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HYDRAULIC MODEL STUDIES OF HORSE MESA DAM ACCESS ROAD STABILIZATION PROJECT

November 1988

Prepared for
Salt River Project
Phoenix, Arizona

by

Engineering Hydraulics, Inc.
Redmond, Washington

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January 18, 1994

U. S. Bureau of Reclamation
Kathy Frizell D-3751
P. O. Box 25007
Denver, CO 80225

Dear Kathy:

Enclosed is the Engineering Hydraulics, Inc. Final Report that you requested.

Please feel free to make a copy if you need one and return the original to me at:

Salt River Project
Dan Pellouchoud - POB012
P. O. Box 52025
Phoenix, AZ 85072-2025

Sincerely,

Dan Pellouchoud

DAN PELLOUCHOUD
Manager
Hydro Generation Division

ph

Lower access road
Stabilization
1 -05.02

FINAL REPORT

Hydraulic Model Studies of Horse Mesa Dam Access Road Stabilization Project

EHI Project No. 5830-002

Prepared for:

SALT RIVER PROJECT
Phoenix, Arizona

November 1988

ENGINEERING HYDRAULICS, INC.

REDMOND, WASHINGTON

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1. EXECUTIVE SUMMARY

The lower access road to Horse Mesa Dam has been washed away six times in the past twenty years during spillway and tunnel releases in the range of 3,000 cfs to 70,000 cfs. A physical hydraulic model study was conducted to investigate alternative structural and non-structural strategies to protect the Horse Mesa Dam lower access road from destruction due to erosion by flood releases. This report presents the results of the hydraulic model study.

A 1:50 scale model of the Horse Mesa Dam was constructed in the hydraulics laboratory of Engineering Hydraulics, Inc. (EHI) in Redmond, Washington. The area of this model below the dam and in the critical areas of the access road is shown in Plates 1.1 and 1.2. Construction of the model was completed in October, 1987. The model included the forebay, the dam and spillways, and one-half mile of the Salt River below the dam.

The model study investigated four strategies for protecting the access road: 1) optimization of spillway operating sequence, 2) slope protection along the access road, 3) redirection of tunnel alignment, and 4) dissipation of tunnel discharge energy through the use of a flip bucket.

The spillway operation optimization tests showed that the velocities along the left bank could be substantially reduced by limiting the tunnel discharge to 25 percent of the total discharge. However, bank protection would still be required. A combination retaining wall-riprap blanket system was designed and tested. With this system the road was not damaged for flows up to 92,000 cfs while using the optimized spillway operating sequence which had been modified to accommodate the Salt River Project (SRP) decision to avoid the use of the south spillway.

Tests were conducted to determine the feasibility of protecting the access road from releases comprised totally of tunnel discharges. A combination retaining wall-riprap system would be a viable alternative. Redirecting the tunnel alignment did not reduce the level of bank protection required. Test results

indicated that a flip bucket could be used to reduce the size and quantity of riprap that would be required to protect the access road from tunnel releases; however, the effect of spray caused by the flip bucket on the adjacent talus slope is unknown and testing of this alternative was not completed to the extent that either the flip bucket or riprap system design were finalized.

Construction cost estimates were prepared for all viable bank protection plans. Estimated costs ranged from $2.3 million for a low height retaining wall and riprap to protect the access road only from tunnel discharges to $7.2 million for a full-length retaining wall and riprap system without operational limitations.

In summary, the viable alternatives which were derived from the model test data were:

ALTERNATIVE	STRUCTURAL FEATURES	DESIGN FLOW, cfs	OPERATIONAL CONSTRAINTS
Spillway Operation	2 ft rock on road fill face from Sta -2+50 to 0+00	approaching 40,000	EHI sequence (Table 5.1).
Riprap Design 1 (Figure 7.3)	Retaining Wall: ◦ Top el. 675 from Sta - 2+50 to +1+00 ◦ Transition top el. 1675 from Sta 1+00 to el. 1658 at Sta 2+75 ◦ Top el. 1658 from Sta 2+75 to 9+92 Riprap: ◦ 6-8 rock ft from Sta -2+50 to 9+92	92,000	None.

ALTERNATIVE	STRUCTURAL FEATURES	DESIGN FLOW, cfs	OPERATIONAL CONSTRAINTS
Riprap Design 2 (Figure 7.4)	Retaining wall per above Riprap: ◦ 4-6 ft rock from Sta -2+50 to 2+50 and 6+50 to 9+92 ◦ 2-3 ft rock from Sta 2+50 to 6+50	55,000 or 70,000	SRP sequence (Table 5.1). EHI sequence.
Riprap Design 6 (Figure 9.1) With toe retaining wall (not tested)	Retaining Wall: ◦ Top el 1658 from Sta 4+00 to 6+00 Riprap: ◦ 6-8 ft rock from Sta 1+00 to 9+00	46,500 92,000*	Powerhouse and tunnel spillway releases only. 40,000 cfs tunnel 6,500 cfs powerhouse. 35,500 cfs north spillway.

* Road fill was lost upstream from Station 1+00

2. INTRODUCTION

2.1 Terms of Reference

The studies described herein were performed by Engineering Hydraulics, Inc. (EHI), for Salt River Project (SRP) under SRP Purchase Order No. VV24044CSS, SRP File No. NC-301.

2.2 Project Background

The facilities at Horse Mesa Dam include: a 266 ft high concrete arch dam; two gated overflow spillways, one at each abutment of the dam; a 30 ft diameter tunnel spillway through the right (north) abutment; a powerhouse containing three 10 MW standard turbines; and a separate powerhouse containing a reversible pumped storage unit.

The un-gated design capacity of the spillway is 150,000 cfs. This was termed the PMF flow for the purpose of this study. The actual PMF has been revised by operation of Roosevelt Dam upstream from Horse Mesa.

A portion of the lower access road to Horse Mesa Dam, approximately one-third of a mile in length, has been periodically washed out during releases from the spillway tunnel and/or the north spillway. The road has been destroyed six times in the past twenty years by flood releases ranging from 3,000 cfs to 70,000 cfs, as summarized below:

Date	Estimated Discharge (cfs)
February, 1968	3,000
December, 1978	50,000
January, 1979	65,000
March, 1979	40,000
February, 1980	70,000
October, 1983	43,000

The loss of the lower road prevents access to the powerhouse for maintenance and overhaul. Each release has resulted in large

quantities of road fill material, including boulders and gravel, being deposited in the river and in the tailrace. The deposition of material in the riverbed blocks the tailrace and precludes the usage of the pump back unit. The costs of rebuilding the road and of lost revenue due to the shut down of the powerhouse have been approximately $5.0 million.

2.3 Study Objectives

Salt River Project (SRP) is considering several alternative solutions to the lower access road erosion problems. The primary alternatives are:

a) Modification of spillway operation,
b) Slope protection,
c) Tunnel energy dissipation,
d) Redirection of tunnel flows, and
e) Various combinations of these alternatives

SRP contracted EHI to perform a hydraulic model study investigate solutions to the problems. The objectives of the study were specifically to:

1. Fabricate and calibrate a hydraulic model to be used as an interactive design tool in the investigation. The model was to be fabricated in such a manner that it could be dismantled and shipped to SRP upon completion of the work;

2. Test alternative solutions to the hydraulic problems;

3. Develop details of the hydraulic design features of the effective solutions; and

4. Prepare budget level cost estimates for the technically feasible alternatives.

3. MODEL SIMILITUDE

3.1 Introduction

Scale models of hydraulic structures are used as interactive tools to replicate the kinematic and dynamic properties of fluid motion through full-scale structures. Complete similitude requires that the model and prototype are kinematically, dynamically, and geometrically similar. It is not usually possible to achieve complete similitude between the model and prototype. However, once the two dominant forces affecting the fluid motion of primary interest in the particular study are known, a model can be designed such that the ratios between the two dominant forces will be the same in the model and the full-scale structure.

The principal forces that influence fluid motion in most hydraulic engineering problems are: gravity, inertia, pressure, viscosity, and surface tension. The scaling laws for the force ratios relevant for the Horse Mesa Dam model study are discussed in the following sections.

3.2 Scaling Laws

Gravity and inertia are the forces principally responsible for the flow patterns of both the full-scale and model Horse Mesa Dam. A unit force due to inertia may be written:

$$f_i = \rho V^2/L$$

and a unit force due to gravity may be expressed:

$$f_g = \Delta\gamma$$

where:

ρ	=	unit mass of the fluid
v	=	velocity
L	=	characteristic length
$\Delta\gamma$	=	the change in unit weight of fluids at their interface

The ratio of these forces becomes:

$$\frac{f_i}{f_g} = \frac{V^2}{L \frac{\Delta\gamma}{\rho}}$$

In this model; the water was in contact with the atmosphere, so $\Delta\gamma$ may be replaced by γ, the unit weight of water.

Since

$$\frac{\gamma}{\rho} = g$$

where g = the acceleration of gravity.

the force ratio becomes:

$$\frac{f_i}{f_g} = \frac{V^2}{Lg}$$

which is the square of the Froude Number, **F**.

Therefore, in order to maintain the same inertia-gravity force ratio in the model and prototype, the Froude Number of the model must equal that of the prototype.

The scale, or length ratio of the model is defined as:

$$L_R = \frac{L_p}{L_m}$$

where

L_p = a certain prototype dimension

L_m = a corresponding model dimension

If the model is geometrically similar to the prototype, then the velocity and discharge ratios can be derived to yield:

$$V_R = L_R^{\frac{1}{2}}$$

and

$$Q_R = L_R^{2.5}$$ respectively.

The Horse Mesa Dam model study was operated as a Froude model.

The Reynolds Number, **R**, is an indicator of the relative dominance of inertia forces over viscous forces. It is defined as:

$$\mathbf{R} = \frac{VL}{\nu}$$

where:

ν = kinematic viscosity

Since the Horse Mesa model was operated under Froude similitude criteria, and since water is the fluid used in both the model and prototype, then the prototype and model Reynolds Numbers cannot be equal. However, the model Reynolds Number exceeded 100,000 everywhere downstream of the forebay. This means that fluid viscosity did not influence flow patterns or drag forces in the model more than it does in the prototype.

The Euler Number, **E**, is defined as:

$$\mathbf{E} = \frac{V}{2\ \Delta P/\rho^{\frac{1}{2}}}$$

where: ΔP = pressure difference.

It is the ratio of the inertia forces to pressure forces. The Euler Numbers of the model and prototype were equal. Therefore, the model could be used to develop a rating curve for the spillway gates, and the pressure distribution in the model flip bucket could be scaled to estimate the pressure distribution in the full-scale flip bucket.

The only fluid motion phenomenon of interest that the model did not accurately replicate was the production of spray and mist.

The ratio of inertia forces ($\rho V^2/L$) to surface tension forces (σ/L^2) is the square of the Weber Number, **W**.

$$W^2 = \frac{\rho V^2 L}{\sigma}$$

where: σ = force of surface tension at the fluid interface.

Since water was used in the model

$$\frac{\rho}{\sigma}_m = \frac{\rho}{\sigma}_p$$

where m and p refer to model, and prototype, respectively. Therefore, the ratio between prototype and model Weber Number is:

$$W_R = \frac{W_p}{W_m} = \frac{V^2 L_p}{V^2 L_m} = V_R{}^2 L_R = (50)(50) = 2500$$

Inasmuch as the size of spray droplets and quantity of spray produced is related to the ratio of kinetic energy to surface energy per unit volume, the above derivation shows that the relative surface tension forces in the model were much stronger

than those of the prototype and that the production of spray and mist will be considerably less than in the prototype.

