Golder Associates

Consulting Mining and Geotechnical Engineers

Golder Associates
CONSULTING MINING AND GEOTECHNICAL ENGINEERS

SURFACE FACILITIES GRADING PLAN
BELINA MINE AREA

PREPARED FOR

VALLEY CAMP OF UTAH
HELPER, UTAH

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1.0 INTRODUCTION

This report presents the results of continuing geotechnical studies carried out by Golder Associates to date at the Belina Mine Site, operated by Valley Camp of Utah, Helper, Utah. The work was initiated based on a proposal to Valley Camp dated July 19, 1979 and accepted per letter from Valley Camp dated August 2, 1979. Subsequent to the proposal, the work has been extended through verbal agreements.

The work has involved the development of the Belina Mine Area surface facilities in regard to the operational requirements of Valley Camp and the regulatory requirements of the State of Utah Division of Oil, Gas and Mining (DOGM), and the Office of Surface Mining (OSM) of the U.S. Department of the Interior.

At the start of the work (August 1979), the Belina No. 1 Mine was in operation under an approved 30CFR211 and Utah Division of Oil Gas and Mining Mine Plan and Permit. Valley Camp's operational plan called for the continued development of the Belina No. 1 Mine, including the development of surface facilities for the No. 1 Mine and the construction of a coal carrying conveyor system and load storage facility for the No. 1 Mine, as set out in general in the originally approved plans. Golder Associates has provided engineering services relative to the geotechnical engineering and hydrologic control structures relative to the proposed development and the current OSM Regulations. This work has included the design of cut and fill slopes and pads, drainage measures, and a sedimentation pond. In addition, construction control and periodic inspection of the work have been carried out at appropriate intervals when required.

2.0 GENERAL SITE CONDITIONS

The Belina Mine Area is located in the upper reaches of Whisky Gulch, a tributary to Eccles Canyon and eventually Pleasant Valley Creek located in the Scofield Drainage of western Carbon County, Utah. Whisky Gulch in the vicinity of the Belina Mine area is an intermittent stream under the definitions provided by the OSM in that it obtains its flow from both surface runoff and ground water discharge at least some part of the year.

The surface facilities area for the Belina mines ranges from an elevation of about 8800 to 9200 feet above sea level. The terrain is steep, and heavily forested with stands of aspen and conifers. Soil cover is thin, in the range of 2 to 10 feet maximum depth, and is in general colluvial material derived from the parent sandstones and siltstones which comprise the rock exposed in the area.

Average annual precipitation at the mine site is about 25-30 inches. Of this, approximately 8 inches occurs as rainfall, generally from May through September.

The general vicinity of the Belina Mine is shown on Figure 2-1. As can be seen, the mining area is on what is known as the Connelville Block, located between the Connelville fault and the O'Connor Fault. The rock units which comprise the Whisky Gulch Area are members of the Blackhawk Formation.

As shown in Figure 2-1, the Belina No. 1 facilities are located on the western flank of Whisky Gulch. The coal handling facilities, first phase, as approved will occupy the gulch itself and portions of the eastern flank. Existing natural slopes in the area are in the range of 20° to 25°.

At the time Golder Associates became involved with the Belina Mine Area (August 1979), a large earthfill had been placed in Whisky Gulch to provide access and work area for the coal storage area. The fill pad was at approximate elevation 8940 at its upper end, sloping down to an existing sedimentation pond constructed at the downstream crest of the fill. At this time, the sediment pond area was exhibiting some local signs of instability in the form of tension cracks, and water was observed seeping out along the toe of the fill. Also, the slopes extending from the Belina No. 1 mine area to the fill pad were steep and exhibiting signs of sloughing.

In order to stabilize the sediment pond area, as well as the Belina slopes, and to provide areas for the surface facilities, Golder Associates recommended that the downstream portion of the existing valley f will be removed and replaced in an engineered manner with a rock toe buttress and a graded filter. The sediment pond was then located at the top of the fill. Additionally, recommendations were made concerning stabilizing the existing slopes from the No. 1 mine area down to the coal storage area. This work resulted in a surface facilities grading plan as presented in Figure 2-2. missing 1992

To the extent possible, all grading was carried out in conjunction with the requirements of the OSM regulations. In the few areas where this was not possible, engineering analyses of variation were carried out to insure the overall stability and satisfactory performance of the structures.

3.0 DRAINAGE CONTROL

3.1 INTRODUCTION

Control of drainage, both surface water and groundwater, is critical to the overall development of the surface facilities. This includes surface precipitation water, snowmelt, ground water intercepted in cuts seeps or springs, and ground water intercepted and collected in the mine workings and pumped to the surface for ultimate discharge. This section presents the methodologies used to predict the flows expected for the various watersheds encountered in the Belina Mine Area.

3.2 DRAINAGE QUANTITIES

Tom

Peak flow on the small watersheds in this study area were estimated using the Rational Method (Grey, 1973). This method is based on the criteria that for storms of uniform intensity, evenly distributed over the watershed, the maximum rate of runoff occurs when the entire watershed is contributing at the outlet and that this rate of runoff, or flow, is proportional to the rainfall intensity. The equation is:

$$Q = ciA$$

where:

c = Runoff coefficient, determined empirically

i = Maximum rainfall intensity, in/hr, whose duration is equal to the time of concentration of the watershed

A = Area of watershed, acres

Q = Peak flow, cfs.

TABLE 3-1

ESTIMATED PRECIPITATION DEPTHS FOR VARIOUS RETURN PERIODS AND DURATIONS AT CLEAR CREEK, SUMMIT, UTAH (FROM RICHARDSON, 1971)

	DURATION									
	5 Min	10 Min	15 Min	30 Min	1 Hr	2 Hr	3 Hr	6 Hr	12 Hr	24 Hr
1	.10	.16	.20	.28	.35	.46	.57	.84	1.08	1.33
2	.12	.19	.25	.34	.43	.57	.70	1.04	1.34	1.65
5	.16	.24	.31	.43	.54	.72	.90	1.34	1.73	2.14
10	.19	.29	.37	.51	.65	.86	1.06	1.55	1.99	2.45
25	.24	.38	.48	.66	.84	1.08	1.31	1.88	2.39	2.92
50	.25	.38	.48	.67	.85	1.13	1.40	2.07	2.67	3.29
100	.27	.42	.53	.73	.93	1.24	1.54	2.29	2.96	3.65

Maximum rainfall intensities for specified return periods were calculated using precipitation data from a gauging station at Clear Creek, Summit, Utah, as reported by Richardson (1971), (Table 3-1). The duration time used when choosing the rainfall intensity was equal to the time of concentration of the particular section of the drainage basin. A 10 year return period was used when choosing the rainfall intensity, as specified by the OSM regulation for permanent diversions.

The time of concentration was determined using the Upland Flow Method (Kent, 1972). Types of flow considered in the Upland Method are: overland, through grassed waterways, over paved areas, and through small upland gullies. Upland flow employed in this method can be a combination of these various surface runoff conditions. The velocity of flow is determined using Figure 3-1. The time of concentration of the drainage basin then is equal to the sum of the times required for water falling on the farthest point of the watershed, flowing over various types of terrain, to reach the outlet.

The runoff coefficient in the Rational Method is dependent on the topography, soil type and vegetation of the watershed. Values of the runoff coefficient can be found in Grey (1973). An estimate of the coefficient was made from these charts assuming:

Topography - Hilly Land
Soil - Open Sandy Loam c = 0.03
Cover - Woodlands .3?

The drainage area of concern at the Belina Mine site was divided into six sections (see Figure 3-2). The drainage characteristics and outflow were calculated for each section. Table 3-2 lists the values used and outflow for each section. Outflow from section I and VI is to be diverted outside the area of concern while the outflow from sectins II-V is to be carried via a culvert under the mine site and out of the drainage area. Water falling within the mine area is to be treated in a settling pond and then discharged.

TABLE 3-2
CALCULATION OF FLOW RATES BY SECTION

SECTION NUMBER	I	II	III	IV	V	VI
AREA (acres)	23.6	28.3	29.8	36.5	36.1	8.3
tc (min)	8.6	14.7	15.3	24.3	24.2	17.4
i (in/hr)	1.74	1.48	1.48	1.25	1.25	1.48
c RUNOFF COEFFICIENT	0.3	0.3	0.3	0.3	0.3	0.3
Q =ciA (cfs)	12.3	12.6	13.2	13.7	13.5	3.7

do they interpolate on chart or take
To. $\frac{8.6}{10.0}$ x .29 x
$\frac{60}{8.6}$ = Tc.

4.0 SEDIMENT POND

4.1 HYDROLOGIC DATA

Hydrologic analyses were made to calculate the runoff volume, from the disturbed area, that would be treated in the sediment pond (see Figure 3-2). The calculated storage volume was based on the 10-year 24-hour storm as required by OSM Regulations. Previous experience has indicated that a 48-hour detention time is required to insure adequate settling.

Hydrographs for the Belina Mine Area were obtained from a report to Valley Camp from Vaughn Hansen Associates. The Hydrograph presented in the report was for a 25-year 24-hour storm. The Hydrograph for the 10-year 24-hour storm is assumed to be of similar shape. The peak inflow into the pond was calculated assuming the hydrograph could be represented by a triangle. The peak runoff for the 10-year 24-hour hydrograph, shown in Figure 4-1, is determined according to the equation:

$$q_p = \frac{484\ AQ}{T_p}$$

where,

q_p = Peak Runoff, cfs
A = Area, sq. miles
Q = Net storm rain, inches
T_p = Time to peak runoff, hours.

The volume of runoff is then defined as the area under the hydrograph.

4.2 SEDIMENT POND DESIGN

The outflow hydrograph of the sediment pond for the design storm was constructed based on a constant outflow from the principal spillway (spillway design will be discussed in Section 4.3). Since the outflow from the sediment pond will be constant and the detention time has been specified as 48 hours, a direct determination of the outflow hydrograph for this design storm is possible, the constant outflow can be determined from the equations:

$$Vi = Vo$$
$$Vi = q_o \times tb$$

where,

Vi = Volume Inflow
Vo = Volume Outflow
q_o = Constant Outflow Rate, cfs
tb = Duration of hydrograph, hours.

The calculated outflow hydrograph is shown with the inflow hydrograph in Figure 4-1. The constant outflow required for the design storm and a 48 hour detention time is 0.94 cfs.

The storage required for the 10-year 24-hour storm can be determined from the inflow and outflow hydrographs. The difference between the area under the inflow hydrograph and that under the outflow hydrograph during the time of inflow is the storage volume; this is shown as the shaded area in Figure 4-1. The pond storage volume necessary to hold the runoff for 48 hours from the design storm is 6.7 acre-feet. The pond must be of sufficient volume to hold both the accumulated sediment and the design storm runoff.

