The Gravity Method of Stress and Stability Analysis

A- 1. ***Example of Gravity Analysis- Frian t Dam.*** -The example presented in this appendix was taken from the gravity analysis of the revised Friant Dam. Friant Dam was constructed during the period 1939 to 1942 and is located in the Central Valley of California. A plan, elevation, and sections of the dam are shown on figure A- 1.

The assumptions and constants used for the analysis are given below:

(1) Unit weight of water = 62.5 pounds per cubic foot.

(2) Unit weight of concrete = 150 pounds per cubic foot.

(3) Unit shear resistance of both concrete and rock = 450 pounds per square inch.

(4) Coefficient of internal friction of concrete, or of concrete on rock = 0.65.

(5) Weight of l&foot drumgate = 5,000 pounds per linear foot.

(6) Top of nonoverflow section, elevation 582.

(7) Crest of spillway section, elevation 560.

(8) Normal reservoir water surface, elevation 578.

(9) Tailwater surface, elevation 305.

(10) Horizontal component of assumed earthquake has an acceleration of 0.1 gravity, a period of vibration of 1 second, and a direction which is at right angles to axis of dam.

Note. Figure A-2 is a graph showing values of the coefficient K_E, which was used to determine hydrodynamic effects for the example given. However, this procedure is not consistent with current practice. A discussion of the coefficient C_m , which is presently used to determine hydrodynamic pressures, is given in sec tion 4-3 4.

(1 1) Vertical component of assumed earthquake shock has an acceleration of 0.1 gravity and a period of 1 second.

(12) For combined effects, horizontal and vertical accelerations are assumed to occur simultaneously.

(13) Uplift pressure on the base or on any horizontal section varies from full-reservoir pressure at the upstream face to zero, or tailwater pressure, at the downstream face, and is considered to act over two-thirds the area of the section. Uplift is assumed to be unaffected by earthquake shock, and to have no effect on stresses in the interior of the dam.

Note. This uplift assumption is no longer used by the Bureau of Reclamation. See section 3-9 for uplift assumptions now in use.

A-2. ***List of Conditions Studied.*** -A list of conditions studied for Friant Dam for both the nonoverflow and the overflow section is tabulated below:

(1) Reservoir empty.
(2) Reservoir full.
(3) Reservoir empty plus earthquake.
(4) Reservoir full plus earthquake.

Loads for reservoir empty are dead loads consisting of the weight of the dam and gates. Loads for full-reservoir operation include, in addition to dead loads, the vertical and horizontal components of normal waterloads on the faces of the dam.

Loads for earthquake effects with reservoir empty include inertia forces caused by acceleration of the mass of dead loads. Loads for earthquake effects with reservoir full include, in addition to the above, the inertia force of the mass of water and the hydrodynamic force caused by the movement