3.3 Modeling Riprap

The following relationships were used to relate model to prototype riprap sizes. [1]

$$\frac{\left(W_r\right)_m}{\left(W_r\right)_p} = \frac{\left(\gamma_r\right)_m}{\left(\gamma_r\right)_p}\left(\frac{L_m}{L_p}\right)^3\left[\frac{\left(S_r\right)_p - 1}{\left(S_r\right)_m - 1}\right]^3$$

and

$$\left(l_r\right)_m = k_\Delta\left[\frac{\left(W_r\right)_p}{\left(\gamma_r\right)_p}\right]^{.33}\left(\frac{L_m}{L_p}\right)$$

where

W_r	=	weight of riprap stone, lb.
γ_r	=	specific weight of stone, pcf
w	=	water
m	=	model
p	=	prototype
L_m/L_p	=	scale ratio
S_r	=	specific gravity of riprap (γ_r/γ_w)
l_r	=	characteristic length of riprap particle
k_Δ	=	layer coefficient, 1.15 for rough quarry stone

[1] Technical Report HL-82-26; Revetment Stability Study, Fort Fisher State Historic Site, North Carolina, U.S. Army Engineer Waterways Experiment Station.

3.3 Spillway Rating Curves

Discharge rating curves were developed for the north, south and tunnel spillways. The curves were developed by setting a known discharge and gate opening, waiting for the forebay water level to stabilize, then recording the forebay water level. This process was repeated for each gate opening for each spillway. The results are plotted in Figures 3.1 through 3.3. These curves were used to set flow rate combinations, knowing the forebay water level and the desired flow split among the spillways. The combined capacity of the spillways was confirmed at 152,000 cfs at a forebay level of el. 1920 ft (the top of the parapet wall).

3.4 Model Calibration

The water surface profile in the Salt River in the reach covered by the physical model is a function of the pool level of Mormon Flats Reservoir and the discharge rate in the river. Discussion with SRP staff revealed that there are no stage vs. discharge data for the river at the downstream limit of the model. During the field program that was conducted prior to the construction of the model, stage data were collected for two relatively low flow rates, i.e., less than 5,000 cfs. The U.S. Army Corps of Engineers Water Surface Program HEC-2 was used to estimate the water levels in the river for the range of flow rates to be run in the physical model. The selection of the roughness coefficient, Manning's n, to be used in HEC-2 was based on an empirical relationship between n and the ratio of the hydraulic radius, R, and the D_{84} size of the bed material. The methodology was presented by Burkham and Dowdy, 1976 [2]. The Manning's n value was estimated to be 0.041.

The pool elevation of 1655.5 for Mormon Flats Reservoir was assumed to be a representative value. Using a Manning's n of 0.041 in the HEC-2 program, the water surface elevations were computed for the station corresponding to the downstream limit of the model, and for Station 8+80. The water surface profile was computed for

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[2] Burkham, D.E., and Dawdy, D.R., "Resistance Equation for Alluvial Channel Flow," Journal of the Hydraulics Division, ASCE, vol. 102, No. HY 10, October, 1976.

the following discharges: 2,600 cfs, 4,500 cfs, 15,000 cfs, 40,000 cfs, 70,000 cfs, 92,000 cfs, and 150,000 cfs. Figure 3.4 includes a plot of the computed water surface elevation at Station 8+80.

The roughness calibration of the physical model was checked by setting a known discharge, adjusting the tailwater control gate at the downstream limit of the model to set the downstream water surface elevation, then measuring the water surface elevation at Station 8+80. The model roughness is considered to be correct if the measured water surface elevation matches the computed values. Figure 3.4 compares the measured water levels with the computed values. The agreement between the computed and measured water levels is very close through approximately 40,000 cfs. At 92,000 cfs, the measured water level is about two feet higher than the computed value. This difference is probably due to fluctuating wave heights and non-uniform flow in the model. Overall, the calibration was considered to be good.

4. MODELING METHODOLOGY

4.1 Model Construction

A 1:50 scale model of the Horse Mesa Dam was constructed at the hydraulics laboratory of EHI. The model included a section of the reservoir beginning at a point 500 ft upstream of centerline of the dam. This section of the reservoir is referred to as the forebay in this report. The river model extended to a point 2,400 ft downstream of the foot of the dam. Figure 4.1 shows the layout of the model.

Two pumps of 5 cfs capacity each were used to circulate water through the model. A water storage tank was constructed below the river model section.

Salt River Project (SRP) specified that, after the completion of the model study, the model would be dismantled and all of the model components would be shipped to SRP. Accordingly, the model forebay and the river model were constructed using 4x4x12 ft. panels of lightweight insulating foam.

The topography was formed in the panels using the following technique. First, a male mold of the panel's topography was made in a box containing oiled sand. The sand was shaped using female templates suspended over the sand. The templates were removed, and insulating foam was sprayed over the mold. After the foam panel had hardened, the panel was removed from the sand-filled box, and the sand was remolded for the next panel. The sand residue adhering to the surface of the panel provided model surface roughness.

The river model panels were mounted on plywood decking above the water supply reservoir. The forebay model panels were installed on an elevated platform constructed of concrete blocks and timber joists. The joints between the panels were then sealed and trimmed.

The dam model was constructed of timber studs placed in a circular arc and faced with 0.25-inch thick PVC sheet on the upstream and downstream faces.

The models of the north spillway, south spillway, and the bypass tunnel were constructed of clear acrylic sheet. All spillway and tunnel gates were designed to be operated manually.

The detailed drawings used to construct the model are contained in Figures 4.1 through 4.14. All model drawings are based upon copies of the construction drawings for the dam that were provided by SRP.

4.2 Instrumentation

4.2.1 Velocities

Velocity measurements were made with a Nixon Model 403 low speed velocity probe. The sensing element is a 0.40-inch diameter plastic rotor. An underwater electrical field is established between the probe housing and an electrode. A pulse is generated each time a rotor blade passes the electrode (5 times per revolution). The probe is designed to operate in the velocity range of 2.5 to 150 cm/sec. A Novonic Streamflow digital counter was used to count and display the pulse frequency. The pulse frequency is proportional to velocity. A linear regression equation was used to convert frequency to velocity.

4.2.2 Wave Heights

Wave heights were measured with a custom fabricated capacitance wire probe. The probe can be used to measure a peak to trough dimension of up to six inches. The probe was connected to a Bell & Howell Model Datagraph 5-144 recording oscillograph. The voltage output of the probe is proportional to the depth of immersion of the probe. The output of the recording oscillograph has the units of volts. A calibration curve of output voltage versus depth of immersion was made. A linear regression equation was used to convert output voltage to wave height.

4.2.3 Water Surface Elevations

During the operation of the model, water surface elevation measurements were made in the forebay and in the river upstream of the adjustable tailwater control weir. Measurements were made with Lory Type "A" point gages, accurate to 0.001 feet.

4.2.4 Flow Rates

Two 5 cfs pumps were used to circulate water through the model. Each pump was connected to a 10-inch discharge pipe. One of the 10-inch lines had a 6-inch bypass line to feed water to the model powerhouse. The 10-inch and 6-inch pipes were fitted with 10x7.515-inch and 6x4.55-inch orifice plates, respectively. The orifice plates were fabricated according to ASME specifications for concentrical square-edged orifice plates.

The pressure taps were installed per ASME specifications, and were connected to Meriam Model 20AA25WM U-tube manometers. The pressure taps for the 10-inch orifice plates were connected to separate manometer stands, each equipped with two manometers. One manometer was filled with blue meriam manometer fluid for measuring low flows, the other filled with mercury for measurement of high flows. The taps to the 6-inch orifice plate were connect to a manometer filled with blue meriam fluid.

4.2.5 Visual Documentation

All testing was documented with 35 mm still photographs and video tape. The video camera used to document all model tests was a Sony BMC 1000 Super Beta Camcorder. Two Sony Sl-HF 1000 editing VCRs were used for post processing of video recordings.

4.3 Test Program

The overall objective of the hydraulic model study was to investigate alternative structural and non-structural strategies to protect the Horse Mesa Dam access road from destruction due to erosion by flood water releases through the tunnel spillway or over the north spillway. The non-structural measures consisted of

modifying the operational sequence of the spillways. The structural alternatives tested included riprap protection of the banks, redirecting the tunnel alignment, and retrofitting the tunnel outlet with a flip bucket.

Six major categories of tests were completed during the testing phase of the model study. The categories of tests are summarized below using the task numbering system and names employed in the purchase order specification:

Task Number	Task Name	Objective of Task
1	Model Construction	Construct permanent, transportable model.
2	Calibration Tests	Check roughness of model river bed. Ensure model reproduces the stage vs. discharge relationship of the prototype. Establish rating curves for north, south, and tunnel spillways.
3	Spillway/Tunnel Operation Tests	Determine the sequence for operating the spillways that will result in least erosive shear stress on access road. Measure and record velocities along the bank for optimum operating sequence.
4	Erodible Bed Tests	Using the optimum spillway operating sequence, document the scour and depositional patterns of the river bed. Test discharge rates were: 40,000 cfs, 70,000 cfs, and 92,000 cfs.
5	Slope Protection Plan Tests	The results of the test runs made during Tasks 3 and 4 were used to design a slope protection system. In this task, the slope protection system was installed in the model and tested at the flow rates of 40,000 cfs, 70,000 cfs, and 92,000 cfs. The design was a combination of concrete retaining wall and riprap.

Task Number	Task Name	Objective of Task
6	Tunnel Energy Dissipation Tests	Several flip bucket designs were fitted to the end of the tunnel. The purpose of the flip buckets was to reduce the erosion of the access road by diffusing the concentrated jet that presently attacks the left bank of the river. The flip buckets were used to direct the high velocity flow into the air in a steep trajectory while simultaneously spreading the flow in a wide arc between the river banks.
7A	Re-Direction of Tunnel Flow Tests	The extended centerline of the tunnel intersects the opposite shoreline at an angle of approximately 40 degrees. The angle between the thalweg of the river and the tunnel is about 15 degrees. In this series of tests, the azimuth of the tunnel was changed to reduce the angles between the jet and the bank, and between the jet and the thalweg.
7B	Slope Protection-Tunnel Flow Only	The final series of tests were conducted to determine the feasibility of protecting the access road from a 40,000 cfs tunnel spillway discharge, with no discharge from the other spillways. The resulting design was checked for failure at higher flows.

Each of the test series summarized above are discussed in detail in the following sections.

5. SPILLWAY AND TUNNEL OPERATION STUDIES

5.1 Objective

The discharge from any one of the spillways produces a unique overall flow pattern in the river. During the simultaneous operation of more than one spillway, the flows from each spillway interact to produce different flow patterns. The objective of the Spillway and Tunnel Operation Tests was to determine the potential for reducing the erosive attack on the lower access road by modifying the operational sequence of the spillways and tunnel.

EHI first conducted a series of tests to study the flow patterns caused by the operation of individual and multiple spillways. The results were used to develop a spillway operating plan that minimized the velocities and wave heights along the access road. Significant design flows were documented using this sequence.

This section also describes some of the spillway operating criteria presently used by SRP and how they would modify the optimum operating sequence.

5.2 Optimization Tests

The spillway and tunnel operation optimization tests were conducted in two parts. The first part involved setting a given total flow rate, then experimenting with different combinations of gate openings on each of the spillways in order to observe the resulting flow patterns. The purpose of this part was to learn how flow from each spillway influences the overall flow pattern in the river. Numerous flow conditions were set. Velocity data were recorded for twelve of the test runs. Figures 5.1 through 5.12 show the flow combinations that were tested, and the velocities at key locations for each combination.