Estimates of the volume of sediment were made from the 0.1 acre-feet per acre of disturbed land value recommended by the OSM Regulations. This results in a total sediment volume of 3.6 acre-feet. The total volume required for the storage pond would be the water volume plus the sediment volume which is 10.3 acre-feet.

The stage-storage curve for the existing pond is shown in Figure 4-2. The volume required for both the sediment and the water storage are marked on the curve as well as the spillway elevation. The existing pond with a storage volume of 11.2 acre-feet is sufficient to hold both the design sediment load and design runoff for a 48 hour detention time. It is understood that the existing decant is at elevation 8856. This is satisfactory for retention of 60% of the maximum allowable sediment level in addition to retaining the design storm for 48 hours. In accordance with the regulations, the accumulated sediment must be removed when it reaches 60% of maximum. It should be noted that, with the fixed decant system, it will be necessary to dewater the pond by pumping after precipitation events in order to provide storm surcharge capacity.

In accordance with Mine Safety and Health Administration Regulation (30 CFR 77.216-1) a permanent identification marker should be located on or immediately adjacent to the sediment pond. This marker should be at least 6 feet high and show the identification number of the impounding structure as assigned by the District Manager, the name associated with the impounding structure and name of the person owning, operating, or controlling this structure.

It is understood that the sediment pond may be used to collect and discharge mine water, as well as surface runoff. If this should be required, we recommend that the fixed elevation decant be replaced by a floating orifice type decant (see Figure

4-3). This design provides a constant discharge at an easily controlled rate and also skims water from the top of the pond, which tends to increase the effectiveness of settling pond performance (Skelly and Loy, 1979). Further, this type of decant will dewater the pond, thereby eliminating the need for pumping, and constantly providing storm surcharge capacity.

The collapsible pipe, shown in Figure 4-3, must extend from maximum pool level to permanent pool level (i.e., from elevation 8849.5 feet to 8856.6 feet). The inlet of the floating orifice is attached to floats so that it is submerged to give a desired outflow rate. The required outflow rate of 0.94 cfs can be accomplished using a variety of combinations of pipe diameter and head (see Figure 4-4). Any pipe diameter that is available could be used for the inlet so long as the decant is constructed with the appropriate head. The top of the pipe must be protected from logging by a trash rack or other suitable device.

4.3 SPILLWAY DESIGN

The emergency spillway was designed to handle the OSM Regulation's design storm of 25-years 24-hours. The emergency spillway elevation is at 8856.0 feet and the crest of the embankment if at 8860.0 feet. OSM regulations require at least 1.0 feet of clearance between the maximum elevation of water in the emergency spillway and the crest of the embankment. This limits the maximum depth of flow in the emergency spillway to 1.0 feet. At this flow depth the flow rate in the emergency spillway calculated by the Manning equation would be 85.0 cfs (see Figure 4-5). The flow rate required to handle the 25-year 24-hour storm was calculated to be 15.9 cfs, from equation 4-1. In conclusion, the emergency spillway can handle flow rates in excess of the 25-year 24-hour storm while still satisfying the OSM Regulations concerning freeboard.

4.4 CONSTRUCTION

Construction of the valley fill and sediment pond structure was commenced in September 1979 and essentially completed in December 1979. The construction of the valley fill was carried out under the full-time inspection of a geotechnical field engineer, who was responsible for insuring that the construction was carried out in conjunction with standard engineering practices and the requirements of the Division of Oil, Gas, and Mining. This consisted of material inspection, stripping and removal of topsoil and organic debris, and placement of the fill materials, including the rock buttress. The granular filter and underdrains, and the general embankment fill.

4.4.1 Construction Materials

An as-built schematic longitudinal section of the valley fill and sediment pond is shown in Figure 4-6. There are essentially five different earth materials involved in the structure.

These materials are as follows:

I. Rock Toe Buttress, consisting of boulders and cobbles ranging from approximately 4 to 1 in mean dimension.

II. Graded Filter, consisting of sand and gravel blended to provide protection against piping of the general embankment fill.

III. Rockfill Drain, consisting of small boulders and cobbles ranging from approximately 2 feet to 6 inches in mean dimension.

IV. General Embankment Fill

V. Foundation Material, consisting of the undisturbed material soil and rock materials comprising the valley floor and walls.

The location and geotechnical strength characteristics of the various materials are presented on Figure 4-6. Particle size gradations of the general embankment fill and the graded filter are shown on Figure 4-7.

4.4.2 Construction Control

The construction of the earthfill and sediment pond was inspected in the field by a qualified geotechnical field engineer. The inspections consisted of determination of material suitability, material placement techniques, and in-place density testing to determine the degree of compaction obtained in the materials. Observation of the sequential phases of the construction are discussed individually below:

under supervisor of certified?

1. Rock Toe Buttress

   Prior to placement of the rock toe buttress, all areas to receive fill were stripped of topsoil and loose or surficial materials. The rock toe buttress was then placed by end dumping or moving with dozers in order to insure interlocking and proper resting of the individual boulders.

2. Graded Filter

   Upon completion of the rock toe buttress, the graded filter material was placed. Placement was carried out in 6 inch to 12 inch lifts, with the material spread by dozers and compacted in place. No compaction tests were carried out in the filter materials due to their granular nature, but careful observations were made throughout the filter placement to insure continuity of material and adequacy of the placement techniques. ok

3. <u>General Embankment Fill</u>

After the graded filter material was placed, general embankment fill was compacted in lifts to form the valley fill and sediment pond embankment. Compaction testing was carried out as the fill was placed to insure that the material was compacted to at least 95% of Modified Proctor Density, per ASTM D-1557. Results of the compaction testing are presented in summary form on Figure 4-8. 90%

4.4.3 Stability Analyses

Stability analyses of the as-built section have been carried out to insure that the fill has an adequate static factor of safety against failure. The analyses were carried out using the <u>Bishop Method</u>, which essentially assumes a circular failure arc through the mass and computes the stability of forces along that arc. This is then carried out on a variety of failure arcs until the minimum factor of safety is determined. The analysis was carried out using the impact parameters and fill geometry shown on Figure 4-6, and various water seepage paths through the embankment and fill. The results of the analyses are presented in Figures 4-9 through 4-12. Critical surfaces are shown on the figure, with the computed Factor of Safety. As can be seen, the minimum computed static factor of safety for the composite embankment is 1.8; the critical failure arc is located in the lower portion of the embankment. This is within the requirements set forth by OSM.

A cross-section of the valley fill (Figure 4-13) shows the valley walls, the embankment fill, and the emergency spillway for the sediment pond.

4.5 POND MAINTENANCE

Sediment ponds must be periodically maintained to remove deposited sediments so that trap efficiency can be preserved. The Federal Regulation require that this occur when the design sediment storage volume has been 60 percent displaced (see Figure 4-2).

Since the required volume is 3 years of sediment or 0.1 acre-feet per acre of disturbed land, the maintenance schedule should require cleaning the pond at least every 21 months (60 percent of 3 years). It is advisable, however, to reduce this to a maximum of every 12 to 18 months because the sediment will not be deposited evenly over the 3 year period (Skelly and Loy, 1979).

A thorough inspection of the sediment pond and embankment should be undertaken at least once per year. When examining for stability and general inspection the inspector should be looking for any of the following conditions:

- Seepage from anywhere on the down-stream side of the embankment but especially around the discharge pipe
- Erosion of embankment slopes
- Continuity of emergency spillway
- Erosion around entrance or exit of discharge pipe
- Clogged principal or emergency spillway
- Check slope stakes for obvious slope movement

- Level of sediment
- Placement of wave erosion protection
- Erosion at spillway discharges
- Clogging of dewatering device.

Monitoring for embankment movement (Skelly and Loy, 1979) should also be a part of this inspection. This can be performed by setting stakes in the embankment, along the toe and several rows proceeding up from the toe. The original position and elevation should be recorded with reference to a permanent landmark. These positions should be checked during inspection. If unstable or potentialy unstable conditions exist, corrective measures should be taken immediately.

5.0 DIVERSION DITCHES AND CULVERTS

5.1 LOCATION

Figure 3-2 shows the drainage area of concern. In an effort to minimize further land disturbance, it is recommended that existing roads be incorporated into the diversion ditch scheme. This will result in the construction of only one additional ditch that must be constructed on undisturbed land. This ditch is labeled on the plan as I-J.

5.2 ANALYSIS AND DESIGN

Diversion channels were conceptually designed using the Chezy-Manning equation:

$$Q = \frac{1.486\ AR^{2/3}S^{1/2}}{n}$$

where:

Q = flow, cfs

n = Mannings Roughness Coefficient

A = Cross sectional area

R_h = Hydraulic radius

S = Slope

Typical values of the rougness coefficient ranges between 0.022 and 0.030 for excavated or dredged earth channels, straight and uniform with short grass and few weeds (Grey, 1973). An average value of 0.027 was used for the calculations.

Channels were designed, where possible, for ease of construction and maintanence. A trapezoidal cross section with sides sloping at 45 degrees and a base width of 1.0 feet was used. A typical cross section of ditch along side of the existing road and an enlarged cross section through a ditch are shown on Figures 5-1 and 5-2, respectively.

Table 5-1 shows the quantity of flow that will be carried by each ditch and the particular drainage sections that contribute to the flow. Table 5-2 summarizes the calculated depths of flow in each channel. OSM regulation requires a minimum free board of 0.3 feet for diversion ditch design. This criteria would be satisfied if all ditches were constructed to a depth of 1.0 feet.

It is recommended that the channels be seeded to aid in the prevention of erosion during peak flows.

Velocities, calculated from Mannings equation, indicate that they would be in the range of 10-12 feet per second at peak flow. Measures needed to reduce the velocities (cutting new ditches in undisturbed areas with reduced slopes) or to prevent erosion (lining the channels with rocks sufficient to resist erosion) are felt to be too destructive to justify the gains.

OSM regulation require energy dissipators at ditch stream interfaces if velocities of entering ditches are greater than that of the receiving stream. A situation of this nature occurs at only one place in the study area. This is where the outflow of Ditch A-B enters an existing stream. Here we would recommend that a rock check-dam be placed at the interface to be used as an energy dissipater.

