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EUREKA ENERGY COMPANY

A SUBSIDIARY OF PACIFIC GAS AND ELECTRIC COMPANY

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JIM

MAR 16 1982

March 12, 1982

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DIVISION OF
OIL, GAS & MINING

James W. Smith
Coordinator of Mined Land Development
State of Utah
Natural Resources & Energy
Oil, Gas, and Mining
4241 State Office Building
Salt Lake City, Utah 84114

Re: Sage Point-Dugout Canyon Project

Dear Mr. Smith:

Attention: Sally Kefer

In answer to your letter dated February 4, 1982 we submit the following enclosures that will become part of our SMCRA Permit Application. The pages, tables, and figures were taken from a "Preliminary Design Report" on Water and Wastewater System, prepared for Eureka Energy Company by Dames & Moore, dated April 10, 1980.

1. Section 6.2 provides information on the wastewater disposal and the sewage lagoon.

2. Section 6.3 deals with surface runoff control including: runoff volumes, sediment transport, sediment pond capacities and cleanout rates, and typical cross-sections of sediment ponds and drainage ditches.

3. Appendix A pages A-8 through A-10 and A-23 through A-27 deals with precipitation giving drainage areas and stream flows.

I would like to point out again that all of the above information is preliminary and final design will provide the detail necessary to determine if the runoff control structures meet the requirements of Utah Mining Codes. For your information, the methods used to determine the final runoff flows and storage volumes will be: 1) for small disturbed areas, the "Rational Method" will be used; and 2) for larger areas the "Soil Conservation Service SCS-TR55 Method" will be implemented; 3) for open channel flow and pipe flow, "Manning's equation" will be used.

Regarding the total suspended solid effluent concentration question, enclosed is a letter from R. A. Urbanowski of Dames & Moore which explains the method used and quantities listed in Tables III-B.8 through III-B.30 included in SMCRA permit application.

The above enclosures should satisfy the concerns raised in your February 4th letter, with the exception of the certified statement. A certified statement which depicts a schedule for detailed submission will be filed with the Division by April 2, 1982. The final detailed designs will be submitted a minimum of 120 days prior to construction.

In response to Everett Hooper's December 17, 1981 letter, enclosed are revised topsoil stockpile drawings and a typical cross section of the stockpile.

If you have any additional questions or requests, please contact me.

Sincerely yours,

Nicolas Temnikov

Nicolas K. Temnikov

NKT:hla/34/N

cc: S. O. Ogden
T. L. Wylie

Enclosures

recommended before developing sources outside the portal area. In Section 7.0, designs for Hoffman Creek show the system including the Coal Creek source and force main. If studies show a well or infiltration line is a suitable source, this portion of the system will be unnecessary.

Industrial storage for the Central Facilities was sized based on industrial, potable, and fire requirements estimated as for the Fish and Dugout Canyon mine portals. Storage was allowed for one shift using a peaking factor supplied by Eureka Energy of 1.4 applied to estimated shift usage including 90 gpm for the coal preparation plant. Storage is provided for 10,000 gallons to be treated and used for potable purposes, and 20,000 gallons for fire protection. Minimum flow and pressure was assumed to be 50 gpm at 50 psi. A potable storage tank sized for one day's usage is provided in the potable system.

Although final selection of materials will be a final design and bidding function, certain assumptions have been made to facilitate cost estimation. Small diameter pipelines (less than 4 inch diameter) will probably be plastic, while larger lines may be coated or otherwise cathodic protected metal pipe such as ductile iron or transite. Insulated, above grade steel tankage will probably be used for water storage tanks, except for the buried cistern at Hoffman Creek portal, where concrete may be used.

6.2 WASTEWATER DISPOSAL

6.2.1 Introduction

This section includes discussions of the systems which will be used for wastewater disposal, including conveyance facilities, the design constraints and assumptions involved in preparing preliminary designs, and cost estimates. As stated in Section 5.1, the recommended wastewater disposal systems include lagoon treatment at a joint site for Fish Creek and Dugout Canyon, septic sytems for the Central Facilities and Pine Canyon, and trucking of wastewater from Hoffman Creek to the lagoon.

6.2.2 Wastewater Conveyance

Wastewater will be conveyed mainly by gravity pipelines at the facility areas and between facility areas and disposal points. The exceptions are trucking of wastewater from Hoffman Creek to the lagoon and pumping of wastewater through force mains along the Dugout Canyon outfall, and from Pine Canyon portal area to the leachfield site.

The gravity pipelines were sized to maintain sufficient velocity to prevent solids deposition (2 feet per second) and to flow no more than one-half full at peak flow. Sewer manholes will be provided at junctions of two or more building sewers and at sharp changes in alignment and/or slope. It was assumed that gravity sewers smaller than 8-inch diameter (state requirement for municipal systems) could be used, since significant amounts of grease and the odd assortment of large-sized items found in public sewers will not be present in project wastewaters. Also, on long, straight sewer reaches with adequate slopes, manhole spacing will be increased to 1000 feet from 400 feet (state requirement for municipal systems) and cleanouts will be placed at 200-foot intervals.

Gravity sewers as small as 4-inch diameter will only be used in short lengths of building sewers, and in septic tank effluent sewers, where most of the solids have been removed from the flow. Manholes and other concrete work will be kept to a minimum on septic tank effluent lines due to the corrosive properties of the anaerobic wastewater. Plastic pipes are highly recommended for all of the sewer lines, due to their resistance to corrosion and ease of construction.

Pump stations and force mains for Dugout Canyon and Pine Canyon will require quite different final designs. The Dugout Canyon pump station will lift raw sewage from a point along Dugout Road to the highest point along the proposed conveyor route to the west (alignment uncertain). Total dynamic head requirements should be in the range where standard wet well or wet well/dry well package designs can be utilized. However, the Pine Canyon pump station will pump septic tank effluent to the leachfield site located about 320 feet above the portal, requiring low output, high head pumps not generally available in standard package designs.

Preliminary designs for both stations are constrained by the following requirements:

1) Each of two pumps must have output equal to 0.9 times peak flow (six times average daily water use); 70 gpm for Pine Canyon, 108 gpm for Dugout Canyon;
2) Minimum force main velocity of 2 feet per second at 0.9 times peak flow.

The truck conveyance system for Hoffman Creek has been discussed in Section 6.1.5. Building sewers and a buried holding tank will also be part of the conveyance system.

6.2.3 Wastewater Disposal Systems

Total Containment Lagoon

A nondischarging waste stabilization lagoon will treat wastewaters originating at Fish Creek, Dugout Canyon and Hoffman Creek portal areas. The preliminary lagoon design also includes capacity for chemical toilet wastes and water treatment plant chemical sludges.

Chemical toilet facilities designed for use in underground mines are available which allow disposal of wastes on a weekly basis. This could be accomplished by pumping the spent chemical wastes out of the units into a large transportable tank, removing the chemical holding tanks from the units for transport, or removing the units from the mines and transferring the wastes to a tank truck at the portal facility. Water treatment plant sludges will be pumped from plant waste dumps and transported by tank truck to the lagoon. Sludges and backwash water from the Fish Creek and Dugout Canyon plants could possibly be transported through the sewer outfalls to the lagoon, but this practice is not recommended due to potential clogging problems.

For the preliminary lagoon design, it was assumed that a multiple cell design is required to provide operational flexibility and avoid excessive odor problems. A three cell series operation system was chosen to control BOD loading and fit the slope and area limitations of the proposed site in the southeast quadrant of Section 28.

A BOD loading of less than 40 lb per acre per day was used for the primary cell. Also, based on reconnaissance level geotechnical investigations, an allowance for seepage, as well as net evaporation, was made in setting the overall area requirements of the lagoon system, as follows:

1) annual net evaporation of 27 inches;
2) daily seepage losses of 1/16 inch in the primary, 1/12 inch in the secondary, and 1/8 inch per day in the tertiary cell; and
3) maximum allowable seepage is 1/4 inch.

Preliminary geotechnical investigations also indicated that soils in the proposed site area would be suitable for both dike fills and bottom sealing. Experience in the region indicates the possibility of moisture sensitivity in the site soils. Soil testing in Phase II will show the need for special construction techniques, if any.

