

Contains CEII Information – Do Not Release

Exhibit F Design Drawings and Supporting Design Report

Section 5.18(a)(5)(iii) of Title 18 of the Code of Federal Regulations (CFR) (revised 4/1/10) refers to Section 4.51 (License for Major Project – Existing Dam) for a description of information that an applicant must include in Exhibit F of its license application.

Exhibit F consists of general design drawings of the principal project works described under paragraph (b) of this section (Exhibit A) and supporting information used as the basis of design. If the Exhibit F submitted with the application is preliminary in nature, applicant must so state in the application. The drawings must conform to the specifications of 18CFR §4.39.

- (1) The drawings must show all major project structures in sufficient detail to provide a full understanding of the project, including:
 - (i) Plans (overhead view);
 - (ii) Elevations (front view);
 - (iii) Profiles (side view); and
 - (iii) Sections.
- (2) The applicant may submit preliminary design drawings with the application. The final Exhibit F may be submitted during or after the licensing process and must show the precise plans and specifications for proposed structures. If the project is licensed on the basis of preliminary designs, the applicant must submit a final Exhibit F for Commission approval prior to commencement of any construction of the project.
- (3) Supporting design report. The applicant must furnish, at a minimum, the following supporting information to demonstrate that existing and proposed structures are safe and adequate to fulfill their stated functions and must submit such information in a separate report at the time the application is filed. The report must include:
 - (i) An assessment of the suitability of the site and the reservoir rim stability based on geological and subsurface investigations, including investigations of soils and rock borings and tests for the evaluation of all foundations and construction materials sufficient to determine the location and type of dam structure suitable for the site;
 - (ii) Copies of boring logs, geology reports and laboratory test reports;
 - (iii) An identification of all borrow areas and quarry sites and an estimate of required quantities of suitable construction material;

Contains CEII Information – Do Not Release

- (iv) Stability and stress analyses for all major structures and critical abutment slopes under all probable loading conditions, including seismic and hydrostatic forces induced by water loads up to the Probable Maximum Flood as appropriate; and
- (v) The bases for determination of seismic loading and the Spillway Design Flood in sufficient detail to permit independent staff evaluation.
- (4) The applicant must submit two copies of the supporting design report described in paragraph (g) (3) of this section at the time preliminary and final design drawings are submitted to the Commission for review. If the report contains preliminary drawings, it must be designated a "Preliminary Supporting Design Report."

The following information provides documentation that existing structures associated with the Middle Fork American River Project (Project) are safe and adequate to fulfill their stated functions. Refer to the following drawings for detailed dimensions and information related to Project facilities.

(1) Drawings

General design drawings for Placer County Water Agency's (PCWA) Project as described in Exhibit A to this Application for New License (License Application) are provided in the exhibit drawings listed below. Reduced drawings showing all major structures, including plans, elevations, profiles, and sections are provided.

Exhibit L No.	Drawing Title
French Meadows Powerhouse and Associated Facilities	
3	French Meadows Dam Plan
4	French Meadows Dam Sections and Details
5	French Meadows Spillway Plan, Profile and Sections
6	French Meadows Dam Geology
7	French Meadows-Hell Hole Tunnel Plan, Profile and Details
8	French Meadows Power Plant and Penstock
9	French Meadows Power Plant General Plan and Outdoor Arrangement
10	French Meadows Power Plant Powerhouse Plans
11	French Meadows Power Plant Powerhouse Plans
12	French Meadows Power Plant Powerhouse Sections
1	Duncan Creek Diversion Plan and Sections
2	Duncan Creek-Middle Fork Tunnel Plan and Profile
45	Duncan Creek Diversion Streamflow Maintenance Pipe Intake Trashrack

Contains CEI Information – Do Not Release

Exhibit L No.	Drawing Title
Hell Hole Powerhouse	
1	Hell Hole Powerhouse Project Area
2	Hell Hole Powerhouse General Arrangement: Plan, Tunnel Profile, Penstock, and Powerhouse
14	Hell Hole Dam Plan
15	Hell Hole Dam Sections and Details
16	Hell Hole Spillway Plan Profile and Sections
17	Hell Hole Dam Geology
18	Hell Hole-Middle Fork Tunnel Plan and Profile
19	Hell Hole-Middle Fork Tunnel Intake and Surge Shaft
Middle Fork Powerhouse and Associated Facilities	
20	Long Canyon Diversion South Fork Dam and Shaft
21	Long Canyon Diversion North Fork Dam and Shaft
22	Middle Fork Power Plant Penstock Plan and Profile
23	L.J. Stephenson Power Plant (Middle Fork) General Plans and Outdoor Arrangement
24	L.J. Stephenson Power Plant (Middle Fork) Floor Plans
25	L.J. Stephenson Power Plant (Middle Fork) Floor Plans
26	L.J. Stephenson Power Plant (Middle Fork) Powerhouse Sections
29	Middle-Fork-Ralston Interbay Dam Plan and Sections
Ralston Powerhouse and Associated Facilities	
30	Middle Fork-Ralston Tunnel Plan and Profile
31	Ralston Power Plant Penstock Plan and Profile
32	Ralston Power Plant General Plan and Outdoor Arrangement
33	Ralston Power Plant Powerhouse Plans
34	Ralston Power Plant Powerhouse Plans
35	Ralston Plant Powerhouse Sections
37	Ralston Afterbay Dam Plan and Sections
Oxbow Powerhouse and Associated Facilities	
38	Oxbow Power Plant General Plans and Outdoor Arrangement
39	Oxbow Power Plant Floor Plans
40	Oxbow Power Plant Powerhouse Sections

(2) Preliminary Design Drawings

This application is for the relicensing of an existing Project, and therefore, includes as-built drawings in Section 1 rather than preliminary drawings. Information related to the retrofit of dams associated with this Project is provided in the Federal Energy Regulatory Commission (FERC) Part 12 Safety Inspection Reports (see (3) below).

Contains CEI Information – Do Not Release

Preliminary drawings of the Hell Hole Reservoir Seasonal Storage Increase Improvement and the infrastructure modifications at the small diversions (Duncan Creek, North Fork Long Canyon, and South Fork Long Canyon diversions) are included in this License Application. Final Exhibit F drawings will be submitted for FERC approval prior to commencement of construction. The preliminary drawings are listed below:

Drawing Number	Title of Preliminary Drawing
Hell Hole Reservoir Seasonal Storage Increase Improvement	
HH-1	Hell Hole Reservoir Seasonal Storage Increase Layout
Duncan Creek Diversion Dam Modification	
DC-1	Duncan Creek Diversion Site Plan
DC-2	Duncan Creek Diversion Plan and Profile View of Intake and Weir
DC -3	Duncan Creek Diversion Site Plan Profile View of Intake "A-A"
DC-4	Duncan Creek Diversion Site Plan Intake Sections "B-B" & "C-C"
DC-5	Duncan Creek Diversion Site Plan Primary View of Instream Flow Maintenance Facility
DC-6	Duncan Creek Diversion Site Plan Tunnel Entrance & Box Culvert
North Fork Long Canyon Diversion Dam Modification	
NF-1	North Fork Long Canyon Diversion Site Plan
NF-2	North Fork Long Canyon Diversion Site Plan - Plan and Profile View of Intake and Weir
NF-3	North Fork Long Canyon Diversion Site Plan Profile View of Intake "A-A"
NF-4	North Fork Long Canyon Diversion Site Plan Intake Sections "B-B" & "C-C"
South Fork Long Canyon Diversion Dam Modification	
SF-1	South Fork Long Canyon Diversion Site Plan
SF-2	South Fork Long Canyon Diversion Site Plan - Plan and Profile View of Intake and Weir
SF 3	South Fork Long Canyon Diversion Site Plan Profile View of Intake "A-A"
SF 4	South Fork Long Canyon Diversion Site Plan Intake Sections "B-B" & "C-C"

(3) And (4) Supporting Design Report

PCWA's recent FERC Part 12 Independent Consultants Dam Safety Inspection Reports for Hell Hole Dam, French Meadows Dam, and Ralston Afterbay Dam fulfill the requirements of 18 CFR § 4.41 (g)(3) and (4) for the filing of a Supporting Design report with an application for a new license. All of the Projects's Independent Consultants Dam Safety Inspection Reports are on file with the FERC. The most recent report for Hell Hole Dam was submitted to the FERC on December 29, 2006, with the next submittal scheduled for December 2011. The most recent report for French Meadows Dam was submitted to the FERC on January 2 and 3, 2007. The next submittal is due in January 2012. The most recent report for Ralston Afterbay was submitted to the FERC on May 27, 2010 and the next submittal is scheduled for May 2014.

Contains CEII Information – Do Not Release

Middle Fork Interbay Dam is classified as a “low hazard” dam since no reasonably foreseeable project emergency would endanger life, health, or property. Accordingly, the dam is exempt from the FERC Part 12 Independent Consultants Dam Safety Inspection requirement. However, FERC conducts annual inspections of the dam and prepares a letter report documenting the inspection findings. In addition, FERC requires partial operation of spillway gates on an annual basis and to full design height at last once every five years. PCWA tests the Middle Fork Interbay gates during the maintenance period when the reservoir level is below the spillway crest elevation. At the end of each calendar year, PCWA submits an Annual Spill Gate Testing Report to FERC demonstrating license compliance. In accordance with FERC requirements, PCWA also thoroughly inspects and evaluates the spillway gates every 10 years. Information regarding the design and construction of Middle Fork Interbay Dam including soil borings depicting the geology at the dam site is provided in Exhibit L Drawing Number 29/FERC Drawing Number 2079-348. No borrow areas are located at the Middle Fork Interbay Dam site. The stress and stability analysis used in the preliminary design of the dam is provided in Attachment F-1 (Report on the Design of Small Concrete Gravity Dams).

A Supporting Design Report for the Hell Hole Reservoir Seasonal Storage Increase Improvement has been prepared to demonstrate that the structures are safe and adequate to fulfill their stated functions. This report is provided in Attachment F-2 of this Exhibit F as: “Exhibit F: Preliminary Supporting Design Report, Middle Fork American River Project.”

Contains CEI Information – Do Not Release

Application for New License

Middle Fork American River Project (FERC Project No. 2079)

General Design Drawings

Contains CEI Information – Do Not Release

Application for New License

Middle Fork American River Project (FERC Project No. 2079)

Preliminary Design Drawings

Contains CEI Information – Do Not Release

Application for New License

Middle Fork American River Project (FERC Project No. 2079)

ATTACHMENT F-1

Report on the Design of Small Concrete Gravity Dams

REPORT ON THE DESIGN
OF SMALL CONCRETE GRAVITY DAMS

CONTENTS

Introduction

Duncan Creek Diversion

Summary of Stresses and Design Conditions

Calculations

Middle Fork - Ralston Interbay Dam

Summary of Stresses and Design Conditions

Calculations

REPORT ON THE DESIGN OF SMALL CONCRETE GRAVITY DAMS

Introduction

This report includes the basic criteria and calculations used in the preliminary design of two of the small concrete gravity dams included in the Middle Fork of the American River Project.

In general, the standards and procedures recommended by the Bureau of Reclamation for small dams have been adopted. The probable maximum flood discharge was used for the spillway design and for stability analysis.

In the stability analysis, the forces considered acting on the dam included external water pressure, internal water pressure or uplift, silt pressure, horizontal and vertical earthquake, and the weight of the structure. These forces were combined as shown in the stability analysis summary sheets. The structures were proportioned for stability against overturning, sliding and overstressing.

MIDDLE FORK - RALSTON INTERBAY DAM

Project No 1033

Sheet — of —

Title of **MIDDLE FORK -**
 Computatic **RALSTON INTERBAY DAM**
STABILITY ANALYSIS

Calc by **RLB**
 Chkd by **VF**

Date **1-2-62**
 Date

SUMMARY

DESIGN CONDITIONS

1. RESERVOIR EMPTY

2. RESERVOIR W.S. EL 2529
 NORMAL OPERATING
 LEVEL.

NO TAILWATER
 SILT TO EL. 2475
 100% UPLIFT AT
 UPSTREAM FACE
 ZERO UPLIFT AT
 DOWNSTREAM TOE.
 EARTHQUAKE
 SHOCK UPSTREAM
 ($\lambda_H = 0.10$) AND
 DOWNWARD ($\lambda_V = 0.05$)

3. FLOOD STAGE.
 W.S. EL. 2532.8
 GATES OPEN.

UPLIFT VARIES FROM
 HEADWATER PRESSURE
 AT UPSTREAM FACE
 TO TAILWATER
 PRESSURE AT DOWN-
 STREAM TOE.
 TAILWATER TO
 EL. 2480

SILT TO EL 2475
 $Q(\text{FLOOD}) = 49,000 \text{ cfs.}$

SPILLWAY DAM MAXIMUM SECTION - 45' BELOW CREST					
CASE	ΣH KIPS	ΣV KIPS	STRESSES (PSI)		
			f _{TOE}	f _{HEEL}	AV. SHEAR
1	0	373.5	+7	+62	—
2	184.4	216.8	+35	+6	17
3	128.7	196.6	+16	+21	12

SPILLWAY DAM SECTION 30' BELOW CREST					
CASE	ΣH KIPS	ΣV KIPS	STRESSES (PSI)		
			f _{TOE}	f _{HEEL}	AV. SHEAR
1	0	220.3	+9	+42	—
2	110.4	129.1	+26	+3	13
3	79.8	138.2	+15	+16	8

SPILLWAY DAM SECTION 15' BELOW CREST					
CASE	ΣH KIPS	ΣV KIPS	STRESSES (PSI)		
			f _{TOE}	f _{HEEL}	AV. SHEAR
1	0	100.6	+8	+23	—
2	54.4	60.3	+17	+1	8
3	28.4	64.0	+8	+11	4

NON OVERFLOW DAM SECTION 40' DEEP					
CASE	ΣH KIPS	ΣV KIPS	STRESSES (PSI)		
			f _{TOE}	f _{HEEL}	AV. SHEAR
1	0	97.3	+2	+52	—
2	40.8	69.1	+34	+4	11
3	33.6	71.7	+27	+13	9

NON OVERFLOW DAM SECTION 14' DEEP					
CASE	ΣH KIPS	ΣV KIPS	STRESSES (PSI)		
			f _{TOE}	f _{HEEL}	AV. SHEAR
1	0	25.2	+15	+15	—
2	3.2	22.5	+21	+5	2
3	1.4	22.7	+15	+11	1

Project No 1033

Title of RALSTON INTERBAY DAM
 Computation STABILITY COMPUTATIONS

Calc by **ECB**
 Chkd by **V.F**

Sheet 1 of 10

Date 11/20/61

Date 12/19/61

FOR VALUES OF $Y = 5, 10, 15$ etc.

FIND x

$-y = \frac{x^2}{80}$; $x^2 = -80y$
 for parabola in + quad.
 $x^2 = 80y \therefore x = \sqrt{80y}$

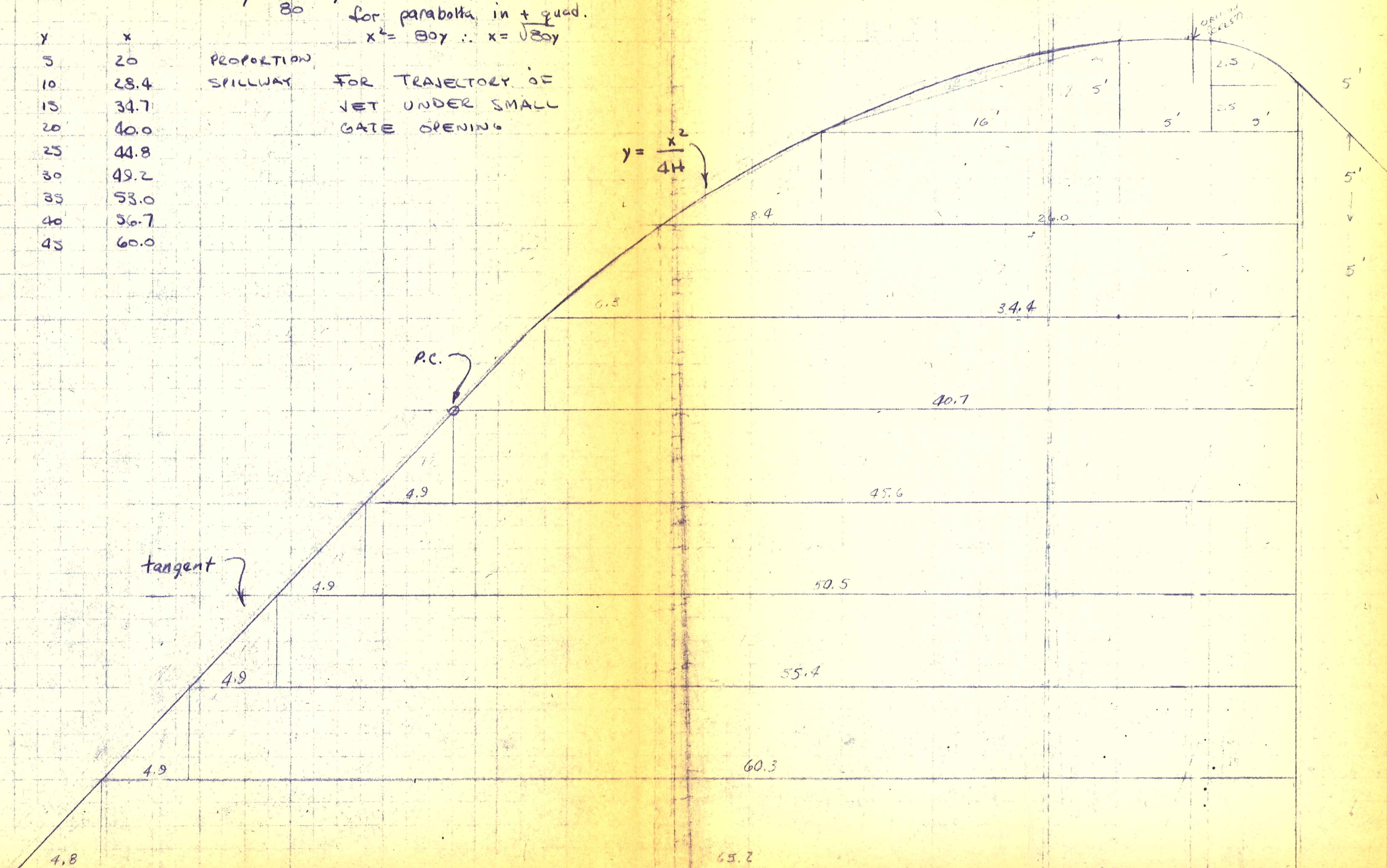
Y	x
5	20
10	28.4
15	34.7
20	40.0
25	44.8
30	49.2
35	53.0
40	56.7
45	60.0

PROPORTION

SPILLWAY

FOR TRAJECTORY OF
 JET UNDER SMALL
 GATE OPENING

$y = \frac{x^2}{4H}$



tangent

P.C.

GATE OPENING

4.8

4.9

4.9

4.9

4.9

12

6.3

65.2

60.3

55.4

50.5

45.6

40.7

34.4

26.0

8.4

16'

5'

5'

2.5

2.5

5'

5'

5'

5'

Project No 1033
 Title of Computation RALSTON TERBAY DAM
 STABILITY COMPUTATIONS

Calc by RCB
 Chkd by V.F

Sheet 2 of 10
 Date 11-20-61
 Date 12/19/61

SECTION	SHAPE	DIMENSIONS	AREA	\bar{x}	x	\bar{y}	y	Ax	Ay	Σx	ΣAx	ΣAy
1		5x2.5	8.35	1.88	3.12	1.0	1.5	26.0	12.5	8.35	26.0	12.5
2		5x2.5	12.50	2.5	2.5	1.25	3.75	31.2	46.8	20.85	57.2	50.3
3		5x2.5	25.0	2.5	7.5	2.5	2.5	187.5	62.5	45.85	244.7	121.8
4		16x5	40.0	5.33	15.33	1.67	3.33	614.0	133.0	85.85	858.7	254.8
5		26x2.5	130.0	13.0	13.0	2.5	7.5	1690.0	975.0	215.85	2548.7	1229.8
6		8x4.5	21.0	2.7	28.7	1.67	8.33	602.0	175.0	236.85	3152.7	1404.8
7		34x4.5	172.0	17.2	17.2	2.5	12.5	2955.0	2145.0	408.85	6105.7	3549.8
8		6x3.5	15.75	2.1	36.5	1.67	13.33	575.0	210.0	424.60	6680.7	2759.8
9		4x7x5	204.0	20.35	20.35	2.5	17.5	4150.0	3570.0	628.60	10820.7	7322.8
10		4x9x5	12.25	1.63	42.33	1.67	18.33	518.0	223.0	640.85	11348.7	7554.8
11		45x4.5	228.0	22.8	22.8	2.5	22.5	5200.0	5130.0	863.85	16548.7	12604.8
12		4x9x5	12.25	1.63	47.23	1.67	23.33	578.0	286.0	881.10	17426.7	12970.8
13		5x2x5	252.0	25.25	25.25	2.5	27.5	6350.0	6030.0	1133.10	23476.7	19000.8
14		4x2x5	12.25	1.63	52.13	1.67	28.33	639.0	347.0	1145.35	24115.7	20247.8
15		55x4.5	277.0	27.7	27.7	2.5	32.5	7670.0	9000.0	1422.35	31735.7	22047.8
16		4x9x5	12.25	1.63	57.03	1.67	33.33	699.0	418.0	1434.60	32434.7	29655.8
17		6x3x5	302.0	30.15	30.15	2.5	37.5	9100.0	11320.0	1736.60	41534.7	40975.8
18		4x9x5	12.25	1.63	61.93	1.67	38.33	759.0	469.0	1748.85	42343.7	41444.8
19		65x2x5	326.0	32.6	32.6	2.5	42.5	10610.0	13840.0	2076.85	52243.7	55284.8
20		4x3x5	12.20	1.60	66.90	1.67	43.33	815.0	528.0	2087.05	53788.7	55312.8
21												
22												

DATA

SPILLWAY PROPORTIONED FOR GATED OPERATION
 ASSUMING PARABOLIC TRAJECTORY FOR JET
 OF WATER FOLLOWS: $y = \frac{x^2}{4H}$ WHERE
 THE ORIGIN OF COORDINATES IS THE CREST.
 H = ^{MAX.} HEAD ON ORIFICE (SMALL GATE OPENING)
 REFERENCE - SMALL DAMS U.S.B.R. 1960

DESIGN CONDITIONS

1. RESERVOIR EMPTY - TEMPORARY CONDITION WHICH EXISTS ONLY DURING CONSTRUCTION.
 ∴ NO EARTHQUAKE IMPOSED
2. NORMAL W.S. EL 2529 GATES CLOSED
 100% UPLIFT AT HEEL, 0 AT TOE,
 NO TAILWATER, SILT TO EL 2475 (TUNNELING),
 EARTHQUAKE - SHOCK UPSTREAM ($A_g = 1g$) †
 DOWNWARD ($A_v = 0.05$) W.S. ASSUMED TO GATE TOP. EL 2530
3. FLOOD STAGE - W.S. EL 2532.8 - MAX.
 GATES OPEN, 100% UPLIFT AT HEEL, 100% T.W. AT TOE,
 EL 2480 TAILWATER, SILT TO EL 2475, $Q = 40,000 \text{ cfs}$

McCREARY · KORETSKY · ENGINEERS

Project No 1033

Sheet 3 of 10

Title of Computation **RALSTON INTERBAY DAM**
WEIGHT OF GATES & PIERS

Calc by **RB**
 Chkd by **RR**

Date **11-17-61**
 Date

GATES

4 - 20x30' TANTNER GATES

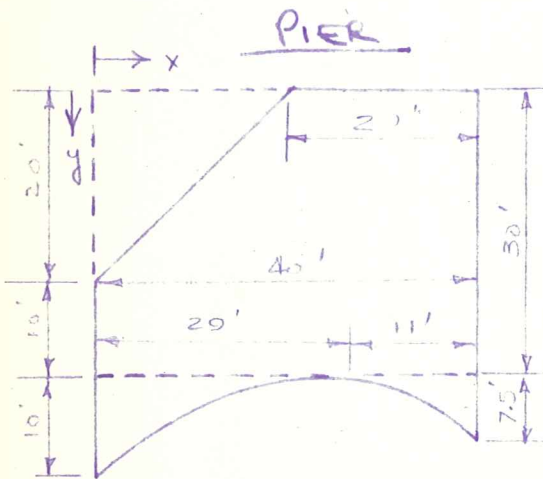
REF HYDRO-ELECTRIC HANDBOOK, (REAGER & JUSTIN)
 p.p. 524-26

GATES -

H = HEIGHT OF GATES - (20')
 W = WIDTH " " - (30')
 h = HEAD ON & GATE (10')

$$\frac{W^2 H h}{1000} = \frac{(30)^2 (20)(10)}{1000} = 180$$

FROM CHART p. 526 $W_T = 50$ kips (est.)