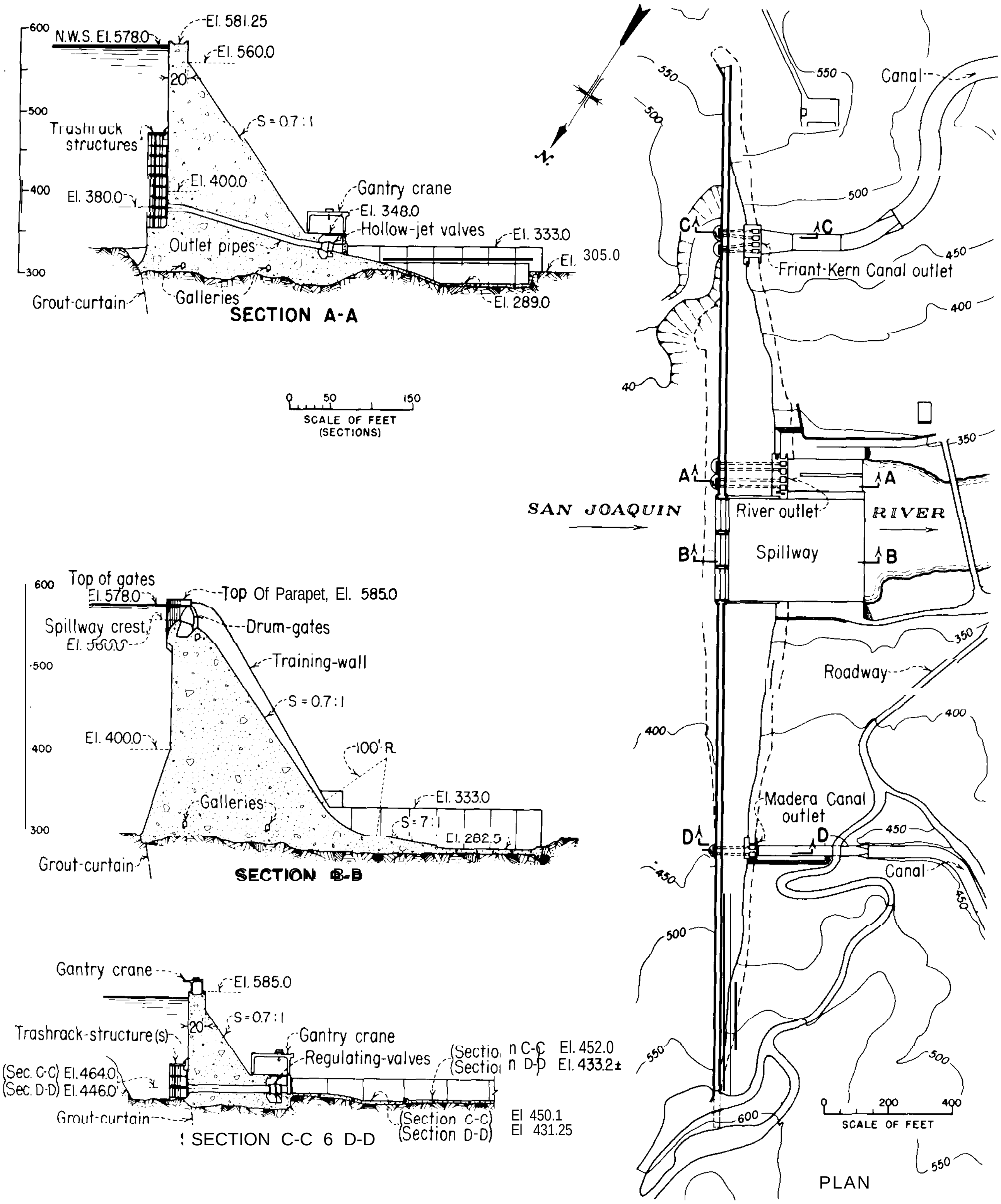

Figure A-1. Friant Dam—plan and sections. -288-D-3156

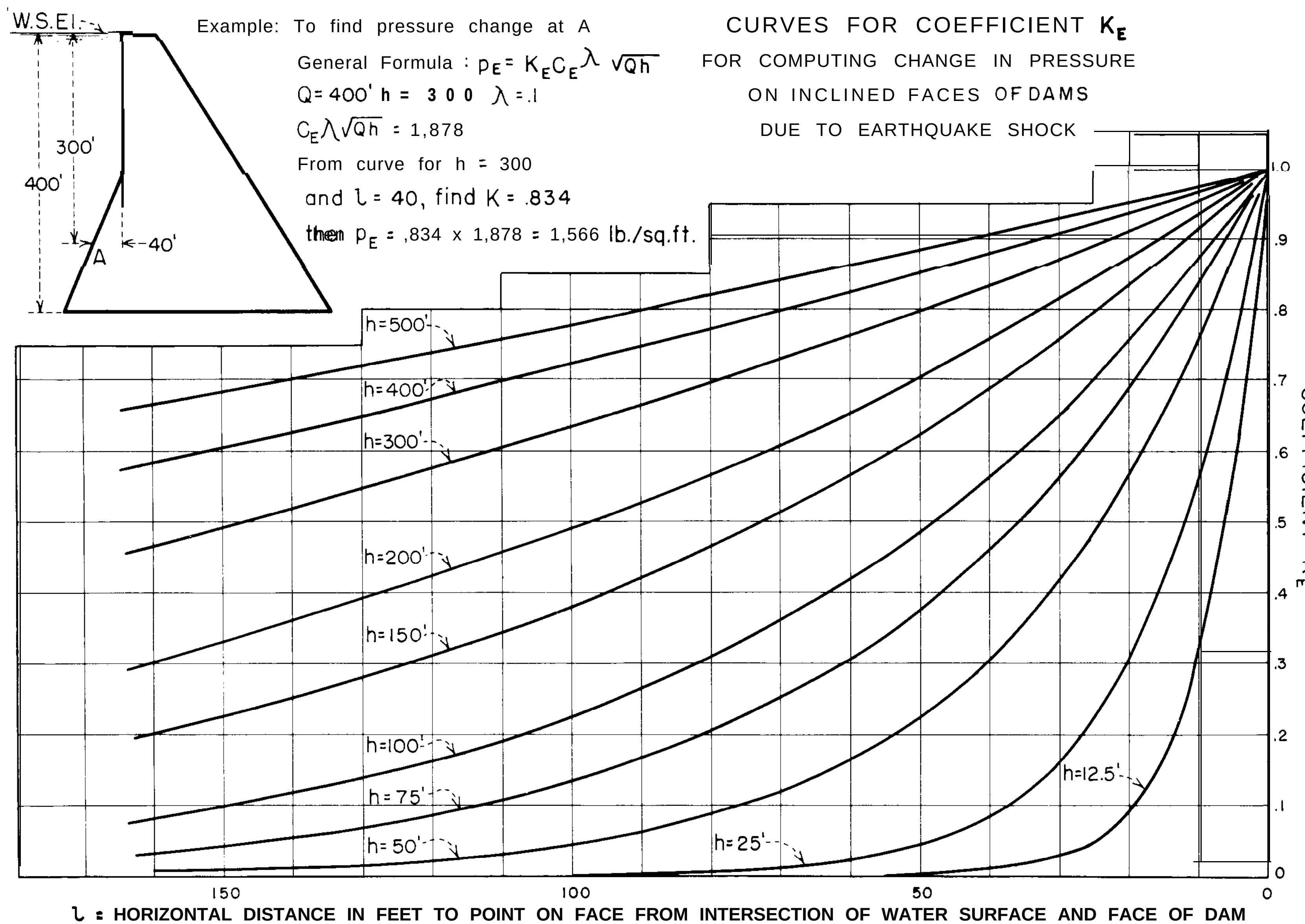

Figure A-2. Curves for coefficient K_E for computing change in pressure due to earthquake shock. -288-D-3157 of the dam against the water of the reservoir. Uplift forces are assumed to be unaffected by earthquake shocks.

The effects of earthquake were studied for each of the following directions of the acceleration:

(1) Horizontal upstream.

(2) Horizontal downstream.

(3) Vertical upward.

(4) Vertical downward.

(5) Horizontal upstream plus vertical upward.

(6) Horizontal upstream plus vertical downward.

(7) Horizontal downstream plus vertical up ward.

(8) Horizontal downstream plus vertical downward.

A-3. ***Computations and Forms.*** -Computations for the gravity analysis of the nonoverflow section of Friant Dam are shown as figures A-3 to A-9, inclusive. These are for reservoir-full conditions with earthquake accelerations upstream and upward. Equations used are shown at the top of the forms. Standard forms are used.

A-4. ***Final Results.*** -Final results are given on figures A-l 0 to A- 18, inclusive, which show normal and shear stresses, stability factors, and principal stresses for each loading condition on the overflow and nonoverflow sections.

A-5. ***Summary and Conclusions.*** -Following is a summary of results and conclusions obtained from the gravity analysis of Friant Dam. These are presented for the purpose of showing the type of information usually obtained from such an analysis.

(1) The analyses of the maximum nonoverflow and spillway sections of Friant Dam indicate stresses and stability factors within safe limits for all loading conditions.