~~Septic systems proposed for the Central Facilities and Pine Canyon are of two different types. A standard septic tank and serial leachfield (Figure 6-2) is assumed to be suitable for the Central Facilities topography, ground water, and soils conditions. A soil percolation rate of 40 minutes per inch was assumed based on available information. Field tests in Phase II will be used to set the final leachfield designs for the Central Facilities. Two separate septic tank/leachfield systems were designed for the two water usage centers at the Central Facilities to reduce pipeline costs and keep the systems close to the main activity areas for ease of operation and maintenance.~~

7.1.10 Fish Creek/Dugout Canyon Lagoon

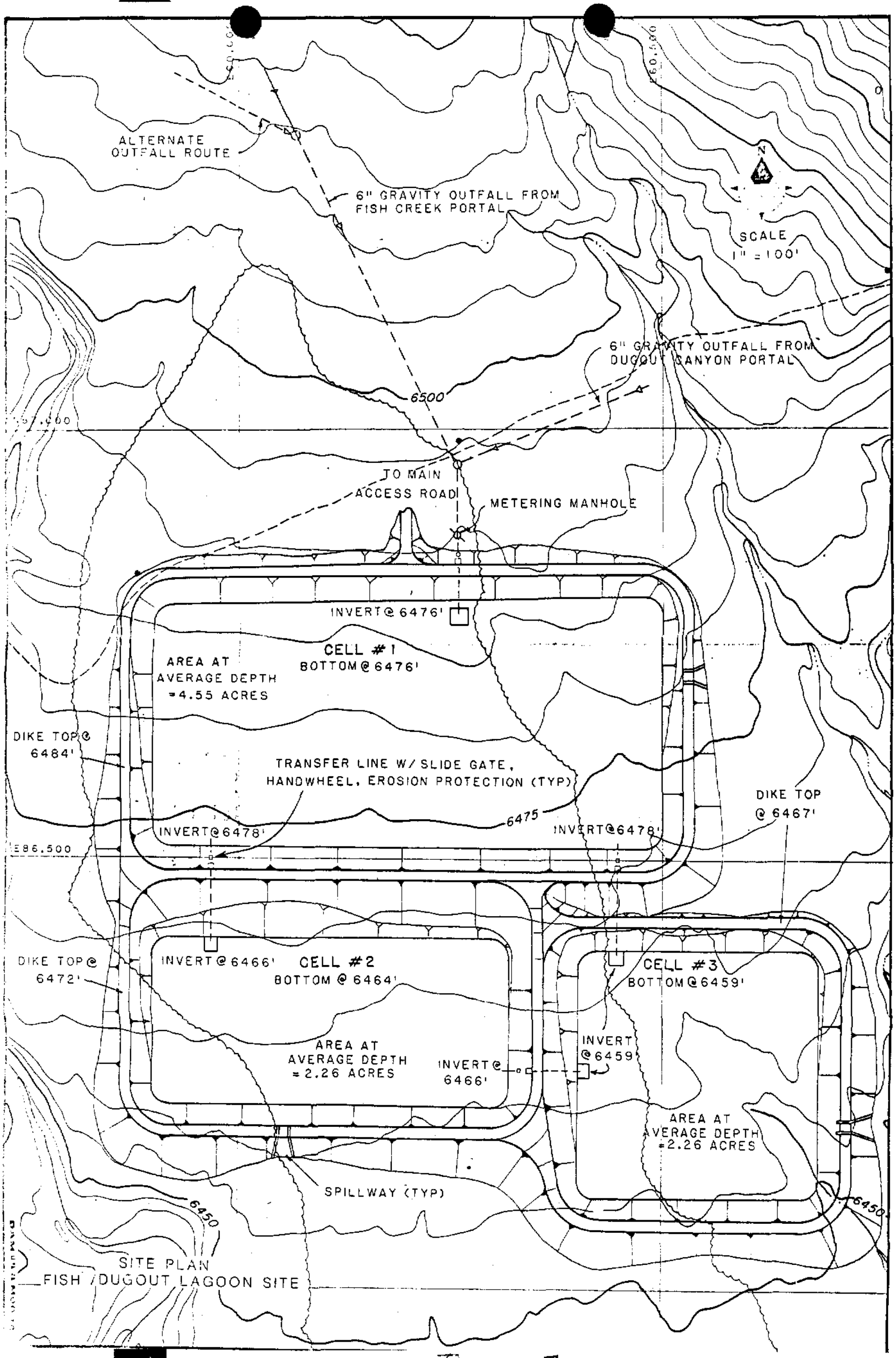
The portions of the project water and wastewater facilities which link and serve mine complex surface facilities are shown in Figure 7-1. A 9.1 acre lagoon facility in Section 28 will receive sewage from Fish Creek and Dugout Canyon portal areas. The lagoon is designed for 27 inches per year net evaporation and 1/12 inch per day net seepage over the three-cell system. Figure 7-24 shows a site plan of the lagoon, while Figure 7-25 illustrates the configuration of the lagoon excavation and transfer piping in a cross-sectional view.

Sewer outfalls from Fish Creek and Dugout Canyon are also shown in Figure 7-1. For Fish Creek flows, a six-inch gravity sewer will follow the proposed Fish Creek road down from the portal to an elevation of about 6740 feet, and then along the proposed conveyor route to the lagoon. The alternate route shown follows Fish Creek road down to an elevation of about 6625 feet and then across open terrain to the lagoon.

The six-inch Dugout Canyon sewer outfall will follow the canyon road down to an elevation of about 6700 feet, where a lift station will provide the increase in head necessary to transport Dugout Canyon flows through a four-inch force main along the proposed conveyor route. At an elevation of about 6580 feet along the conveyor, the force main will empty into a six-inch gravity sewer for transport to the lagoon. Portions of the force main traversing steep slopes will be attached to the conveyor above grade.

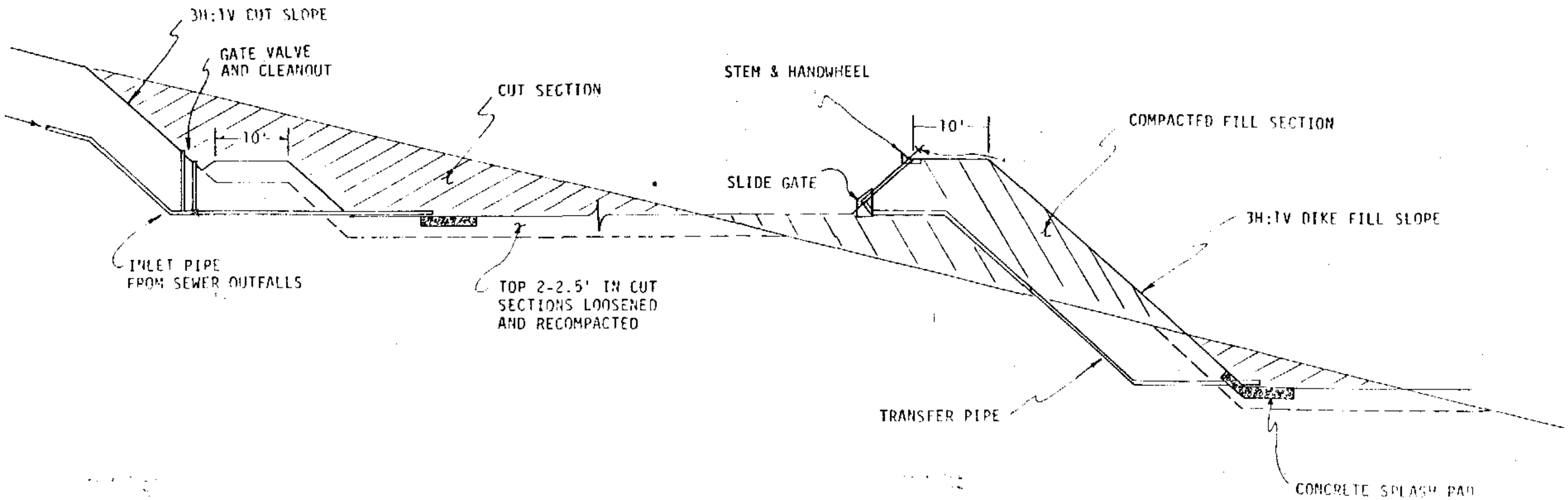
7.2 PRELIMINARY COST ESTIMATES

~~Estimated construction costs for the water storage, supply, treatment, wastewater treatment and sediment control facilities have been tabulated in Table 7-1. The table is divided into several categories representing different separable portions of the project. Each category has been subdivided into major work items. A different arrangement of categories and further subdivision will be necessary for construction bid items. A summary of project costs in shown in Table 7-2.~~