	A	x	Ax
40x30	1200	20	24000
20x20	-200	6.67	-1333
29x19/3	96.6	7.25	700
11x7.5/3	27.5	3.67	101
	1124.1		23,468

$$\bar{x} = \frac{23,468}{1124.1} = 20.9' \text{ PIER ONLY}$$

WEIGHT OF PIER:

6' THICK,
 $A = 1124.1 \text{ ft}^2$

$$W_p = 6 \times 1124.1 (150) = 1012 \text{ KIPS}$$

$$W_{\text{GATE}} = 50 \text{ kips}$$

TOTAL 1062 KIPS

PRORATA PER FOOT

$$\frac{1062}{36} = 29.5 \text{ k/ft}$$

ASSUME C.G. @ $x = 20$

GATE & PIER $y = 18$

	A	y	Ay
1200	1200	15	18,000
-200	-200	6.67	-1333
96.6	96.6	33.0	3190
27.5	27.5	32.25	888
	1124.1		20,745

$$\bar{y} = \frac{20,745}{1124.1} = 18.4' \text{ PIER ONLY}$$

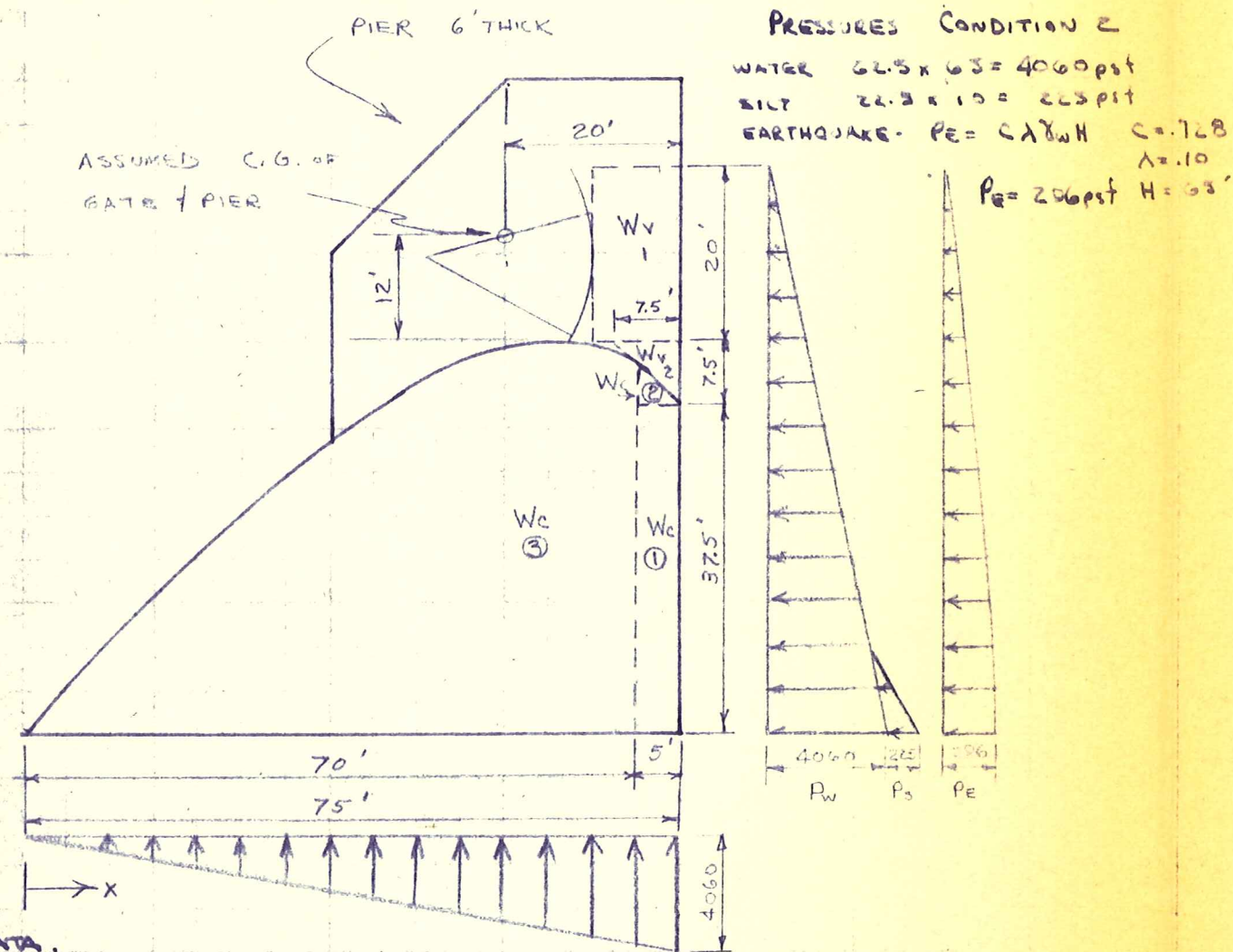
DESIGN CONDITIONS:

1. RES. EMPTY
2. W.S. 2530 + SILT EL. 2475
+ UPLIFT (NO T.W.) + EARTHQUAKE
3. W.S. 2532.8 (FLOOD)
+ SILT + UPLIFT (GATES OPEN)
+ TAILWATER

Project No 1033

Title of RALSTON INTERBAY DAM
Computation STABILITY ANALYSIS & STRESSES

Calc by RLB
Chkd by



PRESSURES CONDITION 2
WATER 62.5 x 63 = 4060 psf
SILT 22.5 x 10 = 2250 psf
EARTHQUAKE - $P_E = C_A \gamma_w h$ $C = 0.128$
 $\lambda = 1.0$
 $P_E = 206 \text{ psf}$ $H = 63'$

DATA.

$\gamma_w = 62.5 \text{ pcf}$
 $\gamma_{\text{silt}} = 22.5 \text{ pcf}$
 $P_E = C_A \gamma_w h$ (EARTHQUAKE EFFECT)
 $\lambda = 1$ horiz. IN WATER U.S.B.R.
 $\lambda = 0.05$ VERTICAL

REFERENCES - SMALL DAMS - USBR
ELEMENTS OF HYDRAULIC ENGR
LINSLEY & FRANZINI

Σ M TOE					
ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
Wc ①	5 x 37.5 x .150	28.1 ↓	75 - 2.5	72.5	+ 2040
Wc ②	5 x 5/2 x .150	1.87 ↓	75 - 3.33	71.67	+ 134
Wc ③	2087 x .150	314.0 ↓	70 - $\frac{52.77 \times 7}{2087}$	44.3	+ 13,900
Wg + P	SEE SHEET	29.5 ↓	75 - 20	55.0	+ 1622
		$\Sigma F_y = 373.47 \downarrow$	$\Sigma F_x =$	$\Delta M = +17,676'k$	
$x = \frac{\Delta M}{\Sigma F_y} = \frac{17,676}{373.47} = 47.4'$		$f = \frac{P}{L} \left(1 \pm \frac{6e}{L}\right) = \frac{373.47}{75} \left(1 \pm \frac{6(9.9)}{75}\right)$		$f_{\text{top}} = 1.07 \text{ ksf (7psi)}$	
$e = 47.4 - 37.5 = 9.9'$				$f_{\text{base}} = 8.92 \text{ ksf (62psi)}$	
				$v_{\text{av.}} = 0$	
ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
$\lambda_v Wc ①$.95 (29.1)	26.7 ↓	75 - 2.5	72.5	+ 1940
$\lambda_v Wc ②$.95 (1.87)	1.78 ↓	75 - 3.33	71.67	+ 127
$\lambda_v Wc ③$.95 (314.0)	298.0 ↓	70 - 25.7	44.3	+ 13,200
$\lambda_v Wg + P$.95 (29.5)	29.0 ↓	75 - 20	55.0	+ 1510
$\lambda_v Wv_1$.95 (20.0 x 3.5)	13.0 ↓	75 - 5.5	69.5	+ 905
$\lambda_v Wv_2$.95 (7.5 x 3/2 x 62.5)	1.66 ↓	75 - 2.5	72.5	+ 120
F_{wu}	75 x 4060/2	152.3 ↑	2/3 x 75	50.0	- 7620
F_{wh}	65 x 4060/2	132.0 ←	1/3 x 65	21.67	- 2850
F_s	22.5 x 10/2	1.12 ←	1/3 x 10	3.33	- 4
$\lambda_h Wc ①$.1 x 28.1	2.81 ←	37.5/2	18.75	- 53
$\lambda_h Wc ②$.1 x 1.87	.19 ←	37.5 + 5/3	39.17	- 7
$\lambda_h Wc ③$.1 x 314.0	31.4 ←	45 - $\frac{52.77 \times 7}{2087}$	18.25	- 573
$\lambda_h Wg + P$.1 x 29.5	2.95 ←	45 + 12	57.0	- 168
E_w	$V_e = .726 P_e \gamma = .726 (206) (63)$	14.0 ←	$M_e = .250 P_e \gamma^2$	$250 (206) (63)^2$	- 374
		$\Sigma F_y = 216.54 \downarrow$	$\Sigma F_x = 184.37 \leftarrow$	$\Delta M = +6173'k$	
$x = \frac{\Delta M}{\Sigma F_y} = \frac{6173}{216.54} = 28.5'$		$f = \frac{216.54}{75} \left(1 \pm \frac{6(9.0)}{75}\right)$		$f_{\text{top}} = 4.970 \text{ ksf (35psi)}$	
$e = 37.5 - 28.5 = 9.0'$				$f_{\text{base}} = 0.810 \text{ ksf (6psi)}$	
				$v_{\text{av.}} = 2.460 \text{ ksf (17psi)}$	
average shear = $v = \frac{\Sigma F_x}{L} = \frac{184.37}{75} = 2.460 \text{ ksf (17psi)}$				$\tan \phi = \frac{\Sigma F_x}{\Sigma F_y} = \frac{184.37}{216.54} = .857$	

PROCEDURE -

MOMENTS OF ALL FORCES TAKEN ABOUT TOE OF DAM SECTION.
WEIGHT & CENTROIDS OF SPILLWAY DAM FROM TABULAR SHEET.
LOCATION OF RESULTANT (K) MEASURED FROM TOE.
SLIDING CONSIDERED POSSIBLE WHEN $\tan \phi$ IS GREATER THAN .7

SAFETY FACTOR = $\frac{17832}{11659} = 1.53$ COND. 2

McCREARY · KORETSKY · ENGINEERS

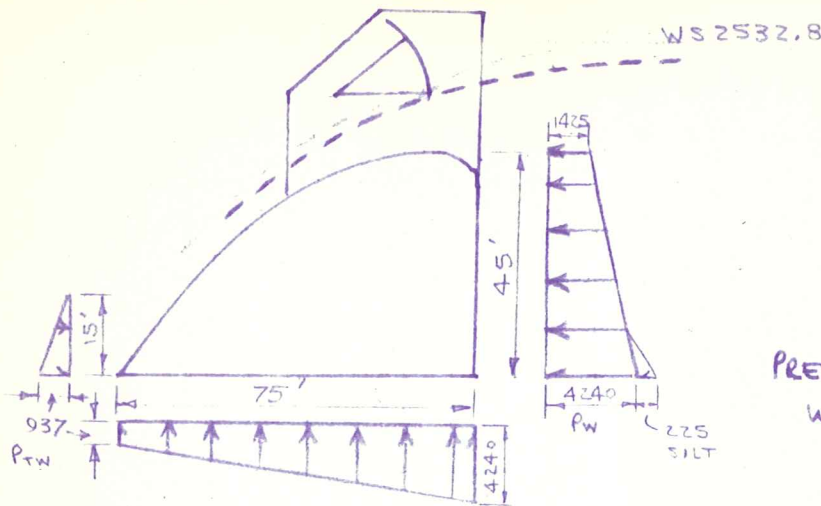
Project No 1033

Sheet 5 of 10

Title of
Computation **RAUSTON INTERBAY DAM**

Calc by **RCB**
Chkd by **VF**

Date **12-27-61**
Date



CONDITION 3 FLOOD STAGE
GATES OPEN
T.W. EL. 2480

PRESSURE-

WATER: 22.8 (62.5) = 1425 psf
67.8 (62.5) = 4240 psf
15.0 (62.5) = 937 psf

$\Sigma M TOE$

	ITEM	CALC. FORCE	FORCE	CALL. ARM	ARM	MOMENT
CONDITION 3	Wc①	SEE CONDITION 1	28.1 ↓	PREV. COND.	72.5	+2040
	Wc②	" " "	1.87 ↓	" "	71.67	+ 134
	Wc③	" " "	314.0 ↓	" "	44.3	+13900
	Wg+P	" " "	29.5 ↓	" "	55.0	+ 1622
	Wv①	22.8 × 11.0 × 62.5	15.65 ↓	" "	69.5	+1089
	Wv②	7.5 × 7.5/2 × 62.5	1.75 ↓	" "	72.5	+ 127
	Fuw	937 × 75 + (4240 - 937) $\frac{75}{2}$	194.3 ↑			-8840
	Fhw	1425 × 45 + (4240 - 1425) $\frac{45}{2}$	127.4 ←			-2393
	Fs	SEE CONDITION 2	1.12 ←		3.33	- 4
FHTW	937 × 15/2	7.03 →	15/3	5.0	+ 35	

$\Sigma F_y = 196.57 \text{ k}$ $\Sigma F_x = 128.72 \text{ k}$ $\Delta M = 7710 \text{ k}$

$x = \frac{\Delta M}{\Sigma F_y} = \frac{7710}{196.57} = 39.3$

$e = 39.3 - 37.5 = 1.8'$

$f = \frac{196.57}{75} \left(1 \pm \frac{6(1.8)}{75} \right)$ $f_{top} = 2.240 \text{ ksf (16 psi)}$

$f_{heel} = 3.000 \text{ ksf (21 psi)}$
 $v_{av.} = 1.715 \text{ ksf (12 psi)}$

average shear $v = \frac{128.72}{75} = 1.715 \text{ ksf (12 psi)}$ $f_{reqd} = \frac{128.72}{196.57} = .655 < .7$

INVESTIGATE SECTION 30' BELOW CREST

Project No 1035

Title of RALSTON INTERBAY DAM
Computation STABILITY ANALYSIS
STRESS ANALYSIS

Calc by RUS
Chkd by JF

Sheet 6 of 10

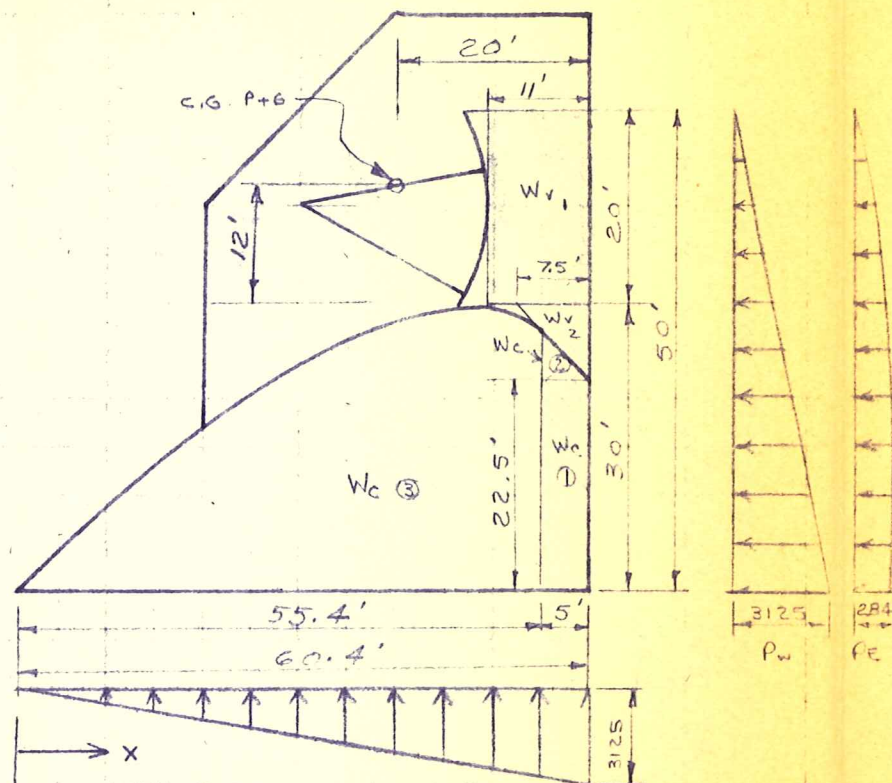
Date 12-29-61
Date

DRAWING OF CONDITION 2

PRESSURES:

CONDITION 2 WATER
50 x 62.5 = 3125 psf
CONDITION 3 WATER
52.8 x 62.5 = 3300 psf
22.8 x 62.5 = 1425 psf
EARTHQUAKE COND. 2
 $PE = C \lambda \delta_w h$
 $y = 50, h = 65$
 $\frac{y}{h} = \frac{50}{65} = .77$
 $\therefore C = .70$

$PE = .70(10)(62.5)(65)$
 $PE = 284 \text{ psf}$



ΣM_{TOE}

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
$W_{c①}$		16.90 ↓		57.9	+ 980
$W_{c②}$		1.87 ↓		57.1	+ 107
$W_{c③}$		172.0 ↓		34.4	+ 5920
W_{P+G}		29.5 ↓		40.4	+ 1190
W_{v1}		15.65 ↓	60.4 - 5.5	54.9	+ 860
W_{v2}		1.75 ↓	60.4 - 2.5	57.9	+ 101
F_{wV}	$60.4/2 \times 3300$	99.5 ↑		40.3	- 4020
F_{wH}	$1425 \times 90 + 1875 \times 30/2$	70.8 ←	$28.1 \times 10 = 281$ $42.7 \times 15 = 640.5$		- 921

$\Sigma F_y = 138.17 \downarrow$ $\Sigma F_x = 70.8 \leftarrow$ $\Delta M = +4217 \text{ k}$
 $x = \frac{4217}{138.17} = 30.5$
 $e = 30.5 - 30.2 = 0.3$
 $f = \frac{138.17}{60.4} \left(1 \pm \frac{6(0.3)}{60.4}\right)$ $f_{top} = 2.230 \text{ ksf (15 psi)}$
 $f_{heel} = 2.360 \text{ ksf (16 psi)}$
 $v = 1.172 \text{ ksf (8 psi)}$
 $\text{over. shear} = \frac{70.8}{60.4} = 1.172 \text{ ksf}$ $\tan \phi = \frac{70.8}{138.17}$ $\tan \phi = .512 < .7$

ΣM_{TOE}

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
$W_{c①}$	$22.5 \times 5 \times 150$	16.90 ↓	60.4 - 2.5	57.9	+ 980
$W_{c②}$	AREA CALL.	1.87 ↓	60.4 - 2.3	57.1	+ 107
$W_{c③}$	117.35×50	172.0 ↓	$5.4 - \frac{24115.7}{1145.35}$	34.4	+ 5920
W_{P+G}	AREA CALL.	29.5 ↓	60.4 - 20	40.4	+ 1190

$x = \frac{8197}{210.27} = 36.8'$ $\Sigma F_y = 220.27 \downarrow$ $\Sigma F_x = 0$ $\Delta M = +8197 \text{ k}$
 $e = 36.8 - 30.2 = 6.6'$ $f = \frac{220.27}{60.4} \left(1 \pm \frac{6(6.6)}{60.4}\right)$ $f_{top} = 1.271 \text{ ksf (9 psi)}$
 $f_{heel} = 6.102 \text{ ksf (42 psi)}$
 $\text{over. } v = 0$
 $\tan \phi = 0$

ΣM_{TOE}

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
$\lambda_v W_{c①}$		16.05 ↓		57.9	+ 930
$\lambda_v W_{c②}$		1.78 ↓		57.1	+ 102
$\lambda_v W_{c③}$		163.1 ↓		34.4	+ 5615
$\lambda_v W_{P+G}$		28.0 ↓		40.4	+ 1130
$\lambda_v W_{v1}$		13.0 ↓	60.4 - 5.5	54.9	+ 714
$\lambda_v W_{v2}$		1.66 ↓	60.4 - 2.5	57.9	+ 96
F_{wV}	$60.4/2 \times 325$	97.5 ↑	$60.4 \times 2/3$	40.3	- 3810
F_{wH}	$50.0/2 \times 225$	78.1 ←	$57/3$	16.61	- 1300
$\lambda_H W_{L①}$		1.60 ←	$\frac{22.5}{2}$	11.25	- 19
$\lambda_H W_{L②}$		0.19 ←	$22.5 \times 1/3$	24.17	- 5
$\lambda_H W_{c③}$		17.2 ←	$30 - \frac{20247.8}{1145.35}$	12.3	- 212
$\lambda_H W_{P+G}$		2.95 ←	30 + 12	42.0	- 124
E_w	$V_E = 72'$ ($y = 726 \times 284$)	10.91 ←	$M_E = 209 \text{ ft}$ ($f^2 = 209(4)$)		- 210

$\Sigma F_y = 129.09 \text{ k} \downarrow$ $\Sigma F_x = 110.44 \text{ k} \leftarrow$ $\Delta M = 2907 \text{ k}$
 $x = \frac{2907}{129.09} = 22.5'$
 $e = 30.2 - 22.5 = 7.7'$
 $f = \frac{129.09}{60.4} \left(1 \pm \frac{6(7.7)}{60.4}\right)$ $f_{top} = 3.750 \text{ ksf (26 psi)}$
 $f_{heel} = 0.503 \text{ ksf (3 psi)}$
 $\text{over. } v = 13 \text{ psi}$
 $\text{over. shear} = \frac{110.44}{60.4} = 1.93 \text{ ksf (13 psi)}$ $\tan \phi = \frac{110.44}{129.09} = .855 > .7$

McCREARY · KORETSKY · ENGINEERS

Project No. 1033

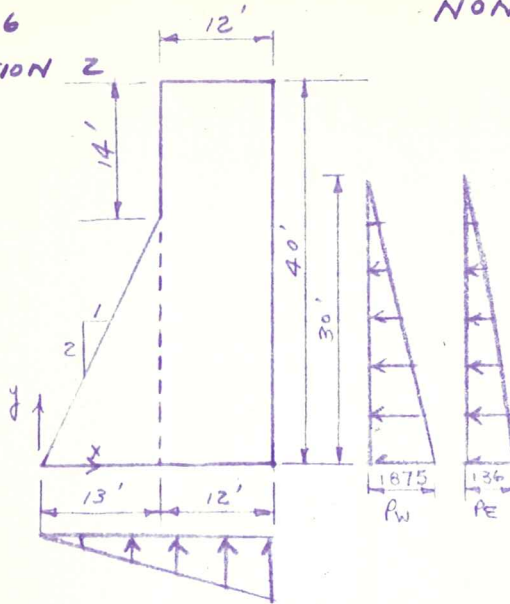
Sheet 8 of 10

Title of Computation **RALSTON INTERBAY DAM
STABILITY & STRESS ANALYSIS**

Calc by **RCB**
Chkd by **VF**

Date **12-5-61**
Date

DRAWING OF CONDITION 2



NON OVERFLOW SECTION

A	x	Ax
$12 \times 40 = 480$	10	9120
$13 \times \frac{24}{2} = \frac{169}{2}$	8.67	$\frac{1462}{2}$
$\Sigma 649$		$\Sigma 10582$

$$x = \frac{10582}{649} = 16.3'$$

A	y	Ay
$12 \times 40 = 480$	20	9600
$13 \times \frac{24}{2} = \frac{169}{2}$	8.67	$\frac{1463}{2}$
$\Sigma 649$		$\Sigma 11063$

$$y = \frac{11063}{649} = 17.1'$$

ΣM_{TOE}

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT	CONDITION
W_c	$649 \times .150$	$97.3 \downarrow$	ABOVE	16.3	+1586	
$e = 16.3 - 12.5 = 3.8'$ $f = \frac{97.3}{25.0} \left(1 \pm \frac{6(3.8)}{25.0} \right)$ $f_{toe} = 0.342 \text{ ksf (2psi)}$ $f_{heel} = 7.45 \text{ ksf (5psi)}$ av. $v = 0$ $\tan \phi = 0$						