5.3 CULVERT DESIGN

Existing culverts were checked to see if they could carry the required 10-year storm. The 24-inch culvert at a road/stream

TABLE 5-1
FLOW RATES TO BE CARRIED BY PROPOSED DITCHES

Ditch	CD	EF	GH	FH	AB	IJ
Drainage Section	I	I	I	I	II	VI
Q (cfs)	12.3	12.3	12.3	12.3	12.6	3.7

TABLE 5-2
SUMMARY OF CALCULATED FLOW DEPTHS
IN PROPOSED DITCHES

Ditch	Cross Section	Depth of Flow (ft.)
CD	Trapezoidal*	0.64
EF		0.64
GF		0.64
FH		0.64
AB		0.70
IJ		0.67

* Base = 1.0 ft, sides slope 45°

intersection southwest of the mine area will receive outflow from sections II and III resulting in a total flow of 25.8 cfs. The 24-inch culvert will carry this flow if the head water elevation is equal to 2.0 times the culvert diameter. Culvert flow quantities were determined using monographs for inlet controlled culverts in the Handbook of Steel Drainages and Highway Construction Products, 1971. The maximum flow through this culvert will depend on the type of entrance inlet of the culvert. Our design was based on an end section conforming to the fill. A cross section of this inlet is shown in Figure 5-3.

The other culvert which carries flow underneath the mine area is 42 inches in diameter. Outflow from sections II, III, IV and V would be carried by this culvert. Computations by others indicate that the culvert has a total capacity of approximately 52 cfs, and that the maximum flow as a result of a 100-year, 24-hour storm and 100 year, 6-hour storm are approximately 17 cfs and 19 cfs, respectively.

Respectfully submitted,

GOLDER ASSOCIATES

Charles W. Lockhart

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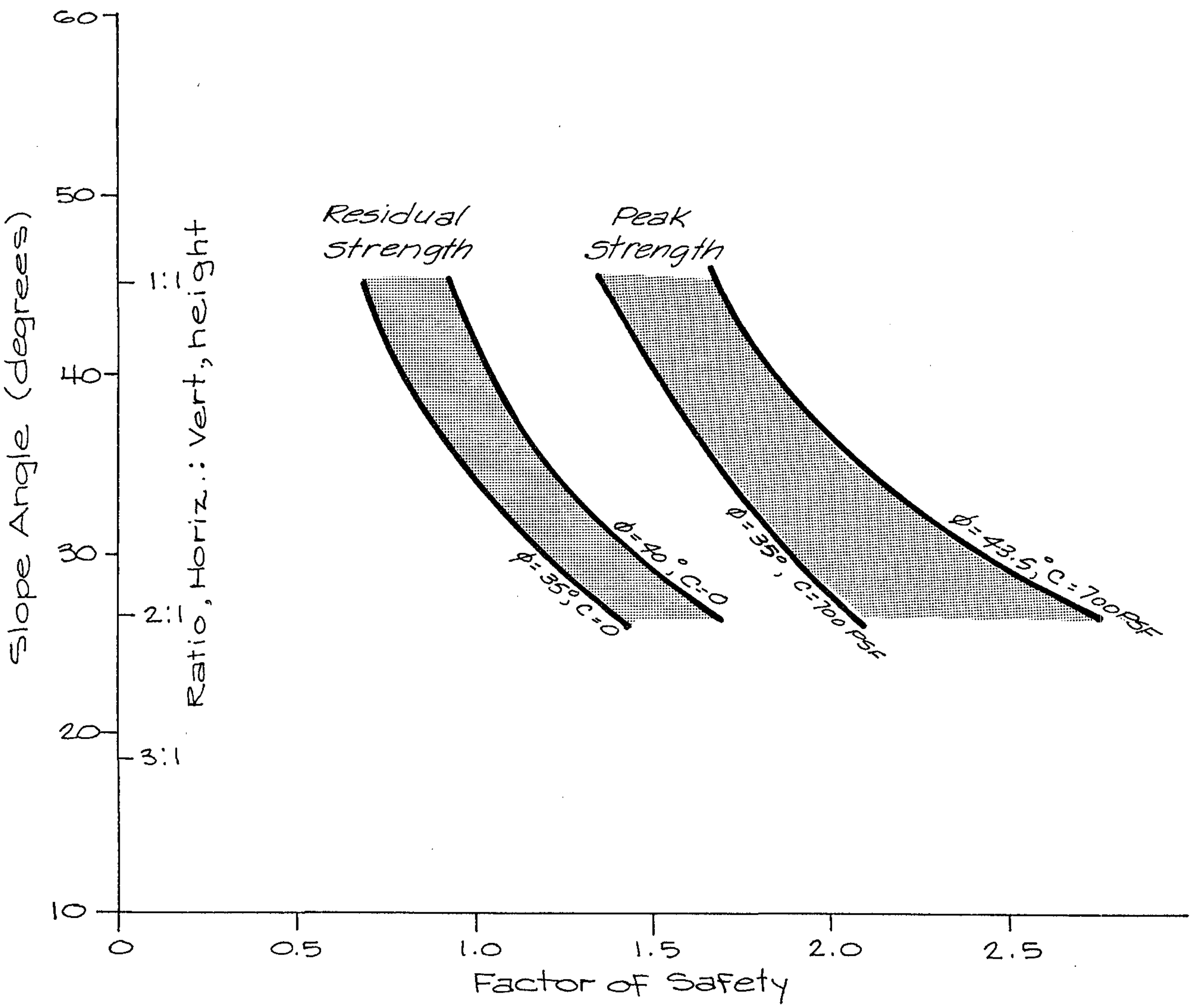
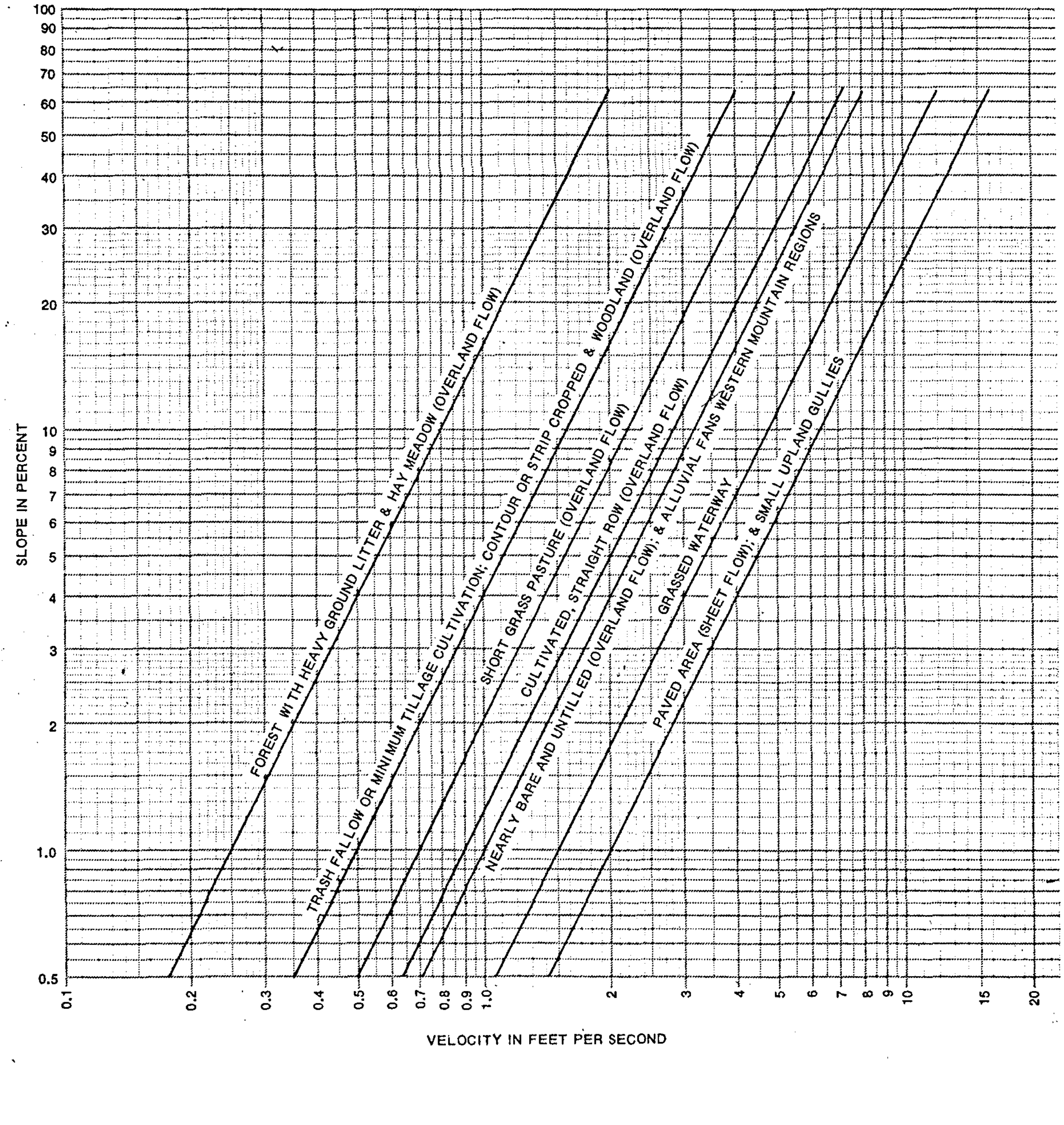
INFLUENCES OF SLOPE ANGLE AND STRENGTH PARAMETERS ON THE FACTOR OF SAFETY