ALTERNATE OUTFALL ROUTE
6" GRAVITY OUTFALL FROM FISH CREEK PORTAL
SCALE
1" = 100'
6" GRAVITY OUTFALL FROM DUGOUT CANYON PORTAL
6500
TO MAIN ACCESS ROAD
METERING MANHOLE
INVERT @ 6476'
CELL #1
BOTTOM @ 6476'
AREA AT AVERAGE DEPTH = 4.55 ACRES
DIKE TOP @ 6484'
TRANSFER LINE W/ SLIDE GATE, HANDWHEEL, EROSION PROTECTION (TYP)
6475
INVERT @ 6478'
INVERT @ 6478'
DIKE TOP @ 6467'
E86.500
INVERT @ 6466'
CELL #2
BOTTOM @ 6464'
DIKE TOP @ 6472'
CELL #3
BOTTOM @ 6459'
INVERT @ 6459'
AREA AT AVERAGE DEPTH = 2.26 ACRES
INVERT @ 6466'
AREA AT AVERAGE DEPTH = 2.26 ACRES
SPILLWAY (TYP)
6450
6450
SITE PLAN
FISH / DUGOUT LAGOON SITE

FIGURE 7-24



TYPICAL LAGOON SECTION

DAMES & MOORE

FIGURE 7-25

Figure 7-25

~~The Pine Canyon system includes a standard septic tank with a mounded leachfield located above the portal facility on the best available site. The mounded leachfield will include a horizontal gravel bed design with a pressure dosing tank and pump to evenly distribute wastewater to the bed. The gravel bed and sand fill were sized for a loading rate of 1.24 gallons per day per square foot, corresponding to a medium sand with a percolation rate of about 15 minutes per inch. The mound design assumes that 1-3 feet of permeable (3-29 minutes per inch) natural soil occurs above bedrock at the site. Field testing in Phase II will be used to confirm these conditions although the exact natural soil percolation rate will not be used in design.~~

~~The submersible dosing pump was sized for a total dynamic head equal to three times the difference in elevation between the dosing tank low level and the gravel bed to account for friction losses in the distribution pipe, manifolds and perforated laterals.~~

6.3 SURFACE RUNOFF CONTROL

The proposed system for surface runoff control consists of sedimentation ponds for each of the portal areas, the Central Facilities, and for the coal preparation waste storage site. The system includes construction of diversion ditches at the Central Facilities and Pine Canyon. Contrary to the results of Chapters 4 and 5, diversion ditches were included in runoff control systems for Fish Creek and Dugout Canyon portals, at the request of Eureka Energy. No runoff control system has been developed for Hoffman Creek, since surface facility designs are not available. Also, the coal preparation waste storage site has not been designed, so the information shown is still based on conceptual design parameters.

6.3.1 Hydrologic Conditions

To calculate the required sedimentation pond capacities, it was necessary to specify the hydrologic conditions of the sites. Thus, from limited soils and ground cover information, each drainage area was classified according to the hydrologic soils groups defined by SCS. Table 6-4 presents the assigned hydrologic soils group, hydrologic condition and resulting curve number for each location. In those areas where soils information was insufficient to assign a hydrologic soil group, those soils with high runoff potential were assumed for conservative design purposes.

6.3.2 Runoff Storage Volume

Runoff volumes were determined using the SCS method for natural drainage areas, and the Rational Method for the disturbed portions. Details of the runoff calculations are presented in Appendix A.

To apply the Rational Method to the disturbed or developed portions of the drainage areas, it was necessary to determine the coefficient of runoff, C. This coefficient accounts for infiltration, depression storage and channel storage, and depends upon rainfall intensity, land use, percentage of impervious area, soil type and vegetative cover. A close examination of these factors was undertaken to refine the C values that were used for the analysis of alternatives. On the steeper portal slopes of Fish Creek, Dugout Canyon and Pine Canyon a runoff coefficient of 0.82 - 0.83 was calculated, whereas at the Central Facilities, where the slopes are not so steep, a coefficient of 0.74-0.76 was used.

6.3.3 Sediment Transport

Sediment loads from the disturbed portions of each drainage area were calculated using the Universal Soil Loss Equation (USLE). Details of the calculations are given in Appendix A. At each location the total disturbed area was subdivided into smaller units and the gradient, slope-length, K factor, and size were determined for each unit. Soil loss was estimated for each sub unit and summed to produce a total soil loss estimate. Because the USLE was applied only to the disturbed areas where ground cover is minimal, the cropping management factor, C, was held constant at 1.0. It was also necessary to estimate soil loss from natural rangeland areas where the USLE is not applicable. Analysis of

TABLE 6-4

HYDROLOGIC SOIL CONDITIONS

LOCATION	HYDROLOGIC SOIL GROUP	HYDROLOGIC CONDITION	CN
Fish Creek	D	Poorest	91
Dugout Canyon	D	Poorest	91
Central Facilities - East	B	Poor to Medium	64
Central Facilities - West	B	Poor to Medium	64
Pine Canyon - North	C	Medium	73
Pine Canyon - South	C	Medium	73
Coal Prep Waste Storage Site	C	Poor	68

Source: USDA, Soil Conservation Service, National Engineering Handbook, 1972 wind, sheet and rill erosion data obtained in the National Resource Inventories indicates that rangelands in the state of Utah experience an average of 2-2.9 ton/ac/yr soil loss. Therefore, to determine sediment loss volumes from the natural portions of each drainage area a conservative value of 3 tons/ac/yr was used in the design calculations. The sediment loads from the disturbed and natural areas were summed and the total sediment storage volume required at each location is shown in Table 6-5.

6.3.4 Sediment Pond Capacities and Cleanout Rates

The calculated sediment storage and runoff storage volumes were used to determine total pond capacity requirements and the resultant sizes. These values are presented in Table 6-5. In Chapter 5, the recommended pond system included a scheme for pond sediment cleanout for each three-year interval. The total sediment storage volumes associated with this scheme were calculated and are shown in Table 6-5.

Total pond area requirements for Central Facilities East differ from the size used in the conceptual analysis (Table A-5). The sharp increase in Central Facilities East pond size stemmed from detailed sediment calculations for the coal storage piles. This change had no impact on the relative rankings of the alternatives.

6.3.5 Runoff Control Designs

The criteria used in the design of the sediment ponds and diversion ditches are listed in Table 6-6. As stated in Appendix A these parameters are based on regulatory requirements and requirements specific to this project. Spillway designs will involve principal and emergency outlet pies sized to pass the 25-year storm. Typical dike and diversion ditch cross-sections are presented in Chapter 7.

TABLE 6-5

SEDIMENT POND DESIGN DATA

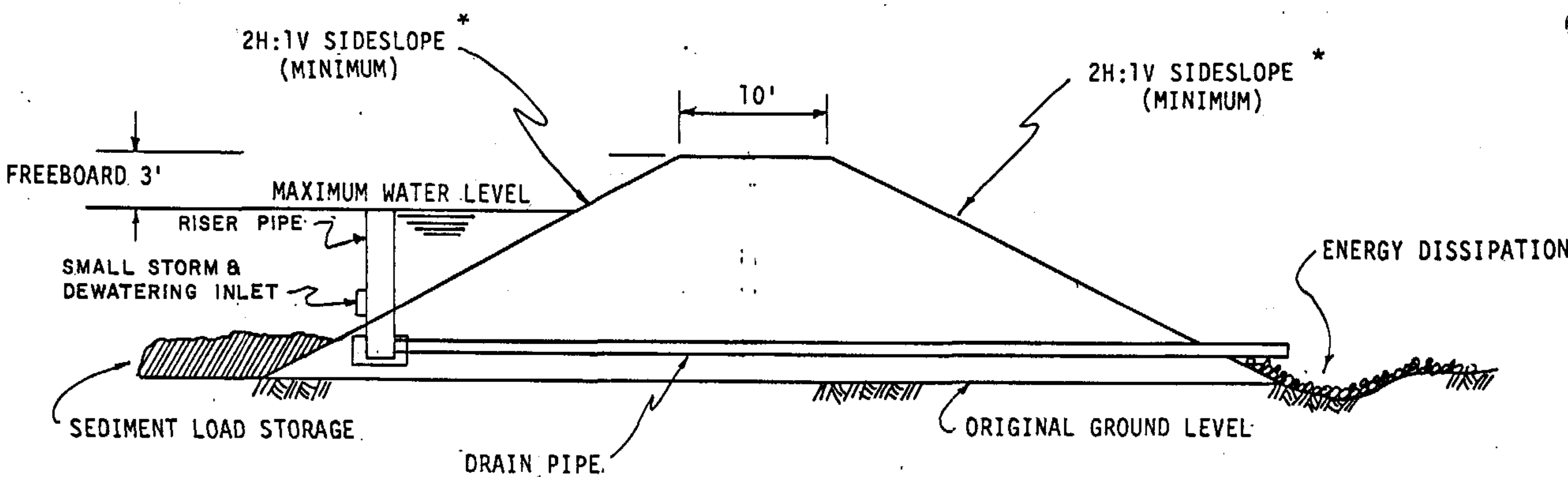
LOCATION	DRAINAGE AREA (acres)	RUNOFF VOLUME (ac-ft/yr)	SEDIMENT VOLUME (ac-ft/yr)	WATER DEPTH (ft)	SEDIMENT POND SIZE[1] (acres)
Fish Creek	12.31	1,230	0.660	10	0.194
Dugout Canyon	12.70	1.270	0.570	10	0.188
Main West	30.10	2.020	0.290	8	0.294
Main East	39.90	1.970	4.675	10	0.661
~~Pine Canyon - North~~	~~33.95~~	~~0.201~~	~~0.056~~	~~6~~	~~0.044~~
~~Pine Canyon - South~~	~~72.04~~	~~0.269~~	~~0.076~~	~~6~~	~~0.060~~
Coal Prep Waste - East	101.60	5.410	0.117	10	0.553
Coal Prep Waste - South	190.01	12.120	0.205	10	1.232