WATER PRESSURES (CONDITION 2)

$\lambda_h = .10$ $h = 30'$ $P_w = 30(62.5) = 1875 \text{ psf}$
 $C = .726$ (SEE REF.) $P_e = C \lambda \gamma_w h = .726(.10)(62.5)(30) = 136 \text{ psf}$

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT	CONDITION
$\lambda_h W_c$	$.95 (97.3)$	$92.5 \downarrow$		16.3	+1508	
F_{wu}	$25/2 \times 1875$	$23.4 \uparrow$	$2/3 \times 25$	16.67	-390	
F_{wh}	$30/2 \times 1875$	$28.1 \leftarrow$	$30/3$	10.0	-281	
$\lambda_h W_e$	$.1 (97.3)$	$9.73 \leftarrow$	ABOVE	17.1	-166	
E_w	$V_e = .726 P_e y = .726(136)(30)$	$2.96 \leftarrow$	$M_{E_w} = .299 P_e y^2$	$299(136)(30)^2$	-37	
$\Sigma F_y = 69.10 \text{ k} \downarrow$ $\Sigma F_x = 40.79 \text{ k} \leftarrow$ $\Delta M = +634' \text{ k}$ $x = \frac{634}{69.10} = 9.2'$ $e = 12.5 - 9.2 = 3.3'$ $f = \frac{69.10}{25} \left(1 \pm \frac{6 \times 3.3}{25} \right)$ $f_{toe} = 4.95 \text{ ksf (34psi)}$ $f_{heel} = 0.58 \text{ ksf (4psi)}$ av. shear $= v = \frac{40.79}{25.0} = 1.63 \text{ ksf}$ $\tan \phi = \frac{40.79}{69.10} = .59 < .7$ $v = 11 \text{ psi}$						

Project No 1033

Sheet 9 of 10

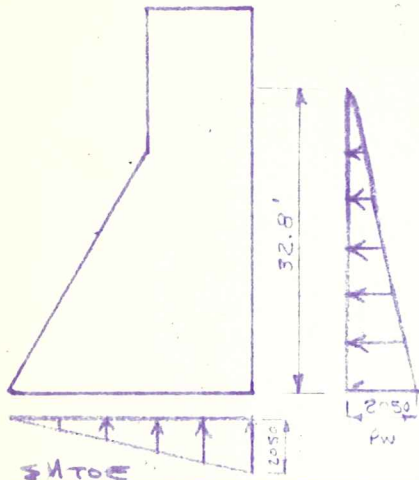
Title of Computation **RALSTON INTERBAY DAM
STABILITY & STRESS ANALYSIS**

Calc by RCB
Chkd by VF

Date 12-5-61
Date

NON OVERFLOW SECTION

CONDITION 3 - GATES OPEN - FLOOD STAGE



WATER PRESSURE

$h = 32.8 \quad 32.8 \times 62.5 = 2050 \text{ psf}$

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
Wc		97.3 ↓		16.3	+1586
Fwu	$25/2 \times 2050$	25.6 ↑		16.7	-427
Fwu	$32.8/2 \times 2050$	33.6 ←		10.9	-367

$x = \frac{792}{71.7} = 11.0$

$e = 12.5 - 11.0 = 1.5$

$\sum F_y = 71.7 \text{ k} \downarrow \quad \sum F_x = 33.6 \text{ k} \leftarrow \quad \Delta M = +792 \text{ k}$

$f = \frac{71.7}{25} \left(1 \pm \frac{6(1.5)}{25} \right) \quad f_{top} = 3.9 \text{ ksi} (27 \text{ psi})$

$f_{heel} = 1.84 \text{ ksi} (13 \text{ psi})$

$\tan \phi = \frac{33.6}{71.7} = .46 < .7; \quad v = \frac{33.6}{25} = 1.34 \text{ ksi} (9 \text{ psi})$

200-029

Project No 1033

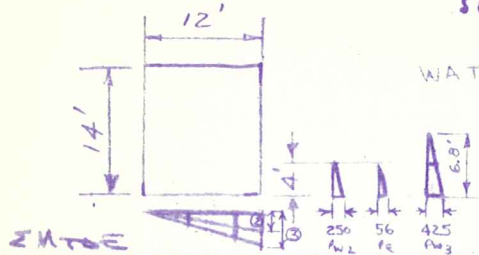
Sheet 10 of 10

Title of Computation **RALSTON INTERMAY DAM**
STABILITY & STRESS ANALYSIS

Calc by **RCB**
 Chkd by **VF**

Date **12-5-01**
 Date

NON OVERFLOW SECTION
SECTION 14' FROM TOP OF DAM



WATER PRESSURES -
 COND. 2 $4 \times 62.5 = 250 \text{ psf}$
 COND. 3 $6.8 \times 62.5 = 425 \text{ psf}$
 EARTH PRESSURE $y = 4, h = 30, y/h = .133, C = .3$
 $PE = .3(.13)(62.5)(30) = 56 \text{ psf}$

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
Wc	12 x 14 x 150	25.2 ↓			
$f = \frac{25.2}{12} = 2.1 \text{ ksf (15 psi) heel \& toe}$ $v = 0$ $\tan \phi = 0$					

EMTDE

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
$\sum Wc$.95 (25.2)	24.0 ↓	12/2	6.0	+144
F_{WU}	12/2 x 250	1.5 ↑	2/3 x 12	8.0	-12
F_{WH}	4/2 x 250	.5 ←	4/3	1.33	-7
$\sum Wc$.1 x 25.2	2.52 ←	14/2	7.0	-18
$E \cdot W$	$\sum W_e = .726(P_e y) = .726(56)(4)$.16 ←	$M_e = .299(P_e y^2)$	$.299(56)(4)^2$	-
$\sum F_y = 22.5 \text{ k} \downarrow, \sum F_x = 3.18 \text{ k} \leftarrow, \Delta M = +107 \text{ 'k}$ $x = \frac{107}{22.5} = 4.75'$ $e = 6.0 - 4.75 = 1.25'$ $f = \frac{22.5}{12} \left(1 \pm \frac{6(1.25)}{12}\right), f_{top} = 3.05 \text{ ksf (21 psi)}$ $f_{heel} = 0.705 \text{ ksf (5 psi)}$ $\tan \phi = \frac{3.18}{22.5} = .141 < .7, \text{ av. } v = \frac{3.18}{12} = .256 \text{ ksf (2 psi)}$					

EMTDE

ITEM	CALL. FORCE	FORCE	CALL. ARM	ARM	MOMENT
Wc		25.2 ↓		6.0	+151
F_{WU}	12/2 x 425	2.55 ↑		8.0	-20
F_{WH}	6.8/2 x 425	1.44 ←		1.33	-3
$x = \frac{128}{22.65} = 5.65$ $e = 0.35$ $\sum F_y = 22.65 \text{ k} \downarrow, \sum F_x = 1.44 \text{ k} \leftarrow, \Delta M = +128 \text{ 'k}$ $f = \frac{22.65}{12} \left(1 \pm \frac{6(0.35)}{12}\right), f_{top} = 2.21 \text{ ksf (15 psi)}$ $f_{heel} = 1.50 \text{ ksf (11 psi)}$ $\tan \phi = \frac{1.44}{22.65} = .064 < .7, \text{ av. } v = \frac{1.44}{12} = .12 \text{ ksf (1 psi)}$					

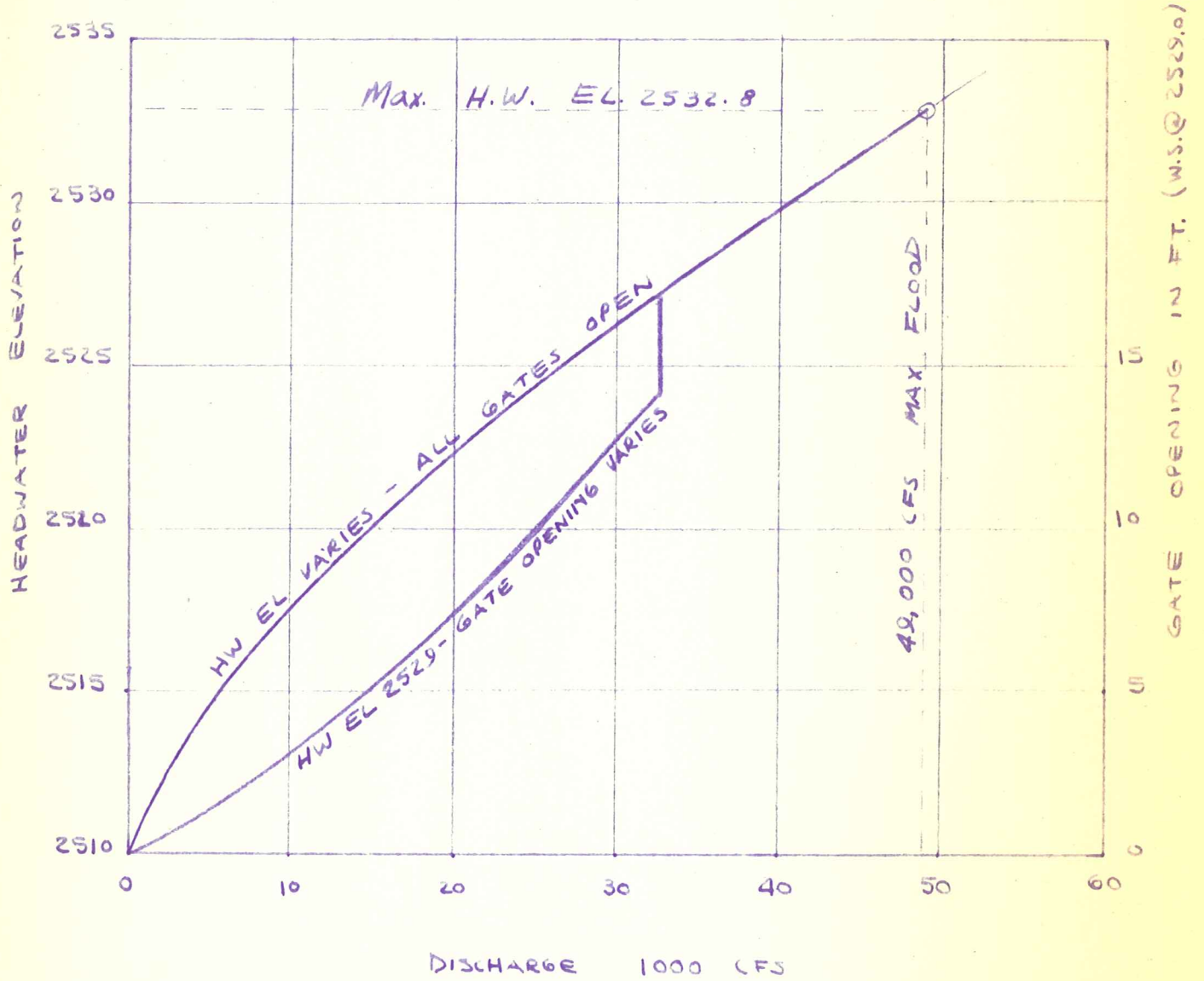
Project No 1033

Sheet 1 of 2

Title of Computation RALSTON INTERBAY DAM
SPILLWAY DISCHARGE CURVE

Calc by RCB
Chkd by RR

Date 12-5-61
Date



SPILLWAY DISCHARGE CURVE
4- 20'x30' TAINTER GATES

Project No 1033

Sheet 2 of 2

Title of Computation RALSTON INTERBAY DAM
SPILLWAY DISCHARGE

Calc by RCB
Chkd by RR

Date 11-23-61
Date

DESIGN FLOOD - Q = 49,000 cfs

1. ORIFICE FLOW UNDER GATE 19' HEAD ON CREST
REF. CORPS OF ENGINEERS HYDRAULIC DESIGN - 311-1

$$Q = CA \sqrt{2gh} \quad h = \text{head on gate} \text{ \& } \\ A = 120d \quad C = .685 \text{ from geometry of gate.}$$

$$d = 5' \therefore h = 16.5'$$

$$Q = 5 \times 120 \times .685 \times \sqrt{2g \times 16.5} = 13,800 \text{ cfs}$$

$$d = 10' \therefore h = 14.0'$$

$$Q = 10 \times 120 \times .685 \times \sqrt{2g \times 14} = 25,300 \text{ cfs}$$

$$d = 15' \therefore h = 11.5'$$

$$Q = 15 \times 120 \times .685 \times \sqrt{2g \times 11.5} = 34,400 \text{ cfs}$$

for 19' head on uncontrolled crest the upper nappe constriction leaves 14.1' actual height to water surface at crest.

$$y = .74 (19) = 14.1'$$

\therefore Beyond discharges under 14.1' opening the gate should be swung clear of water surface.

2. Uncontrolled flow $Q = C_o L H^{3/2}$
compare coef. ...

$$C_o = .685 \left(\frac{2}{3}\right) (\sqrt{2g}) = 3.68$$

since no hindrance from gate... assume C = 3.75

$$\text{@ 19' head} \quad Q = 3.75 (120) (19)^{3/2} = 37,400 \text{ cfs}$$

$$\text{@ 15' head} \quad Q = \quad \quad \quad (15)^{3/2} = 26,000 \text{ cfs}$$

$$\text{@ 10' head} \quad Q = \quad \quad \quad (10)^{3/2} = 15,000 \text{ cfs}$$

$$\text{@ 5' head} \quad Q = \quad \quad \quad (5)^{3/2} = 5,000 \text{ cfs}$$

$$\text{for } Q = 49,000 \text{ cfs} \quad H = \left(\frac{49,000}{3.75 \times 120} \right)^{2/3} = 22.8' \text{ MAX FLOOD}$$

Contains CEI Information – Do Not Release

Application for New License

Middle Fork American River Project (FERC Project No. 2079)

ATTACHMENT F-2

Preliminary Supporting Design Report

Placer County Water Agency

Middle Fork American River Project
FERC Project No. 2079

Application for New License
Exhibit F

Hell Hole Reservoir
Seasonal Storage Increase

Preliminary Supporting Design Report

CONTAINS CEII – DO NOT RELEASE

August 2010

Submitted By
Christensen Associates Inc

Placer County Water Agency
P.O. Box 6570
Auburn, CA 95604
(530) 823-4889
www.pcwa.net

Middle Fork American River Project
FERC Project No. 2079

Application for New License
Exhibit F

Hell Hole Reservoir
Seasonal Storage Increase

Preliminary Supporting Design Report

CONTAINS CEII - DO NOT RELEASE

August 2010

Submitted By
Christensen Associates Inc
810 Fifth Avenue, Suite 210
San Rafael, CA 94901
(415) 485-1440
www.caiworld.com

Contents

1. INTRODUCTION.....	1-1
1.1 PURPOSE	1-1
1.2 SCOPE OF SUPPORTING DESIGN REPORT	1-1
2. DESCRIPTION OF FACILITY	2-1
2.1 LOCATION.....	2-1
2.2 RESERVOIR	2-1
2.3 DAM.....	2-2
2.4 HELL HOLE SPILLWAY	2-2
3. RESERVOIR RIM STABILITY	3-1
4. SITE INVESTIGATIONS AND GEOLOGY	4-1
4.1 SOURCES OF DATA	4-1
4.2 REGIONAL GEOLOGY.....	4-1
4.3 SITE GEOLOGY	4-2
4.3.1 <i>Site Geology and Local Foundation Conditions</i>	4-2
4.3.2 <i>Landslides, Loose Rock or Adverse Bedding</i>	4-5
4.3.3 <i>Sinkholes, Karst, or Solutioning Issues</i>	4-6
4.3.4 <i>Weak Seams</i>	4-6
4.3.5 <i>Geologic Artesian Sources</i>	4-6
5. BORROW AREAS AND CONSTRUCTION MATERIAL.....	5-1
6. STABILITY AND STRESS ANALYSIS.....	6-1
6.1 ANALYSIS METHODOLOGY AND CRITERIA.....	6-1
6.1.1 <i>Geometry</i>	6-1
6.1.2 <i>Criteria</i>	6-1
6.1.3 <i>Loading Conditions</i>	6-2
6.2 METHODOLOGY.....	6-3
6.3 RESULTS OF STABILITY ANALYSES	6-4
6.4 CONCLUSIONS	6-4
7. SEISMIC LOADING.....	7-1
7.1 FAULTING AND MAP OF FAULT TRACES	7-1
7.2 MAXIMUM CREDIBLE AND DESIGN BASIS EARTHQUAKE DEVELOPMENT.....	7-4
8. SPILLWAY DESIGN FLOOD ANALYSES.....	8-1
8.1 PURPOSE	8-1
8.2 BACKGROUND	8-1
8.3 HYDROLOGIC ANALYSES	8-1
8.3.1 <i>Watershed</i>	8-1
8.3.2 <i>Climate</i>	8-2
8.3.3 <i>Floods</i>	8-3
8.4 HYDRAULIC ANALYSES.....	8-4
8.4.1 <i>Purpose of Analysis</i>	8-4
8.4.2 <i>Existing Spillway</i>	8-4
8.4.3 <i>Spillway Rating Curve</i>	8-4
8.4.4 <i>Evaluation Procedure</i>	8-4
8.4.5 <i>Results of Evaluation</i>	8-5
8.4.6 <i>Updated Routing of All Season PMF</i>	8-5

8.4.7	Routing of Local Storm PMF	8-6
8.4.8	Tailwater Determination	8-6
8.5	REFERENCES.....	8-6

Tables

Table 6-1	- Stability Analysis Results	6-4
Table 7-1	- Table of Faults, MCE and PGA.....	7-3
Table 8-1	- Summary of Climate Data at Georgetown Ranger Station.....	8-2
Table 8-2	- Average Snow Depth and Water Content (in)	8-3

Figures

Figure 2-1	- Aerial View of Hell Hole Dam and Spillway	2-4
Figure 2-2	- View from upstream of Hell Hole Dam and Spillway.....	2-4
Figure 2-3	- Hell Hole Spillway Approach Channel	2-5
Figure 2-4	- Hell Hole Spillway Weir	2-5
Figure 2-5	- Spillway Discharge Channel showing Invert Roughness @ Bridge Crossing.....	2-6
Figure 2-6	- Top of Stepped Chute.....	2-6
Figure 4-1	- Regional Geology for Hell Hole Reservoir.....	4-7
Figure 4-2	- Hell Hole Dam Geologic Plan and Sections.....	4-8
Figure 4-3	- Hell Hole Dam Logs of Core Borings.....	4-9
Figure 4-4	- Spillway Weir Foundation Excavation	4-10
Figure 4-5	- Spillway Weir Grout Curtain	4-11
Figure 6-1	- Topographic Mapping - Hell Hole Abutments and Upper Spillway	6-5
Figure 6-2	- Spillway Ogee Section.....	6-6
Figure 6-3	- Loading Conditions - Load Cases 1 and 3.....	6-7
Figure 6-4	- Loading Conditions - Load Case 2	6-8
Figure 7-1	- Potential Earthquake Fault Sources in Region of Hell Hole Dam	7-7
Figure 7-2	- Historic Earthquakes in Region of Hell Hole Dam.....	7-8
Figure 8-1	- Hell Hole Spillway Rating Curve Comparison	8-8
Figure 8-2	- Detailed Topography of Hell Hole Spillway.....	8-9
Figure 8-3	- Hell Hole Dam Abutments & Upper Spillway	8-10
Figure 8-4	- Spillway Ogee Section.....	8-11
Figure 8-5	- Water Surface Profile of Spillway with 21,600 cfs flow	8-12

1. INTRODUCTION

1.1 PURPOSE

Placer County Water Agency (PCWA) is in the process of relicensing its Middle Fork American River Project. This multi-purpose water supply and hydroelectric project was constructed in the 1960's to meet the water supply needs of Placer County and to supply electric power to Pacific Gas and Electric Company's customers in California.

The present Federal Energy Regulatory Commission (FERC) license for the Project expires in 2013. PCWA is applying for a new license to improve and continue operation of the project. As part of the relicensing process, PCWA is investigating potential betterments to the project to enhance its productivity.

One proposed betterment is to increase the seasonal water storage capacity of Hell Hole Reservoir to increase water supply yield and hydroelectric power generation. PCWA proposes to achieve this by adding six-foot high pneumatically operated steel flap gates to the crest of the existing concrete overflow weir that presently controls outflow from the reservoir to the spillway channel.

This preliminary supporting design report is part of Exhibit F of PCWA's application for a new license. The report presents the findings of engineering analyses carried out on the existing spillway and proposed modifications. Its purpose is to provide information regarding the safety and adequacy of the facility.

1.2 SCOPE OF SUPPORTING DESIGN REPORT

The required scope of the preliminary design report is prescribed in FERC regulations as follows:

(3) Supporting design report. The applicant must furnish, at a minimum, the following supporting information to demonstrate that existing and proposed structures are safe and adequate to fulfill their stated functions and must submit such information in a separate report at the time the application is filed.

The report must include:

- (i) An assessment of the suitability of the site and the reservoir rim stability based on geological and subsurface investigations, including investigations of soils and rock borings and tests for the evaluation of all foundations and construction materials sufficient to determine the location and type of dam structure suitable for the site;*
- (ii) Copies of boring logs, geology reports and laboratory test reports;*
- (iii) An identification of all borrow areas and quarry sites and an estimate of required quantities of suitable construction material;*
- (iv) Stability and stress analyses for all major structures and critical abutment slopes under all probable loading conditions, including seismic and hydrostatic forces induced by water loads up to the Probable Maximum Flood as appropriate; and*
- (v) The bases for determination of seismic loading and the Spillway Design Flood in sufficient detail to permit independent staff evaluation.*

(4) The applicant must submit two copies of the supporting design report described in paragraph (g)(3) of this section at the time preliminary and final design drawings are submitted to the Commission for review. If the report contains preliminary drawings, it must be designated a "Preliminary Supporting Design Report."

The outline of this preliminary supporting design report follows these requirements.