(2) The maximum compressive stress, maximum horizontal shear stress, and minimum shear-friction factor all occur for normal full-reservoir operation during earthquake accelerations "horizontal upstream" and "vertical upward."

(3) The maximum tensile stress occurs for reservoir-empty conditions combined with earthquake acceleration "horizontal downstream" acting alone or in conjunction with earthquake acceleration "vertical upward."

(4) The maximum sliding factor occurs for normal full-reservoir conditions combined with earthquake accelerations "horizontal upstream" and "vertical downward."

(5) Points of application of resultant forces on the bases and horizontal sections of the nonoverflow and spillway sections are well within the middle-third for most loading conditions.

(6) Maximum stresses occur at the downstream face of the maximum nonoverflow section; the maximum compressive and shear stresses occur at the base elevation, and the maximum tensile stress occurs at elevation 400. Maximum direct stresses all act parallel to the face.

(7) The maximum sliding factor occurs at elevation 400 and the minimum shear-friction factor occurs at the base elevation of the nonoverflow section.

(8) The maximum compressive stress is 409 pounds per square inch and the maximum tensile stress is 46 pounds per square inch.

(9) The maximum horizontal shear stress is 192 pounds per square inch. The maximum sliding factor is 0.999, and the minimum shear-friction factor is 5.45.

(10) Since tensile stresses occur at points not subjected to water pressure, the possibility of uplift forces acting in tension cracks is eliminated.

Complete results for nonoverflow and spillway sections are tabulated in table A-l.

FRIANT DAM NONOVERFLOW SECTION STUDY No. 3

GRAVITY STRESS ANALYSIS OF MAXIMUM PARALLEL-SIDE CANTILEVER

(NOTE: Origin at downstream face) VALUES AND POWERS OF **y** By J.T.R. Date 1-30-40