[1]At mid-water depth

TABLE 6-6

DESIGN CRITERIA

TYPE STRUCTURE	DESIGN PARAMETER	VALUE
Sedimentation Ponds	Hydraulic design	24-hr detention of 10-hr 24-hr storm
	Pond freeboard	2 ft
	Pond water depth	10 ft (maximum)
	Embankment top width (H<15 ft)	10 ft
	Embankment sideslopes	2H:1V (minimum)
	Combined upstream/downstream sideslopes	5H:1V
	Manning's Coefficient	.02
Diversion Ditches	Hydraulic design	10-yr 24-hr storm
	Freeboard	0.5 ft
	Ditch sideslopes	½H:1V (minimum)
	Ditch bottomwidth	(varies)
	Maximum velocity (unlined)	3 fps
Spillway Structures	Hydraulic design	25-yr 24-hr storm



* MINIMUM COMBINED SIDESLOPE 5H:1V

SEDIMENT POND

TYPICAL EMBANKMENT CROSS-SECTION
(NOT TO SCALE)

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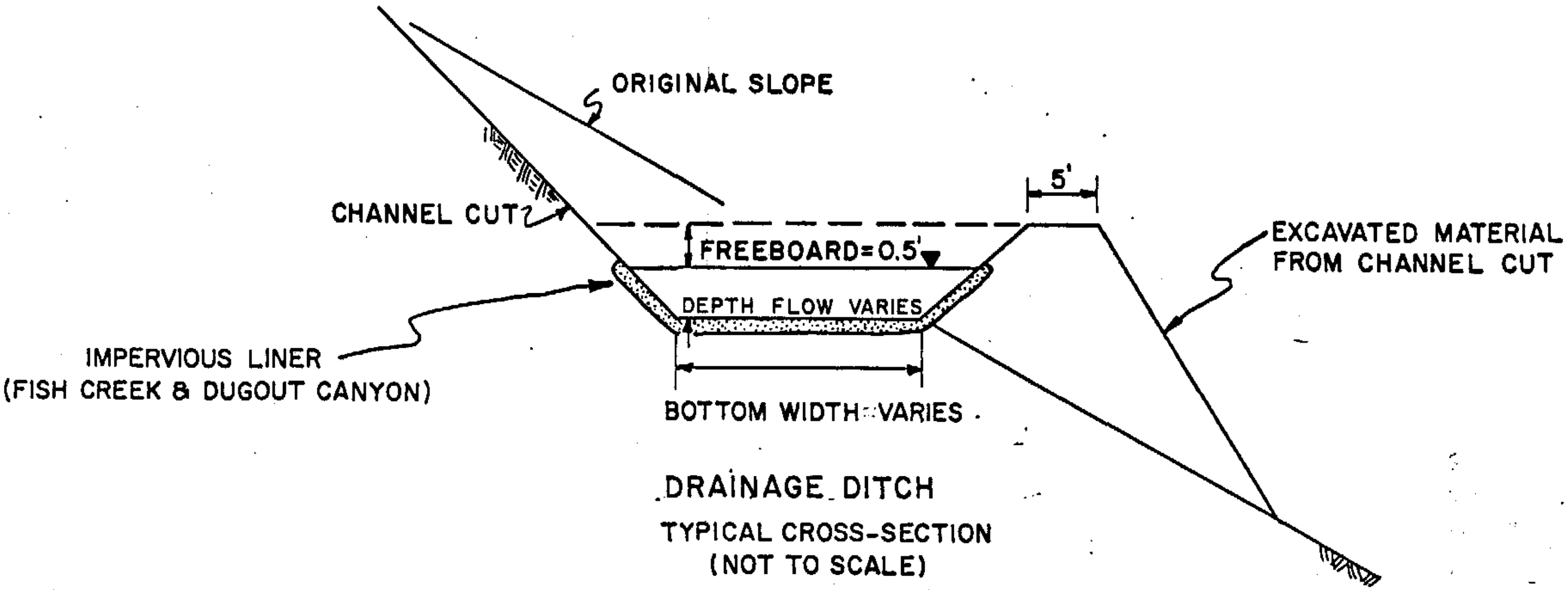
FIGURE 7-1



DRAINAGE DITCH

TYPICAL CROSS-SECTION
(NOT TO SCALE)

DAMES & MOORE

FIGURE 7-13

A method of relating precipitation, apparent evapotranspiration, and runoff for several hydrologically similar watersheds was used to calculate average annual runoff.

This method utilized a water budget approach. The annual water budget for a drainage basin including its soil and aquifer storage components is:

$$P - Q - E_t - G = \Delta S$$

in which P = precipitation on the drainage basin

Q = streamflow from the drainage basin

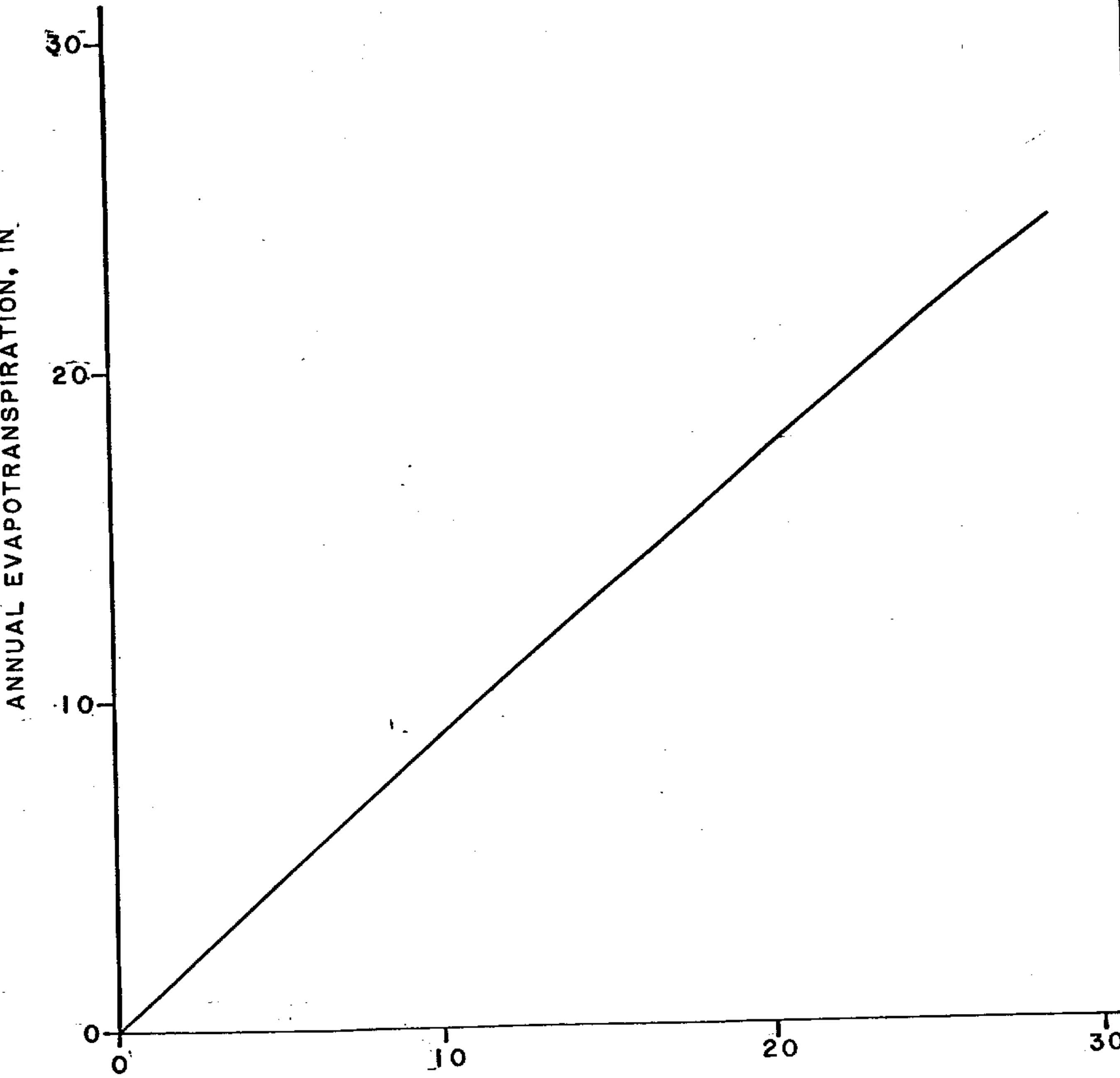
E_t = evapotranspiration from the drainage basin