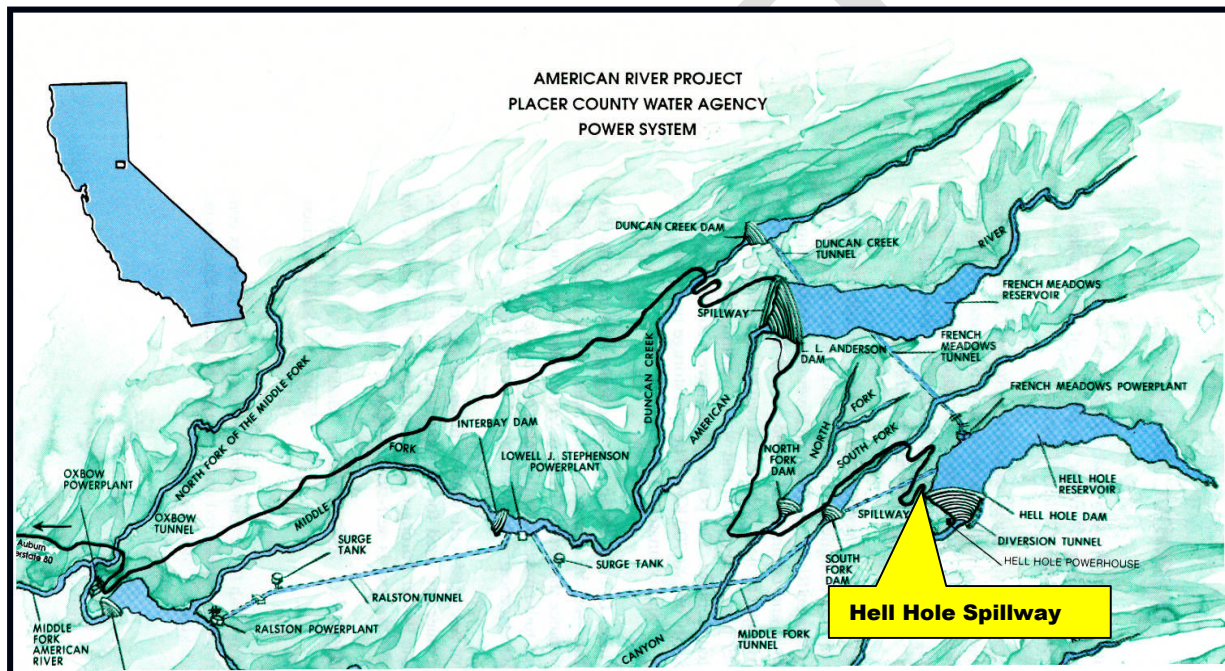
DRAFT

2. DESCRIPTION OF FACILITY

2.1 LOCATION

Hell Hole Dam and reservoir is located in the headwaters of the American River watershed on the west-facing slopes of the Sierra Nevada Mountains. The dam is located on the Rubicon River which is a major tributary of the Middle Fork of the American River.

Hell Hole Dam is located about 60 road miles east of Auburn, California. Primary road access is from State Highway 80 at Auburn, via Foresthill Divide Road to Foresthill, then via Mosquito Ridge Road which passes French Meadows Reservoir before turning south to reach Hell Hole Reservoir.



The following report sections briefly describe Hell Hole Reservoir, Dam and Spillway. Photographs of the existing dam and spillway can be found at the back of this chapter.

2.2 RESERVOIR

Hell Hole Dam impounds the Rubicon River and Five Lakes Creek to form Hell Hole Reservoir. The reservoir has 207,590 ac-ft of gross storage and an active storage capacity of 205,057 ac-ft. The maximum operating water surface elevation is El. 4630⁴ ft. and the minimum is El. 4340 ft.

The intake for the Middle Fork Power Tunnel is on the western shore of the reservoir about one half mile north of the dam. French Meadows power plant is also located on the west shoreline of the reservoir about 1.4 miles north of the dam. This powerhouse discharges water from French Meadows Reservoir into Hell Hole Reservoir.

⁴ In this report elevations are in feet and the datum is NGVD29.

2.3 DAM

Hell Hole Dam is a 1,570 foot-long rock fill embankment dam with a sloping core. The overflow spillway is located on the right (west) abutment of the dam. Outlet works are located at the toe of the dam.

The crest of the dam is 35 feet wide and varies in elevation from about El. 4,650 ft. at the abutments to El. 4,657 ft at the center (due to camber). The maximum dam height above the stream channel is about 410 feet. The top of the impervious core is shown to be 5 feet below the dam crest.

The average upstream slope of the embankment dam is 2.5 horizontal/ 1 vertical and the average downstream slope is 1.4 horizontal/1 vertical.

2.4 HELL HOLE SPILLWAY

Hell Hole Spillway is an open channel spillway excavated in rock through the right (west) abutment of Hell Hole Dam. The channel is about 2700 feet long from its entrance in the reservoir to its confluence with the Rubicon River downstream of Hell Hole Dam.

The centerline of the spillway approach channel extends from the reservoir about 550 ft. downstream to the concrete weir. The width of the channel tapers from about 450 feet wide at its entrance to about 350 feet wide at the weir. The channel invert elevation is about El. 4600 ft at its entrance and throughout much of its length then it abruptly rises to a 30 -foot wide bench at El. 4625.5 ft. immediately upstream of the concrete weir.

The concrete weir is about 350 feet long and 22 ft. wide (upstream/downstream) with a nominal crest elevation of El. 4630.0 ft. Based on a recent field inspection it is apparent that the condition at the downstream toe of the concrete weir section is not depicted accurately on existing engineering design drawings. However, the shape of the weir ogee crest does appear to be accurately depicted. The drawings also show minimal reinforcing with No. 11 anchor dowels at 6 ft. centers extending into the foundation rock.

The discharge channel extends about 300 feet from the concrete weir to the top of the stepped chute. At the upstream end of the channel, it is about 350 feet wide and the invert elevation is about El. 4624 ft. at the downstream face of the weir. Then there is a short channel section that drops 2-3 feet in an area of scour immediately downstream of the weir before the channel invert levels off to a slope of about 1% for a length of about 300 feet downstream of the weir to the beginning of the stepped chute. The floor of the discharge channel is extremely rough due to the uneven surface resulting from the rock blasting employed to excavate the channel through the hard Gabbro bedrock and hydraulic plucking of loose rock by flow over the spillway. Some larger loose rocks have remained in place also contributing to surface hydraulic roughness.

A curved road bridge supported by seven (six in the channel and one at the right abutment) round concrete piers traverses the channel about 100 feet downstream of the weir. At the abrupt transition from the discharge to the chute the channel has tapered in to about 250 feet wide. The stepped chute is about 1900 feet long and descends over 400 ft in a series of steps to its confluence with the Rubicon river about 2000 feet downstream of Hell Hole Dam. The chute

varies in width from about 200 to 250 feet and has been eroded to an irregular cross section shape and invert profile by the spillway discharge.

DRAFT

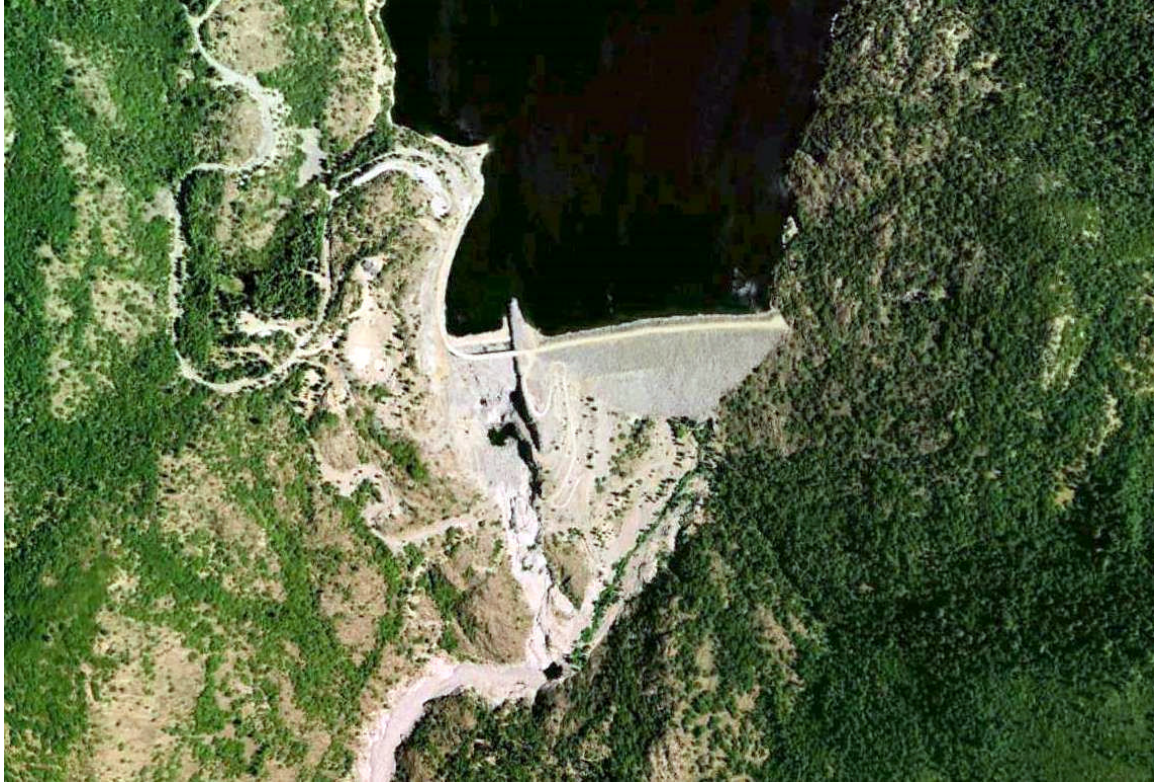


Figure 2-1 - Aerial View of Hell Hole Dam and Spillway



Figure 2-2 - View from upstream of Hell Hole Dam and Spillway



Figure 2-3 - Hell Hole Spillway Approach Channel



Figure 2-4 - Hell Hole Spillway Weir



Figure 2-5 Spillway Discharge Channel showing Invert Roughness @ Bridge Crossing



Figure 2-6 - Top of Stepped Chute

3. RESERVOIR RIM STABILITY

Hell Hole Reservoir is located in a steep sided valley in the Sierra Nevada Mountains at elevation 4630 feet above sea level. The reservoir was impounded in 1965 by construction of Hell Hole Dam, a 410 feet high embankment dam. The reservoir has a gross storage volume of 207,590 acre feet at its present maximum operating water surface elevation of 4630 ft. and covers an area of 1253 acres.

The inundation zone for the proposed seasonal storage increase lies within the lower part of the existing flood pool of Hell Hole Reservoir. Since the reservoir has been in existence for almost one half century, most typical shoreline development processes within the inundation zone have already occurred.

Increased duration of summer storage levels in the inundation zone between elevations 4630 ft. and 4636 ft. will change the reservoir shoreline by causing localized wave erosion in areas where there are overburden deposits or talus slopes. Normal shoreline development processes of wave erosion, deposition and vegetation growth are expected to result in an armored shoreline that is resistant to wave erosion.

No significant long term incremental impact on reservoir rim stability is anticipated due to the proposed seasonal storage project.

4. SITE INVESTIGATIONS AND GEOLOGY

4.1 SOURCES OF DATA

The geologic information summarized in this chapter and seismic information summarized in Chapter 7 are taken largely from data and discussions presented in the PCWA's *Supporting Technical Information Document, Hell Hole Dam, FERC Project 2079-CA (STID)*, which was prepared as part of the Potential Failure Mode Analysis (PFMA) conducted for Hell Hole Dam in 2006.

The STID was originally submitted to FERC in December 2006 but later revised in April 2008 to incorporate findings from recently completed seismotectonic studies for the Nevada Irrigation District's (NID) and Sacramento Municipal Utility District's (SMUD) hydroelectric projects.

Numerous data sources were used to compile the information presented in the STID. These sources included:

- the previous Five-Year Dam Safety Inspection reports from 1986 through 2001 which were based on reviews of construction drawings, published geologic literature and other data presented in various project specific reports prepared since dam construction;
- background data for the PFMA, which included the various project reports and drawings, as well as construction and inspection documents from California Division of Safety of Dams (DSOD) and Federal Energy Regulatory Commission (FERC) files;
- fault maps from the Nevada Bureau of Mines and Geology;
- the USGS Earthquake Hazards Program Circular Area Earthquake Search web site;
- *Seismotectonic Review and Investigation for the Upper American River Project, FERC Project No. 2101* for Sacramento Municipal Utility District (Hamilton and Harlan 2006)
- *Seismotectonic Review and Evaluation for Nevada Irrigation District Dams, Phase I - Identification and Characterization of Potential Seismic Sources and Phase II - Seismic Ground Motions for NID Dams* for Nevada Irrigation District (Hamilton 2007);
- *A seismic characterization study for Stampede, Prosser Creek and Boca dams* (URS/USBR 2005);
- on-line probabilistic seismic hazard maps updated by the USGS in 2008;
- regional earthquake data from the University of Nevada Reno;
- field observations made during a geologic reconnaissance on October 4, 2005.

This data review was supplemented by a geologic field reconnaissance of the existing spillway weir foundation area, conducted on August 11, 2009

4.2 REGIONAL GEOLOGY

Hell Hole Dam is located in the central-northern portion of the Sierra Nevada Mountains, just west of the range crest and approximately 15 miles west of Lake Tahoe, in Placer County, California. Figure 4-1 provides a geologic map for the vicinity of Hell Hole Reservoir.

Basement rocks in this region of the Sierra Nevada are mainly Mesozoic age granitic rocks, with a localized area of gabbro (a more mafic variety of Mesozoic intrusive rock) underlying the Hell Hole dam site. Some remnants of Paleozoic and Mesozoic-age metamorphosed sedimentary rocks are also present in the region. These basement rocks are overlain by abundant Miocene and Pliocene-age volcanic deposits, which in the vicinity of the project are primarily andesitic pyroclastic flows with lesser rhyolitic tuff. Minor amounts of Eocene alluvial channel deposits ("auriferous gravels") are locally present between the basement rocks and overlying volcanic deposits (e.g., on the upper left abutment of L.L. Anderson Dam). Pleistocene glacial deposits are common throughout the region.

Compressional tectonics in the late Jurassic (the first of two episodes of major tectonic activity postulated for the northern Sierra Nevada) resulted in the accretion of oceanic sedimentary and volcanic rocks to the North American continent, and the subsequent intrusion of plutonic rocks. The Foothills fault system, consisting of the Bear Mountain and Melones fault zones, is considered to represent the zone along which these oceanic rocks were accreted onto the continent.

Extensional and transtensional tectonics, represented by the uplift and westward tilting of the Sierra Nevada, commenced in the late Tertiary and continues in the present tectonic regime. The reorientation of regional stress from compressional to extensional both activated normal displacement on some older Mesozoic faults and initiated new faulting in the region, most of which is concentrated near the east margin of the Sierra block

4.3 SITE GEOLOGY

4.3.1 Site Geology and Local Foundation Conditions

Hell Hole Dam and Reservoir are located on the Rubicon River, just downstream from where it bends sharply to the southwest from its northwest-draining course through Rockbound Valley. No faults have been mapped in the immediate vicinity of the dam and reservoir.

The following sections describe foundation conditions in the reservoir, at the existing dam and at the existing spillway. Figures showing foundation conditions and borelogs are found at the end of this report section.

4.3.1.1 Reservoir

Most of the reservoir area is underlain by granitic rock, including the slope underlying the French Meadows penstock upstream of the dam on the north bank of the reservoir. The reservoir flanks are variably wooded, and numerous outcrops of hard rock are exposed above both sides of the reservoir and downstream of the dam. Varying amounts of talus and colluvium mantle the valley slopes.

4.3.1.2 Dam

The dam foundation is predominately medium to fine-grained gabbro. During construction, in-place gabbro was either exposed at the surface or encountered at depths of up to about 15 feet beneath talus, loose rock and/or colluvium on the abutments and beneath alluvium in the channel. All loose material was reportedly removed from the dam foundation during construction, except for occasional talus blocks under the rock fill shell.

Limited geologic mapping data for the dam foundation is presented on the available construction drawings. No geologic map of the as-built foundation could be located in any of the files reviewed for the 2006 PFMA. However, the logs of core holes drilled for the design exploration just upstream of the dam axis indicate the shallow foundation rock was hard and variably fractured, ranging from closely fractured to massive with iron-oxide stained but typically tight joints.

The deeper foundation rock is shown on these logs as consistently fresh, hard and massive below depths ranging from 16 to 66 feet. A summary description of the dam foundation presented in a periodic appraisal of the dam by DSOD, dated May 20, 1987, indicated that "moderate surface jointing" characterized the rock to a depth of 5 feet, below which the rock "is fresh and massive", and that the core zone foundation was excavated to 10 feet deep (this description was presumably based on DSOD foundation construction inspection reports). As-built sections across the final excavation (not included) indicate that the core foundation is frequently level with the adjacent shell foundation, and that an actual core trench, where present, is at depths of up to about 5 to 10 feet lower than the adjacent shell.

Three prominent joint sets were noted along the downstream groins and at the left and right ends of the dam crest during the October 2005 reconnaissance. These joints include a near-vertical set striking approximately parallel to the dam axis (N70°-75°E, dipping 78°S to vertical), and a moderately to steeply-dipping set that is roughly transverse to the axis (N5°E to N43°W, dipping 50°W to 80°W). This latter set forms locally unsupported blocks where it dips out of the near vertical cut left of the powerhouse at the toe of the dam, and given its orientation would likely constitute the primary seepage path through the foundation. A third set consisting of low angle joints (N20°-30°E, dipping 15°-20°W) was noted on the lower right and left abutments.

The foundation gabbro is locally intruded by both mafic and felsic, fine-grained dikes, some of which are near vertical and approximately normal to the axis of the dam (these dikes were noted on a rock knob immediately downstream of the dam on the lower right abutment during the October 2005 reconnaissance).

Several shear zones transverse to the dam axis were encountered during preparation of the core foundation, and were variously treated by excavating, backfilling with slush grout and dental concrete, and/or consolidation grouting and various combinations of these techniques. DSOD construction records indicate that one of the more notable shears was located at the toe of the lower left abutment, along the left side of the channel. In addition, several areas of the core foundation were damaged by excessively heavy blasting that was performed for the purpose of foundation shaping, and were similarly treated by removal of loose rock and placement of slush grout, dry pack mortar and dental concrete, with subsequent grouting. These blast-damaged areas were located partway up the left abutment between Stations 17+60 and 18+50, and on the upper right abutment between Elevations 4615 and 4630 feet; both areas were treated to the satisfaction of the DSOD field engineer according to the DSOD records.

A predominately single row grout curtain was installed in the foundation along the designed alignment of the core trench (i.e., beneath the core/foundation contact). Primary holes were spaced on approximately 40-foot centers, and were drilled to depths of up to about 40 feet on the upper abutments to maximum depths of typically 120 feet through the channel. Most

secondary and tertiary holes between the primary holes were located on 10-foot centers and drilled to depths of 10 or 15 feet. An upstream and downstream row ("left" and "right", respectively) were each drilled and grouted across the lower abutments and channel section, at a distance of about 20 to 30 feet from the main curtain and with wider hole spacing. Grout takes were typically very low, and areas of high take were mostly concentrated in the upper stage portions of holes along the uppermost abutments. A plan and profile of the foundation grouting, depicting hole locations, depths, and grout takes, are available in PCWA files. Generally, low seepage from the toe of the rockfill embankment attests to the overall low permeability of the foundation. No seepage was noted from the abutment rock adjacent to the lower downstream groins of the dam during the October 2005 reconnaissance.

4.3.1.3 Spillway

The spillway was excavated through fresh, strong and hard gabbro. In the area of the spillway crest weir structure, that rock is typically massive with some localized closely jointed zones. Most joints are near-vertical to vertical and generally tight, with one prominent joint set trending approximately N30°W to N60°W. A less prominent secondary set of near-vertical joints trends ENE, across the prominent NW-trending set. These intersecting joints form several partially detached blocks on the vertical left wall of the spillway cut, at the east end of the weir structure. The presently unsupported blocks may require either removal or support via anchoring, with low pressure grouting or dental concrete to seal the fractured rock mass at the left end of the proposed raised spillway gate. The vertical spillway cut at the right end of the weir appears mostly unfractured and massive.

The foundation excavation for the weir structure was mapped during construction (see Figure 4-3). This geologic mapping indicates that rock joints in the foundation were generally tight, except where disturbed by blasting for the excavation. An approximately 120-foot section of the weir foundation, within the central-right portion of the weir from Stations 3+80 to 5+00, was heavily fractured ("broken") by poorly controlled blasting. This area was reinforced with 3 rows of 10-foot deep rock bolts.

A grout curtain was installed along the entire crest structure foundation, after construction of the weir (see Figure 4-5). Grout takes were generally low except within the blast-fractured section along the center-right portion of the structure, where 1,800 sacks of cement were placed within the shallow-stage grout holes (i.e., within 10 feet of the concrete/foundation contact). Bechtel drawing No. 61-07-202 indicates that "this section contains steep joints opened by blasting and partially filled with quarry fines that cross cut the weir foundation".

It is unclear if this heavily fractured section of the weir foundation might be prone to increased levels of seepage under the increased head when water is seasonally stored behind the proposed raised spillway structure; however, given the large volume of grout that was placed along this reach during construction, and the apparent lack of notable seepage from the downstream toe of the existing weir during past periods of high storage, it is considered likely that this area will not be adversely affected by the nominally increased storage.

Examination of the contact between the concrete weir and the gabbro rock foundation indicates that, where visible, the contact is tight with the concrete well-bonded to the underlying rock. Bechtel's foundation map does however note that a 50-foot section of the weir foundation was

not mapped prior to its being covered with concrete, along the left side of the weir from approximately Stations 5+75 to 6+25. It was noted during the August 2009 field visit that a considerable amount of angular rock debris (up to about 6-inches diameter) is encased within the weir concrete immediately above the foundation along this section. It is unknown whether this area was adequately cleared of debris, or inspected, prior to concrete placement.

Although past performance of the existing spillway weir has not revealed any indications of excessive foundation seepage or problems with inadequate foundation strength (e.g., as might be revealed by settlement or cracking of the concrete weir structure), it is unclear if the blast-damaged area of the foundation (with blast-opened fractures partially filled with quarry fines, as reported by Bechtel) would be susceptible to increased seepage or even removal/erosion of loose material that might be present within the foundation (i.e., quarry fines). The quality of the weir/foundation contact, both within the 50-foot left side section and along the blast-damaged section further right, could be further evaluated via drilling of several core holes through the weir concrete and into the foundation.

The spillway cuts have for the most part remained stable with the exception of several large rock slides that occurred, for a period of time after construction, along an undulating, east-dipping joint surface that dips out of the right side of the spillway excavation.

4.3.2 Landslides, Loose Rock or Adverse Bedding

The Rubicon River valley in the area of the dam and reservoir was extensively glaciated. As a result, the reservoir flanks are mostly exposed or lightly wooded bedrock mantled with varying amounts of talus and colluvium. The reservoir slopes are generally moderate, but are locally steep to near vertical in some areas of exposed rock. No significant landslides or areas of natural slope instability have been mapped in the area of the reservoir and dam, and none were noted during the October 2005 field inspection.

As mentioned in Section 4.3.1.2, a moderately to steeply-dipping joint set dips out of the near vertical cut left of the outlet and powerhouse at the toe of the dam. Several localized, unsupported and potentially unstable blocks overlie these outdipping joints; damage to the Howell-Bunger valve, downstream end of the outlet or the stream maintenance pipe could be caused by rockfall from this cut.

In the period shortly following construction, several large-volume rock slides occurred from the east-facing vertical cut above the right side of the spillway, due to removal of support for the rock mass above a prominent, undulating, 40° to 50° east-dipping joint (this removal of support was a result of the excavation for the spillway, which daylighted/undercut the out-dipping joint surface). Examination of the northward extent of this joint/discontinuity, where it drops down to the level of the access road, suggests that weathered dike material may contribute to low shear strength along the joint (a soft, weathered quartz-mica dike, up to about one-inch thick, was noted at this location along the joint surface; the failure surface was previously described as a shear during construction inspection). The main failures along this surface occurred above and mostly downstream of the access road and spillway bridge, over a period during and after construction. The majority of the rock failed during construction in 1965, with an initial slide estimated at 150,000 cubic yards (cy). The last significant sliding occurred in 1972, and the total accumulated volume of slide material up through that time has been

estimated at about 230,000 cy. Smaller rock falls have occurred since 1972, including during the storms of February and March, 1986 and in the early and mid 1990s.