ELEV.		y, y^2, and y^3 (Feet)												
		VERTICAL PLANE												
		U.S.	6	5	4	3	2	1	D.S.					
	y													
	y^2													
	y^3													
	y	27.3		7.3					0					
550	y^2	745.29		53.29										
	y^3	20,346,417		389.017										
	y	62.3		42.3	2.3				0					
500	y^2	3,881.29		1,789.29	5.29									
	y^3	241,804.37		75,686.97	12.167									
	y	97.3		77.3	37.3				0					
450	y^2	9,467.29		5,975.29	1,391.29									
	y^3	921,167.32		461,889.92	51,895.117									
	Y	132.3		112.3	72.3	32.3			0					
400	y^2	17,503.29		12,611.29	5,227.29	(043.29								
	y^3	2,315,685.27		1,416,247.87	377,933.07	33,698.267								
	Y	182 3		147 3	107.3	67.3	2 7 3		0					
350	y^2	33,233.29		21,697.29	11,513.29	4,529.29	745 29							
	y^3	6,058,428.77		3,196,010.82	1,235,376.02	304,821.22	20,346.417							
	y	217.3	211 8	171.8	131.8	91.8	51 8	11.8	0					
315	y^2	47,219.29	44,859.24	29,515.24	17,371.24	8,427.24	2,683.24	139.24						
	y^3	10,260,752	9,501,187	5,070,7182	2,289,529.4	773,620.63	138,991.83	1,643.032						
	y													
	y^2													
	y^3													
	y													
	y^2													
	y^3													
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Figure A-3. **Friant Dam study-values and powers of *y*. –DS2-2(6)**

FRIANT DAM NONOVERFLOW SECTION. RESERVOIR W.S. EL. 578 ... TAILWATER EL. NONE .. STUDY No. 3 .....

GRAVITY STRESS ANALYSIS OF MAXIMUM PARALLEL-SIDE CANTILEVER

INCLUDING EFFECTS OF TAILWATER AND HORIZONTAL EARTHQUAKE

EARTHQUAKE ←

NORMAL STRESS ON HORIZONTAL PLANES $\sigma_z = a + by$ By J.T.R. Date 2-5-40

NOTE: ω_c = .150 ..

ω = .434.03

$a = \sigma_{ZD} = \frac{1}{T}(\Sigma W) - \frac{6}{T^2}(\Sigma M)$ $b = \frac{12}{T^3}(\Sigma M)$ Check: (for y = T), $\sigma_{ZU} = \frac{1}{T}(\Sigma W) + \frac{6}{T^2}(\Sigma M)$

ELEV.	T	$\frac{1}{T}$	$\frac{6}{T^2}$	$\frac{12}{T^3}$	ΣW	ΣM	b	σ_z Pounds per Square Foot — VERTICAL PLANE: us.	6	5	4	3	2	1	D.S	σ_{ZU}
				Reservoir	Full											
550		.036,630,036	.008,050,557,2	0.589,784,4	111,880	-170,700	100.676,20	2,708.4	4,733 5						5438	2,723.94
500		.016,051,364	.001,545,877,7	0.049,626,893	481,490	2,947,200	146.260,38	3, 122	56	6,042	11,833 2				12,285	3,172.56
450		.010,277,492	0.633,761,04	0.013,026,948	1,139,800	14,328,000	186.650,11	2,618 75	6,344 8	13,896 8					20,144	2,633 76
400		.007,258,579,0	0.342,792,70	0.005,182,051,3	2,086,900	39,701,000	205.732,62	2,115	9244 8		14,101 7	22,758.0			29,204	2,164 78
350		.005,485,463,5	0.180,541,86	0.001,980,711,6	3,594,000	86,900,000	172.123,84	4,028	66	10,070	16,0[illegible]	[illegible]8.065.0	30,213.9		35,246	4,025 67
315		.004,601,932,8	0.127,066,71	0.001,169,450	4,925,000	132,780,000	155,286,87	5,740.60	6,646 7	12,889.2	19,132.6	25.176,1	31,219	37,704 1	39,275	5,792.60

Figure A-4. **Friant Dam study-normal stresses on horizontal planes. —DS2-2(7)**

FRIANT DAM NONOVERFLOW SECTION. RESERVOIR W. S. EL. 578 TAILWATER EL. NONE STUDY No. ?

GRAVITY STRESS ANALYSIS OF MAXIMUM PARALLEL-SIDE CANTILEVER

INCLUDING EFFECTS OF TAILWATER AND HORIZONTAL EARTHQUAKE

EARTHQUAKE ←

SHEAR STRESS ON HORIZONTAL AND VERTICAL PLANES $\tau_{zy} = \tau_{yz} = a_1 + b_1 y + c_1 y^2$ By J. T. R. Date 2-6-40

$\tau_{zyU} = -(\sigma_{zU} - p \pm p_E)\tan\phi_U$ $a_1 = \tau_{zyD} = (\sigma_{zD} - p' \pm p'_E)\tan\phi_D$ $b_1 = -\left[\frac{6}{T^2}(\Sigma V) + \frac{2}{T}(\tau_{zyU}) + \frac{4}{T}(\tau_{zyD})\right]$ $c_1 = \frac{6}{T^3}(\Sigma V) + \frac{3}{T^2}(\tau_{zyU}) + \frac{3}{T^2}(\tau_{zyD})$

(‡ Use (+) sign if horizontal earthquake acceleration is upstream.) (‡ Use (−) sign if horizontal earthquake acceleration is upstream.)