G = net ground water flow from the drainage basin's aquifers that is not included in streamflow

ΔS = change in the amount of water stored in the drainage basin's soil and aquifer

On an annual basis, it can be assumed that ΔS is approximately zero for small watersheds. Usually, the subsurface flow out of the drainage basin is a small fraction of the evapotranspiration. Then, it is convenient to call the sum of E_t and G the apparent evapotranspiration. The average annual apparent evapotranspiration for the selected small drainage basins are given in Table A-2. These values were obtained by subtracting the average annual streamflow from the average annual precipitation.

Annual drainage basin evapotranspiration is closely related to annual precipitation. In years when the precipitation is low, evapotranspiration is a large percentage of the precipitation. In years when the precipitation is high, drainage basin evapotranspiration is a smaller percentage of the annual precipitation. The relation between annual precipitation and annual evapotranspiration for the seven drainage basins is shown in Figure A-3.



APPARENT ANNUAL EVAPOTRANSPIRATION
(FROM SELECTED USGS DRAINAGE BASINS)

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FIGURE A-3

TABLE A-2

PRECIPITATION AND EVAPOTRANSPIRATION

Drainage Basin	Precipitation Station	Adjustment Factor	Drainage Basin Precipitation in/year	Apparent Evapotranspiration in/year
West Fork Avintaquin Creek near Fruitland	Soldier Summit	0.96	-	-
Sowers Creek near Duchesne	Nutters Ranch	1.60	18.94	17.88
Minnie Maud Creek near Myton	Price Warehouses*	1.50	14.36	12.16
White River near Soldier Summit	Soldier Summit	1.06	-	-
White River below Tabbyune Creek	Soldier Summit	1.02	-	-
Beaver Creek near Soldier Summit	Scofield Dam	0.93	13.50	11.47
Willow Creek near Castle Gate	Price Warehouses*	1.58	15.23	13.66

*Price Warehouses includes the records of Price and Price Game Farm stations.

An estimate of annual precipitation for drainage basins near the project site is given in Table A-3. The period of record is from calendar years 1931 to 1977, inclusive. The estimate was based on the precipitation record at Price Warehouses gage for which the long-term mean annual precipitation is 9.40 inches. The ratio between mean annual precipation at the Price Warehouses gage to annual precipitation in the small drainage basins is 1.33. The estimate of the mean annual precipitation on the small drainage basins is 12.50 inches.

FISH CREEK?

Corresponding estimates of annual streamflow for the Soldier Creek, Dugout Creek and Pace Creek drainage basins are also given in Table A-3. These estimates were generated by using the annual precipitation values and the evapotranspiration versus precipitation curve presented in Figure A-3.

For the simulated streamflow record given in Table A-3, average annual runoff is 1.5 inches which appears to be the best estimate of annual streamflow for the project drainage basins.

A.5 DETERMINATION OF ANNUAL FLOW RECORDS

The annual runoff sequence generated in Table A-3 was compared to actual sequences in nearby drainage areas. The coefficient of variation is 0.36 which is much less than for the measured streamflows in other drainage basins. USGS stations in the vicinity generally have coefficients of variation of from 0.5 to 0.9; therefore, the synthetic streamflows of Table A-3 do not appear to represent a good record, and are not used to generate the annual streamflow sequence.

Another runoff sequence for the drainage basins in the vicinity of the project site was calculated using the White River streamflow records. Annual runoff for the White River for the period of record 1940 to 1978 is given in Table A-4. The coefficient of variation for this record is 0.59. Therefore, it is assumed to represent a better runoff sequence.

reduce reservoir size and project costs, but requires better flow records than those currently available. Risk factors and records are further discussed in Appendix B.

A.9 RUNOFF CONTROL

Runoff storage is required for the 10-year 24-hour storm volume, with a theoretical detention time of 24 hours. Diversion ditches are to be designed for a 2-year flow. For the natural areas, runoff was calculated using the method described in SCS National Engineering Handbook, Section 4, Hydrology. From this information natural runoff storage requirements and diversion ditch designs were calculated. The Rational Method was used to determine runoff volumes from the disturbed areas.

Sediment loads were calculated using the Universal Soil Loss Equation (USLE). Factors used in the USLE equation and to determine sediment yield were obtained from background materials referenced in the OSM regulations. The basic equations and values are as follows:

$$A = R\ K\ L\ S\ C\ P$$

where A = sediment transfer rate; tons per acre per year (t/ac/yr)

R = erosion index = 20 t/ac/yr

K = soil erodibility factor = 0.24

L = slope length factor

S = slope factor

C = cropping management factor; from 0.18 to 0.23

P = conservation practice factor = 1.0

$$Y = A\ Dr\ Ad$$

where Y = sediment yield, acre-feet per year (af/yr)

A = sediment transfer rate, t/ac/yr

Dr = delivery ratio = 0.5

Ad = area of disturbed terrain, acres

0.00046 = conversion from weight to volume, acre-feet per ton (density = 100 lb/ft^3)

The soil erodibility factor K was taken from SCS soil survey data. An average K factor was used which is considered to be representative of the common soil type of the area. The delivery ratio (Dr) was based on sheet or rill erosion on very small drainage basins, taken from results of studies reported in Sedimentation Engineering (ASCE M&R No. 54). A delivery ratio of 0.5 corresponds to a drainage basin of 0.05 square miles. Although the delivery ratio is known to vary widely based on the sources, texture of eroded material, basin characteristics, and depositional areas, the average value is considered the best available estimate for the range of conditions in the project area. The cropping management factor, C, varied from 0.18 for the main facility areas to 0.23 for the steeper portal slopes. This factor is based on a representative type of vegetative cover and 20-40 percent ground cover of the project area. Conservation practice factor, P, was held constant at 1.0 since the use of erosion control practices in the reclamation plan is uncertain. An average topographic factor, LS, was determined by calculating an average gradient and slope length for each of the drainage areas. Sediment volume calculations were generalized for the alternative analysis.

Neither evaporation nor percolation was considered in the calculation of pond storage volume. Although small volumes will be lost, these losses were not considered in calculating conceptual design volumes. Design criteria and assumptions are based on regulatory design requirements, as well as requirements specific to this project.

Several methods were used to determine required sedimentation pond volumes. Permanent sediment storage, as well as temporary storage volumes, were calculated and compared to storage requirements based on federal regulations. Conceptual designs and cost comparisons have been prepared for sedimentation ponds at each of the site areas. The use of diversion structures to limit runoff and sediment losses was considered and ponds were sized with and without the use of these diversions.

The following alternatives were considered for pond design:

Alternative 1: Sedimentation pond volume based on accumulated sediment volume from drainage area over the 40-year life of pond; and runoff storage for 10-year 24-hour storm.

Alternative 2: Sedimentation pond volume based on accumulated sediment volume from drainage area to the pond for 3 years, with 60 percent sediment removal at the end of each 3 year period; and runoff storage for 10-year 24-hour storm.

Alternative 3: Sedimentation pond volume based on federal requirement of 0.1 acre-feet for each acre of estimated area within the upstream drainage area; sediment removal as in Alternative 2; and runoff storage for 10-year 24-hour storm.

