Slope....	0.0191 ft/ft
Percent Full......	26.65 %
Full Capacity.....	3.86 cfs
QMAX @.94D........	4.15 cfs
Froude Number.....	1.44 (flow is Supercritical)

Assume 12:1 bottom Slope for calcs.

$D_{50} = 0.1' =$ 1 inch

Open Channel Flow Module, Version 3.2 (c) 1990
Haestad Methods, Inc. * 37 Brookside Rd * Waterbury, Ct 06708

Rectangular Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: Prep Plant Culverts

Comment: CGC-8

Solve For Depth

Given Input Data:

Bottom Width.....	10.00 ft
Manning's n......	0.024
Channel Slope....	0.0100 ft/ft
Discharge........	9.88 cfs

Computed Results:

Depth............	0.34 ft
Velocity.........	2.89 fps
Flow Area........	3.41 sf
Flow Top Width...	10.00 ft
Wetted Perimeter.	10.68 ft
Critical Depth...	0.31 ft
Critical Slope...	0.0134 ft/ft
Froude Number....	0.87 (flow is Subcritical)

With the velocity low (< 3 fps) no riprap is required.