Flow from the north spillway impacts the river near the left bank of the plunge pool. The flow follows the left bank, and the highest velocities are observed there. Flow from the south spillway impacts the river near the north bank of the plunge pool.

The highest velocities are along the right bank of the river. It appeared that the operation of the south spillway would be much less destructive to the access road than the operation of the north spillway. Tunnel discharges impact the plunge pool along a line rotated counterclockwise about 15 degrees from the thalweg of the river downstream from Sta 7+00. Most of the tunnel jet crosses the plunge pool on the surface and impacts the left bank of the river. The flow splits near the bank. The right half of the flow follows the river bank in the downstream direction. The left half follows the river bank in the upstream direction. The operation of the tunnel spillway causes a counterclockwise eddy to be formed in the plunge pool between the jet impact point and the left bank.

The strength of the eddy and the magnitude of the velocities along the left bank increase with increasing flow rate from the tunnel. Conversely, the addition of flow from the north and south spillways counteracts the forces that produce the eddy, with the result that the strength and velocities in the eddy are reduced. The rotation of the eddy decreases as the ratio of tunnel flow to total flow decreases.

The eddy disappears when the tunnel discharge is limited to approximately twenty-five percent of the total discharge. This condition is generally true for the range of flow rates tested, up to 92,000 cfs. It also produces the lowest velocities along the left bank.

5.3 Constraints on Spillway Operation

During the first witness test, EHI demonstrated to SRP that the operation of the south spillway results in lower velocity and turbulence intensity along the access road than does the operation of the north spillway. The effectiveness of limiting the tunnel flow to 25 percent of the total flow, while using the south spillway to capacity before the north spillway is used, was also demonstrated. This combination of spillway operating measures was termed the "EHI Sequence." SRP agreed that the EHI Sequence significantly reduced the velocities along the bank. However, SRP has a preferred sequence for operating the spillways that is based as much on the environmental effects of spillway operation as it

is on the hydraulic effects. Traditionally, the tunnel spillway was used before either of the spillways on the dam. Currently, the north spillway is operated first, then the tunnel spillway is operated. The operation of the north spillway generates spray and strong winds in the canyon near the powerhouse. The electrical gear on the roofs of the powerhouses is exposed to the wind-driven spray as are the elevator and stairway on the downstream dam face. These are the only operation's access to the powerplant during spillway operation. The south spillway, located directly above the powerhouse, has never been used. SRP has assumed that the wind and spray generated by the south spillway would be more severe than that generated by the north spillway. The south spillway will be used only after the north spillway and tunnel spillways have both reached capacity.

Their concerns about the spray generated by the operation of the south spillway, prompted a modification of the EHI Sequence, which was called the "SRP Sequence." The SRP Sequence limits the tunnel flow to 25 percent of the total flow, but uses the north spillway to capacity before the south spillway is used. In addition, the west gates on the spillway are used before the east gates. This is to keep the source of the spray as far away as possible from the powerhouse. The gate operating schedules for EHI Sequence and SRP Sequence are summarized in Tables 5.1 and 5.2, respectively.

5.4 Documentation of Flow Conditions for EHI Sequence

Flow conditions were documented for the EHI sequence, which had been derived from the results of earlier tests. The following flow rates were set in the model and documented on video tape and in still photographs: 6,500 cfs, 11,500 cfs, 15,000 cfs, 20,000 cfs, 30,000 cfs, 40,000 cfs, 50,000 cfs, 70,000 cfs, 92,000 cfs, and 150,000 cfs. Pictures of flow conditions for the 40,000 cfs, 70,000 cfs and 150,000 cfs cases are included in Plates 5.1 through 5.3.

Velocity data were collected for the following discharges: 20,000 cfs; 40,000 cfs; 70,000 cfs; and 92,000 cfs. These flow rates were selected for more detailed documentation because they were

potentially significant design flows. The maximum tunnel discharge is 40,000 cfs. The flood of record is about 70,000 cfs. The 200-year flood discharge of 92,000 cfs has been recommended as a design discharge by SRP's consultant, Mr. Jacob Douma.

The velocity data are contained in Figures 5.13 through 5.16. Similar data were not collected on the fixed bed model for the SRP sequence. However, conditions were compared for the two alternative sequences during erodible bed tests described in the following chapter.

6. BASELINE SCOUR STUDIES (SRP AND EHI SEQUENCES)

6.1 Objectives

During this task, a portion of the rigid bed river channel was removed and replaced with an erodible bed section. The purpose of the baseline scour tests was to determine the equilibrium bed profiles for a range of flow rates, while using the optimized spillway operating sequences. The SRP and EHI operating sequences were used for 40,000, 70,000, 92,000, and 150,000 cfs. After each test run, the bed contours were documented by taking cross section measurements. The results of the tests were used during the design of the bank protection system. The results were also used to compare the relative scouring potential of the EHI sequence and SRP sequence.

6.2 Preparation of Erodible Bed

The model rigid bed panels were removed between Station -0+40 and Station 7+50. See Figure 6.1. A deep wooden box was constructed in the place of the panels. The box was filled with a compacted mixture of stiff clay and sand. The surface of the mixture was brought to a finish grade equal to the estimated surface elevation of the bedrock. A mixture of 3/8-inch gravel was placed above the clay to simulate the granular composition of the existing road fill material. The gravel simulated prototype rock with a d_{50} of approximately 24 in. The foam roadway between the powerhouse and Station -0+40, and from Station 7+50 to Station 10+00 was cut out of the model and replaced with 3/8-inch gravel material. The Hilfiker wall system was installed in the clay material from Station 4+00 to Station 7+50. Downstream of Station 7+50, the wall remained embedded in the original foam panel. The foam bed of the plunge pool area below the spillways, upstream of Station -0+40, was not reconstructed using erodible materials. The erodible bed as installed is shown in Plate 6.1.

6.3 Test Results

Discharges of 40,000 cfs, 70,000 cfs, and 92,000 cfs were run using the SRP Sequence. Velocity and wave data were collected and detailed cross section measurements were made after the bed forms had reached equilibrium for each flow rate. Then the next higher flow rate was set, and the process was repeated. After the completion of the 150,000 cfs SRP case, the bed was restored to the original contours, and the EHI Sequence was run. Plate 6.2 shows flow conditions for the 40,000 cfs flow with the SRP sequence. Figures 6.2 through 6.11 and Plates 6.3 and 6.4 show the changes in bed elevation using the SRP Sequence. Figures 6.12 through 6.21 and Plate 6.5 present the cross-sectional changes for the EHI Sequence.

The EHI Sequence caused less damage to the access road from the powerhouse to Station 5+00. In fact, there was little damage to the road, with a simulated 24 in. rock facing, for the lowest flow tests, 40,000 cfs, with the EHI sequence. This sequence may provide acceptable conditions at the road fill for flows approaching 40,000 cfs. Downstream of Station 5+00, the flow pattern along the access road was nearly independent of the flow distribution between the north and south spillways, and the damage caused by each operating sequence was nearly the same. Both sequences showed a tendency to scour deeply in front of the Hilfiker wall system downstream of Station 5+00.

Figures 6.22 through 6.25 present the wave and velocity data collected during testing with the SRP Sequence and Figures 6.26 through 6.29 for the EHI Sequence. The data indicate, as expected from the scour results, that velocities and wave amplitudes along the access road were generally lower with the EHI Sequence, especially for the lower flow ranges, i.e. 40,000 and 70,000 cfs. For these flows, velocities upstream from station 0+00, near the powerhouse, were anywhere from 80% to 33% of those for the SRP sequence and wave amplitudes were on the order of 50%. Farther downstream, in the area nearer Station 0+50, velocities were consistently 50% or more less than those for the SRP Sequence values.

As flow increased to the 200 year flood (92,000 cfs) and the PMF, the differences became less discernable as the operating sequence became the same. The difference in flow patterns was a residual effect of the differing scour patterns which defined the flow boundary geometry.

These data were used in the design of first trial slope protection systems which were subsequently tested.

7. SLOPE PROTECTION (SRP AND EHI SEQUENCES)

7.1 Review of Previous Results

The preceding phases of the model study were used to determine whether alternate spillway operating sequences could produce velocities along the access road that would be significantly lower than those that are caused by the present spillway operating sequence. Up to this point, the spillway operations optimization studies and baseline scour studies have shown that the velocities along the access road can be significantly reduced by operating the tunnel up to a maximum of 25 percent of the total discharge, and by operating the south spillway to capacity before the north spillway is used, the EHI Sequence. SRP indicated that they would prefer to not use the south spillway until and unless the north spillway and tunnel spillway were already at capacity. Accordingly, the bank protection system design and testing were based upon the SRP Sequence.

It was assumed that the materials to be used for the bank protection system would consist of concrete or soil cement walls, riprap, or some combination of these, and that it would be most cost effective to use materials found near Horse Mesa Dam. Several field investigations were conducted to ascertain the availability of the required raw materials near the dam.

The preliminary design of the bank protection system was based upon:

a) the velocity data collected in the preceding phases of the model study, and

b) the knowledge that adequate supplies of riprap, aggregate and sand exist near the dam.

It was determined that a vertical concrete or soil cement wall would be required near the powerhouse because:

a) riprap of the size required to withstand the flow from the north spillway is not available in the area, and would be impractical to place if it were, and

b) the riprap would have to be placed on a slope of 1.5:1 to 2:1, and the riprap slope would fill the narrow river channel in front of the pump back unit, thereby blocking the flows into and out of the pump back unit.

Downstream of the impact area of the north spillway, say to Station 1+50, the slope of the existing ground below the water level is steeper in many place than the stable slope for riprap. Thus, unless the toe of the riprap slope were supported by a retaining wall, the riprap would have to be extended to near the thalweg of the river. This would increase the quantity of riprap required by a large factor. As a compromise, it was decided to test a slope protection system that consisted of a low-height retaining wall, and riprap.

7.2 Objectives and Methods

The objectives of the tests conducted during the slope protection testing were to test the feasibility of the combination retaining wall/riprap scheme, and to determine the size of riprap required, and the longitudinal extent of the required slope protection system.

A model of a retaining wall was installed in the model. Riprap of several representative particle sizes was placed on the slope between the top of the retaining wall and the top of the slope. Discharges of 40,000 cfs, 70,000 cfs, and 92,000 cfs were tested using the SRP and EHI Sequences. Although the SRP Sequence was the design sequence, the EHI Sequence was also run in order to determine whether the size of the riprap above the retaining wall could be reduced.

7.3 Riprap Design

The initial estimate of the riprap size was based upon the Safety Factor Method, as presented in Simons and Senturk, 1977 [3]. The velocities measured near the bottom and bank during the previous testing were used in the determination of the stable riprap size and slope. In order to minimize the quantity of riprap, side slopes as steep as 1.5:1 were investigated. The diameter of stable riprap particles, for a given safety factor against failure, increases with increasing side slope. The estimated riprap diameter that would be stable on a 1.5:1 slope was greater than eight feet. This size may not be available near Horse Mesa Dam in sufficient quantity, and, in any case, would be an impractical size to place. A 2:1 slope will require a nominal riprap diameter of six to eight feet under some conditions. It is likely that rock in this size range may be quarried.

The riprap scaling equations presented in Chapter 3 were used to design the model riprap mixture. The largest riprap size determined by the Safety Factor Method, as scaled according to model theory, was placed all along the access road during the initial testing at various discharge rates between 40,000 cfs and 92,000 cfs. The riprap size was reduced in those areas where subsequent testing revealed it was possible to do so. The following sections discuss the results of the testing in greater detail.