$C = \dfrac{51.0 \text{ lbs./cu.ft.}}{\sqrt{1 - 0.72\left(\frac{Q \cdot Sec}{1000 \cdot N \cdot ft}\right)^2}} =$ $p_E = CK\lambda\sqrt{Qh}$ Q =

$C' = \dfrac{51.0 \text{ lbs./cu.ft.}}{\sqrt{1 - 0.72\left(\frac{Q' \cdot Sec}{1000 \cdot N \cdot ft}\right)^2}} =$ $p'_E = C'K'\lambda\sqrt{Q'h'}$ Q' =

ELEV.	$\frac{2}{T}$	$\frac{4}{T}$	$\frac{3}{T^2}$	$\frac{6}{T^2}$	$\frac{6}{T^3}$	ΣV	$p = \omega h$	p_E	K	$\tan\phi_U$	$p' = \omega h'$	p'_E	K'	$\tan\phi_D$	b_1	c_1	$\tau_{zy} = \tau_{yz}$ Pounds per Sqare Foot — Vertical Plane U.S.	6	5	4	3	2	1	D.S.	τ_{zyu}
				Reservoir Full																					
550	.073260072	.140,520,14	.004,025,786	.008,058,372	$\overset{3}{0}$294892,7	-45,502	1,925			0				.70	-194.955,32	+2,001,369,3	0		1,714.2						0
500	.032,102,728	.064245,456	$\overset{3}{0}$772,95887	.001,545,878	$\overset{3}{0}$024,813,9	-291,880	5,362.5			0				.70	-100.905,42	-595,884,6	0		3,264.7	8,364.0					0
450	.020,534,384	.041,109,968	$\overset{3}{0}$316,88952	$\overset{3}{0}$633,76104	$\overset{3}{0}$106,573,7	-748,720	8,800			0				.70	-123.902,34	-264,138,16	0		3,400.4	9,562.3					0
400	.015,117,158	.030,234,316	$\overset{3}{0}$171,396,30	$\overset{3}{0}$342,793	$\overset{3}{0}$002,591,0	-1,413,200	12,238			0				.70	-137 432,33	-136,314,55	0		3,415.5	9,919 3	15,111				0
350	.010,970,927	.021,941,854	$\overset{3}{0}$090270,90	$\overset{3}{0}$180,542	$\overset{6}{0}$990,356,7	$\bar{2}$,289,400	15,675			.30				.70	-168.786,83	+285,314,75	3,424.80		6,110.9	9,969.8	14,102	20,142		24,172	3,494.8
315	.009,203,856	.018,407,731	$\overset{3}{0}$063,533,57	$\overset{3}{0}$127,067	$\overset{6}{0}$584,752,5	$\bar{3}$,031,500	18,081			.30				.70	-158.170,88	+219,857,91	3626.51	4238.6	6,949.9	10,675	15,194	20,139	25,179	27,192	3,686.5

Note: Values for p_E and K were computed separately and are not shown.

Figure A-5. **Friant** Dam study-shear stresses on horizontal and vertical planes. —DS2-2(8)

FRIANT DAM NONOVERFLOW SECTION. RESERVOIR W.S. EL. 578 TAILWATER EL. NONE STUDY No. 3

ω_c = 150
ω = 434,03

GRAVITY STRESS ANALYSIS OF MAXIMUM PARALLEL-SIDE CANTILEVER

INCLUDING EFFECTS OF TAILWATER AND HORIZONTAL EARTHQUAKE

EARTHQUAKE ← (ACCEL)

PARTIAL DERIVATIVES FOR OBTAINING σ_y

By. H. P. W. Date 2-19-40

$K_1=\frac{4}{T}p-\frac{4}{T^2}\Sigma W\pm^{\ddagger}\frac{4}{T}p_E-\frac{12}{T^3}\Sigma M$ | $K_2=\frac{2}{T^2}\Sigma W\pm^{\ddagger}\frac{2}{T}p'_E-\frac{2}{T}p'-\frac{12}{T^3}\Sigma M$ | $\frac{\partial\sigma_{zu}}{\partial z}=\omega_c+K_1\tan\phi_u+K_2\tan\phi_D+\frac{6}{T^2}\Sigma V$ | $K_3=\frac{12}{T^3}\Sigma M+\frac{2}{T^2}\Sigma W-\frac{2}{T}p\pm^{\ddagger}\frac{2}{T}p_E$ | $K_4=\frac{12}{T^3}\Sigma M-\frac{4}{T^2}\Sigma W+\frac{4}{T}p'\pm^{\ddagger}\frac{4}{T}p'_E$ | $\frac{\partial\sigma_{zD}}{\partial z}=\omega_c+K_3\tan\phi_u+K_4\tan\phi_D-\frac{6}{T^2}\Sigma V$

($\ddagger$ Use (+) sign if horizontal earthquake acceleration is upstream) ($\ddagger$ Use (−) sign if horizontal earthquake acceleration is upstream) ($^{\otimes}\omega$ -omitted if water on face is absent)