The high vertical cut along the access road upstream from a point just north of the spillway bridge, above the approach channel, presently appears stable. This apparent stability exists despite the presence of the rock mass that is remaining above the out-dipping joint that continues along this section. The orientation of the cut along this reach is somewhat less parallel to the strike of the out-dipping joint than along the reach where it previously failed, which may favorably affect the stability of the overlying mass. It is estimated that the remaining potential future rock slide mass above the spillway approach channel is approximately 150,000 to 200,000 c.y.

Several apparently detached blocks or slabs (one approximately 20 feet x 30 feet x 5 feet thick) were noted at the top of the spillway cut in the area generally above the roadway bridge, during the October 2005 geologic field reconnaissance. The top of this cut face should be periodically checked for signs of developing distress (e.g., tension cracks near the edge of the cliff, or large detached blocks).

4.3.3 Sinkholes, Karst, or Solutioning Issues

There is no known sinkhole potential at Hell Hole Dam due to the insoluble and non-erodible nature of the hard gabbro that comprises the foundation.

4.3.4 Weak Seams

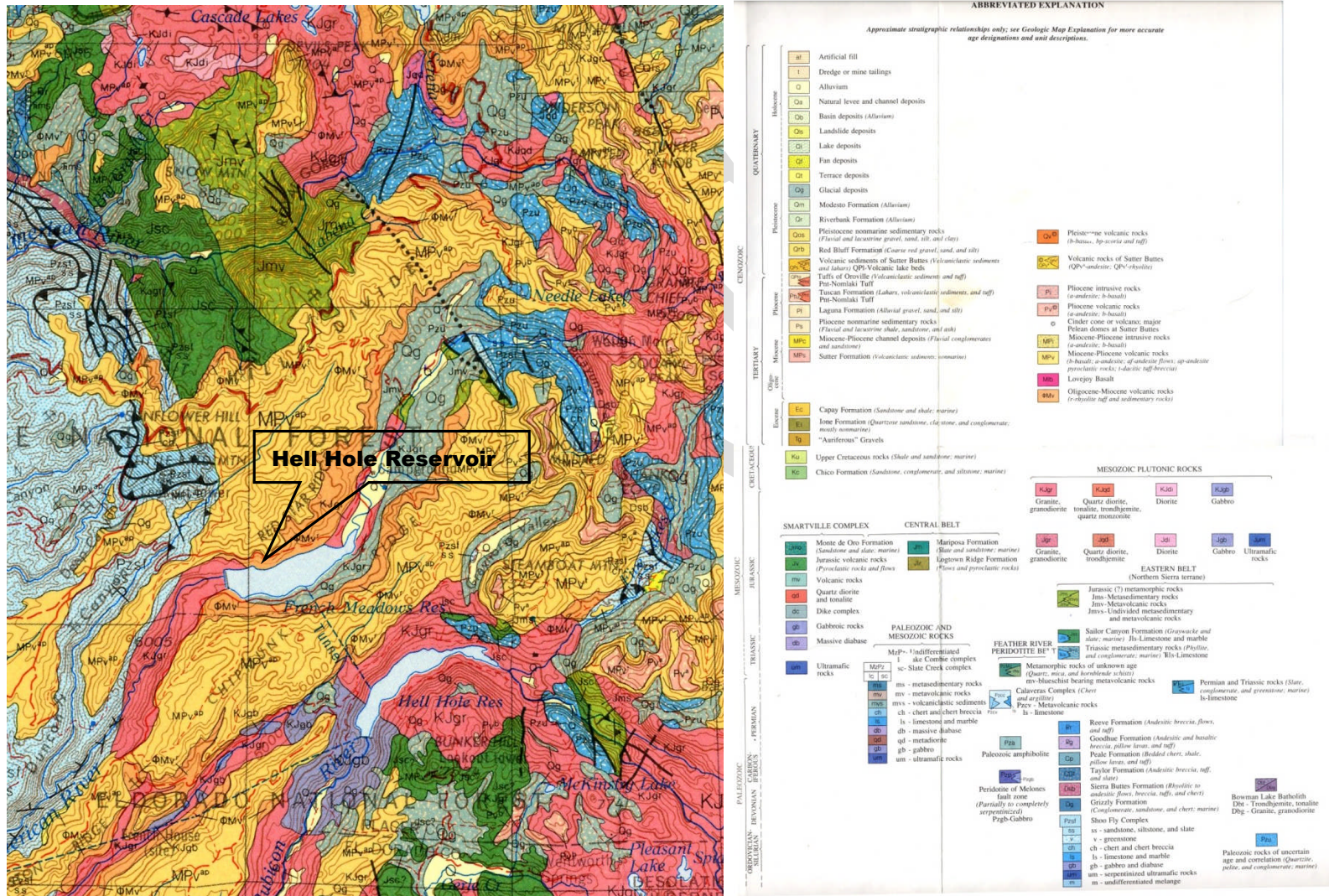
Because of the hard and strong condition of the gabbro comprising the foundation of Hell Hole Dam, the foundation is considered to be grossly stable. In addition, there are no known potentially weak discontinuities in the dam foundation (e.g., weathered dikes or shears, or clay-filled joints). No soluble joint infillings (e.g., calcite or gypsum), weak zones or discontinuities were mapped or otherwise reported in the construction documentation, and no such conditions were noted during the October 2005 geologic field inspection.

As discussed above, the block slide failures that have occurred along the right side of the spillway cut may have been influenced by the presence of a weak, weathered dike or shear along the eastward, outdipping discontinuity at that location. However, those failures are largely attributed to the removal of lateral support for the rock mass above the east dipping joint when that surface was daylighted (exposed) by the vertical spillway excavation.

4.3.5 Geologic Artesian Sources

There are no known artesian sources in the area of the dam site. No problems with spring flows in the foundation were recorded in the construction files reviewed for the PFMA.

Figure 4-1 - Regional Geology for Hell Hole Reservoir



Map Source: Geologic Map of the Chico Quadrangle, California, 1:250,000, Compilation by G.J. Saucedo and D.L. Wagner 1992
 California Division of Mines and Geology, Regional geologic Map Series, Chico Quadrangle - Map No. 7A (Geology), Sheet 1 of 5

Figure 4-2 - Hell Hole Dam Geologic Plan and Sections

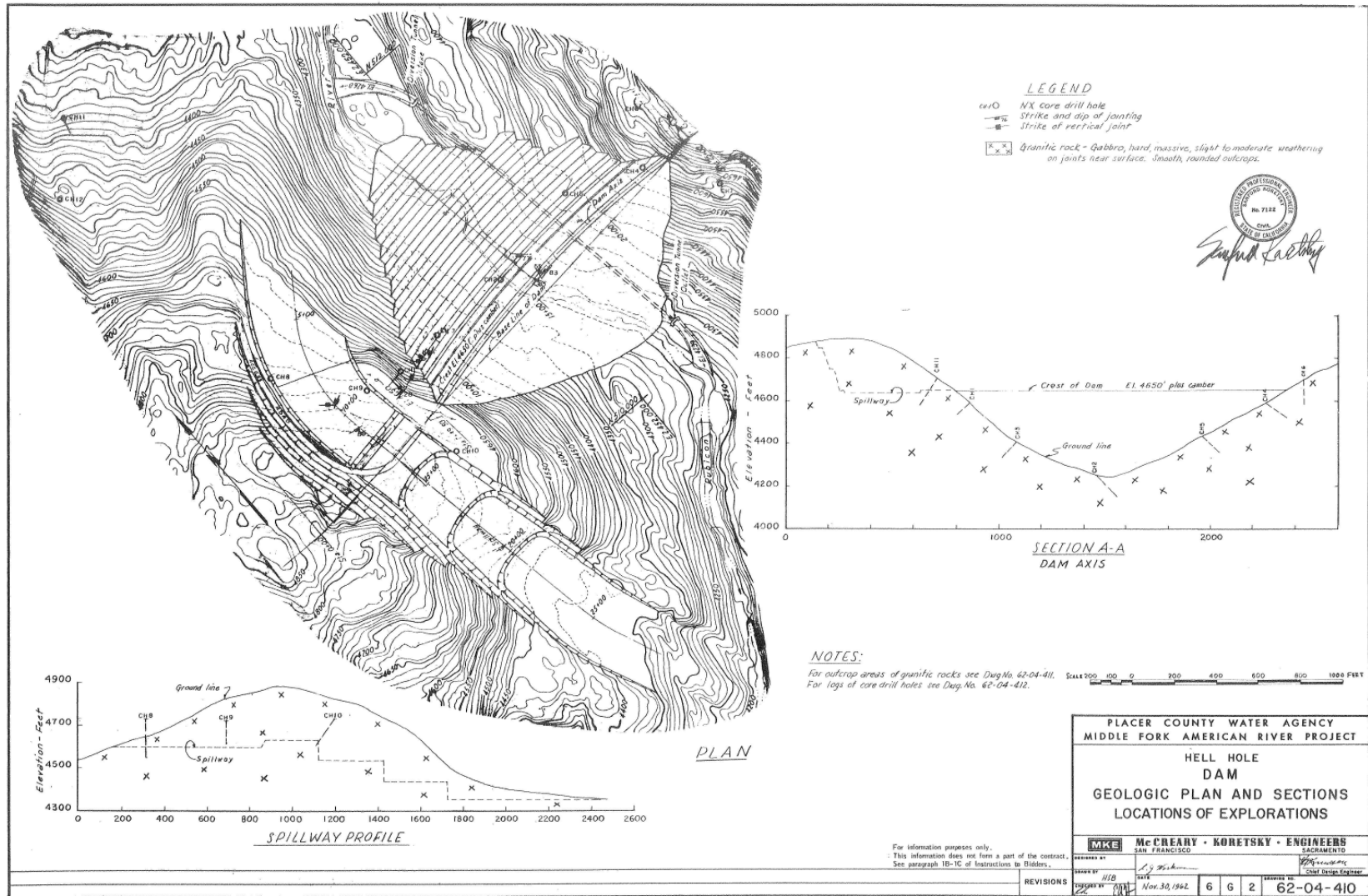


Figure 4-3 - Hell Hole Dam Logs of Core Borings

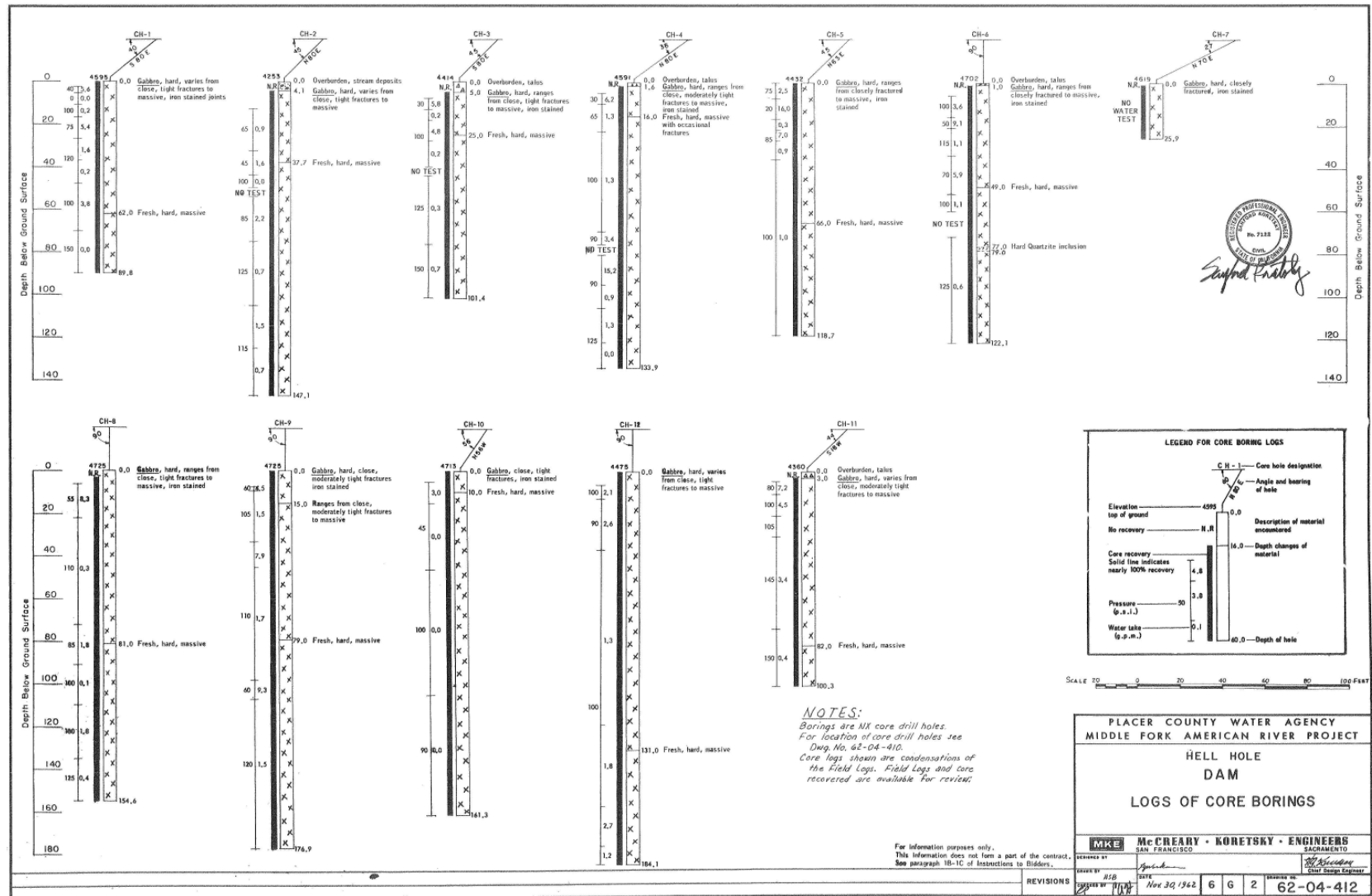


Figure 4-4 - Spillway Weir Foundation Excavation

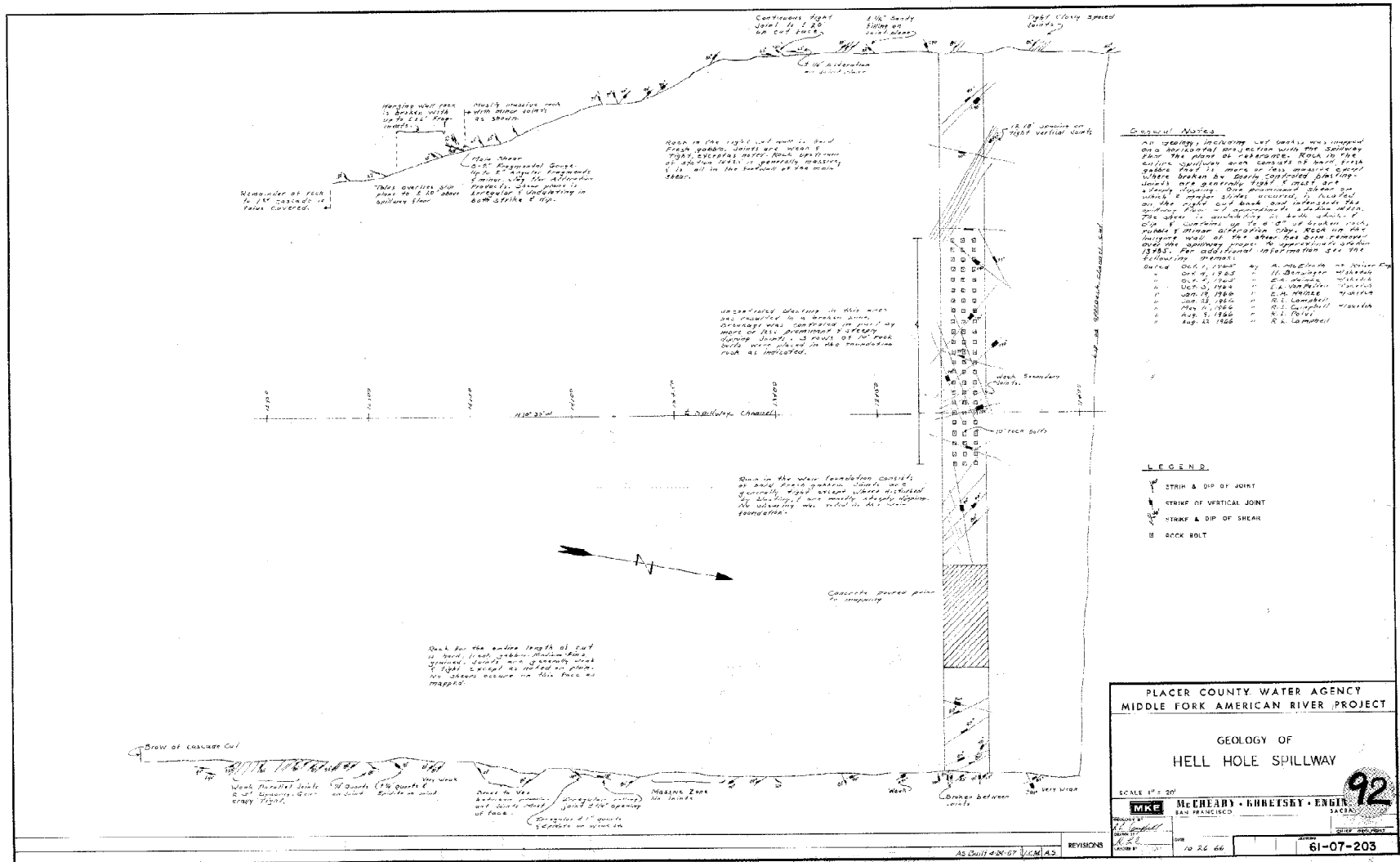
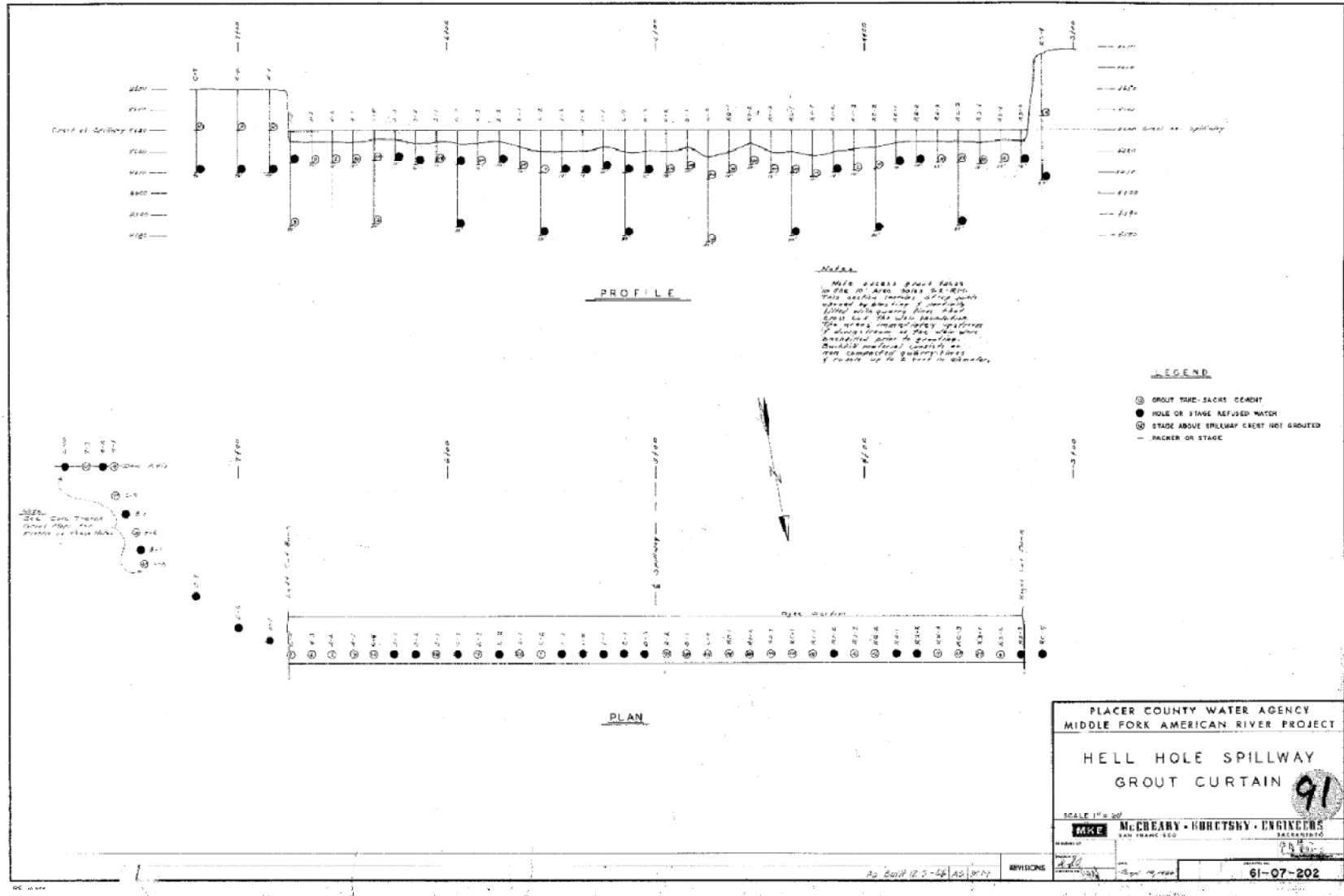


Figure 4-5 - Spillway Weir Grout Curtain



5. BORROW AREAS AND CONSTRUCTION MATERIAL

No borrow areas or in-situ construction material are required for the proposed seasonal storage project.

DRAFT

6. STABILITY AND STRESS ANALYSIS

6.1 ANALYSIS METHODOLOGY AND CRITERIA

6.1.1 Geometry

The physical layout and geometry of the weir is shown on construction drawings which are reproduced on Figures found at the back of this report section. Figure 6-1 shows the location of the concrete ogee section in the spillway channel relative to the approach channel, the roadway bridge and the right abutment. Figure 6-2 is a copy of an American River Constructors drawing showing a cross section through the spillway weir crest.

For analysis purposes the downstream 3 ft. of the concrete was assumed structurally ineffective since the drawing shows that part of the structure to be quite thin. The resulting assumed base width is 19.0 ft. In addition, any horizontal support from foundation rock downstream of the structure or "keying in" of the foundation was neglected. The base of the concrete was assumed to be level at El. 4625.50 ft. as shown on the drawings, although it is likely that the foundation excavation was deeper in some locations.

6.1.2 Criteria

The structural stability of the concrete ogee section for the various loading conditions was evaluated on the basis of permissible stresses and other criteria which are in accordance with current practice as follows:

6.1.2.1 Uplift

Uplift pressure beneath the concrete ogee section varies linearly from full reservoir pressure under the upstream heel to tailwater pressure at the downstream toe.

6.1.2.2 Overturning

The resultant force at the base of the section must fall within the kern of the base. No vertical tensile stresses permitted at the rock/concrete interface at the upstream toe of the ogee section.

Sliding

The evaluation of safety against sliding of the ogee section at its contact with the foundation is computed by use of the standard shear-friction equation. The criteria used for evaluation of sliding safety are as follows:

Loading Case	SFF
Usual Loading	3.0
Unusual Loading	2.0
Seismic Loading	1.3

The expression for the shear-friction factor of safety (SFF) is the ratio of resisting-to driving forces as follows:

$$SFF = \frac{\mu \sum N + A_c \tau_v}{\sum S} = \frac{\text{Resisting Force}}{\text{Driving Force}} = \frac{\text{Capacity}}{\text{Demand}}$$

- where:
- $\sum N$ = Summation of compressive forces, reduced by any uplift forces that are acting normal to the failure plane.
 - $\sum S$ = Summation of shear forces acting on the plane.
 - μ = Coefficient of friction along the assumed failure plane.
Or, $\mu = \tan \phi$, where ϕ is the internal angle of friction for the material.
 - τ_v = Maximum allowable shear stress (cohesion) on the assumed failure plane without normal forces acting.
 - A_c = Compression force area of the plane.