Figure 1



VELOCITIES FOR UPLAND METHOD OF ESTIMATING T_C Figure 3-1



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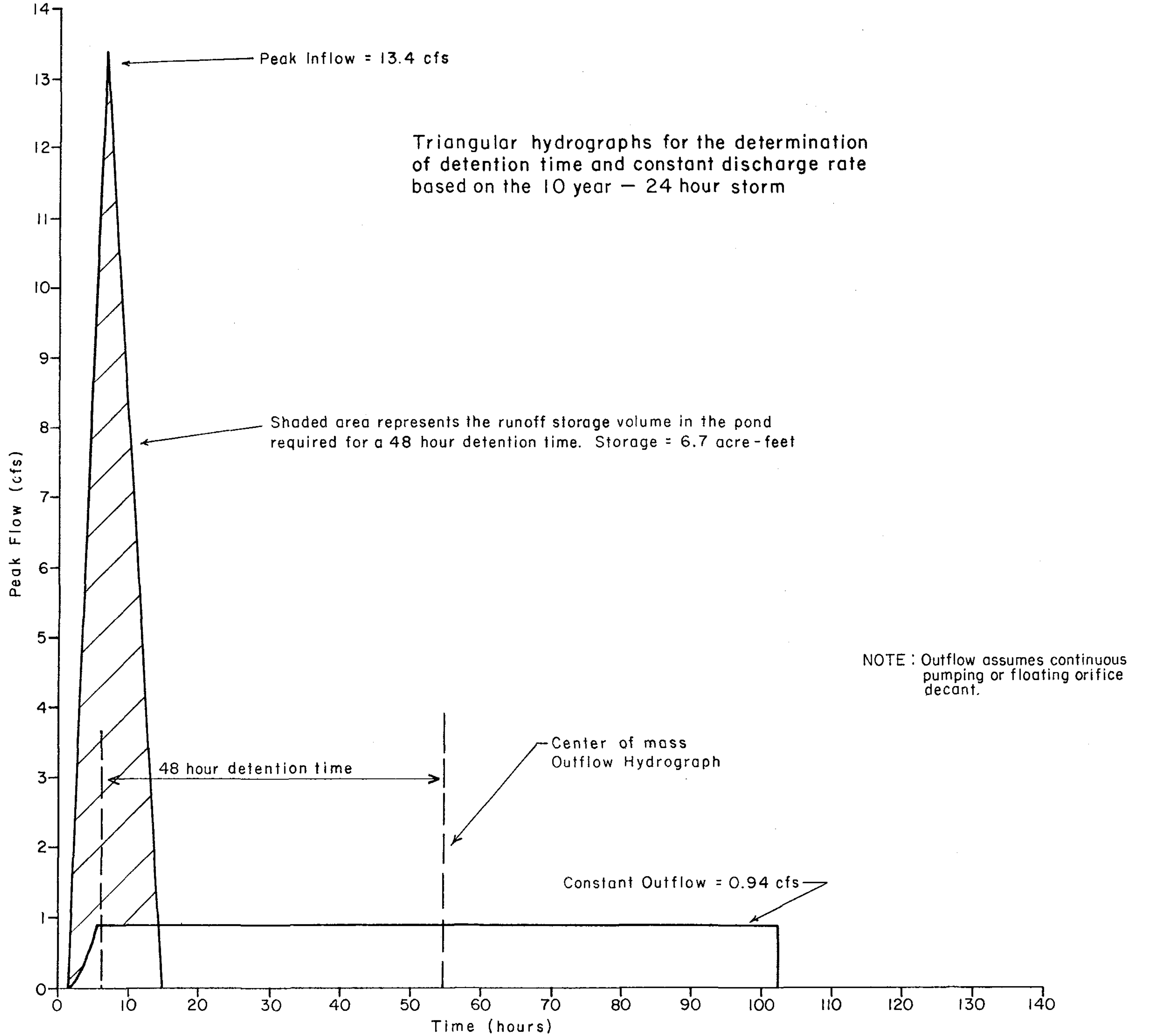

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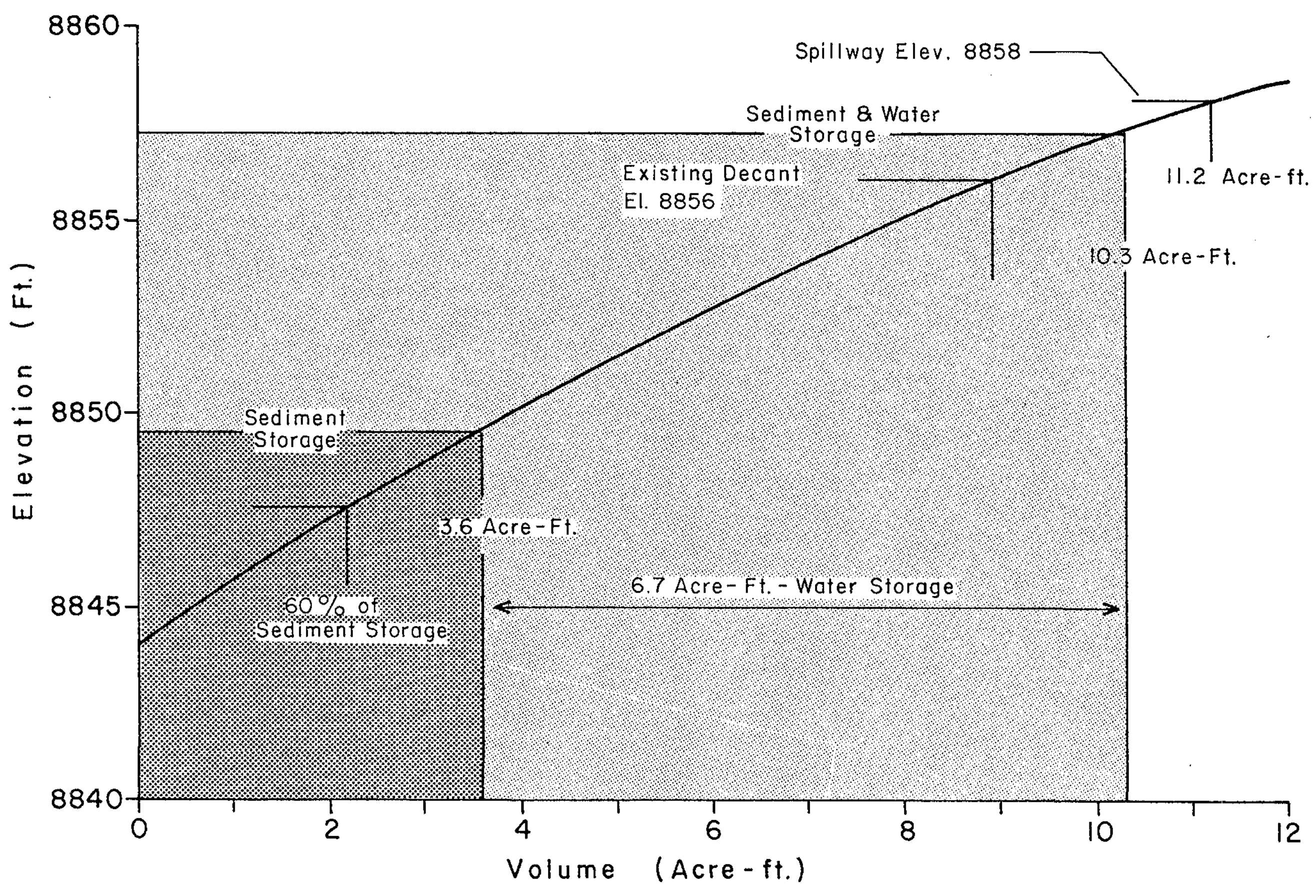
INFLOW AND OUTFLOW HYDROGRAPHS OF SEDIMENT POND

Figure 4-1

STAGE-STORAGE CURVE FOR SEDIMENT POND

Figure 4-2



NOTE: Decant is fixed. Collected water must be removed by pumping.

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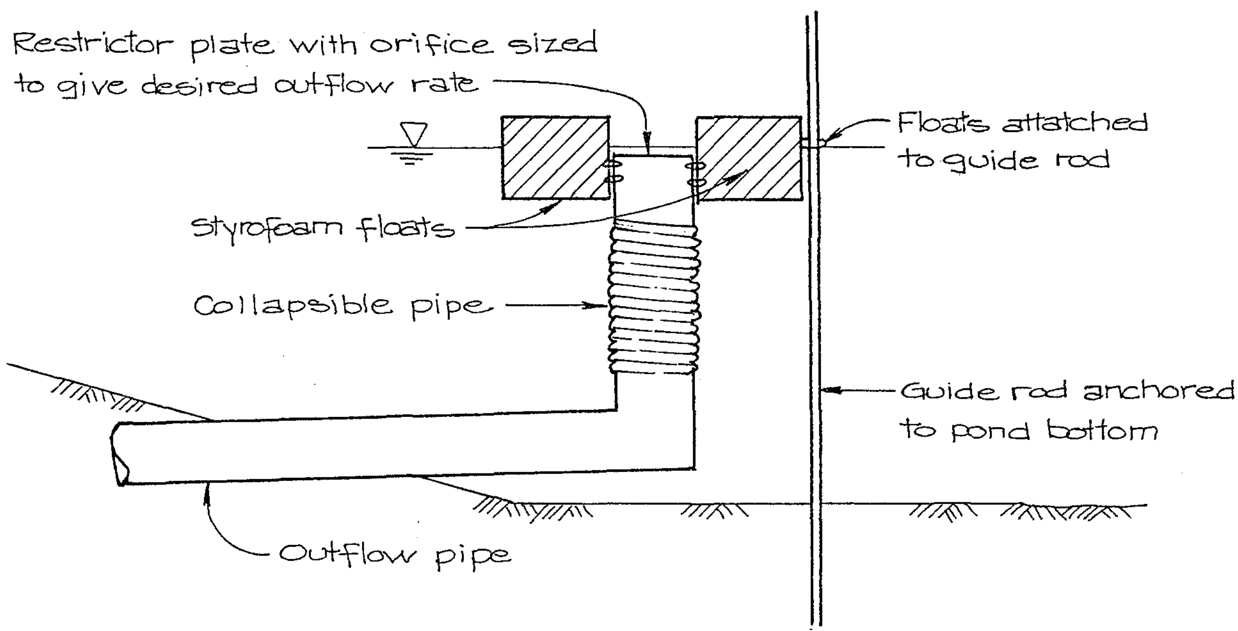
FLOATING ORIFICE TYPE DECANT

Figure 4-3



NOTES

1. Collapsible pipe must extend from maximum pool level to permanent pool level.
2. Inlet is attatched to floats so that it is submerged to give desired outflow rate.