$\frac{\partial\tau_{zyu}}{\partial z}=\tan\phi_u\left(\omega^{\otimes}-\frac{\partial\sigma_{zu}}{\partial z}\pm^{\ddagger}\frac{\partial p_E}{\partial z}\right)+\frac{\partial\tan\phi_u}{\partial z}\left(p-\sigma_{zu}\pm^{\ddagger}p_E\right)$ | $\frac{\partial\tau_{zyD}}{\partial z}=\tan\phi_D\left(\frac{\partial\sigma_{zD}}{\partial z}-\omega^{\otimes}\pm^{\ddagger}\frac{\partial p'_E}{\partial z}\right)+\frac{\partial\tan\phi_D}{\partial z}\left(\sigma_{zD}-p'\pm^{\ddagger}p'_E\right)$

$\frac{\partial\tan\phi_u}{\partial z}=\frac{\Delta\tan\phi_u}{\Delta z}$ | $\frac{\partial\tan\phi_D}{\partial z}=\frac{\Delta\tan\phi_D}{\Delta z}$ | $\frac{\partial p_E}{\partial z}=\frac{p_E}{\Delta z}$ | $\frac{\partial p'_E}{\partial z}=\frac{\Delta p'_E}{\Delta z}$ | $\frac{\partial\Sigma V}{\partial z}=-\left(p-p'\pm^{\ddagger}\lambda\omega_c T\pm^{\ddagger}p_E\pm^{\ddagger}p'_E\right)$ | $\frac{\partial T}{\partial z}=\tan\phi_u+\tan\phi_D$

ELEV.	$\frac{2}{T}$	$\frac{4}{T}$	$\frac{2}{T^2}$	$\frac{4}{T^2}$	$\frac{6}{T^2}$	$\frac{12}{T^3}$	$\frac{18}{T^4}$	K_1	K_2	$\frac{\partial\sigma_{zu}}{\partial z}$	K_3	K_4	$\frac{\partial\sigma_{zD}}{\partial z}$	$\frac{\partial p_E}{\partial z}$	$\frac{\partial\tan\phi_u}{\partial z}$	$\frac{\partial\tau_{zyu}}{\partial z}$	$\frac{\partial p'_E}{\partial z}$	$\frac{\partial\tan\phi_D}{\partial z}$	$\frac{\partial\tau_{zyD}}{\partial z}$	$\frac{\partial T}{\partial z}$	$\lambda\omega_c T$	$\frac{\partial\Sigma V}{\partial z}$
										Reservoir Full												
550											—	−701.140,43	+25.518,153		0	0		0	−25.887,293	7	15 T	2,783. 47
		Note.	K, , K_2 and K_3 not required above El 400 because U.S Face is vertical and multiplier is zero.																			
500								—	—	—	—	−642.476,85	151.476,99		0	0		0	+62.283,893	7	15 T	5046 36
450								—	—	—	—	−668.224,00	+156.752,77		0	0		0	+65976935	7	15 T	−11,219.45
400								—	—	—	- -	−682.648,68	156.580,57		0	0		0	+65.856,399	7	15 T	15,354 50
350								+83.484,103	+388.412,99	+33.601,79	−127.803,97	−604.702,13	+101.699,85		0	+8,669,463,0		0	+27.439,895	1.0	15 T	−19,690.70
315								+70.914,684	+363.888,06	+40.793,158	−113.103,54	−572.489,25	+100.529,19		0	+6.512,052,6		0	+26.620,433	1.0	15 T	−22,716.50

Figure A-6. **Friant Dam study-partial derivatives for obtaining σ_y. –DS2-2(9)**

WITH EARTHQUAKE

INTERMEDIATE COMPUTATIONS FOR OBTAINING STRESSES — GRAVITY ANALYSIS OF FRIANT DAM

FRIANT DAM STUDY NO. 3 - NONOVERFLOW RES. W. S. EL. 578 T. W. S. EL. NONE BY H. P. W. DATE

	$\frac{dt}{dz}$	$\left[\frac{12}{T^3}\Sigma V\right.$	$\frac{2}{T^2}t_{zyU}$	$\left.\frac{4}{T^2}t_{zyD}\right]$	$\frac{6}{T^2}\frac{\partial \Sigma V}{\partial z}$	$\frac{2}{T}\frac{\partial t_{zyU}}{\partial z}$	$\frac{4}{T}\frac{dt_{zyD}}{dz}$	$=\frac{\partial b_1}{\partial z}$	
		Normal		Full Reservoir		Operation			
550	.7	26.836,370	0	20.55,941	22.408,484	0	3.793,0098	21.807,619	
500	.7	14.485,098	0	8.862,218,6	10.892,811	0	3.998,965,8	2.957,829,6	
450	.7	9.753,536,5	0	6.150,173,5	7.110,450,3	0	2.712,309,9	1.875,786,3	
400	.7	7.323,274,9	0	4.700,430,7	5.263,410,5	0	1.991,123,2	1.436,296,4	
350	1.0	4.534,641,1	210,319,23	2.982,875,3	3.554,995,6	.095,112,046	.602,082,17	1.516,354,8	
315	1.0	3.545,354,4	156,144,66	2.344,424,1	2.886,510,9	.059,936,057	.490,021,77	1.291,767,4	