7.4 Test Results

Figure 7.1 shows the approximate alignment of the existing access road. Figure 7.2 shows the alignment of the retaining wall and the changes to the road alignment between Station 2+00 and Station 5+00. Figure 7.3 shows the area of river bank covered by riprap in the first riprap design test. Plates 7.1 and 7.2 show the appearance of this riprap and retaining wall scheme prior to testing on the model. In Riprap Design 1, 6-8 feet diameter riprap was used throughout. Discharges of 40,000 cfs, 70,000 cfs, and

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[3] Simons, D. B., and Senturk F., Sediment Transport Technology, Water Resources Publications, Fort Collins, Colorado, 1977, pp. 418-430.

92,000 cfs, EHI and SRP Sequences were run using this design. At 40,000 cfs SRP Sequence, some of the 6-foot rock overturned in-place near Station -0+50. This would not constitute failure of the fill. No rock moved during 40,000 cfs, EHI Sequence. At 70,000 cfs and 92,000 cfs SRP Sequence, some of the smaller-to-medium boulders were displaced from behind the retaining wall and were deposited in the river. There was no apparent rock movement during the EHI Sequence. The riprap was also stable for release of 40,000 cfs through only the tunnel. Plate 7.3 shows this design following completion of all testing.

Figure 7.4 shows the distribution of riprap by size for Riprap Design 2. The riprap between the powerhouse and Station 2+50, and the riprap between Station 6+50 and 9+92, was reduced to 4-6 feet in diameter. The riprap between 2+50 and 6+50 was reduced to 2-3 feet in diameter. At 40,000 cfs SRP Sequence, 10 percent of the 4-6 feet rock moved between Station -0+25 and 0+25. Otherwise, all riprap was stable. At 55,000 cfs SRP Sequence, 10-15 percent of the riprap between Stations -1+25 to 1+25 had moved, all 2-3 foot rock appeared stable, and the 4-6 foot rock downstream of Station 6+50 appeared stable. At 70,000 cfs SRP Sequence, riprap between Station -2+00 and 2+00 was unstable.

In Riprap Design 3, Figure 7.5, the upstream reach of 4-6 foot diameter riprap was shortened to Station 1+60, and the riprap at the top of the slope downstream of Station 6+50 was reduced to 2-3 feet in diameter. Approximately 10 percent of the 2-3 foot diameter rock moved. The 4-6 foot diameter rock was stable.

In Riprap Design 4, Figure 7.6, the 4-6 foot diameter riprap along the sloping bank downstream of Station 6+50 was replaced with 2-3 foot diameter rock. The 2-3 foot riprap blanket failed in the constricted area downstream of Station 7+00. Downstream of Station 1+50, during discharge of 40,000 cfs through only the tunnel up to 20 percent of the material moved either vertically down slope, or diagonally across the slope in the downstream direction.

In Riprap Design 5, Figure 7.7, the 4-6 foot diameter rock between the powerhouse and Station 1+60 was stable up to about 55,000 cfs, but experienced noticeable movement at 92,000 cfs. The 2-3 foot

diameter riprap on the road was suitable for all flows tested as protection for the top of the slope except at 92,000 cfs and the tunnel only case, in which about 5-10 percent of the rocks moved. This movement is shown by the variation in the rock color patterns in Plates 7.4 and 7.5. The 3-4 foot rock was adequate on the side slope between Stations 1+60 and 9+92 up to 70,000 cfs, but was unstable for 40,000 cfs discharged only through the tunnel.

Figures 7.8 through 7.14 show the velocity field on the bed for the EHI and SRP Sequences for 40,000 through 92,000 cfs, and for the tunnel only case.

The above tests demonstrated that:

1. Riprap Design 1 will operate satisfactorily for any flow combination through 92,000 cfs, and

2. Riprap Design 2 will operate satisfactorily through 55,000 cfs, SRP Sequence. Based upon comparison of wave and velocity data for the EHI and SRP Sequence (see Figures 7.9 and 7.12), it appears that Design 2 will perform satisfactorily through 70,000 cfs, EHI Sequence.

8. BASELINE SCOUR STUDIES (TUNNEL RELEASE SEQUENCE)

8.1 Objectives

The slope protection measures discussed in Chapter 7 will protect the full length of the access road. The design discharge for which complete protection will be provided will depend upon the size of the riprap that is placed between the concrete retaining wall and the top of the slope. This chapter discusses a series of tests that were conducted to determine the feasibility of providing complete access road protection against discharges from only the tunnel. The Tunnel Release Baseline Scour tests were conducted to collect sufficient baseline velocity and scour data that could be compared with the velocity and scour data resulting from re-directing the tunnel alignment.

8.2 Test Results

A baseline scour test was conducted to determine erosion patterns and scour and deposition limits that would result from discharging 40,000 cfs from the tunnel and 6,500 cfs from the powerhouse. Figures 8.1 through 8.10 present the initial cross sections taken before the test run for Stations 0+50 through 9+00 and the final cross sections after running 40,000 cfs in the tunnel and 6,500 cfs through the powerhouse. The duration of the test run was 6.3 hours, at which time it was determined that the granular material had armored the bed of the plunge pool and no further scour was taking place.

Review of Figures 8.1 through 8.10 shows that the access road between the powerhouse and Station 2+00 remained intact, but downstream of Station 2+00, the access road was destroyed. The cross-sectional plots also indicate that the granular road fill materials deposited in the bed of the channel starting at about Station 4+00. The channel was nearly blocked between Stations 6+00 and 8+00. The cross sections for Stations 2+00 through 5+00 show that, except for depositional areas, the slope of the scoured banks was nearly the same as the pre-flood banks.

The plots for Stations 4+00 through 8+00 show that the left bank scoured up to the Hilfiker wall structure. This structure is actually a tied-back concrete base wall supporting a Hilfiker retaining system. The tied-back concrete base wall is anchored to bed rock. The model results indicate that the granular material between the river and the wall could be eroded away, so the potential exists to undermine the wall.

9. SLOPE PROTECTION STUDIES (TUNNEL RELEASE SEQUENCES)

9.1 Objectives

The objectives of studying riprap-wall protection systems for discharges only from the tunnel was to compare the cost of providing complete road protection against releases of 46,500 cfs from the powerhouse and tunnel with the cost of providing complete protection against releases of much longer return internal flows such as 70,000 cfs and 92,000 cfs.

9.2 Riprap Design

Figures 8.11a through 8.11c show the velocity patterns at the surface, mid-depth and bottom respectively. These velocities were measured when the bed forms were at equilibrium.

A riprap bank protection system was designed to protect against tunnel only discharges. The design was based on the velocity data mentioned above, and the results of the bank protection studies discussed in Chapter 7. The main difference between the bank protection measures discussed in Chapter 7, and the present case, is that in the previous tests, a low-height concrete retaining wall was used to anchor the toe of the riprap slope. In the present tests, a trench was excavated to anchor the toe of the slope. Based upon the previous tests with the 4-6 foot rock, it was decided that larger rock would be required to protect against tunnel only flow, without the aid of a low-height retaining wall. Accordingly, 6-8 foot rock was used. Figure 9.1 shows the areal extent of the riprap for the tunnel only flows.

Rock groins projecting perpendicularly to the bank line were also considered to reduce velocities and riprap size requirements at the bank. The effect of the groins would be to deflect the currents away from the bank. The general flow pattern would not be changed; the eddy in the scour hole would be concentrated and velocities past the ends of the groins would be greater than those at the bank without the groins. Therefore, even larger material (than 6-8 ft

rock) would be required for the groins. Quarrying rock of this size was considered impractical, so groin designs were not tested.

9.3 Test Results

Figures 9.2 through 9.9 are cross-sectional plots that compare the riprap/channel boundaries before and after a test run of 4.5 hours. Figure 9.10a presents surface velocities and wave data and Figure 9.10b, bottom velocities for this test condition. The riprap design was generally stable except near Station 5+00. The plot in Figure 9.7 indicates that the slope was too steep and that the toe of the slope failed. It appears that the riprap sloughed down the bank and deposited in the river bottom. A concrete retaining wall would likely stabilize the toe of the riprap slope. If this is done, it appears that 6-8 feet diameter rock in this configuration will be sufficiently stable to protect against tunnel only discharges.

Following completion of documentation of the test described above, 92,000 cfs was passed through the model (6,500 powerhouse flow,; 40,000 cfs tunnel flow; and 45,500 cfs flow over the North Spillway). The access road upstream from the 6-8 foot riprap was lost. Road fill was removed from behind the riprap from Station 1+00 to about Station 1+75. As the fill was removed, the 6-8 foot rock in this region sloughed to a flatter slope and armored the exposed end of the fill. Fill material from the upstream section of the road, which had failed, deposited at the toe of the 6-8 foot rock, protecting it. Other than a continuation of the sloughing failure of the riprap at Station 5+00, noted during the previous test, the remaining road cross-section appeared stable. With the addition of concrete retaining wall to prevent sloughing at Station 5+00, as previously described, it appears this configuration will be stable downstream from station 1+00 for a 92,000 cfs flow.

10. TUNNEL REDIRECTION STUDIES

10.1 Objectives and Methods

One of the objectives of the overall model study was to investigate alternative strategies to reduce erosion of the access road by modifying the flow patterns from the north, south, and tunnel spillways. During operation of the existing tunnel, the water jet plunges into a wide section of the river. The momentum of the jet is sufficiently high that a strong current crosses the river and impacts the opposite bank. In addition, a strong counter-clockwise reverse flow pattern is created that produces a current in the direction of the powerhouse along the access road. An experiment was conducted to alter the alignment of the tunnel spillway so that the azimuth of the flow discharging from the tunnel would more closely parallel the channel of the river, thereby reducing the erosive attack on the access road.

The end of the tunnel was rotated in a clockwise direction, when viewed from above. A baseline scour test was conducted, and velocity fields were documented. The results were compared with the scour and velocity data of the present tunnel

10.2 Tunnel Redirection Design and Test Results

The downstream 92 feet of the tunnel were removed and replaced at an angle of 5.7 degrees clockwise from its present alignment, as shown in Figure 10.1. This is the maximum angle that the tunnel can be deflected without forcing the jet into the right canyon wall.

The erodible bed material and access road were restored to the original topography. A flow of 40,000 cfs was released from the tunnel until the bed forms reached equilibrium. Velocity and wave data were collected. Detailed cross section measurements were taken before and after the run. The cross section and velocity data were compared with the baseline scour data taken from tunnel releases with the tunnel in its existing alignment. Figures 10.2 through 10.11 compare the before and after cross sections. Figures

10.12a through 10.12c show the velocity vectors at the surface, mid-depth and bottom, respectively.

The damage to the access road that was caused by the redirected tunnel flow was nearly the same as that caused by the flows through the existing tunnel. Redirecting the tunnel 5.7 degrees clockwise did not alter the geometry of the system sufficiently to change the flow pattern. The overall velocity pattern for both cases was the same. The peripheral velocities along the access road were nearly identical.

The results of the redirected tunnel tests show that there would be no advantage to altering the alignment of the tunnel, since the same level of riprap bank protection would be required as with the existing tunnel.

11. TUNNEL ENERGY DISSIPATION

11.1 Objectives and Methods

The purpose of the tunnel energy dissipation studies was to develop a flip bucket design that would eliminate the reverse flow eddy and reduce the magnitude of the velocity acting on the left bank. This was to be achieved by spreading the flow into a fan-like pattern, while simultaneously flipping the water into the air in at least a 45-degree trajectory.