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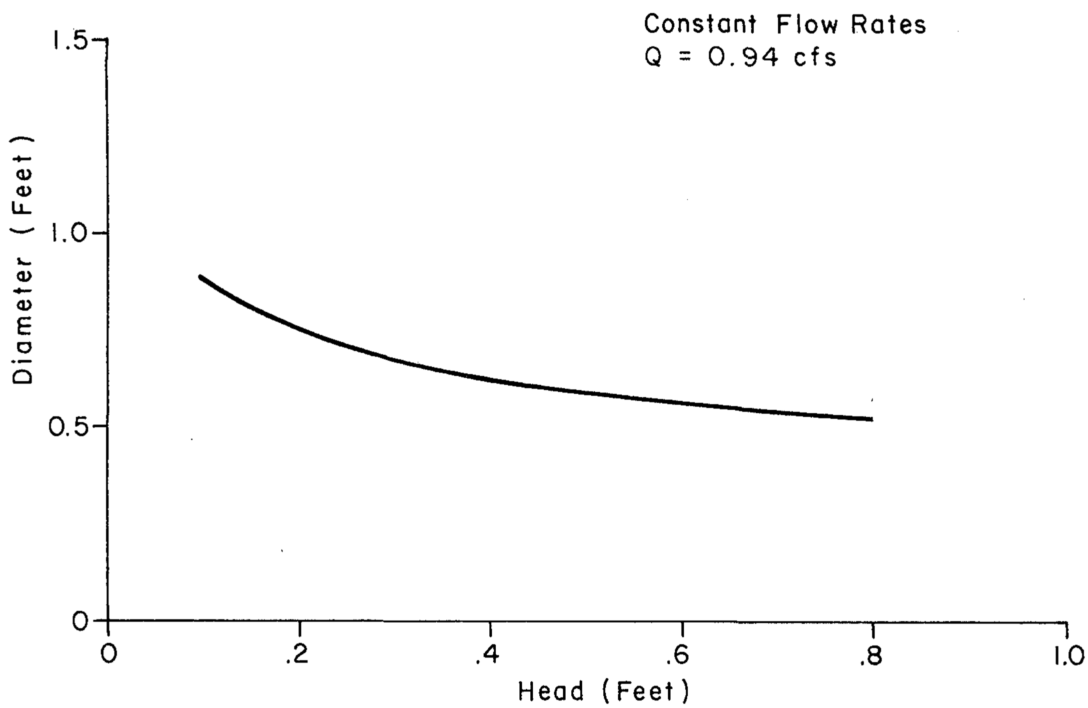
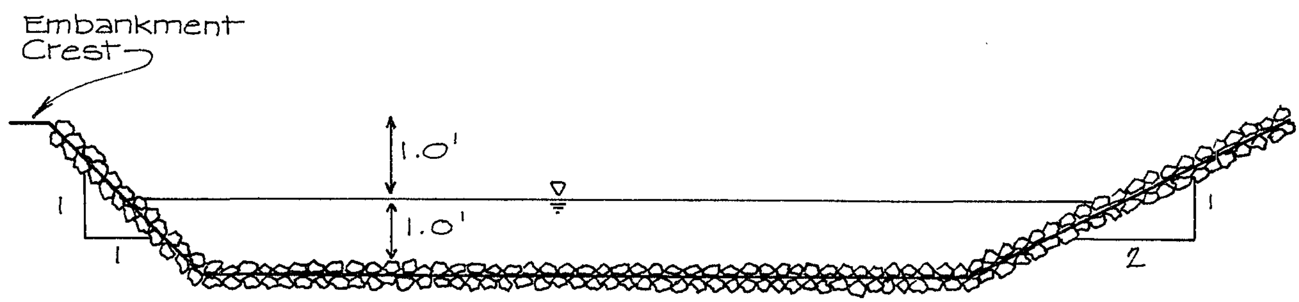
PIPE DIAMETER Vs. HEAD FOR CONSTANT FLOW RATE Figure 4-4



CROSS SECTION EMERGENCY SPILLWAY Figure 4-5



$n = 0.040$
$S = 0.05$
$R = 0.84$ Ft.
$A = 11.50$ Ft.^{2}
$P = 13.65$ Ft.

$$Q_{max.} = \frac{1.486}{n} R^{0.67} S^{0.5} A$$

$Q_{max} = 85.0$ cfs

Scale: 1" = 2.5'

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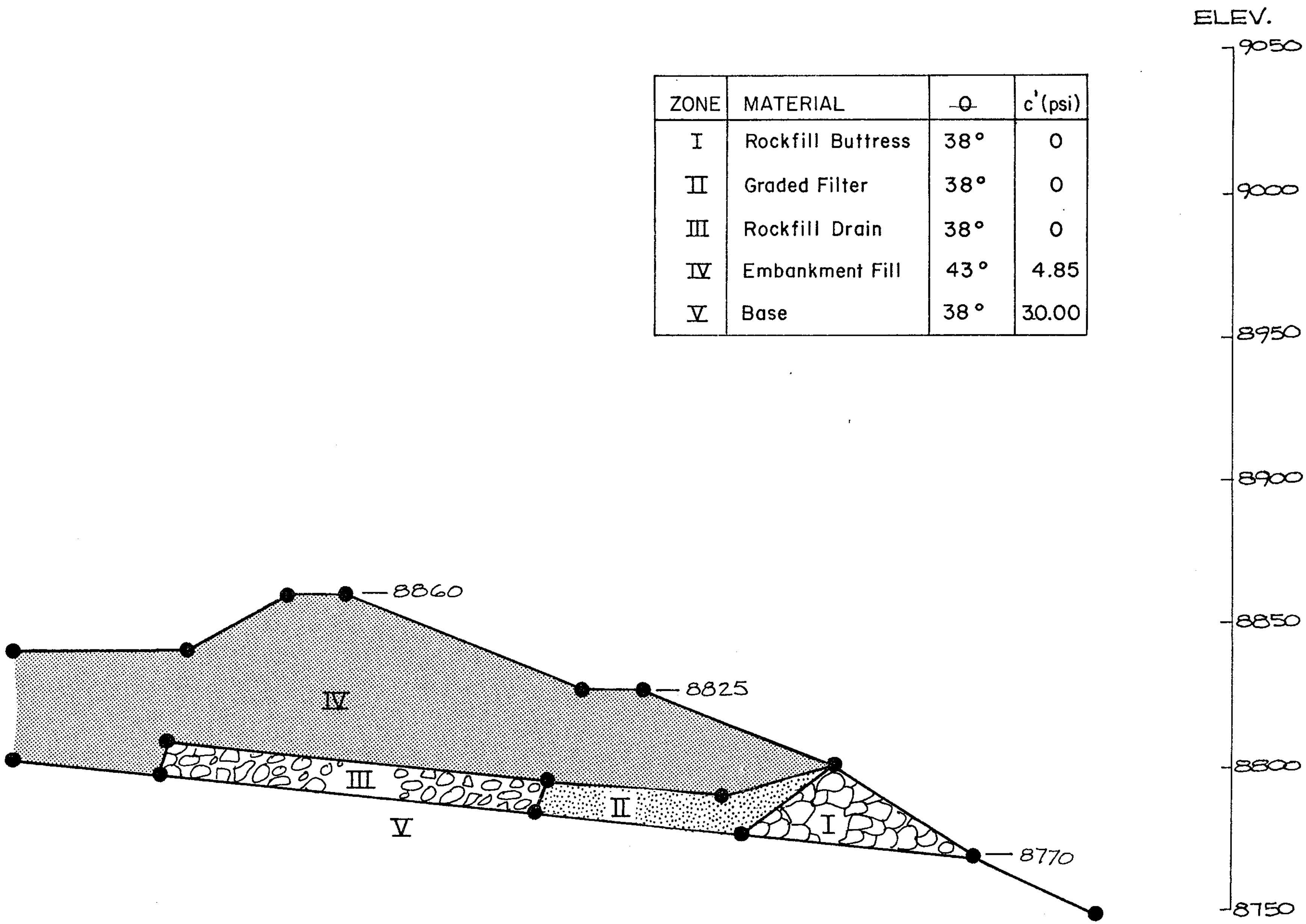

ZONE	MATERIAL	Ø	c'(psi)
I	Rockfill Buttress	38°	0
II	Graded Filter	38°	0
III	Rockfill Drain	38°	0
IV	Embankment Fill	43°	4.85
V	Base	38°	30.00

LONGITUDINAL GEOMETRY FOR VALLEY FILL

Figure 4-6

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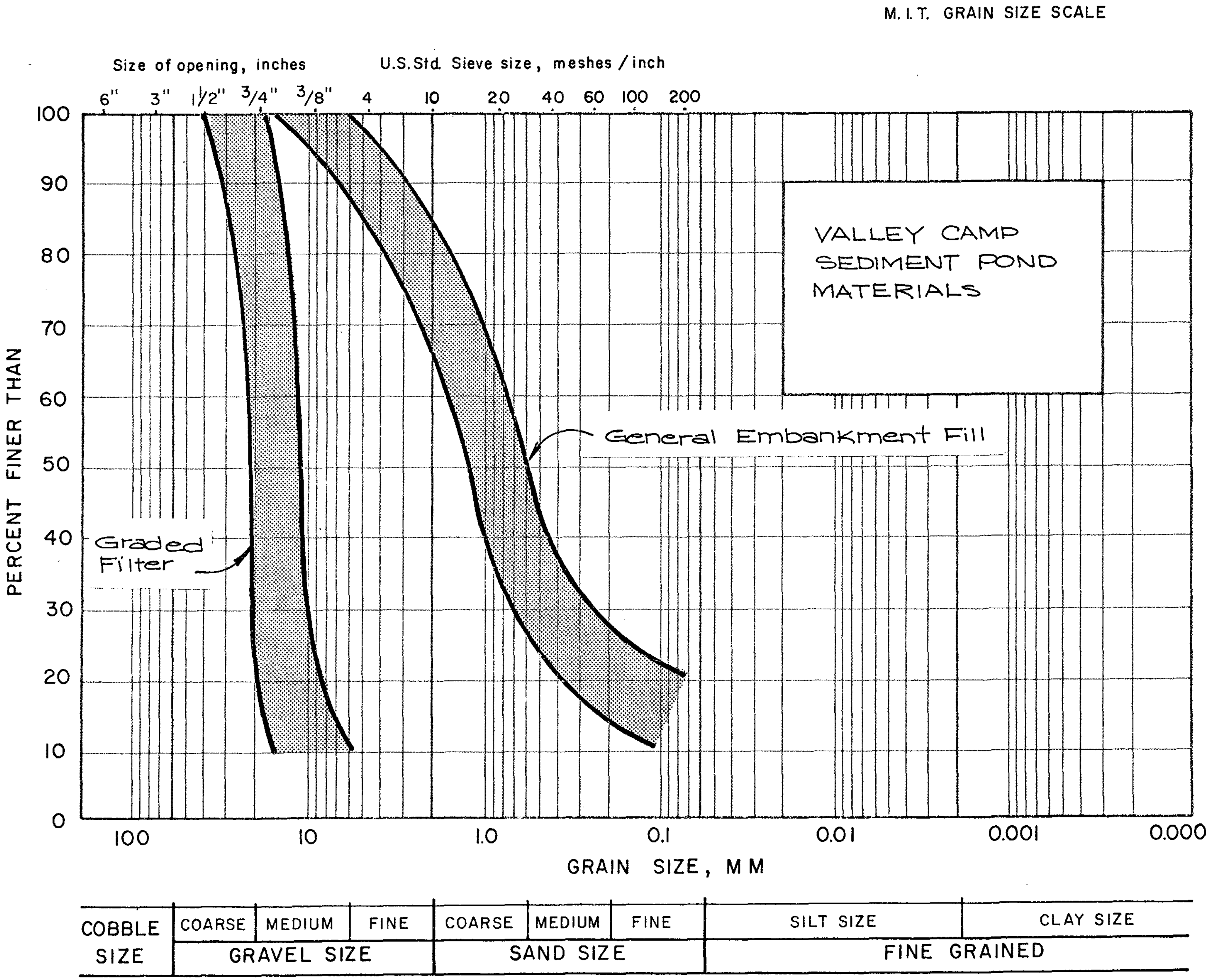