6.1.3 Loading Conditions

The analysis investigated the following loading conditions:

- ▶ **Load Case 1: Normal Summer Reservoir Level – Water level at the top of the proposed 6 ft. high closed gates.**
 - Dead Weight of Concrete (150#/CF)
 - Weight of Water above concrete (62.4 #/CF)
 - Res. Water Surface El. 4636.0
 - Tailwater None
 - Uplift Pressure
- ▶ **Load Case 2: Summer PMF – Overtopping of closed gates by 7.0 ft.⁵**
 - Dead Weight of Concrete (150#/CF)
 - Weight of Water above concrete (62.4 #/CF)
 - Res. Water Surface El. 4643.0 (7 ft. gate overtopping)
 - Tailwater El. 4832.9
 - Uplift Pressure
 - Anchor Force
- ▶ **Load Case 3: Seismic Condition (Proposed gates/flashboards in raised position).**

⁵ See Reference 2 for a discussion of the determination of the reservoir and tailwater levels for the summer PMF

- Dead Weight of Concrete (150#/CF)
- Weight of water above concrete (62.4 lb/CF)
- Res. water surface El. 4636.0
- Tailwater None
- Uplift Pressure
- Anchor Force
- Seismic Loading - Estimated peak ground acceleration (pga) = 0.35

Figure 6-3 and Figure 6-4 show schematic views of the loading diagrams and force vectors.

6.2 METHODOLOGY

The analysis was performed by the two dimensional unit equilibrium method using a spreadsheet model specially developed for the analysis of concrete gravity dams and spillway sections.

The first step of the analysis was to determine the overturning stability. The value and location of the resultant vertical force were determined neglecting the stabilizing effect of the existing anchor dowels and the eccentricity of the resultant about the center of the base was calculated. If the resultant vertical force falls within the kern⁶ of the base, no vertical tensions would develop at the base and no vertical tensile forces would be transferred to the anchor dowels.

For Loading Case 1 - Normal Summer Reservoir Level the resultant force falls well within the kern of the base. For Case 2 - Summer PMF Reservoir Level and Case 3 - Normal Summer Reservoir Level plus Earthquake, the resultant vertical force falls somewhat downstream of the kern, which would cause tensile stresses to develop under the upstream portion of the base.

It is assumed that tensile stresses cannot be transmitted from the concrete into the foundation rock, therefore any tensile forces would be transferred into the anchor dowels. For Load Cases 2 and 3 the value of tensile forces in the anchor dowel that would bring the resultant force to the kern was computed and the tensile stress in the dowels was compared to the yield strength of the steel. The shear-friction factor of safety (SSF) and the bearing pressure at the base of the concrete were then computed.

Earthquake loading for the analysis was estimated using the gravity method and accounting for the hydrodynamic effect of reservoir water by the Westergaard method. This procedure assumes a parabolic shaped portion of hydrodynamic water mass upstream of the dam accelerates along with the dam concrete mass.

⁶ middle third of the base resulting in an eccentricity less than Base Width/6 =3.2 ft.

6.3 RESULTS OF STABILITY ANALYSES

The results of the analysis for the three loading conditions are presented on the following table:

Table 6-1 - Stability Analysis Results

Loading Condition	Case 1	Case 2	Case 3
Overturning Stability			
Eccentricity (ft.) ⁷	0.4	14.1	4.7
Tensile Stress in Dowels (psi) ⁸	0	9400	2500
Sliding Stability			
Allowable Shear Friction Factor of Safety	3.0	2.0	1.3
Cohesion (psi) ^{**9}	3	5	6
Maximum Bearing Pressure (psi)	2	9	9

6.4 CONCLUSIONS

Stability analyses indicate that the concrete weir on Hell Hole spillway is capable of supporting the additional structural loading imposed on the structure by the proposed addition of six foot high crest gates

Under the normal full reservoir condition (Load Case 1) the vertical resultant force remains well within the kern of the base and no tensile forces would be transferred to the steel anchor dowels.

Under the summer PMF and the earthquake conditions (Load Cases 2 and 3) the vertical resultant force lies downstream of the kern and under the assumed conditions and some tensile forces would be transferred to the steel anchor dowels. The yield strength of the steel is unknown, but reinforcing steel commonly used at the time the structure was constructed had a yield strength of 40, 000 psi. The calculated stresses in the steel are less than 25% of this value.

The calculated cohesion at the rock/concrete foundation interface required to meet minimum values of shear-friction factor of safety are very low and well below the expected capacity of the foundation even in the blast damaged areas discussed in Section 4 of this report.

Calculated values of bearing pressure for the three loading conditions investigated are very low and of little concern.

⁷ Eccentricity of the resultant vertical force from the center of the base computed neglecting the anchor dowels.

⁸ Computed stress (psi) in anchor dowels assuming resultant vertical force is at downstream boundary of kern (eccentricity = 3.2 ft.)

⁹ Value of cohesion at the rock/concrete foundation interface required to meet the allowable shear friction factor of safety with the angle of internal friction assumed at 45°

Figure 6-1- Topographic Mapping - Hell Hole Abutments and Upper Spillway

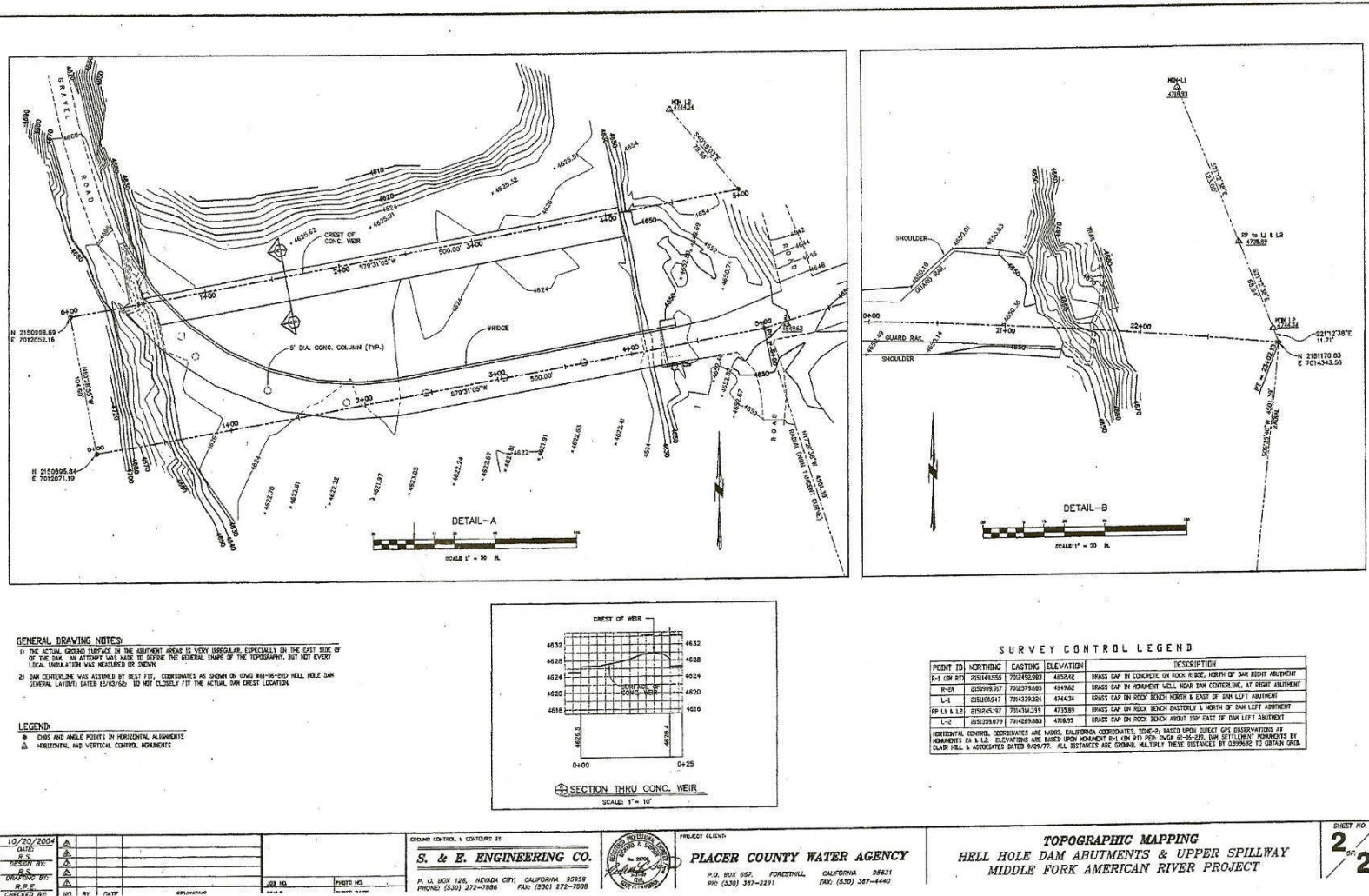


Figure 6-2 - Spillway Ogee Section

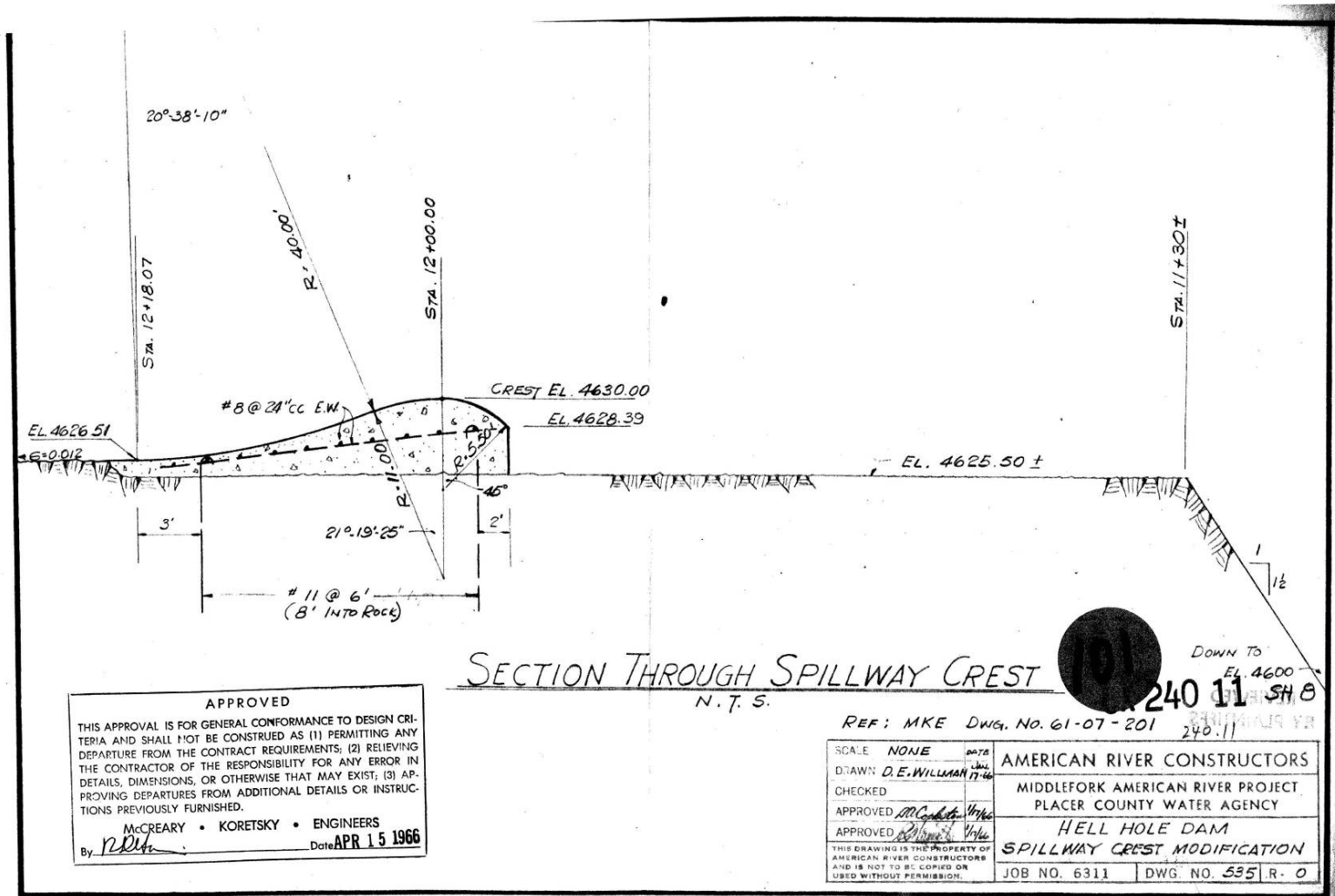


Figure 6-3 - Loading Conditions - Load Cases 1 and 3

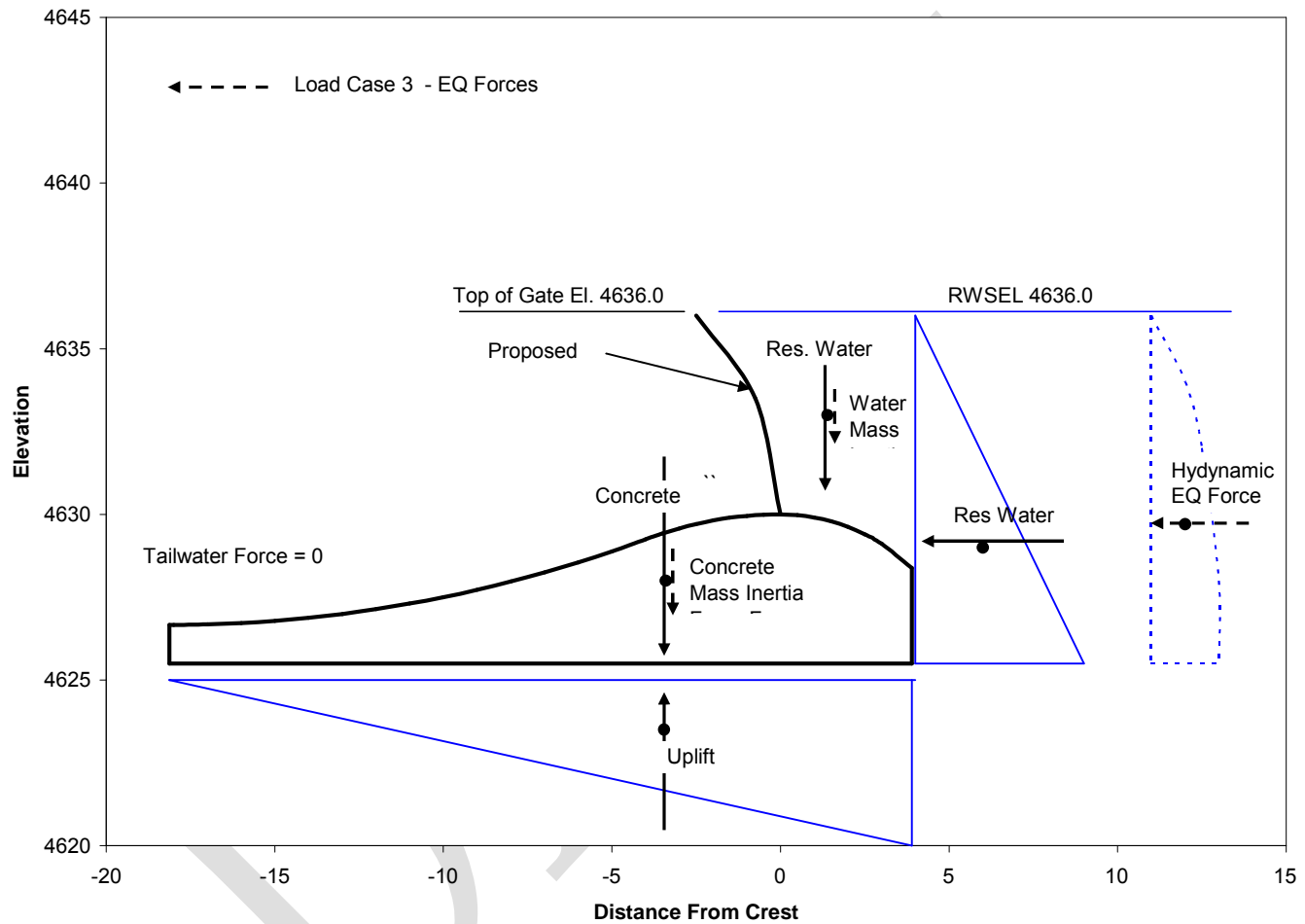
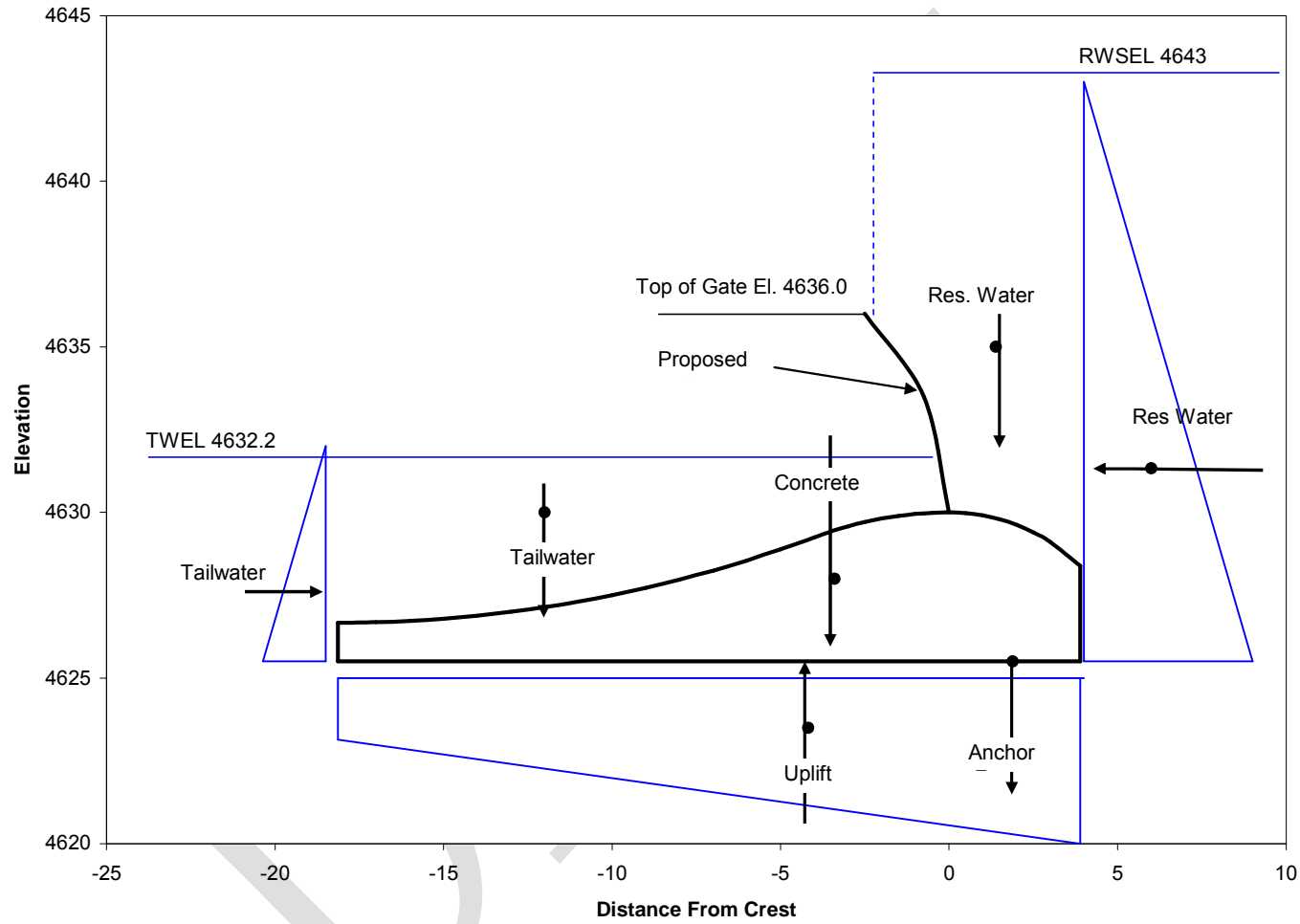


Figure 6-4 - Loading Conditions - Load Case 2



7. SEISMIC LOADING

7.1 FAULTING AND MAP OF FAULT TRACES

Within the project region, the eastern portion of the Sierran block is relatively intact, bounded on the west by the north-northwest trending Foothills fault system and on the east by the northwest trending Sierran eastern frontal fault system. Both Hell Hole and L.L. Anderson dams are located toward the east margin of the block, just west of its transition to the Basin and Range extensional tectonic region.

A considerable amount of seismotectonic research within the Tahoe region and along the east margin of the Sierran block has been completed since the last Five-Year Dam Safety Inspection report for Hell Hole Dam was prepared in 2001. These studies indicate that, at the latitude of Hell Hole Dam, the westernmost-extent of active faulting along the east margin of the Sierran block is located somewhat west of Lake Tahoe along what has been referred to as the "Tahoe-Sierra frontal fault zone" (Schweickert et al, 2000) and the Tahoe-Sierra Eastern Frontal fault zone (Hamilton and Harlan, 2006). This fault zone has been described as accommodating distributed extension (i.e., it consists of segmented normal faults) along the west margin of the Tahoe basin, and is located 18 km (11 miles) east of Hell Hole Dam.

The more continuous West Tahoe-Dollar Point fault zone further east (26 km/16 miles from Hell Hole Dam) is considered to be one of the youngest and most active faults in the Tahoe basin. The Tahoe West Slope fault zone splays northward from the main Eastern Frontal fault zone, between the Frontal fault and the West Tahoe fault, and forms scarps in glacial moraines along Meeks Creek above the west shore of Lake Tahoe (Hamilton and Harlan, 2006).

Bathymetric and surface mapping studies indicate that the faults in the Tahoe basin are more active than previously thought, and may have a combined rate of activity similar to the Carson Range (Genoa) fault system (URS/USBR, 2005). The Genoa fault runs along the eastern front of the Carson Range just east of Lake Tahoe, and is one of the principal faults along the Sierra Nevada - Basin and Range transition zone. It has produced several large magnitude (M 7.2-7.5) late Holocene earthquakes (500 -600 years before present (bp) and 2,000- 2,200 years bp) that were recently defined by trenching studies (Ramelli et al, 1999).

Most of the eastern frontal region/Tahoe basin faults are considered to have relatively high slip rates of over 1 mm/year; Hamilton (2006) estimate a slip rate of about 1.5 mm/yr for the Tahoe-Sierra Eastern Frontal fault zone.

As shown on Table 7-1 additional known but more distant surface fault sources that could generate lower levels of shaking at Hell Hole Dam include the Dog Valley fault and Mohawk Valley fault zone in the region to the northeast of the dam, and the Big Dipper and Downieville zone of faulting (as defined by Hamilton, 2007) within the Foothills fault system to the west. These latter faults within the Foothills fault system, which includes numerous inactive Mesozoic-age suture zones and serpentine belts, are characteristically low slip-rate faults.

Previously suspected seismic sources in the region have included the PG&E-conceived Snow Mountain, Volcanoville and Giant Gap faults, which were shown on the *Fault Activity Map of California* (Jennings, 1994) as late Cenozoic faults. Neither of the Snow Mountain nor Volcanoville postulated faults were evaluated via sufficient field study by PG&E to confirm their existence.

Recent investigations conducted for the SMUD seismotectonic review (Hamilton and Harlan, 2006) conclude that the PG&E-interpreted Snow Mountain late Cenozoic fault does not exist as a geologic fault, and furthermore that postulated fault was not shown as a potential seismic source in the recent seismic source characterization for Stampede, Prosser Creek and Boca dams (URS/USBR, 2005).

The Giant Gap fault was evaluated via some field work including trenching, but this work yielded only equivocal evidence for tectonic faulting. Consequently, none of these PG&E-proposed faults are shown as potential seismic sources on Table 7-1, but are indicated for reference on Figure 7-1.