	$-\frac{dt}{dz}$	$\left[\frac{18}{T^4}\Sigma V\right.$	$\frac{6}{T^3}t_{zyU}$	$\left.\frac{6}{T^3}t_{zyD}\right]$	$\frac{6}{T^3}\frac{\partial \Sigma V}{\partial z}$	$\frac{3}{T^2}\frac{dt_{zyU}}{dz}$	$\frac{3}{T^2}\frac{dt_{zyD}}{dz}$	$=\frac{\partial c_1}{\partial z}$
550	-.7	1.494,525,8	0	1.129,637,7	.820,823,59	0	.104,203,57	.683,605,5
500	-.7	.348,758,36	0	.213,376,03	.174,844,47	0	.048,141,64	.031,935,20
450	-.7	.150,362,84	0	.094,812,538	.073,077,596	0	.020,906,81	.013,285,58
400	-.7	.083,030,327	0	.053,242,86	.039,783,904	0	.011,287,546	.007,680,13
350	-1.0	.037,311,913	.003,461,095	.024,543,681	.019,500,799	.078,260,089	.002,477,024	.006,934,04
315	-1.0	.024,473,22	.002,155,702	.016,183,323	.013,283,530	.041,373,256	.001,691,295,5	.005,044,31

Figure A-7. Friant Dam study-intermediate computations for obtaining stresses. —DS2-2(10)

FRIANT DAM NONOVERFLOW SECTION. RESERVOIR W.S. EL. 578 TAILWATER EL. NONE .... STUDY No. 3

GRAVITY STRESS ANALYSIS OF MAXIMUM PARALLEL-SIDE CANTILEVER

INCLUDING EFFECTS OF TAILWATER AND HORIZONTAL EARTHQUAKE

NORMAL STRESS ON VERTICAL PLANES $\sigma_y = a_2 + b_2 y + c_2 y^2 + d_2 y^3$

EARTHQUAKE ←

By. H.P.W. Date 4-2-40

$$\frac{\partial a_1}{\partial z} = \frac{\partial \tau_{zyD}}{\partial z} \qquad \frac{\partial b_1}{\partial z} = \frac{\partial T}{\partial z}\left[\frac{12}{T^3}(\Sigma V) + \frac{2}{T^2}(\tau_{zyu}) + \frac{4}{T^2}(\tau_{zyD})\right] - \frac{6}{T^2}\left(\frac{\partial \Sigma V}{\partial z}\right) - \frac{2}{T}\left(\frac{\partial \tau_{zyu}}{\partial z}\right) - \frac{4}{T}\left(\frac{\partial \tau_{zyD}}{\partial z}\right)$$

$$\frac{\partial c_1}{\partial z} = -\frac{\partial T}{\partial z}\left[\frac{18}{T^4}\Sigma V + \frac{6}{T^3}(\tau_{zyu}) + \frac{6}{T^3}(\tau_{zyD})\right] + \frac{6}{T^3}\left(\frac{\partial \Sigma V}{\partial z}\right) + \frac{3}{T^2}\left(\frac{\partial \tau_{zyu}}{\partial z}\right) + \frac{3}{T^2}\left(\frac{\partial \tau_{zyD}}{\partial z}\right)$$

(‡ Use (+) sign if horizontal earthquake acceleration is upstream) (‡ Use (−) sign if horizontal earthquake acceleration is upstream)

$a_2 = \sigma_{yD} = a_1 \tan\phi_D + p' \pm^{‡} p'_E$ $\quad b_2 = b_1 \tan\phi_D + \frac{\partial a_1}{\partial z} \pm^{‡} \lambda\omega_c$ $\quad c_2 = c_1 \tan\phi_D + \frac{1}{2}\frac{\partial b_1}{\partial z}$ $\quad d_2 = \frac{1}{3}\frac{\partial c_1}{\partial z}$ $\quad$ Check for $y = T$; $\sigma_{yu} = (p \pm^{‡} p_E) - \tau_{zyu}\tan\phi_u$