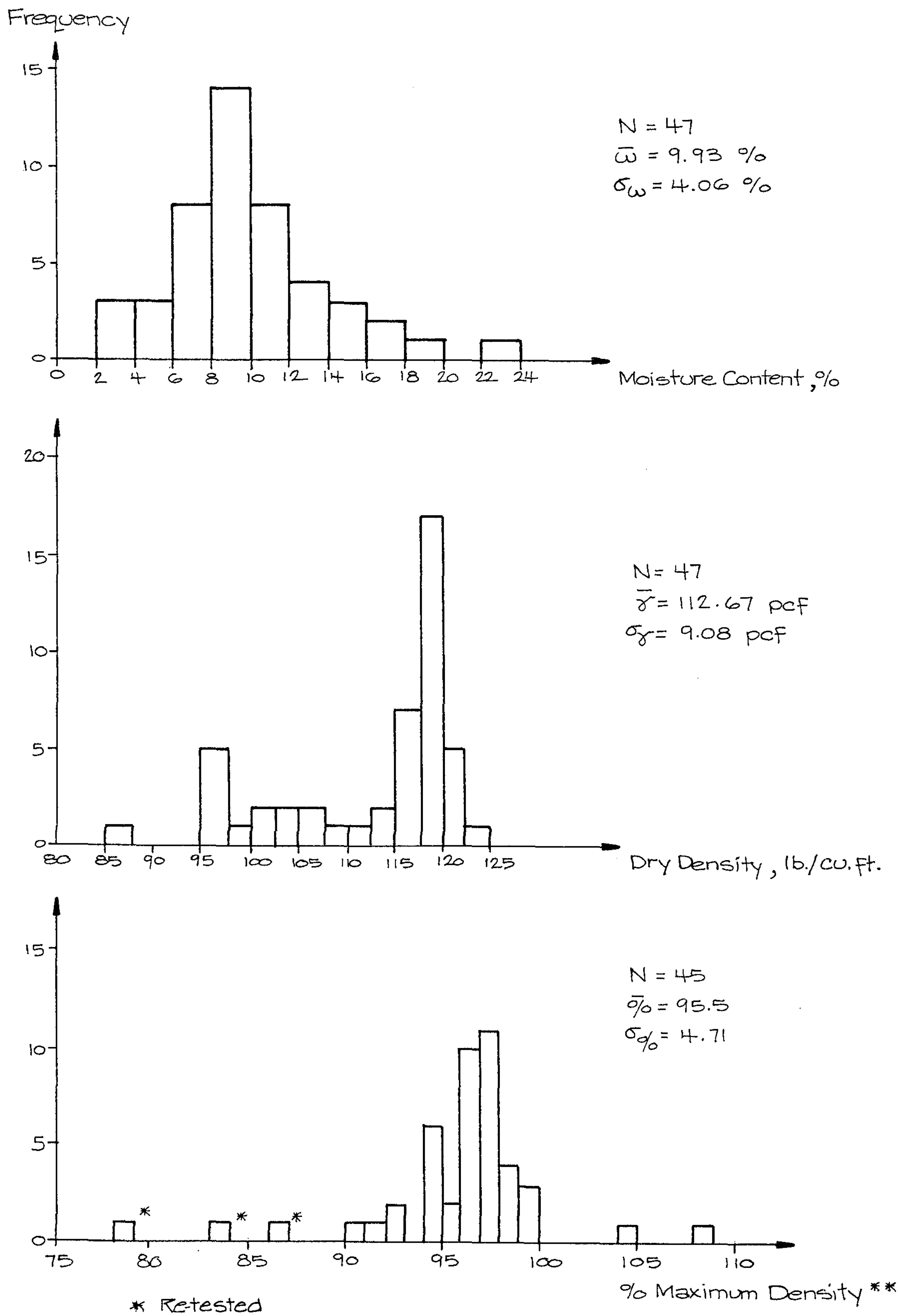
Project No. 793-1156A
Project Valley Camp
Golder Associates
Date 6-1980
By HK

GRAIN SIZE DISTRIBUTION

FIGURE 4-7

COMPACTION TEST RESULTS

Figure 4-8



* Retested

** per ASTM D-1557, modified Proctor density

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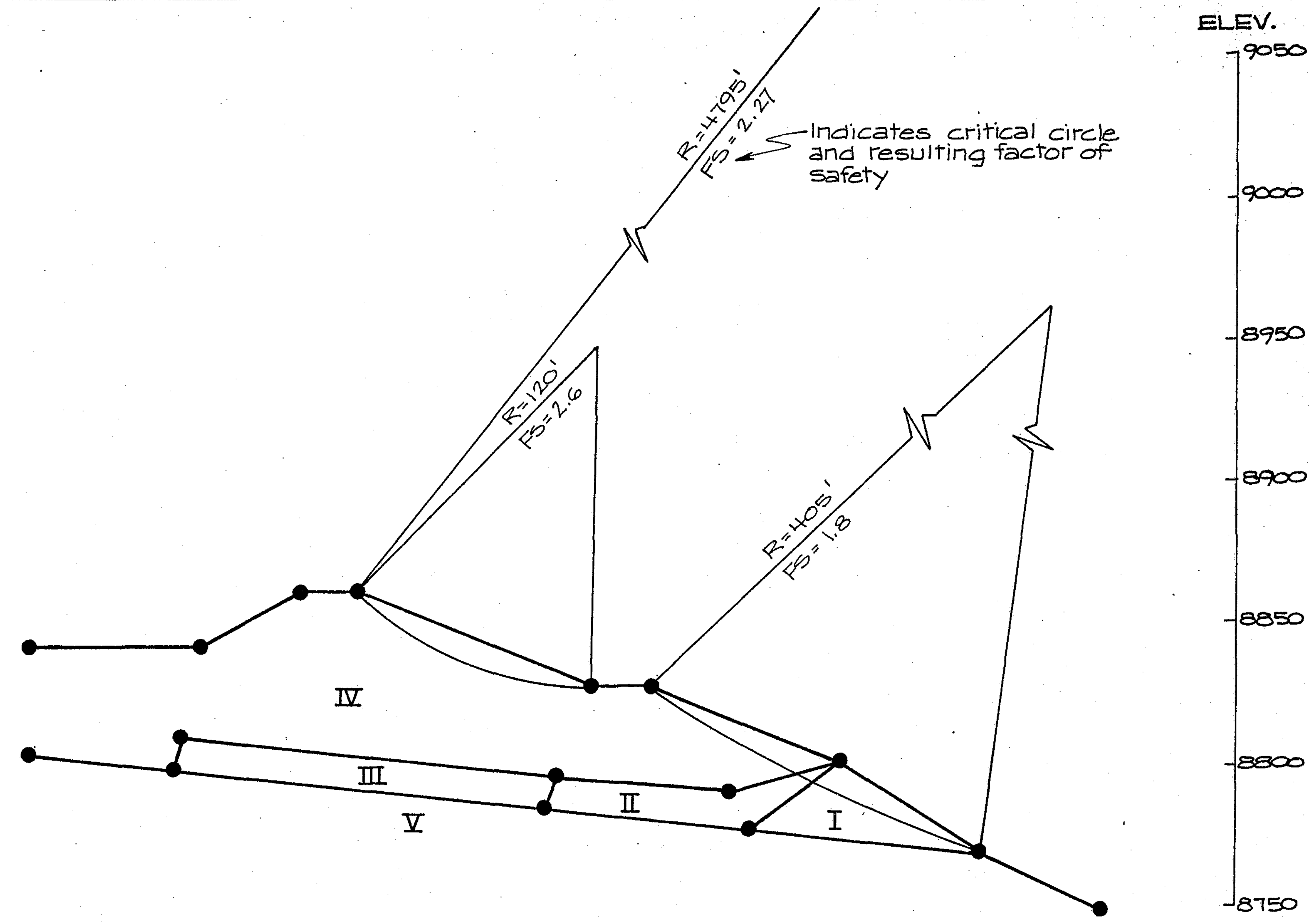

PE STAMP.

EMBANKMENT STABILITY ANALYSES DRAINED

Figure 4-9

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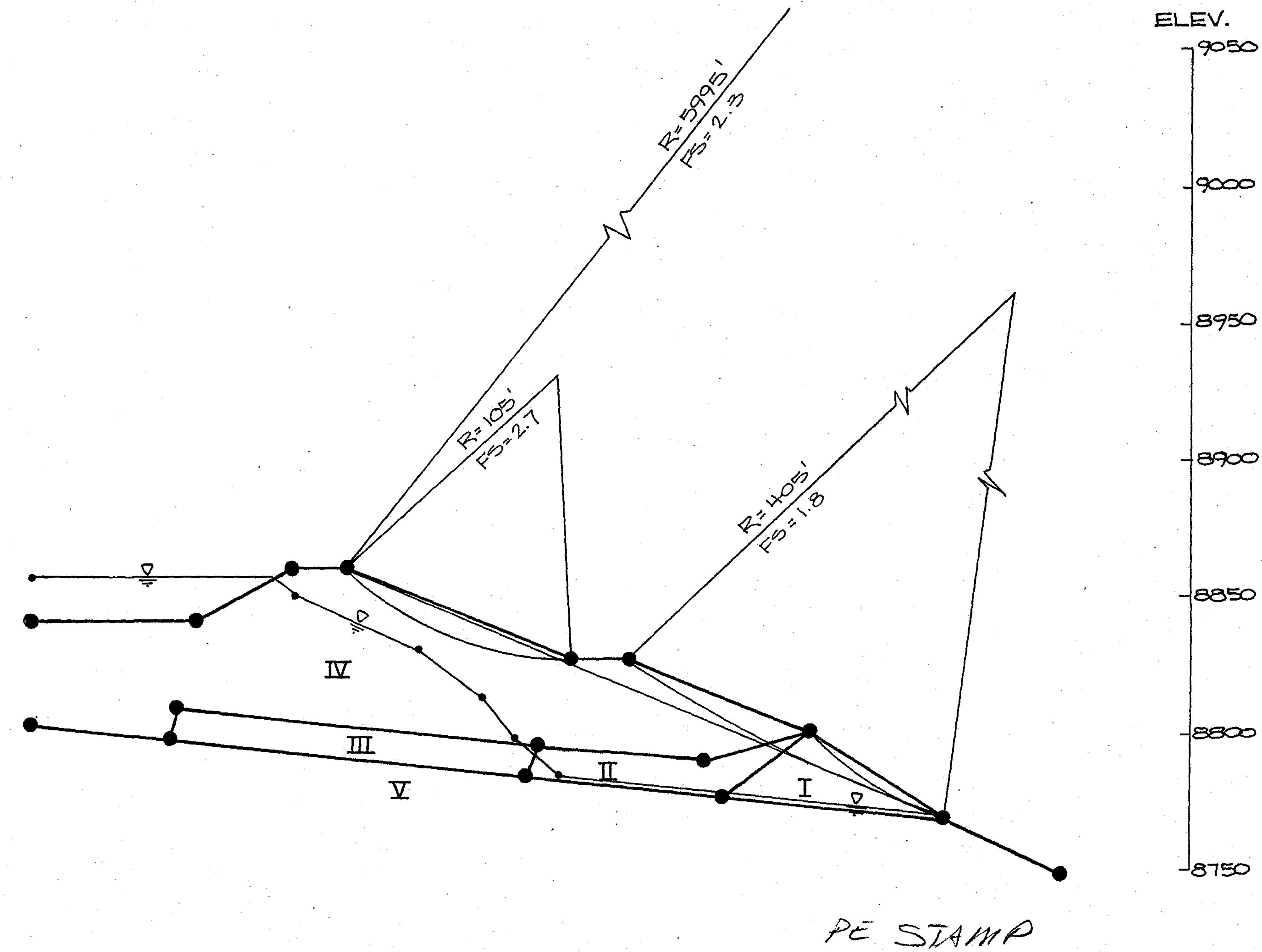