Alternatives 4, 5, and 6 are the same as alternatives 1, 2 and 3 respectively, but diversion ditches are included in the design to convey runoff and sediment from undisturbed areas away from the disturbed areas and sedimentation pond, thereby decreasing the required design volumes.

Table A-7 presents the cost comparison for each alternative pond system. Costs are computed as the present worth of expenditures and are suitable for comparing alternatives. The quantities of excavation, embankment fill and sediment removal were estimated for each alternative and a reasonable unit price applied to calculate total costs. Those costs common to each alternative were not included in the comparative costs and therefore the costs shown cannot be considered total costs. Common costs included diversion ditches at the downhill end of the disturbed area to direct sediment-laden runoff to the ponds.

Cost analysis, using Table A-7, indicates that the Alternative 2 pond design is cost effective at every pond site. Although cost comparison is an important method of alternative selection, several other factors play a significant role in the selection process. The steeper

TABLE A-7

SEDIMENTATION PONDS
TOTAL COSTS-PRESENT WORTH[1]

LOCATION	NO DIVERSIONS ALT 1	NO DIVERSIONS ALT 2	NO DIVERSIONS ALT 3	DIVERSIONS ALT 4	DIVERSIONS ALT 5	DIVERSIONS ALT 6
Fish Creek						
Pond Size	3.82	.717	1.026[2]	.223	.179	.305
Total Cost	143,020	30,730	39,020	52,050	360,000	55,300
Dugout Creek						
Pond Size	5.06	.940	1.318[2]	.203	.177	.305
Total Cost	188,160	40,030	49,880	51,250	350,000	55,290
Cen. Fac./East						
Pond Size	.446	.348	.618	.358	.314	.590
Total Cost	19,140	14,680	25,010	17,510	15,820	26,410
Cen. Fac./West						
Pond Size	.407	.288	.519	.286	.268	.506
Total Cost	16,880	12,350	21,250	16,390	15,710	24,900
Coal Prep. Waste Storage-East						
Pond Size	.850	.581	.98			
Total Cost	34,020	24,040	39,150			
Coal Prep. Waste Storage-West						
Pond Size	1.80	1.29	2.23			
Total Cost	69,240	51,010	78,570			

[1] Total costs represent dollars. Pond sizes are expressed in acres.
[2] Calculations are based on disturbed area defined as the entire upstream drainage area.

slopes in the portal areas limit the allowable size of the sedimentation ponds. For this reason Alternative 1 cannot be used for the design in the portal areas. In the main facilities area, however, size is not a limiting factor.

Benefits not reflected by the costs presented are associated with the use of diversion ditches for runoff and sediment control. Diversion ditches are important for protection of the facilities and equipment against floods. Diversion ditches not only limit the size of sedimentation ponds required, but also protect against the direct flow of runoff and sediment across the impervious facility areas which could cause other environmental problems. The sedimentation pond is also protected against possible washout under extreme flood conditions.

In the portal areas at Fish Creek and Dugout Canyon where steeper slopes occur the problems associated with the use of drop structures and steep channels are reflected by high costs which were not included in the alternative cost analysis.

Because of area topography and drainage patterns, diversion structures were not considered useful in the design of sedimentation ponds for the coal preparation waste storage sites. Structures could not be placed so as to divert significant volumes of runoff and sediment from undisturbed areas away from the disturbed areas. For this reason, Alternatives 4, 5, and 6 were eliminated from the alternative designs at these sites.

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February 22, 1982

Mr. Jerry Davis
Pacific Gas & Electric Company
77 Beale Street - Room 2619
San Francisco, California 94106

Dear Jerry:

This letter is in response to your request (2/16/82) for a "brief" written explanation of suspended sediment concentrations presented in the permit application for Eureka Energy's Book Cliffs project, as prepared by Dames & Moore.

The question, as presented to me, is why do the effluent concentrations from the sedimentation ponds exceed the DOGM standard of 45 mg/l. The answer is complex. I will try to answer the question on two fronts: (1) the regulations, themselves; and, (2) a more thorough explanation of the analyses that produced the numbers in question.

SUMMARY

1. The sediment concentrations presented do indeed exceed the regulatory standard. However, the regulatory standard is very commonly exceeded by streams in a natural, undisturbed condition, and the regulatory standard, therefore, seems unreasonable.
2. The numbers presented are not intended to be taken as definitive. They are intended to show that the sedimentation ponds significantly reduce the suspended sediment loads of the streams involved. The trap efficiencies are on the order of 60 to 70 percent when 24-hour detention times are provided.
3. An accurate determination of sediment concentrations is not practical for design purposes. Flows and sediment concentrations would need to be sampled during rare storm events (e.g., 5-, 10- and 25-year storms) to provide comprehensive data.

Use USLE ! results

Such sampling would require many years (or extreme luck).

REGULATIONS

The allowable effluent limitation of 45 mg/l of suspended sediment from a 10-year, 24-hour storm was adopted by the DOGM in the form of OSM regulations. The OSM has since deleted and revised many of the regulations, and is in the process of developing new effluent limitations. I was confused by the changing regulations when designing the sedimentation ponds and consulted John Nadolski, a hydrologist for the OSM in Denver. He suggested that I use a 24-hour detention time for design criteria since the OSM regulations were unclear and changing. Therefore, I used the detention time as the primary design parameter, and ignored the effluent limitation.

As far as I know, the Utah DOGM has not followed the OSM lead in re-evaluating the regulations. I strongly feel that they should because, as evidenced in the following discussion, the 45 mg/l value seems unrealistic for Utah. That value is normally exceeded by undisturbed, natural streams under normal flow conditions, and it is greatly exceeded under storm conditions. To illustrate this, I have attached Table 5 from:

Mundroff, J.D., 1972. Reconnaissaince of Chemical Quality of Surface Water and Fluvial Sediment in the Price River Basin, Utah, Utah Division of Natural Resources, Technical Publication No. 39, in conjunction with the USGS.

The table presents 97 sediment concentrations from various streams in the Book Cliffs region, and 92 of them exceed 45 mg/l. From this table, it is reasonable to conclude that streams in the Price River basin rarely meet the DOGM standard. I believe the on-site data collected by Eureka Energy shows a similar situation exists on the project area.

As a final note on the regulations, I would like to point out that a 10-year, 24-hour storm is a reasonably severe hydrologic event in comparison to the flows usually encountered in streams. As such, a 10-year storm would probably result in even higher flows and sediment concentrations than those presented in Table 5.

It appears that natural, undisturbed streams regularly violate the DOGM standards, often by one or two orders of magnitude. Therefore, the standards seem unreasonably strict.

ANALYSES

The average effluent concentrations of the sedimentation ponds during a 10-year, 24-hour storm are extremely high, as pointed out by the DOGM. They ranged from a high of 161,000 mg/l to a low of 11,000 mg/l, far in excess of the 45 mg/l standard. However, the design criteria used, a 24-hour detention time, is met.

The sediment concentrations presented are unrealistic; they are presented to illustrate relative improvements only, not actual numbers. For example, sedimentation pond SV-4 has the highest average effluent concentration: 161,000 mg/l. But it also had the highest peak influent concentration, 1,440,000 mg/l, which is attenuated to a peak effluent concentration of 524,000 mg/l, a reduction of 64 percent. The overall trap efficiency of the basin is 68 percent. Thus, the basin is very effective in reducing both sediment loads and reducing peak concentrations. Those points (and the detention times) are what the computer model was intended to show.

As to the numbers, themselves, they are unrealistic for a number of reasons. They are synthetically generated, not based on field data. The total sediment load is determined by the USLE as required by the original OSM regulations. The USLE is used as a sediment predictor because it is, essentially, the only method commonly available. The sediment loads derived with the USLE are not necessarily accurate estimates. It was developed for very small agricultural plots, and using it for relatively large watersheds calls for extrapolation beyond its capabilities. It is independent of flow. For example, basin SV-4 had a sediment load of 91.6 tons, and a 10-year, 24-hour flow volume of 0.14 acre-feet. On the basis of weight, the flow was 48 percent sediment. Converting tons to milligrams and acre-feet to liters yields an average influent concentration of 482,000 mg/l, clearly an unrealistic value. The importance of the model is to show that the average effluent concentration is only 32 percent of the influent concentration.

Thus, the major cause of the unrealistic numbers is the use of the USLE for sediment load. However, two other factors are also involved. The DEPOSITS model creates a sedimentgraph proportional to the hydrograph when a sediment load is given. Only extensive field data could confirm or deny whether or not this is applicable to the watersheds under study. Secondly, to determine how much sediment was trapped in the pond, a grain-size distribution of the suspended sediment was required. A few distributions had been determined by Eureka Energy, and these were used as input to DEPOSITS. However, distributions based on small flow events usually have much higher percentages of clay than those of rare storm events, thereby producing conservative trap efficiencies.