The *Preliminary Map of Pleistocene to Holocene Faults in the Lake Tahoe Basin* (Schweickert et al, 2000) shows a feature identified as the "Rubicon fault" trending northwest along Rockbound Valley and along the uppermost reach of the Rubicon River (that map indicates that there was insufficient data to determine the timing of the most recent movement at the time of the map's preparation, and the basis for showing this feature was reportedly based on an examination of aerial photos without ground checking). This feature forms a pronounced air photo lineament that extends toward the upstream end of Hell Hole reservoir, approximately 3 miles upstream (east) of the dam. At this location the lineament appears to terminate, and does not extend through Chipmunk Ridge which separates Hell Hole and French Meadows reservoirs.

Air and ground reconnaissance examination of this feature, the "Rockbound Valley joint zone", indicate that the lineament results from glacial scouring along a prominent linear joint set, and is not related to faulting (Hamilton and Harlan, 2006).

In addition to the known seismogenic surface faults described above, which are shown on Figure 7-1 and listed on Table 7-1, another potential seismic source zone is defined by a northeast-southwest oriented cluster of earthquakes that have occurred in the region of Snow Mountain, immediately north of L.L. Anderson Dam. This local area of seismicity was evaluated as part of the recent 2006 SMUD seismotectonic study, and is recognized as a source of potentially moderate future earthquakes in that area (Hamilton and Harlan, 2006). As mentioned above, this zone is not associated with the northwest-trending PG&E postulated Snow Mountain fault. Earthquake activity in the vicinity of Snow Mountain is depicted on the Figure 7-2 as the "Snow Mountain-Soda Springs area earthquakes".

Faults and suspected faults of potential significance to Hell Hole Dam are shown on Figure 7-1.

Table 7-1 - Table of Faults, MCE and PGA

Fault/ Source	Approximate Distance to Hell Hole Dam - miles (km)	Estimated Maximum Credible Earthquake Magnitude	Estimated Peak Ground Acceleration - $g^{4,5}$		Slip Rate - mm/yr	Sense of Slip
			median	84 th %ile		
Tahoe-Sierra Eastern Frontal fault zone	11 (18)	7.0 ¹	0.24	0.35	1.5 ¹	normal
Tahoe West Slope fault zone	13 (20)	6.8 ¹	0.20	0.29	1.5 ¹	normal, left-lateral?
West Tahoe-Dollar Point fault zone	16 (26)	7.0 ¹	0.17	0.24	0.1 - 2.0 ²	normal
Snow Mountain-Soda Springs seismicity source zone	7 (11)	6.0 ¹	0.21	-	-	-
Sierran Intrablock background earthquake	3 (5)	5.0 ¹	0.19	-	-	-
Genoa fault zone	29 (47)	7.5 ¹	0.11	0.16	0.3 - 4.0 ²	normal
Mohawk Valley fault zone (unsegmented)	34 (54)	7.3 ²	0.08	0.12	0.3 - 4.0 ²	normal, right-lateral
Dog Valley fault	27 (43)	6.7 ²	0.07	0.11	>0 - 1.3 ²	normal, left-lateral
Junction House segment of Downieville Zone (Foothills fault system)	25 (40)	6.2 ³	0.05	-	<0.05	normal
Big Dipper (Foothills fault system)	24 (38)	6.0 ³	0.05	-	<0.05	normal

Table Notes: _____

1. Maximum magnitude from Hamilton and Pyke, 2006; slip rates from Hamilton and Harlan, 2006.
2. Maximum magnitude and slip rates from URS/USBR, 2005.
3. Maximum magnitude from Hamilton, 2007.
4. Peak ground acceleration (pga) in rock from Sadigh et al, 1997. Note that the 1997 Sadigh et al attenuation relationship was selected since it provides more conservative pga values than those determined by other contemporary attenuation models; the recent Next Generation Attenuation models published in 2008 should be considered for estimating the site peak ground acceleration, as part of the next 5-year dam safety review.
5. Median peak ground acceleration values selected for low slip-rate faults and background seismic source areas without identified associated faulting; 84th percentile pga values shown for higher slip-rate faults in the eastern frontal region and Tahoe basin. No factors were applied for hanging wall or footwall effects in estimation of pga.

7.2 MAXIMUM CREDIBLE AND DESIGN BASIS EARTHQUAKE DEVELOPMENT

The project region is located in the central-north portion of the Sierran block, and is not as seismically active as other regions of California (e.g., the Coast Ranges). However, both Hell Hole and L.L. Anderson Dams are located toward the east margin of the block, near its transition to the Basin and Range extensional tectonic region which is characterized by active faulting.

A number of earthquakes have occurred historically within this marginal area of the Sierra block, and earthquakes will continue to occur within the region. Earthquakes that have been recorded in the region of Hell Hole Dam are shown on Figure 7-2.

An online search (April 2008) of earthquake data available from USGS and CDMG catalogs indicates that the nearest earthquake recorded in the vicinity of Hell Hole Dam was a magnitude 2.9 event in 1965, at a distance of 4 km (2.5 mi) northeast of the dam along Chipmunk Ridge (USGS Earthquake Hazards Program Circular Area Earthquake Search web site, 2008).

No other earthquakes within a distance of 10 km (6 mi) of the dam have been recorded in these USGS/CDMG catalogs. However, as shown on the Figure 7-2, which is based on data from the University of Nevada Reno's Seismological Laboratory (2005), epicenters of several earthquakes of less than magnitude 2 are shown approximately 9 km (5.6 mi) north-northwest of Hell Hole Dam. These earthquakes are likely associated with the several swarms of earthquakes that occurred south of Soda Springs in the vicinity of Snow Mountain between 1977 and 1987. The largest of these earthquakes was a magnitude 5.2 event that occurred in 1980, approximately 27 km (17 mi) north of Hell Hole Dam.

The Soda Springs earthquakes have not been attributed to any recognized surface fault source, but their epicenter distribution indicates their occurrence along at least three northeast-southwest aligned zones at depth. This northeast-southwest aligned pattern "accords with the NE-SW aligned faults of the conjugate fault system that occurs within the right lateral Eastern California shear zone centered along the adjacent Tahoe basin region" (Hamilton and Harlan, 2006). Given the approximately 25 km (15.5 mi) length of the zone of hypocenters that extend to depths of about 20 km (12 mi), and "the absence of any paleoseismic indication of faulting in surface outcrops", Hamilton estimates that a subsurface rupture of this zone would produce a maximum earthquake no greater than about magnitude 6 (ibid).

As previously discussed, PG&E has postulated the existence of the northwest-trending Snow Mountain fault, which is shown on the *Fault Activity Map of California* (Jennings, 1994) as a northwest-trending late Cenozoic fault with its southern end approximately 13 km (8 mi) north of Hell Hole Dam. PG&E indicated that the association of the Snow Mountain fault to the Soda Springs earthquakes is unclear (PG&E, 1994), and the northeast-southwest trending zone of rupture indicated by the distribution of epicenters does not align with the northwest-striking PG&E fault trace.

The PG&E-postulated Snow Mountain fault was not shown as a potential seismic source in the recent seismic source characterization for Stampede, Prosser Creek and Boca dams (URS/USBR,

2005) and, furthermore, investigations conducted for the recent SMUD seismotectonic review conclude that the PG&E-interpreted Snow Mountain late Cenozoic fault does not exist as a geologic fault (Hamilton and Harlan, 2006).

The Snow Mountain area is nevertheless clearly a recognized seismogenic source zone, and is defined as the “Snow Mountain-Soda Springs seismicity source zone” on Table 7-1 and depicted on Figure 7-2 as the “Snow Mountain-Soda Springs area earthquakes”. The closest larger-magnitude historical earthquake occurred in 1966 on the Dog Valley fault near Truckee, with an epicenter approximately 45 km (28 mi) northeast of Hell Hole Dam. This magnitude 6 earthquake produced an approximately 16 km- (10-mi) long zone of northeast-trending ground rupture (Jennings, 1994).

For the period 1735 to the present, nine earthquakes with a magnitude ≥ 5 have been instrumentally recorded or otherwise estimated within a 50 km (31 mi) radius of the dam. Only two of these were measured at magnitude 6, including the 1966 Dog Valley earthquake and an 1857 earthquake with an estimated magnitude and location just north of Lake Tahoe. In 1909, two earthquakes with estimated locations and magnitudes of 5.0 and 5.5 were reported to have occurred south of Downieville about 50 km (31 miles) northwest of Hell Hole Dam. These earthquakes may have originated somewhere along the southern portion of the Downieville zone of faulting as defined by Hamilton (2007).

Both L.L. Anderson and Hell Hole dams are situated within the almost aseismic “Sierran Intrablock domain” (Hamilton, 2007) with an estimated maximum background earthquake of magnitude 5 at 5 km (ibid). However, given the proximity of Hell Hole Dam to the Tahoe-Sierra Eastern Frontal fault zone, a large earthquake originating along that fault or on one of several other faults in the Tahoe basin would likely produce a stronger level of shaking at the dam than would result from a “local” background earthquake (similarly, the close proximity of L.L. Anderson Dam to the Snow Mountain-Soda Springs seismicity source zone makes that dam more susceptible to an elevated level of potential shaking originating under Snow Mountain).

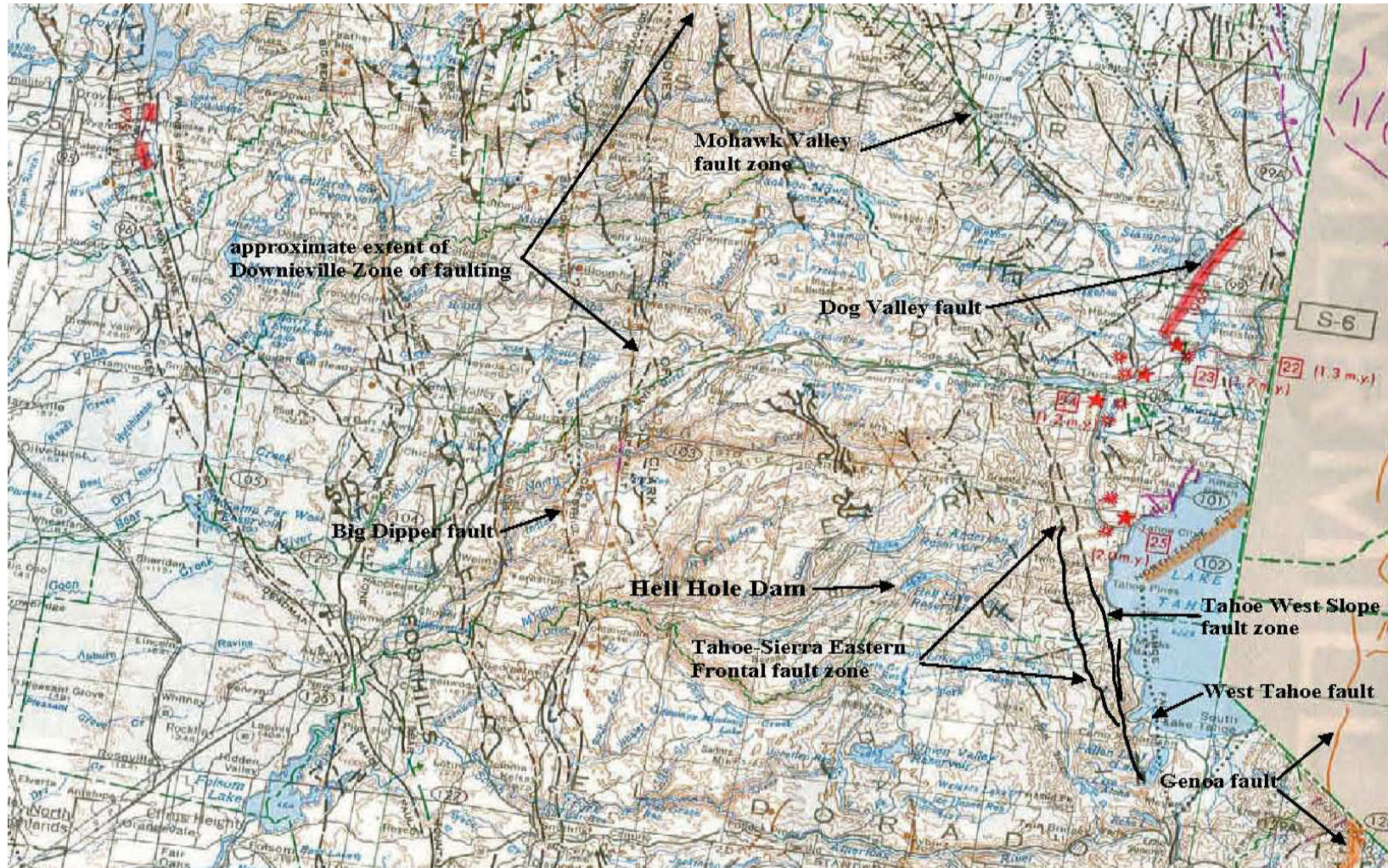
Potential seismogenic sources significant to Hell Hole Dam are listed in Table 7-1. Prior to the 2006 dam safety inspection and PFMA, the controlling maximum credible earthquake (MCE) with respect to ground motions at Hell Hole Dam was considered to be a postulated local earthquake of magnitude 5.0 at a distance of 5 km (the 1991 earthquake deformation analysis used a pga of 0.20g;). However, as indicated on Table 7-1, the recent SMUD and NID evaluations suggest that an earthquake along the Tahoe-Sierra Eastern Frontal fault zone would constitute the controlling MCE, and would produce higher peak ground accelerations at Hell Hole Dam than was previously considered.

As shown on Table 7-1, a pga of 0.35g at Hell Hole Dam (at 84th percentile ground motion) is estimated for the MCE along the Tahoe-Sierra Frontal fault zone. That fault zone, as previously discussed, appears to have a slip rate of about 1.5mm/yr with mainly normal east-side down displacement (Hamilton and Harlan, 2006). Use of the 84th percentile pga value for this fault is therefore considered appropriate given the relatively high slip rate of the faults in the east frontal/Tahoe basin region. No factors were applied for hanging wall or footwall effects in the estimation of pga presented in Table 7-1.

The Design Basis Earthquake (DBE) is a probability-based ground motion defined as having a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years. The California Geological Survey has developed maps depicting probabilistic ground motions for most of California. From the Probabilistic Seismic Hazards Ground Motion Page (California Geological Survey, 2008), the area of Hell Hole Dam is susceptible to peak horizontal ground accelerations of approximately .19 g with an estimated 10 percent probability of exceedance in 50 years.

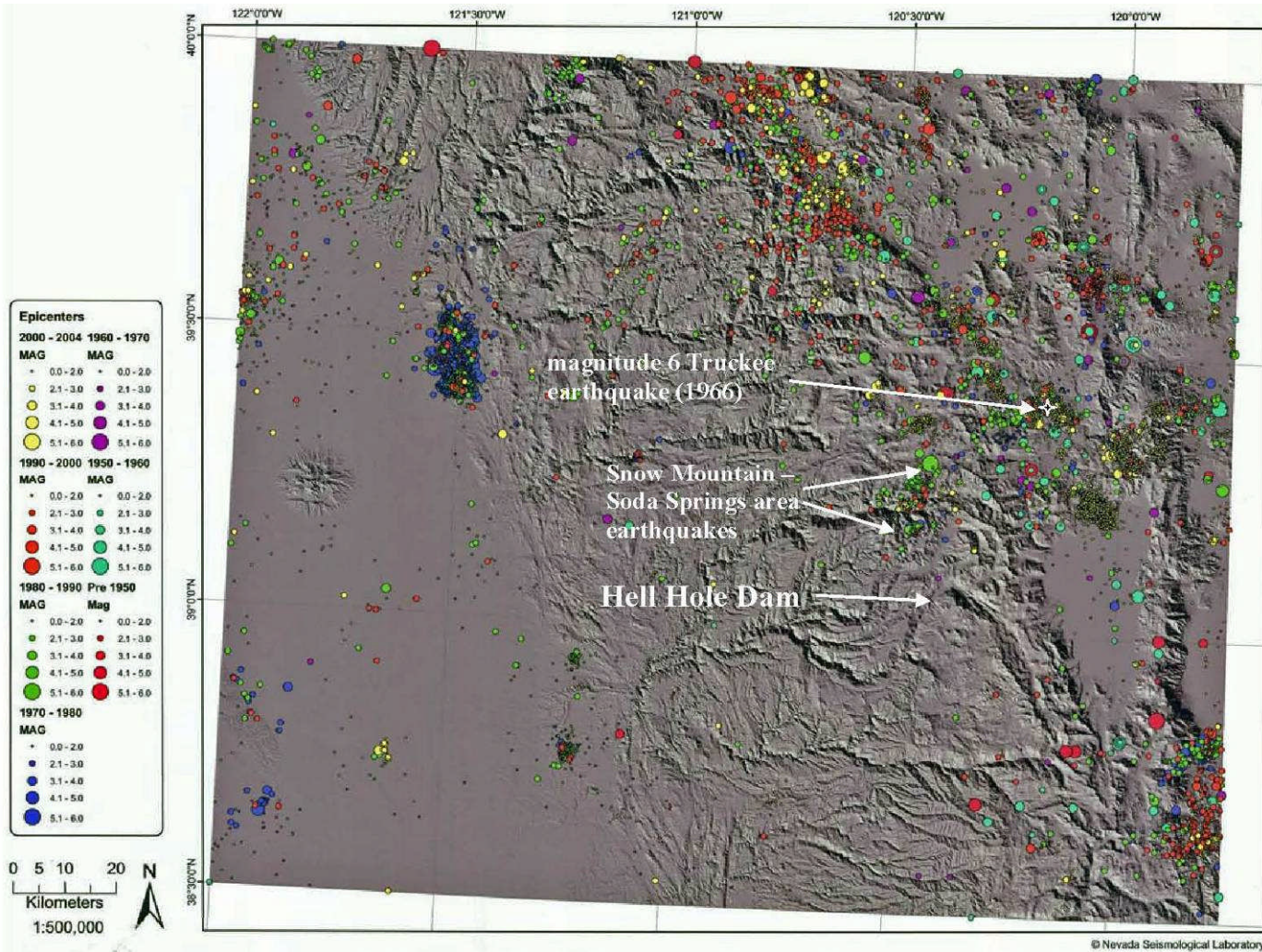
DRAFT

Figure 7-1 - Potential Earthquake Fault Sources in Region of Hell Hole Dam



Modified from: Fault Activity Map of California, 1994, State of California Division of Mines and Geology.

Figure 7-2 - Historic Earthquakes in Region of Hell Hole Dam



Modified from: Earthquakes in north-central region of Sierra Nevada, 1934 to 2005.
Reference: University of Nevada Reno Seismological Laboratory, 2005.

8. SPILLWAY DESIGN FLOOD ANALYSES

8.1 PURPOSE

Hydrologic and hydraulic analyses of the spillway were carried out to determine water surface profiles needed for structural stability analyses. The water surface profiles also provide information regarding the estimated freeboard between the water surface and the top of water retaining structures.

Water surface profiles are developed by routing flood inflow hydrographs developed in hydrologic analyses through the reservoir based on analysis of the hydraulic capacity of the spillway system over its range of operating conditions.

8.2 BACKGROUND

Hell Hole Dam has been determined to be a high hazard dam due to its size, reservoir capacity and likely consequences of dam failure. The spillway design flood is therefore the Probable Maximum Flood (PMF).

At the time the spillway was constructed in the 1960s the peak flow over the spillway during routing of the all-season PMF through the reservoir was determined to be 58,800 cfs. The corresponding maximum reservoir water surface elevation was E. 4643.2 Ft.

Since then the methodology and assumptions used to develop the Probable Maximum Precipitation (PMP) and corresponding PMF inflow hydrograph have evolved resulting in larger estimated flood inflow. In 2005, the All-Season PMP and resulting PMF inflow was reevaluated¹⁰. The routed peak discharge at the spillway was determined to be 98,600 cfs, a 68% increase in peak discharge. The corresponding maximum reservoir water surface elevation was estimated to be 4648.9 ft.

8.3 HYDROLOGIC ANALYSES

8.3.1 Watershed

The Hell Hole watershed is situated in the foothills and mountainous uplands of the western slope of the central Sierra Nevada mountain range, primarily within the Tahoe National Forest (TNF) and Eldorado National Forest (ENF).

The watershed that drains to Hell Hole Reservoir ranges in elevation from 4600 feet to over 9000 feet and has a plan area of 113.4 square miles. Most of the watershed is forested which becomes more sparse at higher altitudes. There are also upstream diversions that divert water out of the watershed to Sacramento Municipal Utility District's Upper American River Project and Georgetown Public Utility District's Stumpy Meadows Project.

¹⁰ Mead and Hunt July 2005.

Runoff from the watershed is stored in Hell Hole Reservoir and released for water supply hydropower generation and to meet minimum instream flow requirements.

8.3.2 Climate

The climate is characterized by hot, dry summers and mild, wet winters, with most of the precipitation falling between October and March. Precipitation falls as rain in the lower elevations and snow at elevations greater than about 5,000 feet. Watershed areas with elevations higher than about 6,000 feet are typically extensively covered by snow until May.

The summer climate in the Hell Hole drainage area is characterized as moderate, with dry days and cool nights. Localized intense rainfall can occur during thunderstorms which normally occur during summer.

Table 8-1 shows average temperatures and precipitation for the climate station at Georgetown Ranger Station in the Eldorado National Forest at an elevation of 3000 feet. The period of record for this station is 1946-2009.

**Table 8-1 - Summary of Climate Data at Georgetown Ranger Station
 (Period of record 1946-2009)**

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual Total
Average Maximum Temperature (F)	52.0	52.6	57.7	64.8	71.9	80.5	89.0	89.3	83.5	73.0	59.2	51.4	68.7
Average Minimum Temperature (F)	35.2	34.9	37.1	41.4	46.9	54.2	60.9	60.3	55.8	48.3	39.7	34.7	45.8
Average Total Precipitation (in)	9.67	7.80	7.36	4.21	1.80	0.68	0.12	0.20	0.74	2.84	6.58	8.37	50.39
Average Total Snowfall (in)	5.2	2.8	2.8	1.4	0.1	0.0	0.0	0.0	0.0	0.0	0.5	2.5	15.3
Average Snow Depth (in)	1	1	0	0	0	0	0	0	0	0	0	0	0

Western Regional Climate Center, <http://www.wrcc.dri.edu/cgi-bin/cliMAIN.pl?ca3384>

Snow depth and water content measurements are made at two snow courses in the upper elevations of the watershed. Snow Course No. 369 Miranda Cabin is located at 6200 feet and Snow Course No. 371 Diamond Crossing is located at 6050 feet. Measurements are made monthly in February through May and sometimes in January and June as well. Table 8-2 shows average snow depth and water content for these two snow courses. The period of record for the data is 1967 through 2008.

Table 8-2 - Average Snow Depth and Water Content (in)

	Jan	Feb	Mar	Apr	May
Miranda Cabin Snow Depth	44.3	61.5	85.0	85.4	59.7
Miranda Cabin Water Content	14.7	21.8	33.1	37.7	29.4
Diamond Crossing Snow Depth	29.2	44.4	55.7	50.7	25.2
Diamond Crossing Water Content	8.7	14.6	20.3	21.0	11.5

8.3.3 Floods

Historically, outflow from Hell Hole Reservoir has been measured at USGS Gage 1142880, Rubicon River below Hell Hole Dam. The water-stage recorder and V-notched, sharp crested weir are located on the right bank about 600 feet downstream from the outlet of the dam.

The period of record for this gage is from November 1965 to the present. Measurement at the gage is likely only calibrated for low flows. It is our understanding that high flows are estimated using the measured reservoir elevation and the existing spillway rating curve. The maximum discharge through the spillway was estimated to be 28,800 cfs on January 2, 1997 based upon a measured water surface elevation at the reservoir of 4637.7 feet.