EMBANKMENT STABILITY ANALYSES
STEADY STATE — FULL DRAINAGE INTO II

Figure 4-10

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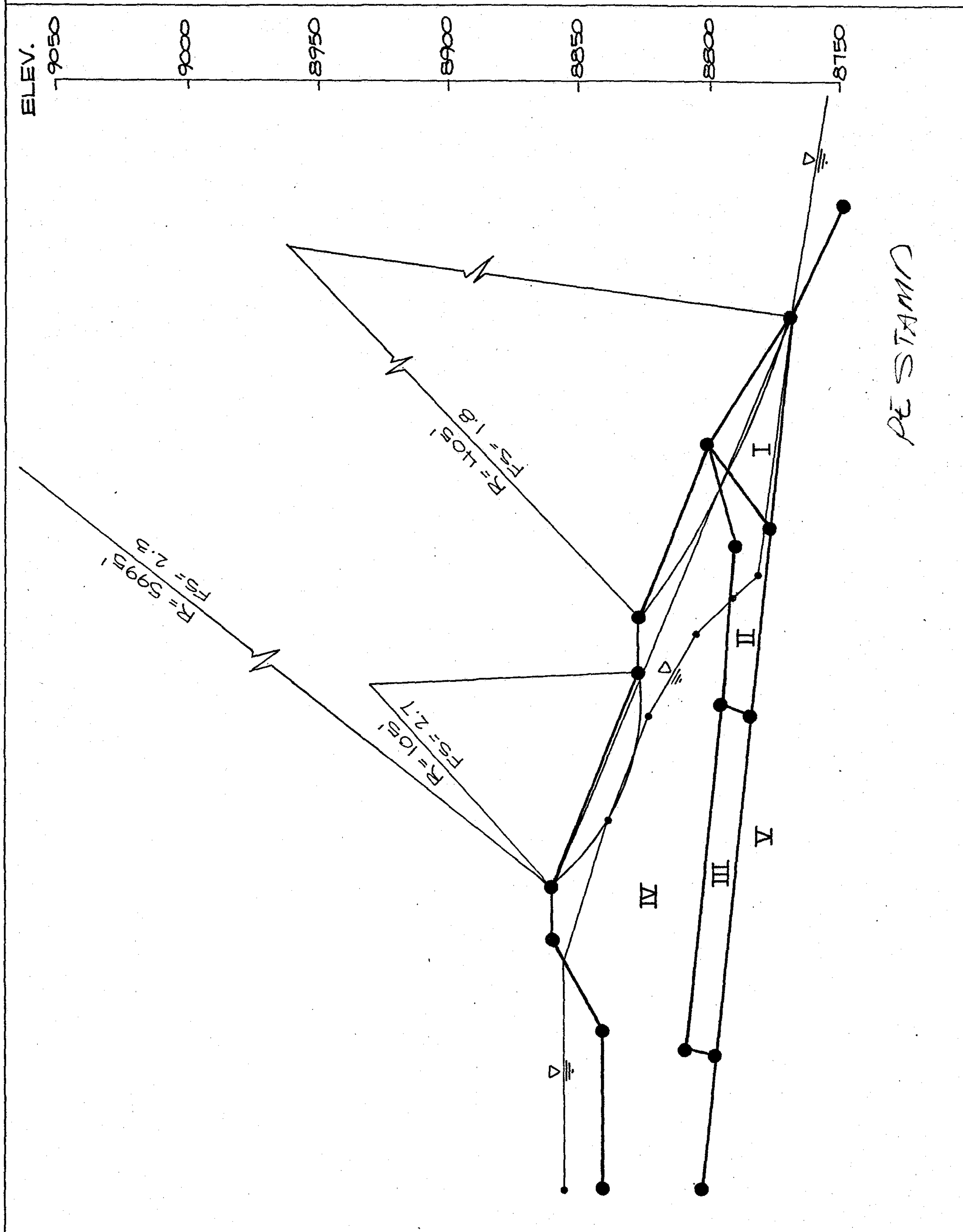
EMBANKMENT STABILITY ANALYSES
STEADY STATE — II PARTIALLY CLOGGED

Figure 4-11



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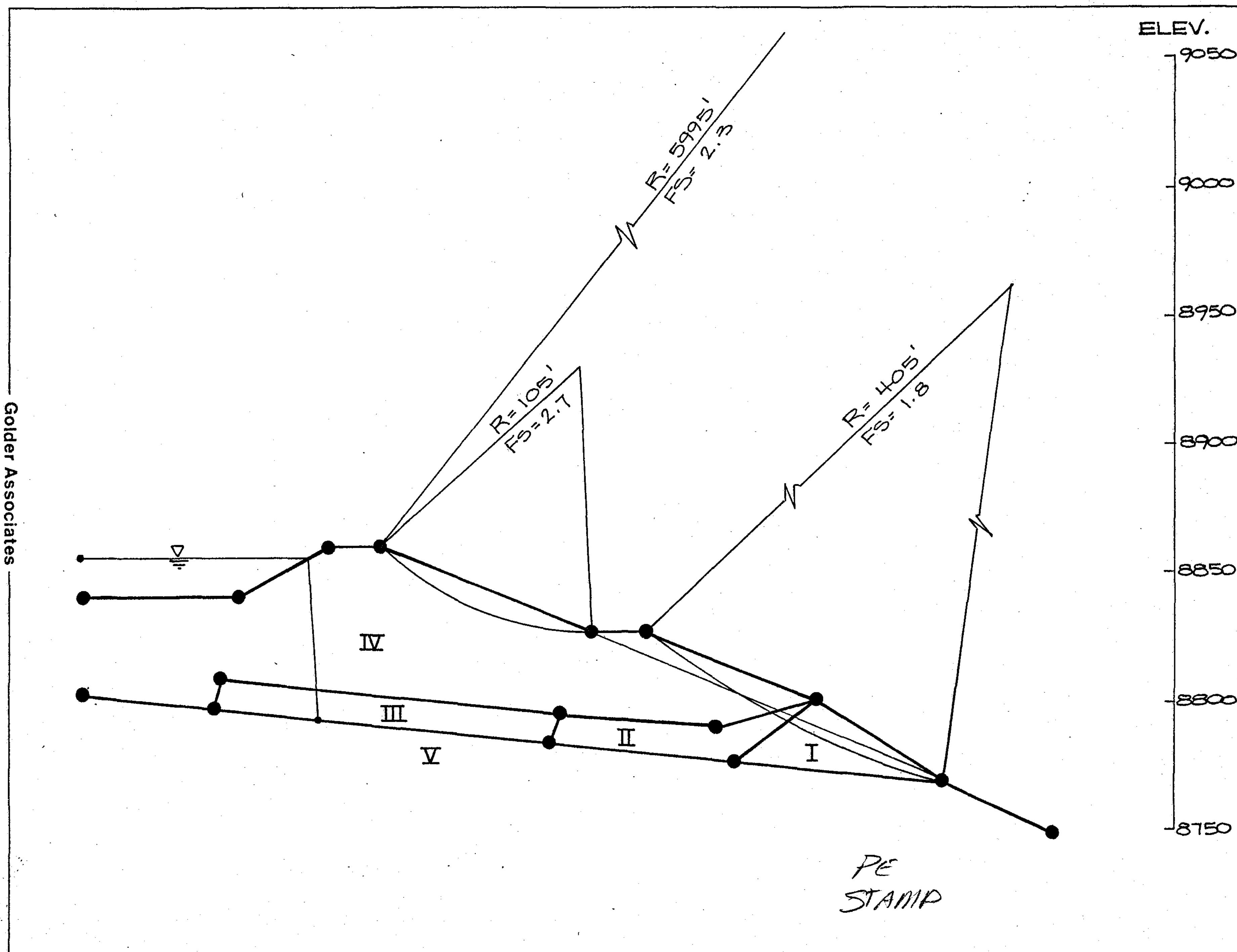
Project No. 793-1156A Reviewed CWL Date 2-80