It is evident from the above discussions that the DEPOSITS model requires extensive field data to perform accurate calculations. However, to produce truly accurate results, a paradox occurs. To have accurate data for input, the project should already be in place, and a 10-year, 24-hour storm must be sampled. This is not possible in the design phase of a project, and one makes do with the limited data available. It is probable that, if the same calculations were done for the watersheds in their undisturbed condition, the resulting sediment concentrations would be similar to those in the disturbed condition. These calculations were not performed for two reasons. First, the analyses would be extensive. Secondly, the results would not be conclusive. The sediment concentrations would be unrealistic, as they are in the analyses already done, thereby showing that, with the limited data available, the sediment loads and concentrations cannot be accurately predicted.

I hope this discussion adequately addresses the questions raised by the DOGM.

Sincerely,

DAMES & MOORE

Richard A. Urbanowski

Richard A. Urbanowski
Staff Engineer

RAU:smh
cc: Mr. A. Prakash - D & M - DN
Mr. S. F. Tarlton - D & M - DN
Mr. J. Keaton - D & M - SL
Mr. W. J. Gordon - D & M - SL
Enclosure

Table 5.—Periodic determinations of suspended-sediment discharge and particle size—Continued

Map number	Sampling site	Date of collection	Time	Temperature (°C)	Discharge (cfs)	Suspended sediment: Concentration (mg/l)	Suspended sediment: Discharge (tons per day)	Percent finer than indicated size, in millimeters: 0.002	0.004	0.008	0.016	0.031	0.062	0.125	0.250	0.500	1.000	2.000	Methods of analysis
32	Unnamed creek at Highway 10, near Price	8-29-69	1215	18	75	130,000	26,300	32	36		59		93	100					C,P,V,W
33	Miller Creek at Highway 10, near Wellington	5-15-70	1155	15	.02	51	(1/)												
34	Miller Creek near Wellington	8-29-69	1210	17	21	4,700	266												
		8-29-69	1400	20	20	5,090	275	19	19		31		73	98	100				C,P,V,W
35	Outflow drain from Olsen (Mounds) *Reservoir*, near Mounds	6- 3-70	1140	16.5	12	57	1.8												
36	Washborad Wash near Mounds	6- 3-70	1145	16	24	1,820	118												
37	Meads Wash at Highway 6 and 50, at Price	8-29-69	1300	19	22	49,000	2,910	39	53		84		98	100					C,P,V,W
		5-15-70	1210	14	1.2	52	.2												
38	Cardinal Wash at mouth, at Highway 6 and 50, at Price	8-29-69	1510	18	7.5	22,700	460												
		5-15-70	1230	18	1.5	84	.3												
41	Coal Creek at Highway 6 and 50, near Wellington	8-27-69	1730	28	1.8	101	.5												
		5-15-70	1320	22.5	7.5	146	3.0												
		6- 3-70	1040	17	4.0	222	2.4												
44	Soldier Creek at Highway 6 and 50, near Wellington	11- 6-69	1330	8	8.0	837	18												
		5-15-70	1325	18.5	6.0	144	1.8												
		6- 3-70	1045	17	2.0	122	.7												
45	Price River at Highway 297, at Wellington	6- 3-70	1025	15	145	875	343												
46	Price River at Highway 296, at Wellington	8-28-69	1730	19	28	167	13												
		8-29-69	1400	20	40	111,000	12,000	18	20		30		70	99	100				C,P,V,W
		8-29-69	1430	19	300	49,700	40,300												
		3-25-70	1150	7	60	7,240	1,170	41	52		75		95	100					C,P,V,W
47	Price River near Wellington	6-28-58	1210	23	63	256	44						82						S
		7-16-58	1220	22	40	101	11						76						S
		7-30-58	1010	21	34	38	3.5						84						S
		8-14-58	1230	26	28	70	5.3						95						S
		3-25-70	1200	5	60	6,950	1,130												
		5-15-70	1305	17	60	67	11												
		6- 3-70	1030	15	145	1,850	724												
52	Desert Seep Wash at outflow from Desert Lake, near Elmo	9-25-69	1600	19	6	983	16	45	61		90		100						C,P,V,W
		5-15-70	1525	22	9	146	3.5												
		6- 3-70	1300	19	18	24	1.2												
53	Desert Seep Wash near Elmo	6- 3-70	1200	16	26	165	12												
55	Price River near Mounds	9-25-69	1500	20	25	239	16												
		11- 6-69	1430	5	42	160	18												
		6- 3-70	1135	18	160	2,240	968												
64	Grassy Trail Creek below junction with Dugout and Rock Creeks, near Dragerton	5-15-70	1340	21	.25	86	.06												
67	Icelander Creek at Highway 6 and 50, near Dragerton	5-15-70	1355	24	1.1	48	.1												
68	Price River above Camel Wash, near Woodside	11- 5-69	1430	7	62	153	26												
69	Price River below Summerville Wash, near Woodside	8-29-69	1345	18	130	78,800	27,700	34	43		74		99	100					C,P,V,W
71	Price River at mouth, near Green River	11- 6-69	1415	4	64	203	35												

1/ Trace (less than 0.05).

Table 5.—Periodic determinations of suspended-sediment discharge and particle size

See plate 1 for locations of map numbers.
Methods of analysis: C, chemically dispersed; P, pipet; S, sieve; V, visual accumulation tube; W, in distilled water.

						Suspended sediment													
								Percent finer than indicated size, in millimeters											
Map number	Sampling site	Date of collection	Time	Temperature (°C)	Discharge (cfs)	Concentration (mg/l)	Discharge (tons per day)	0.002	0.004	0.008	0.016	0.031	0.062	0.125	0.250	0.500	1.000	2.000	Methods of analysis
7	White River below Tabbyune Creek, near Soldier Summit	8-28-69	1730	18	6.4	180	3.1												
		5-15-70	0800	2.5	145	501	196												
		6- 3-70	0840	8.5	5.8	81	13												
8	Price River above White River, near Colton	9-25-69	1020	10	110	46	14												
		5-15-70	0820	3.0	150	16	6.5												
9	White River above Price River, near Colton	9-25-69	1030	9	7	222	4.2												
10	Price River below White River, near Colton	8-28-69	1800	19	175	73	34												
		9-25-69	1015	12	120	48	16												
		5-15-70	0845	4.5	230	394	245												
		6- 3-70	0855	10.5	320	101	87												
11	Beaver Creek near Soldier Summit	5-15-70	0855	3.5	16	809	35												
		6- 3-70	1605	19	10	768	21												
12	Kyune Creek near Colton	5-15-70	0905	2.0	9	637	15												
13	Horse Creek near Castle Gate	5-15-70	0920	7.5	.3	119	.1												
14	Diamanti Canyon at Highway 6 and 50, near Castle Gate	5-15-70	0935	3.5	.8	268	.6												
15	Ford Creek at Highway 6 and 50, near Castle Gate	8-28-69	1600	14	.2	234	.1												
		5-15-70	0940	3.5	1.1	120	.4												
16	Willow Creek near Castle Gate	5-15-70	1010	5.5	9	308	7.5												
17	Willow Creek at mouth, at Castle Gate	8-29-69	1940	18	5.9	96	1.5												
		3-25-70	0900	0	4.0	844	9.1												
		5-15-70	1020	7.0	10	184	5.0												
		6- 3-70	1550	19	10	92	2.5												
18	Price River above Willow Creek, at Castle Gate	5-15-70	1030	5.0	270	740	539												
19	Price River near Heiner	6-28-58	0950	17	334	158	142						51						S
		7-16-58	1045	17	289	70	55						60						S
		7-30-58	0820	18	244	50	33						58						S
		8-14-58	1045	19	219	33	20						62						S
		8-28-69	1950	18	174	619	291												
		9-25-69	1120	12	109	115	34												
		3-25-70	0910	0	109	1,880	553												
		5-15-70	1030	5.0	280	587	444												
		6- 3-70	1555	16	360	308	299												
21	Spring Canyon Creek at Helper	8-29-69	2015	19	.6	2,260	3.6												
		9-25-69	1130	14	.1	158	(1/)												
		11- 6-69	1135	5	1.0	502	1.4												
22	Price River at Spring Glen	5-15-70	1045	7.5	280	695	525												
23	Gordon Creek near Carbonville	3-25-70	1045	4.5	4.0	5,740	62	64	81		100								C,P,W
		5-15-70	1100	14	3.0	1,650	13												
		6- 3-70	0935	16.5	4.5	3,870	47												
24	Pinnacle Creek near Price	5-15-70	1135	14	.2	45	.02												
25	Carbon Canal near Price	8-29-69	0845	17	100	6,470	1,750												
		5-15-70	1115	8.5	135	701	256												
		5-15-70	1115	8.5	135	978	356												
		6- 3-70	0930	12	150	462	187												
26	Carbon Canal at Highway 10, near Price	8-29-69	0930	17	75	1,610	187												
27	Carbon Canal at Highway 10, near Wellington	8-29-69	0950	17	70	1,790	338												
28	Carbon Canal near Elmo	8-29-69	0900	16	60	1,110	180												
29	Price River at Price	9-25-69	1140	16	10	96	2.6												
		3-25-70	1100	4	40	3,450	373	52	67		92		99	100					C,P,V,W
		5-15-70	1140	12	60	268	43												
		6- 3-70	0955	13.5	140	559	211												
30	Price River near Price	6- 3-70	1005	14	140	795	301												
31	Drunkards Wash at Highway 10, near Price	8-29-69	0920	15	2.5	408	2.8												
		8-29-69	1230	14	150	186,000	75,300												
		8-29-69	1230	15	150	183,000	74,100	28	33		57		88	99	100				C,P,V,W
		9-25-69	1210	18	1.5	66	.3												
		5-15-70	1200	16	1.5	281	11												