The combination concrete wall-riprap slope protection system that was tested during the slope protection design task was left in place for flip bucket tests. It was decided to leave the wall in the model until it could be shown that a flip bucket would be sufficiently effective in protecting the road that no riprap would be required on the road embankment.

Four different flip bucket versions were tested. Each flip bucket version underwent several modifications. See Figures 11.1 through 11.4.

11.2 Flip Bucket Test Results

The design of the flip bucket was made more complicated than usual because the stream lines of the flow in the tunnel are not parallel to the centerline of the tunnel. The tunnel turns 32.4 degrees to the left approximately 250 feet downstream of the crest. This causes the flow to ride up on the right wall of the tunnel in the turn, forming a shock wave. Thereafter, the shock wave crosses back and forth between the right and left walls of the tunnel, riding high up on the side wall before it accelerates towards the opposite side. The locations at which the wave is reflected from the side wall vary with the flow rate. This condition made it difficult to design a flip bucket that worked well for a wide range of discharge rates. At 40,000 cfs the shockwave is crossing from the left side toward the right side at the tunnel exit.

Flip bucket design Number 1 and Number 2 were patterned after the Fontana Dam flip bucket designed by Rex Elder at Tennessee Valley Authority. Several versions of each design were tested. The modifications that were tested did not spread out the flow uniformly, and produced locally concentrated areas of high surface velocities and wave heights along the left bank.

The flip bucket was changed to Flip Bucket Design No. 3, as shown in Figure 11.3 to increase the trajectory angle and to spread the flow more evenly acrosss the tunnel plunge pool. The longitudinal dispersion of the jet is shown in Plate 11.1 However, the flow pattern created by this design displaced some of the riprap on the access road and on the embankment slope. The damage was due primarily to high velocities near the surface and to wave action as shown in Plates 11.2 and 11.3. If the access road had not been protected by the combination concrete wall-riprap system discussed above, the damage sustained by the embankment would have been significantly greater than was observed. Figure 11.5a and 11.5b present the velocities at the surface and bottom of the river, respectively, near the left bank while running 40,000 cfs through the Design No. 3 flip bucket and 6,500 cfs through the powerhouse.

The Design No. 3 flip bucket was designed to form a trajectory angle of 60 degrees above horizontal. The trajectory angle could have been increased slightly in order to reduce the horizontal distance between the flip bucket and the point at which the flow impacts the river. This could have decreased the wave heights and velocities at the access road by a small amount. However, it appeared unlikely that any amount of refinement to this design would have resulted in a sufficiently tranquil flow pattern near the access road that no bank protection measures would be required along the access road.

The flip bucket caused the lower end of the tunnel to flow full at low flow rates. This condition resulted in sub-atmospheric pressures in the tunnel as the hydraulic jump which formed in the tunnel entrained and removed the air trapped in the tunnel. If a flip bucket were installed in the end of the existing tunnel, an air vent system would have to be provided.

The model generated considerable spray and a very noticeable breeze in the vicinity of the flip bucket and near the jet from the flip bucket. The spray, mist, and wind generated by the model flip bucket cannot be scaled up directly to estimate those caused by the full scale structure. However, it can be safely stated that the spray, mist and wind in the full scale structure would be very significant and could lead to other bank stability and operational problems.

The findings discussed above were reported to SRP. SRP requested that EHI's consultant, Rex Elder, pursue the testing of the flip buckets and to provide his recommendation as to whether the flip bucket alternative warranted more detailed investigation. Mr. Elder personally both supervised and participated in the creation and testing of numerous flip bucket modifications. The end result is shown schematically in Figure 11.4 as Design No.4. This design produced a reasonably uniform distribution of flow across the river, and reduced the wave heights and velocities along the access road, as compared to the previous flip bucket tests. Figures 11.6a and 11.6b show the surface and bottom velocities and wave heights that resulted from the operation of Flip Bucket Design No. 4. However, Mr. Elder concluded that slope protection measures would still be required in order to stabilize the left bank, but that the amount of required protection should be appreciably less than for the other slope protection schemes. He also felt that the flip bucket design could be refined further and that this, in turn, should further reduce the amount of protection required. Through consultation with SRP, it was decided not to further pursue design of a flip bucket alternative.

12. SUMMARY AND CONCLUSIONS OF MODEL STUDIES

12.1 Operating Sequence

Initial tests confirmed that operation of the tunnel spillway resulted in impingement of the surface jet on the access road fill and established a counter-clockwise return eddy which eroded the fill in an upstream direction. Operation of the north spillway resulted in impingement of flow on the road adjacent to the powerhouse and a high level of turbulence in the flow immediately adjacent to the fill. South spillway operation concentrated flows on the right bank of the river away from the access road fill.

A series of tests were performed to optimize the sequence of spillway operation to minimize the velocities, turbulence and wave activity along the access road. This resulted in what has been termed the "EHI Sequence" found in Table 5.1. This sequence: limited tunnel flows to 25 % of the total flow to counteract formation of the return eddy along the left bank road fill; utilized the south overflow spillway first to prevent direct impingement of spillway flows on the road fill beneath the south spillway and promote flow concentration and higher velocities to occur along the right bank away from the road fill; and utilized a special sequence of gate operations of the north spillway to disperse the falling jet and reduce impingement on the road fill.

SRP operational preferences, based upon environmental considerations of spray formation rather than hydraulic performance, led to modification of the operating sequence resulting in the "SRP Sequence". This sequence, detailed in Table 5.2, again limits tunnel flow to 25 % of the total at the lower flows, but utilizes the north spillway up to its capacity and then the south spillway.

The third operational sequence considered, the historical SRP operation mode, utilizes the tunnel alone up to its capacity of 40,000 cfs, and then the overflow spillways.

In terms of reducing erosion of the access road, the EHI Sequence is the superior. The use of the south spillway may generate unacceptable spray at the pump-back powerhouse. The model cannot predict how much spray will be generated. Watching the model operation certainly raised the question as to whether the spray would be greater with the south spillway. The SRP sequence resulted in almost immediate erosion of the road in the vicinity of the powerhouse by north spillway flows. The historical or tunnel release scheme resulted in the greatest potential for erosion of the road further downstream in the critical area near the Hilfiker wall. None of these sequences was clearly the best, but each must be considered in the context of what other protection is necessary to stabilize the access road.

12.2 Protection Design

Slope protection schemes consisting of a concrete retaining wall at the base of a riprap protected road fill slope were developed which:

1) Were unlimited by operational sequence and performed satisfactorily for flows through the 200 year design flow of 92,000 cfs (Retaining Wall and Riprap Design 1, Figures 7.2 and 7.3);

2) Based on a comparison of wave and velocity data, probably would have performed satisfactorily for the EHI Sequence for all flows through 70,000 cfs (Retaining Wall and Riprap Design 2, Figures 7.2 and 7.4); and

3) Performed satisfactorily for the SRP sequence for flows through 55,000 cfs (Retaining Wall and Riprap Design 2, Figures 7.2 and 7.4).

Tests of a less extensive riprap system designed specifically to withstand releases of up to 46,500 cfs from the powerhouse and only the tunnel spillway were inconclusive. This system, shown in Figure 9.1, utilized a riprap berm keyed into a trench in the river bed to stabilize the toe of the riprap slope, rather than a wall. The system suffered a slope failure during testing. It appeared

that had the toe between stations 4+00 and 6+00 been stabilized with a short wall that the design would have been successful. However, this was not confirmed through further testing. Other than the Station 4+60 to 6+00 slope failure, the remainder of the protection system (from Station 1+00 to 9+00) was stable for flows up to 92,000 cfs, though the road was lost upstream from the protected area (upstream from station 1+00).

Re-direction of the tunnel flow to more closely align with the thalweg of the downstream river channel did nothing to reduce erosion of the access road.

Dissipation and dispersion of tunnel flow energy through the use of a flip bucket was successful, but not to the extent that no slope protection was required to stabilize the access road. The level of protection that would have been required in combination with a flip bucket was not defined through model testing. However, it was apparent that the level of protection required to stabilize the road downstream from Station 1+00 could be appreciably reduced. The flip bucket could be further refined with some additional savings being probable.

The protection systems described in items 1) through 3) at the start of this section can be recommended depending upon the flow rate to which protection is desired and the limitations to be placed on operating sequence.

13. SITE PROTECTION DESIGN AND CONSTRUCTION CONSIDERATIONS

13.1 Site Description

The construction site is located on the south bank of the Salt River immediately downstream of Horse Mesa Dam. Road access to the site is by way of an access road that branches from State Route 88 about 4 miles east of Tortilla Flat. By air, the site is 17 miles northeast of Apache Junction. The access road between State Highway 88 and Horse Mesa measures 5.2 miles. It has steep to intermediate grades and numerous curves. It has a graded stone surface, nominally 20 feet wide, narrowing at one point to 15 feet. The road is well maintained. The shoulders are narrow (in some places soft and sloughing) with steep, deep drop-offs to side canyons bordering the road for most of its length. Rock overhangs and close rock walls occur near the bottom of the road. The last mile of the road, from the Fish Creek Bridge to Horse Mesa Dam, lies on the south side of the Salt River Canyon with short up and down moderate grades. This section of the river is Canyon Lake, formed by the Mormon Flats Dam. In this stretch, the road varies in width from 20 feet to 30+ feet.

The site is also accessible by watercraft. A loading point located on State Route 88 east of Apache Junction before crossing the first bridge on the southern shore of Canyon Lake has been used on previous projects. Land and water access to the site is shown in Figure 13.1. A 30x90 foot barge that is owned by SRP is located on the lake. It can be used for transporting equipment and materials.

The construction areas extend from the parking lot at Horse Mesa Dam adjacent to the pump back unit to a point approximately 1,300 feet downstream. This portion of the road borders the plunge pools formed by the actions of the ski jump and the tunnel spillways. This is the portion of the access road that has been eroded by spillway release. It is bordered on the south by near vertical canyon walls near the west end of the construction area. The work area is approximately 25 feet wide near Horse Mesa Dam opening to

30-40 feet at the upper road retaining wall. River bars and a gravel storage area at the south edge of the river provide additional working space.

The model tests showed that erosion of the road can be substantially reduced or eliminated by the use of large rock riprap. Rock of adequate quality is present in the vicinity of the construction site. Large boulders occur on the talus slopes on the north side of the river and large boulders have been scavenged from the river bottom and incorporated in to the fill forming the access road. Numerous rock outcroppings occur on the plateau behind the residence area. Also, thick dacite strata occurring on both sides of the river are visible from the access road. The strata on the north side have less burden.

Sand and gravel for manufacturing concrete occurs in the area. Horse Mesa Dam and the spillway tunnel were built using local materials. A deposit of aggregate at Fish Creek was identified in the construction report for the spillway tunnel. Sand and gravel mixture is present along the lower access road.

Working space is extremely limited. There is no space in or near the working area for the contractor's camp. Whatever space exists in the working area must be used for concrete batching and materials stockpiling.

13.2 Previous Work, Studies, and Information

Considerable information on the site, materials, and construction conditions exist in the form of construction reports, special studies, and individual experience and "first hand" knowledge. From the construction reports, it is evident that local materials were used in the manufacture of concrete for Horse Mesa Dam and the spillway tunnel. The concrete in both structures is in excellent condition which testifies to the quality of the aggregate available in the area. Although the source of the aggregate was not identified, it had to come from the canyon below Horse Mesa Dam. The Fish Creek source was identified but not used. Eye witness accounts indicate that large amounts of aggregate exist in the river bottom which can be recovered by dredging or by lowering the

level of Canyon Lake and excavating the material with a bulldozer. Samples of aggregate materials from several locations from along the south river bank were taken during a reconnaissance of the area. The sieve analysis indicates that although there is an adequate amount of fine aggregate, it appears to be deficient in some of the size fractions, particularly minus 100. Coarse aggregate was also deficient. However, this could be misleading due to the river bank location and the small size of the samples. It was noted that the fines tended to break down during the sieving. This tendency could cast some question on the quality of the material, however, it can be reasoned that the same aggregate was used in construction of the previous structures with no apparent detrimental results. The tendency to breakdown could help overcome the deficiency of fines.