The current spillway design flood is the all-season Probable Maximum Flood (PMF) determined in a studies prepared by Mead and Hunt in 2004 and 2005.

8.3.3.1 All Season PMF

The Probable Maximum Precipitation (PMP) was computed according to methodology published in Hydrometeorological Report 58 (HMR No. 58). The PMF was routed using the Army Corps of Engineers HEC-1 program to compute the inflow and outflow resulting from the PMP. Hydrologic conditions present in November produce the largest PMF. In these previous flood routing studies, the recomputed all-season PMF peak outflow from Hell Hole Reservoir is 98,600 cfs. The associated peak reservoir stage is 4648.9 feet, relative to the nominal dam crest elevation of El. 4630 ft. and nominal top of core elevation of El. 4625 ft. (The lowest measured elevation on the crest of the dam appears to be (El 4649.62 at BM 2A on the right abutment).

The reduction in dam freeboard due to the increase in magnitude of the Probable Maximum Flood is an existing issue addressed in the existing STID and in correspondence with regulatory agencies. Since this change in minimum dam freeboard is not related to the present proposal to increase seasonal storage it is not addressed in this report. It is assumed that this issue will be addressed separately regardless of whether the seasonal storage project is implemented.

8.3.3.2 Local storm PMF

HMR No. 58 also provides procedures for computing the PMP associated with a localized thunderstorm. Mead and Hunt delineated two distinct watershed areas contributing to inflow

at Hell Hole Reservoir: Upper Hell Hole at 41.9 square miles, draining Five Lakes Creek, and Lower Hell Hole at 70.8 square mile, draining the Rubicon River. To estimate the local storm PMF, a six-hour thunderstorm of magnitude calculated according to procedures set forth in HMR No. 58 was assumed to occur in the entire watershed area.

A local storm in the drainage area contributing to Hell Hole Reservoir as described above is expected to produce a peak inflow of 29,000 cfs approximately eight hours after rainfall begins.

8.4 HYDRAULIC ANALYSES

8.4.1 Purpose of Analysis

The purpose of the analysis was to determine water surface profiles for flood flow through the spillway channel for stability analysis of the spillway weir including the proposed crest gates.

To provide a basis for design of spillway modifications, the hydraulic capacity and rating curve of the existing spillway were reevaluated using the Hydrologic Engineering Center's River Analysis System (HEC-RAS) computer model using the updated topographical survey data.

8.4.2 Existing Spillway

The existing spillway is generally described in Section 2.4. In order to evaluate water surface profiles over the spillway using a mathematical hydraulic model a detailed survey of the spillway, including the approach channel, concrete ogee section, bridge details and downstream section was completed by Andregg Geomatics in November 2009. This survey is shown in Figure 8-2.

8.4.3 Spillway Rating Curve

Previous reservoir routing analyses of inflow design floods utilized a rating curve for the spillway crest based on USBR model studies of irregular spillway crests. In this approach flow over the spillway crest is computed as a function of upstream water level, however, the coefficient of discharge for the spillway crest is reduced to account for sub-optimal approach and discharge conditions over the spillway weir that reduce flow over its crest. The adjustment factors are based on laboratory and field test conducted by the USBR. In the present study the approach and discharge conditions are addressed in a hydraulic model rather than by adjusting the spillway crest discharge coefficients.

8.4.4 Evaluation Procedure

Spillway hydraulics were analyzed using the HEC-RAS (USACE Hydrologic Engineering Center, River Analysis System), a mathematical model designed to perform one-dimensional hydraulic calculations for natural and constructed channels.

The model is capable of simulating sub-critical, supercritical and mixed flow regimes to develop water surface profiles, estimate flow velocities and estimate channel discharge. The HEC-RAS model uses Manning's empirical relationship between channel geometry, slope, surface roughness and fluid velocity. HEC-RAS allows for fluctuations in the weir coefficient to account for upstream energy heads that are either higher or lower than the design head, according to Figure 250 (USBR, 1977). The program also accounts for the effect of high tailwater elevations on flow over the ogee.

The evaluation process was conducted as follows:

1. A computational model of the spillway channel was built using HEC-RAS to evaluate its hydraulic capacity. Updated topographical data was used to develop cross-sections to represent the channel geometry in the model.
2. Weir geometry was input using detailed as-built drawings of the cross-sectional weir shape.
3. Bridge geometry for model input was determined from survey data in conjunction with photographs of the bridge.
4. Surface roughness coefficients (expressed as Manning's n) were estimated from experience data for similar channels in rock.
5. A range of flows from 5000 cfs to the all-season PMF flow of 98,600 cfs was evaluated using HEC-RAS to develop an updated rating curve.
6. Sensitivity analyses were carried out to evaluate the sensitivity of the results to Manning's roughness coefficient n .

8.4.5 Results of Evaluation

The results of the analysis are shown on the rating curve shown on Figure 8-1. The figure shows a comparison between the present and previous estimates of the relationship between water surface elevation in the reservoir and gravity flow down the spillway.

The data show that the hydraulic capacity of the spillway according to the new rating curve is slightly lower than the existing curve. For example, at El. 4634, the previous rating curve indicates a flow of 10,000 cfs while the updated rating curve shows a flow of about 8,600 cfs at the same elevation. This difference increases with increase in water surface elevation.

At high reservoir stages, the spillway ogee operates as a broad-crested weir with high tailwater elevation at the upstream end of the discharge channel. This condition appears to reduce the hydraulic capacity of the spillway.

8.4.6 Updated Routing of All Season PMF

The routing of the All-Season PMP through the reservoir with the modified spillway rating curve resulted in a reduced peak outflow 95,400 cfs and a corresponding peak water surface elevation in Hell Hole Reservoir of 4649.3 ft. This is 0.9 ft higher than previously determined in the 2005 PMF update. This results in minimum dam freeboard (excluding wave run-up) about 3.5 inches below the lowest point on the existing dam crest (El. 4649.62 at BM. 2A). We note that the top of the existing impervious core of the dam is shown on drawings to be 5 feet below the crest elevation which means that the reservoir water surface would be some 4-5 feet above the top of the impervious core.

The issue of minimum freeboard of Hell Hole Dam is addressed in the PFMA and STID and in correspondence between PCWA and FERC. Since this is an existing issue that is directly related to the increase in the magnitude of the inflow design flood and is not related to the proposal to add spillway crest gates - it is not addressed further in this study.

8.4.7 Routing of Local Storm PMF

Consistent with California Division of Safety of Dams (DSOD) policies, the Local Storm PMP was routed through Hell Hole Reservoir with the designed six-foot gates in the closed (or raised position).

The conservative assumption is made that the gates remain closed during the local storm PMF flood due to potential causes such as failure of the operating system.

The resulting outflow over the gates in the raised position is 21,600 cfs with a corresponding peak water surface elevation over the raised gates of 4643.0 ft eleven hours after onset of the local storm PMF. The surcharge over the closed gates resulting from the local storm PMF is 7 ft. A surcharge of 7.0 ft represents the most extreme potential for overtopping after the onset of the local storm PMP taking into account only upstream conditions.

8.4.8 Tailwater Determination

The same HEC-RAS geometry of the channel bottom that was used to update the spillway rating curve was used to determine tailwater elevation just downstream of the concrete ogee section with the spillway gates raised resulting from an outflow of 21,600 cfs.

Figure 8-5 shows a profile of the spillway channel bottom and the resulting equilibrium water surface profile at a flow 21, 600 cfs. A computed tailwater elevation of 4632.9 ft under these conditions was used in stability analyses.

8.5 REFERENCES

1. FERC Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter II, Selecting and Accommodating Inflow Design Floods for Dams, October 1993.
2. FEMA, Federal Guidelines for Dam Safety - Selecting and Accommodating Inflow Design Floods for Dams, October 1998.
3. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Department of the Army, Corps of Engineers, Hydrometeorological Report No. 58, Probable Maximum Precipitation for California - Calculation Procedures, Silver Spring, MD, October 1998
4. Mead and Hunt, 2004, *Probable Maximum Flood Determination, Hell Hole, Interbay, Ralston Afterbay Dams, Middle Fork American River Project*, for PCWA.
5. Mead and Hunt, 2005, *Addendum to the May 2004 Probable Maximum Flood Determination, Hell Hole, Interbay, and Ralston Afterbay Dams*, for PCWA. Mead and Hunt.
6. McCreary Koretsky Engineers, 04/17/1967, Hell Hole Spillway, Plan and Sections. Drawing No. 61-07-201A

7. S&E Engineering, 10/20/2004, Topographic Mapping, Hell Hole Dam and Upper Spillway. Middle Fork American River Project - 2 sheets.
8. Andregg Geomatics 11/2009 Topographic mapping of Spillway including approach channel and discharge channel. Contour and DTM point data files provided.
9. USBR, *Design of Small Dams*, 1977

DRAFT

Figure 8-1 - Hell Hole Spillway Rating Curve Comparison

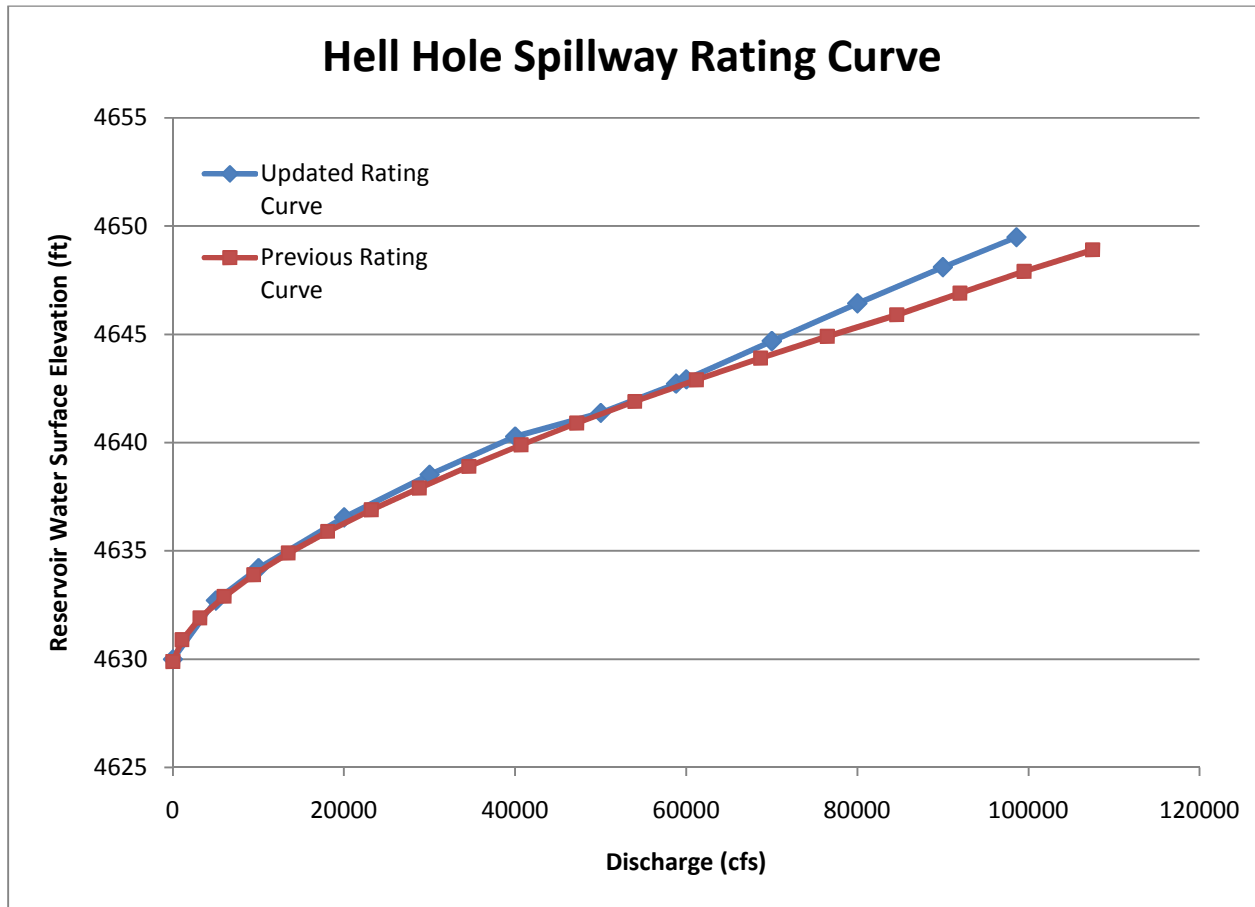


Figure 8-2 - Detailed Topography of Hell Hole Spillway

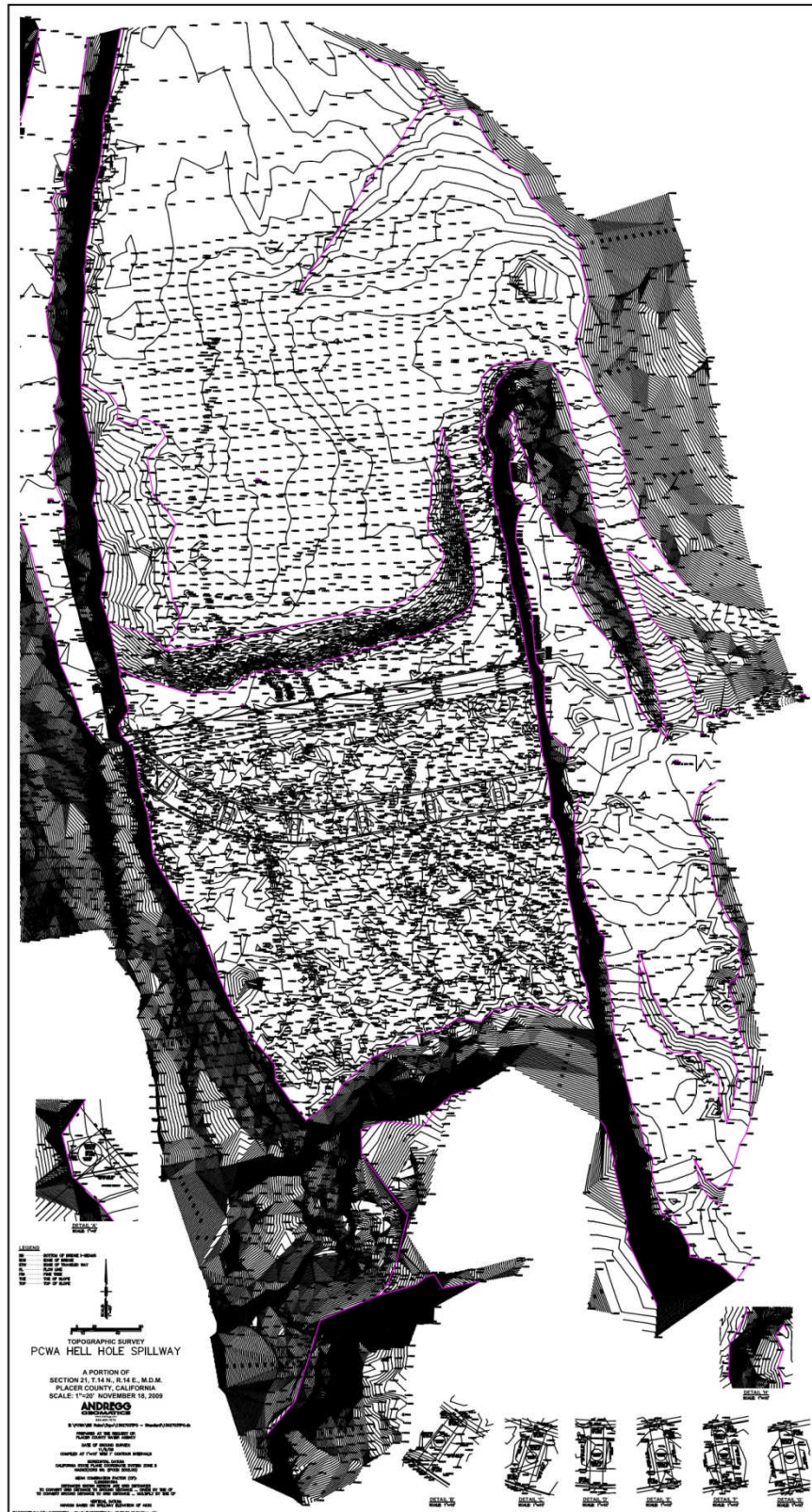


Figure 8-3 - Hell Hole Dam Abutments & Upper Spillway

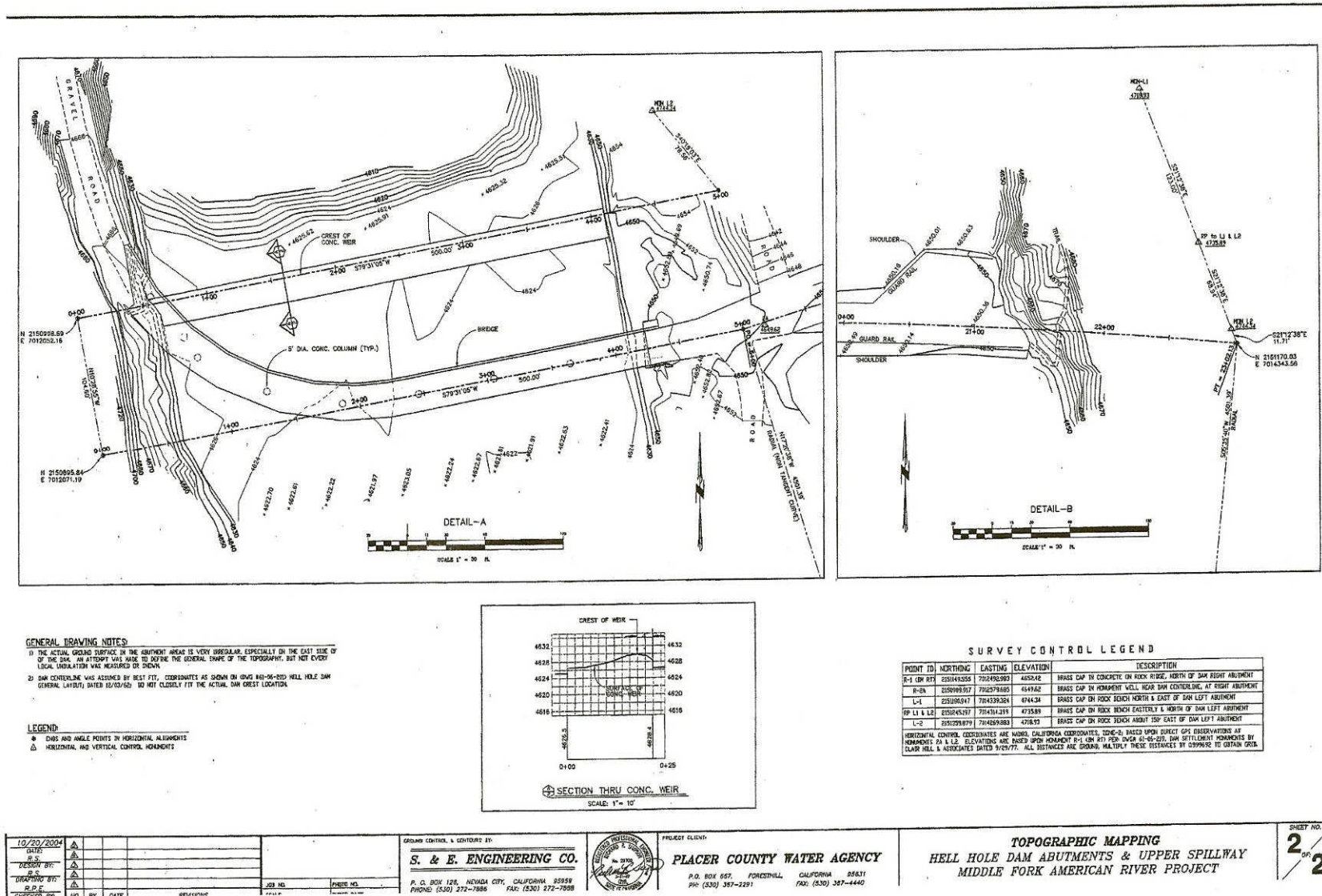


Figure 8-4 - Spillway Ogee Section

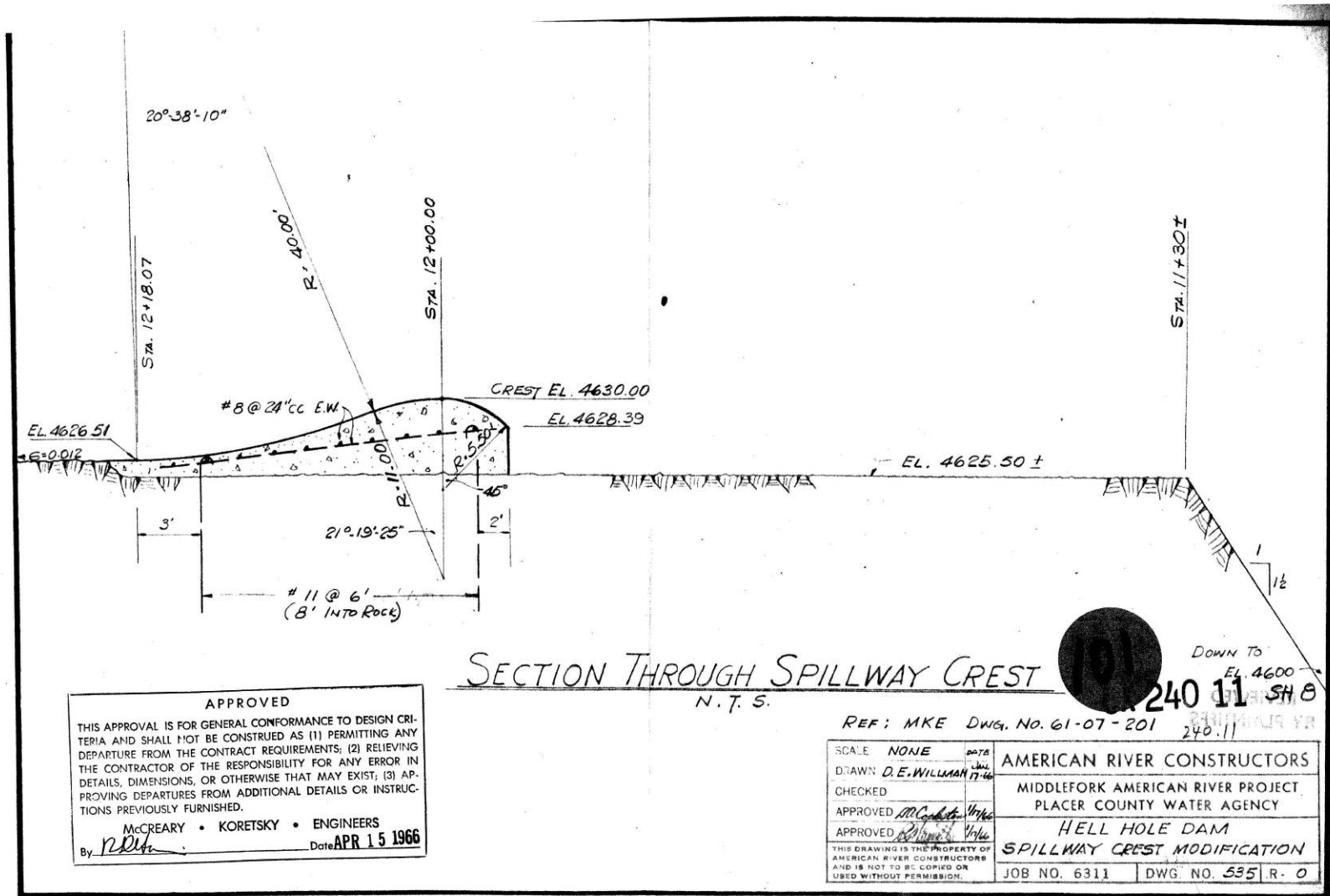


Figure 8-5 - Water Surface Profile of Spillway with 21,600 cfs flow