EMBANKMENT STABILITY ANALYSES
STEADY STATE — FULL DRAINAGE INTO III

Figure 4-12

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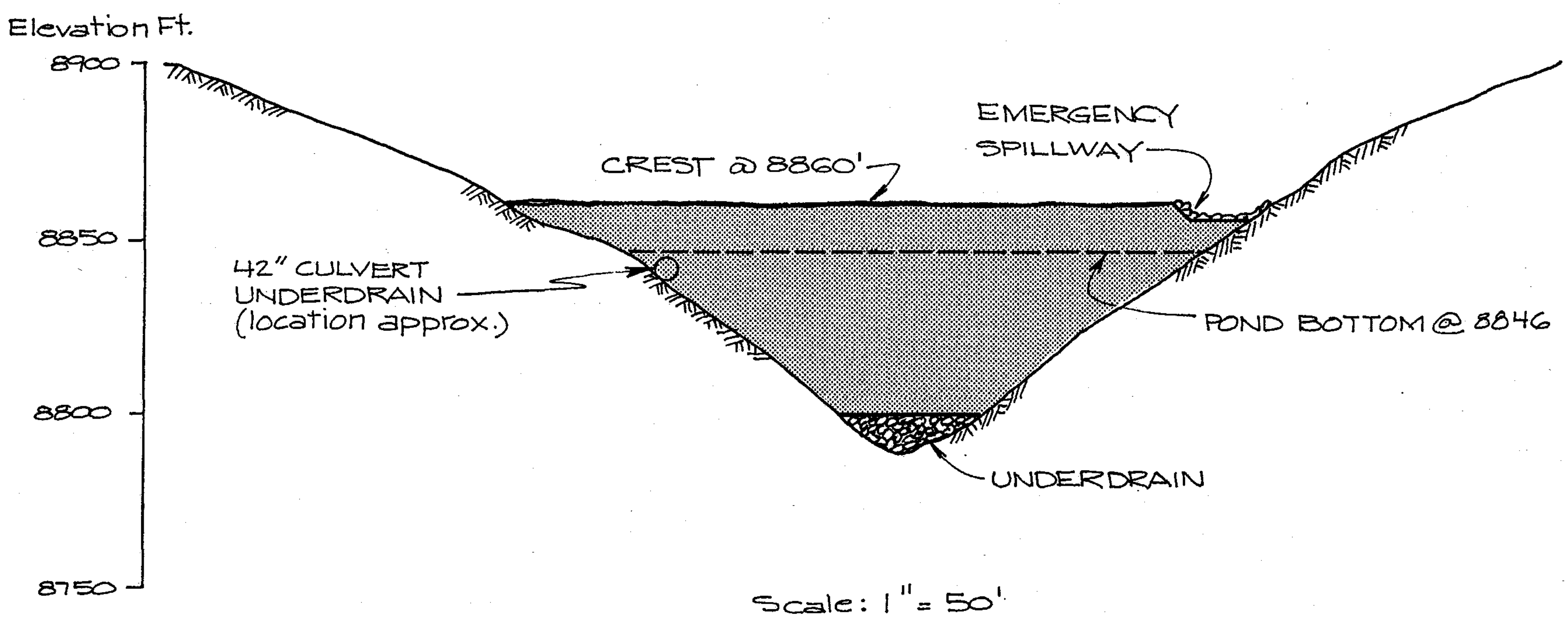

NOTE: Data from field survey undertaken by Valley Camp

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VALLEY FILL CROSS-SECTION

Figure 4-13

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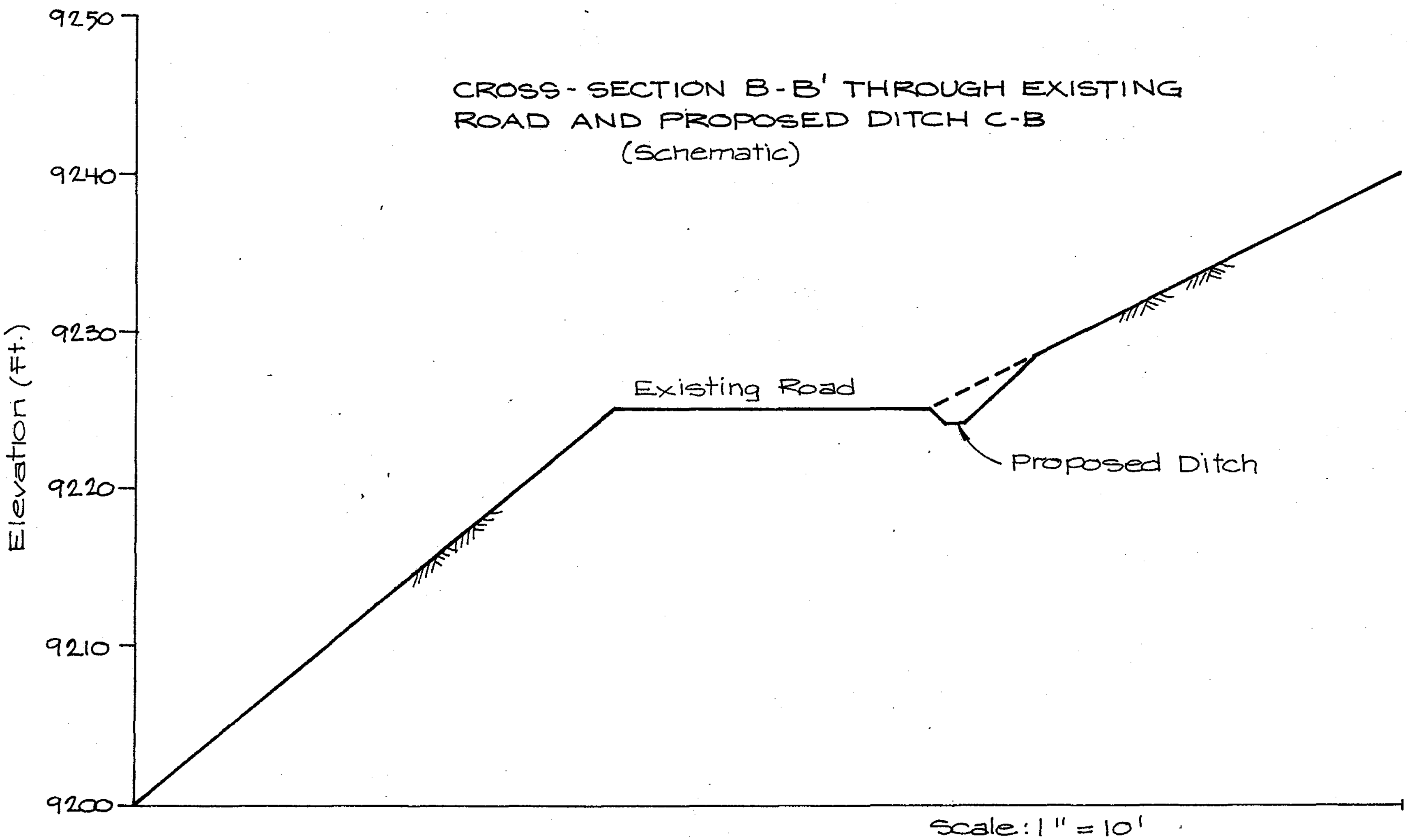

TYPICAL CROSS-SECTION THROUGH EXISTING ROAD AND PROPOSED DITCH C-B

Figure 5-1

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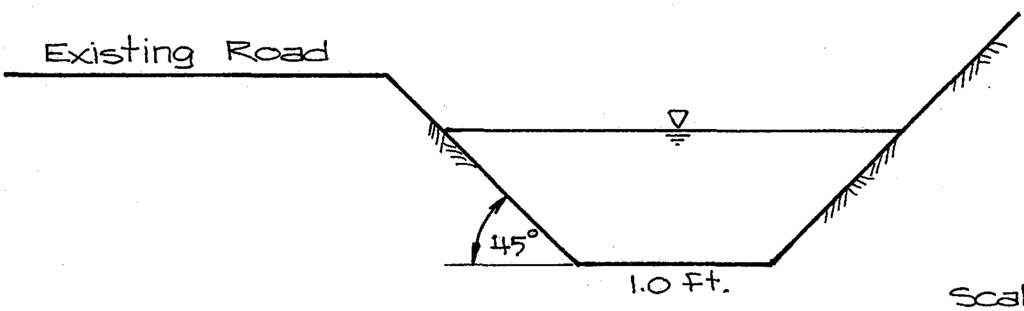
CROSS-SECTION THROUGH DITCH AND TYPICAL DESIGN CONFIGURATIONS

Figure 5-2

DITCH A-B
(Schematic)



Scale: 1" = 1'

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

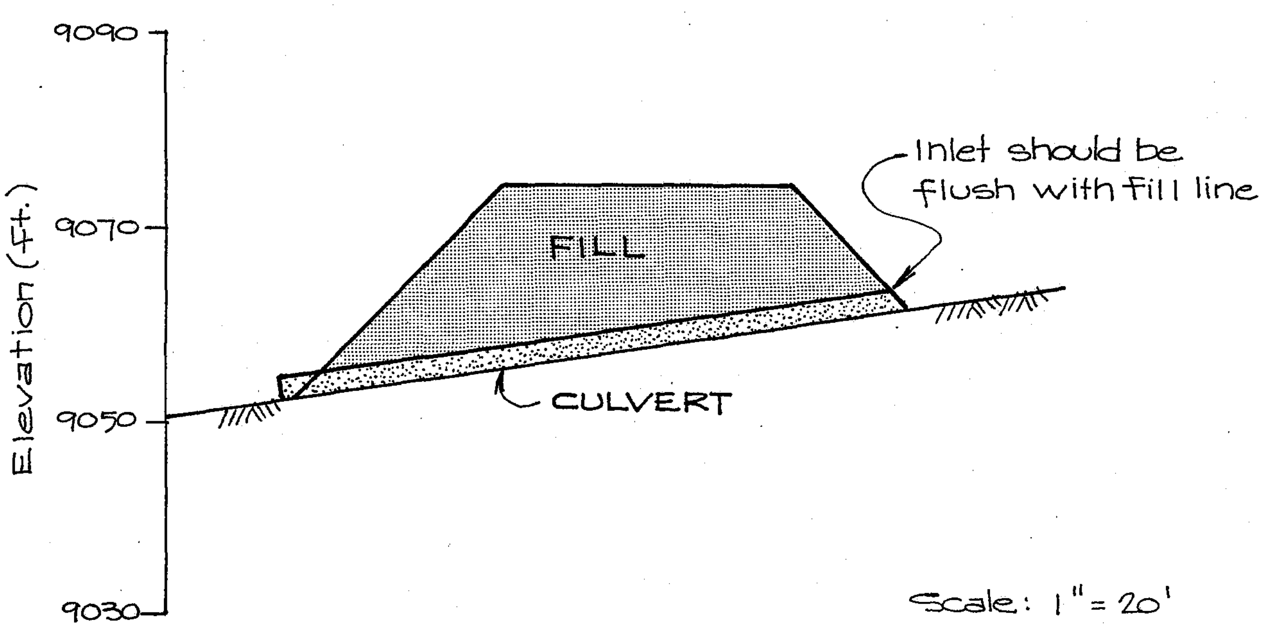
$n = 0.027$
$S = 0.089$
$Q = 12.6$ cfs

Depth of flow y was solved for by trial and error.

$\therefore$ y = 0.70 ft.

INLET CONFIGURATION FOR CULVERT

Figure 5-3



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