A search was made for sources of riprap material. Two sources were located. However, more detailed prospecting should identify additional sources. One of the requirements of riprap for this work is that a supply of 72-inch rock be produced. It is doubtful that every quarry site would produce this size of material due to close bedding and jointing thereby limiting the maximum size of rock that could be produced. Any one quarry site would have to be proven by test blasts. One potential site is a stratum occurring on the north side of the river which appears to be 30-40 feet thick. It was not possible to examine this site closely because of its inaccessibility. However, this stratum appears to continue high above the access road on the south side of the river. The material on the south side was examined although it is covered by several hundred feet of burden. This material is a dacite porphyry which is abundant in the area. The outer surface of this material is badly decomposed for a depth of two to three inches. Once the decomposed material is removed, the base material is sound and intact. No joints were observed. This may be consistent with the large boulders observed on the north side of the river. This observation could not be confirmed because of the lack of access to that side of the river. The specific gravity of this material is less than average rock probably due to its volcanic origin. According to tests conducted by ERTEC in previous studies, dacite porphyry found the area is strong, testing at 9,560 psi or greater. The same material was observed in stream beds in the

general area and was found to occur in large boulders with edges rounded from water wear. From these observations it was concluded that the dacite occurring in the vicinity of the construction site could be obtained and would serve as an adequate riprap material.

Very useful information is provided by the ERTEC report, Horse Mesa Dam Lower Access Road Retaining Walls Geotechnical Investigation of July 1981. This work explored the occurrence of rock in the lower access road in the area subject to spillway erosion. The work consisted of a drilling program and a seismic survey. The dacite porphyry which occurs in rock outcroppings was found in the rock base below the road. The test results of the cores taken in the exploratory program assisted in the evaluation of the rock proposed for making the riprap. The report provides cross sections of the lower access road and map of the rock contours. This information was used in laying out the erosion protection system used to establish technical and cost feasibility.

Interviews were conducted with those familiar with the site and several reconstructions of the road. Video tapes and photographs were examined. From these and the site observations described above, it was learned that the sequence of erosion and reconstruction has resulted in the sorting of the size of material in the access road. The larger rock is at the bottom of the road embankment. Finer material covers the large rock. This contributes to the erodibility of the road material and instability of the side slopes of the embankment. There is presently no protection on the side slopes of the road embankment, and erosion progresses rapidly during spillway operation. The erosion moves the fine materials downstream, depositing them in the riverbed. The erosion process does not remove the very large boulders that have been incorporated in the road fill. These boulders have been scavenged from the talus slopes and the riverbed. From photographs, it is apparent that a very large number of these boulders exceed six feet in size. This leads to the observation that huge rocks occur naturally in the area and that riprap sizes required for protection of the road embankment should be available in the area.

In order to reconstruct the road, it has been necessary to lower the water level in the tailrace exposing the riverbed. Other than the plunge pools, the riverbed is accessible for a considerable distance downstream of Horse Mesa Dam, the elevation of the water level at the unwatered stage was not available. Large quantities of gravel were observed in the exposed river bed. Most of the boulders have been removed from the riverbed and incorporated into the road fill.

13.3 Riprap Options

The model tests have shown that a 2:1 riprap slope using 72-inch and 48-inch riprap is resistant to the erosive forces near, but not under, the spillways. The design of a riprap slope protection system includes:

a) riprap stones of sufficient size and gradation such that the flow energy cannot move them

b) an anchoring system that will prevent undermining or removal of rocks in the toe of the riprap blanket,

c) and a filter system underlying the outer protection system that prevents the removal of the fine embankment material supporting the riprap blanket.

Several options were studied. The simplest system is a riprap blanket covering the full height of the embankment and anchored at its toe with a berm. This is probably the least expensive system to construct since it does not require excavation or dewatering of the tailrace. The anchoring berm is placed as random fill using riprap material. For this system to be successful, the berm must be stable.

Another method of anchoring the toe of the riprap blanket is to excavate a trench at the toe of the blanket below the depth at which erosion could undermine it. The toe trench could be placed in granular bottom materials or into the underlying rock. If the stability of the bottom materials can not be guaranteed, it is advisable to take the trench into rock or bottom material to a

depth that prevents plucking of the anchoring rock from the trench by the erosive forces of turbulent water.

An alternative method of anchoring the riprap blanket must be found if the slope of the finished protective system and the underlying support do not intersect within a reasonable distance of the road embankment. A suitable toe anchor can be provided with a retaining wall. In this plan, a wall is placed at an optimal distance from the shoulder of the riprap blanket with its top at the point of intersection with the riprap. The wall must be built on suitable foundation material and designed as a retaining wall. This concept is illustrated in Figure 13.2.

The concept illustrated in Figure 13.2 is a concrete gravity wall with the sloping side next to the river. This will permit the optimal use of prestressing tendons close to the side adjacent to the fill to help stabilize the wall. With the use of tendons, the amount of earth and rock excavation can be minimized. The weight of the structure and the force in the tendons combine to resist overturning. The stepped foundation helps to reduce rock excavation and prevent sliding. The elevation of the top of the wall will be at the normal operating water level. This will permit flooding of the tailrace before the riprap work is complete. The tendons would be placed when the fill behind the wall is level with the top of the wall.

An alternative to riprap is a full height retaining wall. The fundamental concept of a full height wall is the same as that illustrated in Figure 13.2. The primary difference is in the height of the wall. The obvious advantage of a full height wall is that it does not extend into the river channel. This is an important consideration adjacent to the powerhouse tailrace.

One remaining riprap alternative is the use of soil cement. Soil cement has been used extensively over the last 25 years as a base for secondary roads. Recent applications include: lining of irrigation canals and dam facings in place of riprap. Soil cement essentially uses available natural materials with cement as a bonding agent and is mixed onsite in a pug mill or in place. This is in contrast to roller compacted concrete (RCC), which is a low

slump concrete, prepared using conventional concrete materials and a batch plant. Soil cement would be expected to have a slightly lower compressive strength than RCC, 1,850 to 2,500 psi compared to 3,000 psi, but has been found to give satisfactory service for bank protection with water velocities approaching 30 fps. A properly designed soil cement section would probably serve very satisfactorily in this application. RCC has not been applied historically as riprap protection, but rather as mass structural concrete for gravity dams. The section that is suggested for preliminary consideration is illustrated in Figure 13.3. Soil cement can be placed at a steeper slope than compacted earth or rock riprap because of its inherent strength. The section used for estimating purposes provides for an outer slope of 1:1 and an inside surface slope of 1:1.5. The resulting gravity section is stable against overturning with a saturated fill with no water in the river. The section is characterized by its wide horizontal layers, which are compacted with rollers. The soil cement can be mixed in place or mixed in a central plant, laid down, and compacted in 8-inch lifts. The adjoining layers bond and interlock to form a structural unit. The soil cement material is made from naturally occurring granular material and cement. The optimum mixture is determined by tests but reasonable estimates can be made using a cement content of 10 percent. The material in the road or the river gravels can be used to make the soil cement.

Each of the options discussed above have characteristics that are better adapted to different sections of the embankment than others. The model tested a system that contained a full-height retaining section and a combination wall/riprap section. This arrangement worked successfully. In optimizing the arrangement, it may be possible to substitute one of the options for parts of the tested arrangement, i.e., the soil cement section could be substituted for the full height concrete wall or one of the full faced riprap schemes could be substituted for part of the retaining wall/riprap combination. By looking at various other structural combinations, it may be possible to find an alternate arrangement that will be less costly while providing adequate protection for the road.

13.4 Access Road Alignment

As presently installed, the access road from Horse Mesa Dam to the west end of the reinforced embankment supporting the upper access road (the tied-back wall/Hilfiker system) is nominally 20 feet wide with varying alignment and elevation. The road has been rebuilt several times using eroded materials deposited in the riverbed. No attempt has been made to maintain grade or width except for minimum width. Most drawings of the site indicate a road width of 20 feet. The ERTEC report has used an elevation for the road surface of 1678 and a water level of 1658 in the river.

These studies assumed a road width of 20 feet with a 5 foot shoulder on each side of the road. The grade of the road slopes down from a high point at the upper access road retaining structure to the parking area at Horse Mesa Dam. A water level in the river of elevations 1658 was used. The alignment of the road was reestablished so that estimating quantities could be determined. Although the present alignment was generally followed, some slight realignment was made to provide the maximum space for construction of a slope protection system. The alignment used for these studies is shown in Figure 13.4. Table 13.1 lists the P.I.'s (Point of Intersection of the centerline) defining the alignment shown in Figure 13.4.

13.5 Description of the Wall/Riprap Option

A slope protection system combining a concrete retaining wall and riprap was adopted for model testing. At the time of the testing, the design flow rate was 92,000 cfs using the SRP spillway operating sequence. It has been previously established that a riprap system would be an alternative bank protection method. Review of the literature on the subject showed that resistance of riprap to erosive velocities is a function of slope, size and specific gravity of the riprap material. As the side slope steepens, the required rock size to maintain a given safety factor against failure increases. The present embankment is standing on a slope of about 1.5:1, close to the angle of repose of the material comprising the road fill. Riprap sizes larger than 8 feet in diameter would be required in some areas at a side slope of

1.5:1. At a slope of 2:1 riprap that is 6 feet in diameter will provide the same safety factor. Accordingly, a slope of 2:1 was chosen for testing in the model.

The 2:1 slope chosen for model testing causes the toe of the embankment to extend farther into the river than is presently the case. From the available information, the slope of the river banks becomes steeper with distance from the road. This is the case in the location of the plunge pools. At a 2:1 slope, the embankment will not intersect the river bottom within a reasonable distance from the road berm. Further, the rock contours established in the ERTEC report indicated that the rock is covered by up to 20 feet of burden. All of this indicates that it is not possible to anchor the riprap blanket with either a berm or a toe trench and that it will be necessary to build a retaining wall to anchor the riprap. The placement of the retaining wall is not critical since foundation rock for supporting the wall is available at any distance from the road. The placement of the wall will affect the required height of the wall, however. It is presumed that there is an economically optimum placement but this would have to be determined by an optimization study.

A design was selected for purposes of the model testing. The selected design was based on engineering judgement using the information available at the time. The best arrangement combining structural feasibility and hydraulic performance appeared to be a combination of full height concrete retaining wall section and a riprap/retaining wall section. This arrangement is shown in Figure 13.4 and consists of a full height wall starting at Horse Mesa Dam parking area adjacent to the pump back unit extending downstream 340 feet (road station 4+80). The riprap/retaining wall combination starts at this point with a transition structure following, more or less, the direction of the waterline at the end of the spillway plunge pool. Over the length of the transition, the elevation of the top of the wall changes form 1675 to elevation 1658, nominal water level. The combination riprap/retaining wall section extends from this point to road station 13+83, the end of the protection system. The profile of the proposed wall along its length showing the height of the wall and the cover over the foundation rock is shown in Figure 13.5. This figure shows both

the model testing stationing system and the road/wall design stationing system.