and Dugout Reservoirs, more detailed information is available. Mean pan evaporation for May-October is 52 inches, while annual pan evaporation is 65 inches. The pan coefficient is 0.69. Therefore, the mean total evaporation is 44.83 inches per year. To obtain monthly estimates, comparisons were made to evaporation data from weather stations nearest the project site. Table A-6 presents the estimated mean total evaporation for Anderson and Dugout Reservoirs, along with estimated standard deviations.

For the purpose of estimating reservoir size to compare alternatives, a rough estimate of seepage losses has been made. Based on Dames & Moore's prior experience with Mancos Shale, and on preliminary surficial geology inspections of the sites, seepage may be highly variable. Seepage varies with depth of water, areal coverage, depth to impermeable surface, permeability, and depth to ground water. For a preliminary estimate of seepage for Anderson Reservoir, a steady state, radial well formula was used with an average permeability of $1 \mathrm{x} 10^{-5}$cm/sec, an impermeable soil layer 14 feet beneath the reservoir bottom and surface areas and average depths of water according to the reservoir size. For Dugout Reservoir, preliminary geologic information indicates that bedrock (of unknown type) may lie within 2 feet of the surface, therefore, a constant seepage rate of 10 acre-feet/year was assumed.

Sediment storage volumes for the proposed reservoirs were calculated using procedures described in the Soil Conservation Service publication TR-12 "Procedure-Sediment Storage Requirements for Reservoirs." Data was taken from Eureka Energy's water quality measurements and compared with a regional reservoir sedimentation survey.

This reservoir sediment survey, the USDA Agricultural Research Service Miscellaneous Publication No. 1143, shows a small reservoir near Ferron, Utah, having a sediment yield of 0.14 acre-feet per square mile per year. A total suspended solids concentration of 3000 mg/l, based on limited data from Eureka Energy's sampling program, corresponds to approximately 0.14 acre-feet per square mile.

Because the diversion structures will be designed to bypass the bed load, the 0.14 acre-feet per square mile per year is a conservative figure to use for estimating sediment storage for Anderson Reservoir. At a mean annual runoff of 1.5 inches for the 40-year life of the project, 135 acre-feet was used for sediment storage at Anderson Reservoir.

Dugout and Pace Creeks have less suspended sediment. 1500 mg/l was used to calculate the 40-year sediment storage volume in Dugout Reservoir. Twenty-one acre-feet was used with only Dugout Creek Diversion and 38 acre-feet with both Dugout and Pace Creeks being diverted.

Since the drainage areas above the major project reservoirs are very small (see Figure A-1), flood surcharge is a relatively minor volume. For both Anderson and Dugout Reservoirs, a volume of 30 acre-feet was used. This is approximately the equivalent of a 200 year flood. A more detailed analysis of flood surcharge requirements is presented in Appendix B.

Based on the foregoing analysis, approximate reservoir sizes were determined and cost estimates developed for alternative comparison. Reservoir storage volumes used are shown in Table 4-9 and costs are included under Source Development in Tables 4-2 through 4-8, in the report text.

It is not necessary to size the storage reservoir for the most critical drought. To do so may increase project costs beyond what is economically acceptable. Eureka Energy may wish to allow a certain risk of running out of water or having to decrease water usage. This would

Section	Location in Permit Application	Remarks
784.21(a)	IV-G.4	
784.21(a)(1)	IV-G.4	
784.21(a)(2)	-	N/A
784.21(b)(1)	IV-G.4.1	
784.21(b)(2)	IV-G.4.1	
784.21(b)(3)	IV-G.4.1	
784.22	III-B.5.2 III-B.5.4.3 Drawings: D03-0160 to D03-0169	
784.23(a)	Drawings:	
	D03-0006 to D03-0008	Underground mining activities
	D03-0020 to D03-0037	Lands to be affected throughout the operation
	D03-0070 to D03-0118	Pre- and post-mining topography and cross-sections
784.23(b)(1)	Maps: D03-0002 D03-0020 to D03-0037	
784.23(b)(2)	Maps: D03-0020 to D03-0037 D03-0070 to D03-0118	
784.23(b)(3)	Maps: D03-0020 to D03-0037	Section III-D.9 presents the bonding costs for each of the surface facilities to be constructed.
784.23(b)(4)	Maps: D03-0026 D03-0027 D03-0028 D03-0029	
784.23(b)(5)	Maps: D03-0002 D03-0022 D03-0125 and D03-0126 D03-0132 D03-0134	Topsoil stockpiles
	D03-0035	Preparation plant waste disposal site
	D03-0036 and D03-0037	Underground development waste disposal sites

With the exception of the preparation plant site, soils will be stockpiled separately by lift to maintain the integrity of each lift. Soil material stockpiles will be clearly marked with durable, legible signs identifying the material as "topsoil," "subsoil," or "soil material" in the case of other suitable growth media.

Soils from soil mapping unit number IEE2B will be single lifted to a depth of 2 feet and stockpiled below the land fill upon which the coal preparation plant will be built. The topsoil will be graded, covered with plastic sheeting, and marked with permanent markers. The site will then be surveyed based on a permanent bench mark that will remain undisturbed throughout the mine life. The soil, plastic, and markers will be covered with fill materials.

Following removal of the preparation plant, the fill will be graded to conform with natural contours, the topsoil re-lifted based on survey records and markers, and the soil reapplied.

4.1.3 CONSTRUCTION AND MAINTENANCE OF TOPSOIL STORAGE AREAS

Topsoil, subsoil, and soil substitutes will be stored in sites relatively protected from wind and water erosion, contaminants, and vehicle traffic. The soil stockpiles will generally be rectangular in shape, located on relatively level ground, and will have outslopes not steeper than 3h:1v. The preparation plant stockpile will be segregated from and buried under the preparation plant fill. Maps D03-0124 through D03-0129 show the locations of the soil stockpiles.

Following removal of stockpiled soil, the stockpile area will be graded to conform with natural contours, fertilized, and seeded with a planting mixture designed for that specific vegetation type (see Section IV-F.5, Revegetation Plan).

4.1.4 TOPSOIL STOCKPILE PROTECTION AND STABILIZATION

Planting specifications for the control of erosion from topsoil stockpiles will differ depending upon the life of the stockpile. Stockpiles to remain in place no longer than 90 days will not be revegetated. The surfaces of these stockpiles will be left in a roughened condition to retard wind and water erosion.

Stockpiles to remain in place longer than 90 days but less than one year will be seeded to the appropriate temporary cover crop listed in Section IV-F.5.2, Species and Application Rates for Soil Stockpiles. The surface of stockpiles with slopes less than 3h:1v will be ripped to a depth of 24 inches if necessary. The surface will then be disced, harrowed, or compacted to provide the proper seedbed and then drill seeded using conventional drilling methods (one contour pass). On slopes steeper than 3h:1v the surface will be ripped if necessary and roughened by chaining or other appropriate means for seedbed preparation. The slope will then be broadcast seeded at twice the drill rate and the slope roughened again to cover the seed.