A concrete gravity section was chosen for the retaining wall to simplify construction and to minimize required excavation of the road embankment. It is desirable, both for technical feasibility and cost, not to excavate any more of the road embankment than is necessary. Using a gravity section it is possible to reduce the base size of the retaining wall by the use of prestressing tendons on the back side of the wall (see Figure 13.2). In this case the sloping side of the wall is placed away from the fill so that the tendons can be advantageously placed to resist overturning. The top of the wall is placed at elevation 1658 so that the tendons and backfilling operations will not delay reflooding of the river and resumption of power operations.

The riprap blanket is placed behind the retaining wall on the prepared embankment slope. The slope of the present embankment will be maintained as close as possible to the present grade line. However, it will be necessary to dress the slope to a 2:1 grade so that a uniform riprap blanket of the required thickness can be placed. The riprap design that has been chosen for estimating is a two course system with a top course of 72-inch riprap placed on a single course of 48-inch riprap placed on a 48-inch filter blanket. The filter blanket will consist of two 24-inch layers, a course layer overlaying a fine layer. The fine layer will be placed directly on the embankment slope. The sizing of the filter material is left to detailed design since for estimating purposes it is only necessary to recognize the general size characteristics of the filter. Since the economics of the riprap system is predicated on a local quarry for producing the riprap material, there will be ample small size material as a side product for construction of the filter blanket. The general design of the riprap/retaining wall is illustrated in Figure 13.2

There is insufficient space adjacent to the riverside shoulder of the road to place riprap in the section of the road from Horse Mesa Dam to road station 4+08. In this section, it will be necessary to use a full height retaining wall. Riprap in this section would encroach on the intake/tailrace of the pump back unit.

13.6 Materials for Construction

The principal materials required for construction are fill material for the road embankment, concrete, and riprap size rock. The present road embankment material will be removed, added to as may be required from deposits in the riverbed, and re-used as fill for the road embankment.

Aggregate for concrete will either have to be brought in from off-site sources or produced locally. The source for locally produced aggregate is the deposits in the river channel. These materials are of unknown quality and quantity. Concrete aggregate materials for construction of Horse Mesa Dam were produced locally and general references were made in the construction reports of downstream sources. It is probably safe to assume that local materials are of adequate quality since the concrete structures produced from these materials are in excellent condition. Although off-site materials of adequate quality and quantity can be guaranteed, it would be necessary to transport those materials not at the site. A premium price would have to be paid for off-site materials because of transportation costs. Transportation into the site will be more expensive than usual because of the tortuous nature and traffic limitations of the access to the working area. It is possible to make use of the barge on Canyon Lake to bring materials to the work area which would reduce transportation costs and improve the technical feasibility of using off-site materials. The impact of the sources of aggregate on on the cost of concrete are examined in this study.

Economics demand that riprap be obtained from a local quarry. One potential on-site quarry was located and used in preparing the estimate. Other quarry sites are possible but may be more difficult to access, such as the site located on the side of the river opposite the road. Difficulties arising from environmental considerations can be anticipated in obtaining a quarry permit on the U.S. Forest Service land which surrounds the site. This difficulty is an administrative problem probably affecting cost, but is not a technical barrier to construction.

As pointed out above, a quarry must be located that is capable of producing riprap of adequate size. Although a field reconnaissance was made, the outcroppings in the area selected for the quarry used in this study were not large enough to make a definite conclusion regarding the adequacy of the quarry site. It is certain that the dacite porphyry observed in massive seams in outer areas is present at the selected quarry site. Further, the occurrence of large dacite boulders in the area provide evidence that this size of rock can be produced from local seams of the dacite. Once this project proceeds to design it will be necessary to prove this or another quarry site. Because of its topographical nature and because it is not on U.S. Forest Service land, this quarry site will probably be the least expensive to develop. Therefore, riprap size rock from other sites will be more expensive.

A source of riprap, not considered in the cost estimate, is the material buried in the road embankment. This is an unknown quantity in unknown locations. If any of this material is uncovered in the required excavation of the road embankment, it may be used for riprap. Another source of riprap is the material submerged in the river or on the banks. Any of this material that meets the riprap requirements may be used to reduce the amount that must be obtained from the quarry. None of this material is included in the cost estimate.

14. BASIS OF COST ESTIMATES

14.1 Introduction

This estimate of construction cost and technical feasibility is based on a general design concept and plan of construction. There will be several other designs and construction plans that are possible and, therefore, feasible. The purpose of this study is to determine a design and construction plan that offers at least one approach to the problem without being the most expensive or necessarily the cheapest. Once it is decided to proceed with the project, it will be necessary to develop the design in detail and optimize the various design features. This will include a decision on the source of concrete aggregate and the location and development of the quarry for riprap. The overriding purpose of the cost estimate is to make it comparative so that one option can be compared to another. However, an effort has been made to be definitive enough to reliably establish the order of magnitude of the expected cost.

14.2 Riprap with Concrete Retaining Wall

A road alignment and stationing plan has been established so that construction features can be located and quantities can be determined. In general, the present alignment has been used except as noted in Section 13.4. The stationing used for this study is arbitrarily started at ERTEC drill hole HM-B-1 and is at station 0+69. Stationing is from point to point, there is no allowance for curves. The end of the road is at station 13+83. The stationing used for the road is independent of that used in the model study for mapping hydraulic performance. No special stationing was used for the retaining wall. The length of the wall between road stations is calculated. The length of excavation for the wall corresponds to the length of the wall.

The construction plant used for the estimate consists of two parts, the contractors compound and the construction area. The contractor's office and maintenance facilities, located in the contractor's compound, will be located near the entrance to the

access road immediately off of State Highway 88. The concrete plant and major construction equipment will be located at the working area. The quarry will be located in the plateau behind the school house in the workers residential area. This is illustrated in Figure 14.1.

Opening the quarry will be one of the first construction activities. Quarry work can precede work on the road since this work will not interfere with normal operations. Minimal facilities will be required at the quarry. Preliminary quarrying operations include clearing the work area and disposal of brush, stripping and preparation of disposal areas, preparation of stockpile areas (strippings can be used for this), construction of temporary roads between the quarry and the road to the residential area (strippings or rejected quarry materials can be used for this), and initial production stockpiling or riprap. Scheduling will be required so that road construction activities will permit moving most of the quarry production directly to the point of ultimate deposit.

Stockpiling of concrete aggregate will have to precede most construction activities. The amount of material to be stockpiled will depend on how the aggregate is to be produced. If aggregate is to be trucked into the site, it will be necessary to accumulate sufficient material so as not to delay concrete production once concreting operations commence. It is estimated that the proposed retaining wall will require 10,500 tons of sand and 11,400 tons of gravel. The retaining wall will be poured at the rate of about 380 CY/D of concrete requiring about 260 tons of sand, 310 tons of gravel and 72 tons of cement per day. Construction of the wall will require about eight 5-day weeks. If this material is delivered by road, it will all have to move down the single lane access road at a rate that will supply the concreting operation. The restricted working area limits the storage area at the working site. It will be necessary to create a limited storage area and supply concrete materials during off-construction hours (probably nights and weekends). This estimate provides for the creation of surge storage and working areas in the river by dredging river deposits to widen the bank. Primary storage would be at the contractor's compound.

An alternative method of supplying concrete aggregate is by dredging river deposits. Although this could be done with a dragline, it is probably more practical to use bulldozers while the lake level has been lowered for other construction activities. It will be necessary to process all naturally deposited materials to supply properly graded concrete aggregates. Further, it will be necessary to process excess material in order to supply deficient size fractions. It is expected that this operation could be equipped to supply aggregate at the required rate.

Because the bedrock is covered, it is necessary to excavate a trench in the road embankment for placement of the retaining wall. Since it is necessary to work from the road and to maintain passage on the road, temporary support of the embankment must be provided. The estimate provides for tied-back wood sheet piling to be disposed of in place. The base width and depth of the trench will vary with the depth of cover. Excavated material will be side cast and used to backfill around the wall and to bring the road surface to grade. A key trench will be excavated into the bedrock to anchor the base of the retaining wall.

The concrete retaining wall will be built in the conventional manner using reusable plywood forms. The wall will be built as illustrated in Figure 13-2 and will be lightly reinforced to assure transmission of horizontal stresses between the pre-stressing tendons used to stabilize the wall. Concrete will be delivered to the forms by transit mixers batched from a central plant in the working area. The estimate provides for the use of 8-CYD mixers making 6 pours per hour (48 CY/H) for 384 CY/D.

The design of the retaining wall used for the estimate requires the use of pre-stressing tendons to stabilize the wall. This reduces excavation and concrete requirements. Tendons will be placed in holes formed in the wall during concreting of the wall. Tendons will be placed after the wall is completed and after backfilling and initial riprap placement brings the embankment behind the wall level with the top of the wall. The only drilling that will be required will be drilling into the rock to anchor the tendon. Each tendon will require about 15 feet of anchorage.

Riprap will be placed behind the wall on the prepared embankment slope. The riprap plan is illustrated in Figures 13.4 and 13.5. The filter blanket will be dumped material graded with a small bulldozer. The 48-inch riprap will be placed using a clam shell or other equipment that will not disturb the filter material. The 72-inch riprap will be placed on the 48-inch material using a clam shell bucket, sling, or other equipment that the contractor may devise. The riprap material will be extended onto the road as provided for in the plan illustrated in Figures 13.4 and 13.5 To finish the road surface, road material previously excavated to provide room for the protective riprap will be placed on top of the riprap underlying the road. This will complete the work.

14.3 Soil Cement Replacement

Soil cement is an alternative to the full height concrete wall used in the area of the tailrace of the pump back unit. A suitable soil cement section for this purpose is shown in Figure 13.3. Construction of this section will require the excavation of almost all of the road down to bedrock or deep enough in the underlying soil to eliminate the risk of undercutting the toe of the soil cement by erosion. As noted in Section 13.3, this section is designed to be stable with a saturated embankment and no water in the river. Material excavated from the road will be stockpiled and used for making soil cement and for backfilling. Soil cement will be mixed in a central mixing plant, spread in 8-inch compacted layers, and compacted by sheepfoot rollers. Each layer will be stepped back 8 inches to produce a face slope of 1:1. Riprap with a granular topping will be used to surface the road in the same manner as used in the riprapped sections. This will eliminate the extensive surface erosion that occurs when the road is overtopped from spillway action.

14.4 Construction Sequence and Duration

14.4.1 General

The development of the various road bank protection systems and estimates envisioned certain construction procedures and production rates. The feasibility of these procedures establish the

feasibility of the proposed scheme and permit an estimate for the cost and duration of construction. The sequences of construction activities illustrated in Tables 14.1 through 14.4 are a part of these estimates. Although the actual construction sequence and procedures may differ from those envisioned in preparing these comparative estimates, it is desirable to examine the logic of the envisioned sequences to test the underlying rationale of the estimates. The following descriptions of the envisioned construction sequence is provided in an attempt to identify major interferences and unidentified construction problems that would preclude completion of the work.

14.4.2 Preparations for Construction

Prior to commencement of construction operations it will be necessary to construct certain support facilities and provide access to various parts of the construction area. Major among these is the contractor's base, access roads, storage areas, and concrete batching mixing facilities. Of special importance to this work is the development of a quarry for the riprap that will be required for embankment protection. Since road access to the work area is by way of the access road to the dam, it will be necessary to make those improvements to the road that are necessary for the safe transportation of materials, equipment, supplies, and personnel.

None of this work will require close coordination with other parts of the work, though coordination with power operation's overhaul schedules may be necessary. All of this work should be done well ahead of the start of actual construction and can be done more or less independently. The construction of the contractor's plant will take from four to six weeks depending on the emphasis the contractor wishes to provide. Improvements to the access road into the work area will take approximately two to three weeks since the road has been well maintained and requires relatively little work.

A batch plant is to be used for the manufacture of project concrete. Concrete will be mixed in transit mixers and batched from a central plant near the work area. The batch plant is best located at the west end of the construction area in the general

area of the warehouse presently located on the site. It may be desirable to temporarily move the warehouse to provide adequate room for the batch plant and its operation. At the batch plant it is necessary to provide batching hoppers for sand and gravel and a cement silo for storage and batching of the cement. A storage area for sand and gravel with enough capacity for 1½ days of concrete production needs to be provided. There is not enough room in the work area for storage of all the aggregate material that is required so it will be necessary to replenish the aggregate supply on daily basis. One way of doing this, the method provided for in the estimate, is by stockpiling aggregate in a special storage area provided at the contractor's base. Aggregate for the next day's operation will be provided on the off-shift hours, after the day shift and at night. This will require the movement of about 700 tons of material which can be accomplished with several large trucks on the nonproduction shifts. Depending on the amount of site preparation that will have to be done, the time for installation of the batch plant will take two to four weeks. This work will be done before work on the protective works is started.

14.4.3 Riprap Quarry

The opening of the quarry for production of riprap is another preparatory work that will be done before work on the protective works is started. This work will include clearing and disposal of all vegetative material from the quarry and related working areas, leveling of working and stock pile areas, removal of weathered surface material, construction of a quarry road joining existing access roads, improvement of the access road to handle quarry truck traffic, and stock piling of sufficient quantities of riprap materials to support riprap placement operations. This work is a noncritical item so long as it is done well before riprap is required. The quarry will be located apart from the work area. It will not interfere with other construction operations. It is estimated that quarry preparation will require from six to eight weeks, depending on the emphasis the contractor feels is appropriate for this operation.

14.4.4 Construction of the Full Length Protection System

The anticipated schedule for this construction is included in Table 14.1. Once all the preliminary work is complete, work can commence on the construction of the concrete retaining wall which will anchor the toe of the riprap. The work should proceed from west to east since the concrete batching plant will be located at the west end of the construction site. The plan that was envisioned in preparing the estimate was to first lower the water level in the channel to provide a dry working area for excavation of the toe of the existing access road embankment where the concrete retaining wall will be placed. The sequence of the work will be to excavate the trench for the wall using a back hoe or drag line and side cast the excavated material on the channel side of the trench. The side of the embankment is to be retained with reusable wood sheet piling driven into the bank prior to excavation. Although wood was used in preparing the estimate, steel sheet piling could also be used at higher cost. In fact, if it is found that wood sheet piling cannot be jetted into place in the materials used to build the embankment, it will be necessary to use steel piling. The sheet piling is to be anchored to rock with drilled in steel ties.

Excavation of the embankment materials is to be followed with excavation of the key trench in rock. This will be done with shallow drilling, blasting, removal of the spoil with a small tractor, and hand cleanup. Blasting would require environmental clearances by Federal agencies and must be carefully monitored.

Excavation is to be followed by setting of the forms for the retaining wall. Forms will be set to allow for simultaneously concreting one section while stripping and moving the previous section. The contractor will devise his own plan for this, but it could be set up with one set of forms being stripped, the next set being concreted, one set ready or near ready for the next concreting sequence, and the next set being set up.

After the concrete has attained sufficient strength, fill can be placed adjacent to the wall. The sheet piling can be pulled at this time but it will take additional time and will delay the time

that the river can be rewatered. For this reason, it was decided not to allow for salvage of the sheet piling in preparing the estimate. The fill material would be obtained from the previously side cast excavated material. Material not needed for backfill would be graded in place and left at the toe of the retaining wall. The embankment would next be graded in preparation for the riprap.

Riprap can be brought to the top of the retaining wall to provide a working platform for installation of the tendons. If the concrete wall is ready along it's entire length, the river channel can be refilled. At this point the tendons can be installed in the back side of the retaining wall. Installation of the tendons will require drilling anchorage into the rock below the retaining wall through the holes cast into the concrete for this purpose. The tendons are installed in the holes, the anchorage is grouted, and after the grout has attained sufficient strength the tendons are stressed.

Riprap bedding is placed on the graded embankment slope and is followed by one layer of riprap. This material can be dumped but precautions need to be taken to minimize disturbance of the bedding. The estimate provides for grading this material with an extendable arm grader working from the shoulder of the embankment. The next layer of riprap is then placed. The method of placement depends on the size of material being placed. the 48-inch material can be dumped and moved into place with a bulldozer. The 72-inch material will have to be placed one piece at a time with a crane.

Except for grading of the road surface and cleanup, this completes the work.

As can be deduced from the above description, the plan of work provides for the succeeding work sequences. By working from west to east there will be a minimum of interferences. The excavation work will require little back and forth traffic on the road. Erection of the forms, also, will require minimum back and forth traffic except for supply, which can be preplanned to minimize interference with succeeding operations. The greatest problem with form erection will be moving the forms from behind concreting operation to ahead of it. This will have to be coordinated.

Concreting will require considerable back and forth traffic between the batching plant and the placement site. To minimize the impact on power operations it will be necessary to place about 380 cubic yards of concrete each working day. The working sequence described above places all potentially interfering operations on the end away from concerting operations. Backfilling should follow as close as it can without interfering with concreting operations. The channel can be refilled as soon as backfilling in complete and a suitable work platform for installation of the tendons is ready. Placement of the major portion of the riprap can be done after the river channel has been refilled.

14.4.5 Construction of Soil Cement Riprap in Place of the High Wall

A soil-cement alternative was considered in place of the high concrete wall adjacent to the powerhouse. This scheme requires the excavation of the existing road down to bedrock in the section where the soil-cement is to be placed. In this case it is not practical to use sheet piling to retain the existing fill. Therefore, most of the road will have to be excavated and the material will be stockpiled for making soil-cement and for replacement fill. Where to place the stock pile is a major problem with this work.

The schedule for this scheme is included in Table 14.2. Excavation of the road can begin before the river is drawn down, however, it will be necessary to unwater the river before excavation can be completed. Soil-cement mixed in a pug mill is placed on bedrock in 8-inch compacted layers. Backfill material is laid next to the soil-cement and compacted at the same time. One lift is placed on top of the next but stepped back 8 inches to provide for the 1:1 face slope. The soil-cement face is cured with either a sprayed on curing compound or moist cured by covering with plastic sheet or damp burlap. As far as the soil-cement work is concerned, the river can be reflooded as soon as the working area rises above water level.

14.4.6 Partial Protection - Tunnel Only

The work and sequence of work of protecting only the part of the road subject to erosion from tunnel operation is the same as for the full length system, since the same design and embankment cross-section will be employed (see Figures 13.2 and 13.5, Section 3-3). The duration of the work is shortened as shown in Table 14.3 and the quantities are reduced. It will still be necessary to do about the same amount of work in construction of a base facility and mobilization. It will still be necessary to open a quarry and provide a quarry road to construct a concrete batching plant. The cost to open a quarry for this plan will not be significantly less than the cost to open a quarry for the comprehensive plan.

14.4.7 Berm Anchored Riprap

A berm anchored riprap system could reduce the cost of the work and could be done without unwatering the river or shutting down power operations as shown in Table 14.4. This scheme requires more riprap, but at the same time eliminates the need of a concrete retaining wall to anchor the toe of the riprap.

In a berm anchored system, the berm is constructed with riprap material placed by dumping from a barge. Construction of the berm is the first operation and is constructed using the largest size material. The riprap is placed behind the berm dumping the materials from a barge or by placing with a crane. The material is placed in sequence of size. A measured amount of material is placed in a measured space so that thickness of the resulting blanket is generally uniform. Construction above the water line is the same as described for the riprap placement for the full length protection system.

14.4.8 Trench Anchored Riprap Protection System

A riprap filled trench is another method that might be used to anchor the riprap blanket. The schedule for construction of this alternative is also included in Table 14.4. This method would be used if the berm would not be stable under flood conditions or if

it adversely affected power operations. In this case, a trench is excavated by dredge using either a barge mounted dredge or by a clam shell bucket. This approach can be used only if the bottom materials are thick enough for a trench of adequate depth and stable under flood flows.

The sequence of construction is to excavate the trench and backfill with large riprap using a barge. The riprap blanket is built up from the trench using a barge dump or clam shell to place the underwater material. As in the case of the berm anchored system, construction is controlled by placing a measured amount of material over a measured area in sequence of size. Material placed above the water line is placed the same as for the full length protection system.

15. COST ESTIMATES

15.1 General Considerations

The comparative cost estimates prepared for this study were developed for the systems that were successfully tested individually in the hydraulic model or designs that were derived from combined results of several model tests. Implementation plans of the modeled systems are based on structures and construction procedures that have been used successfully in similar situations. Where necessary, for estimating purposes, detailed design was done. Where special problems could be anticipated, they were either avoided by selection of alternatives, or the design was adapted to deal with them. The overall design, features, and procedures were selected as feasible and reasonable approaches to the problem. Once SRP selects a plan, a design study will be required and construction documents will be prepared based on detailed site exploration. At that time a definitive cost estimate can be prepared.

These cost studies identify the major cost components and their relative importance in accomplishing the objective. For this reason, quantities are calculated and unit costs are determined. It is inevitable that some cost items will not be properly identified and that actual costs will be more or less than those used in developing the estimate. Certain overhead costs have been purposely excluded, such as insurance, bonding, engineering,and contractor central office charges. Base costs, such as field offices and maintenance facilities, construction of temporary structures, and equipment moving and field assembly, have been included. Site problems have been included in the estimates by inclusion of provisions to deal with special situations.

Table 15.1 summarizes the quantitative data used to develop the cost estimate for the full-length combination concrete/riprap bank protection system. Only major items are listed. Production rates, related quantities, and unit prices were applied to these figures. These quantities were used in the preparation of the cost estimates.

15.2 Detailed Estimates

These estimates were developed from unit costs using Means, "Site Work Cost Data, 1987" and the quantities determined from the plans in Figures 13.2, 13.4, and 13.5. Where specific equipment or construction procedures were not covered in Means, information was obtained from other sources, such as contractors, rental agencies, and industrial sources. Where no information source could be found with reasonable effort, an allowance was used based on the best information available. The unit cost given in Means was adjusted to reflect the difficulty of access and work in such a restricted site. This was done by increasing the overhead in the cost items by 20% and decreasing productivity appropriately for working conditions.

The developed estimates identifying Cost Item, Unit Cost, Quantity, and Total Costs are given in Table 15.2. The figures given here are summarized from a more detailed estimate and may not relate to unit prices for similar items in other work. An allowance for contingencies is listed under each major construction phase heading in Table 15.2 so that a reviewer may adjust for uncertainties that are different from those implied by the contingency factors shown. In general, these contingency factors were selected to represent reduction in production caused by the difficulties presented by the remoteness of the site and restricted work area.

The estimated cost for construction of a riprap protected embankment with a retaining wall anchored toe as illustrated in Figure 13.4 is $7,260,514. The estimate is reduced to $5,662,828 if the 34-foot high wall immediately adjacent to the dam parking lot is replaced with a soil cement bank protection system. This estimate is shown in Table 15.3. The estimated cost to protect 800 feet of the access road from tunnel only discharges, using the combination of low-height concrete retaining wall and riprap, is $2,629,017 (see Table 15.4). This latter design, though not tested as an individual alternative in the model, was derived from previous test results.

16. ACKNOWLEDGEMENT

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