ELEV.	$\frac{\partial a_1}{\partial z}$	$\frac{\partial b_1}{\partial z}$	$\frac{\partial c_1}{\partial z}$	b_2	c_2	d_2	σ_y Pounds per Square Foot				VERTICAL PLANE								
							U S	6	5	4	3	2	1	D S	σ_{yu}				
			Reservoir	Full															
550	-25.887,293	+21.807,619	-.683,605,49	-177.356,02	+12 304,768	-.227,868, 5	2,373.97		1914.9					2,619.5	2,373.9				
500	+62.283,893	+2.957,829,6	-.031,935,70	23 349,901	+1.861,795,3	.010,645,07	6,142 86		6143 9	5,941. 2				6,042.5	6,111 9				
450	+65 976,939	1.875,786,3	-.013,285,579	-35.754,699	+.752,996,4	-.004,428,53	9,759 95		906 9.5	9,667.6				10,171.5	9,760.0				
400	+65.856,399	+1.436,296,4	-.007,680,134	-45.346,232	+.627,728,0	-.002,560,05	13,393 0		13,594.1	13,493.9	13,494 5			14,100.8	13,370.0				
350	+27 439,695	+516,354,8	-.006,934,037	-105.710,89	+.957,897,7	.002,311, 35	15,110 75		15,105.4	14,988.3	13,896 6	15,105 9		17,120.9	15,907.8				
315	+26.620,433	+1.219,767,4	-.005,044,313	-99.099,183	+.799,784,2	-.001,681,44	18, 127.0	18,127 7	17,121 4	16,114.13	15,109.7	16,112 8	18,127.1	19,135.9	18,351.0				

Figure A-8. **Friant** Dam study-normal stresses on vertical planes. —DS2-2(11)

. FRIANT . DAM. NONOVERFLOW SECTION. RES. W.S. EL. 578. TAILWATER EL. NONE STUDY No. 3.

GRAVITY STRESS ANALYSIS OF MAXIMUM PARALLEL-SIDE CANTILEVER

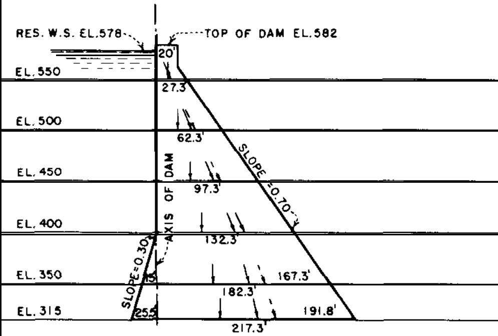
RESERVOIR FULL WITH EARTHQUAKE PRINCIPAL STRESSES By H.P.W. Date 4-6-40.

$$\sigma_{P1} = \frac{\sigma_z+\sigma_y}{2} \pm \sqrt{\left(\frac{\sigma_z-\sigma_y}{2}\right)^2 + \left(t_{zy}\right)^2}$$ {If $(\sigma_z-\sigma_y) > 0$, use (+); If $(\sigma_z-\sigma_y) < 0$, use (−)} {Alternate sign gives σ_{P2} which is perpendicular to σ_{P1}} $$\phi_{P1} = \frac{1}{2}\arctan\left(-\frac{t_{zy}}{\frac{\sigma_z-\sigma_y}{2}}\right)$$

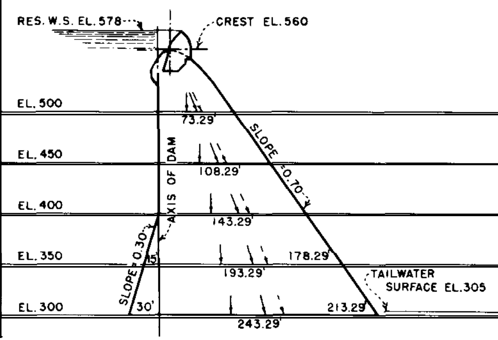
LEV.		VERTICAL PLANE									Stress in pounds per sq. inch							
		U.S.	6	5	4	3	2	1	D.S.		U.S.	6	5	4	3	2	1	D.S.
550	$\frac{1}{2}(\sigma_z+\sigma_y)$	2,548.96		3,345.7					4,076.9									
	$\frac{1}{2}(\sigma_z-\sigma_y)$	174.99		1,391.8					1,395.5									
	σ_{P1}	2,723.95		6,219.4					8,153.8		19		43					57
	σ_{P2}	2,373.97		472.0					0		16		3					0
	$\tan 2\phi_{P1}$	0		$\bar{1}$.806,438					$\bar{2}$.745,023									
	ϕ_{P1}	0		−30°31′					−35°00′									
500	$\frac{1}{2}(\sigma_z+\sigma_y)$	4,642.21		9					9.152.0									
	$\frac{1}{2}(\sigma_z-\sigma_y)$	7,469 172 56 65		2,86 6,111 -14.05	17,88,959.7 6 5				18,132.6	304	22		20	124				127
	σ_{P2}	6,111.86		9,376 6	778				0		42		65	1				0
	$\tan 2\phi_{P1}$	0		232 362	$\bar{2}$.798,728				$\bar{2}$.745,071									
	ϕ_{P1}	0		+44°53′	−35°10′				−35°00′									
450	$\frac{1}{2}(\sigma_z+\sigma_y)$	6,196.85		8,123.2	11,753.2				15,492.1									
	$\frac{1}{2}(\sigma_z-\sigma_y)$	$\bar{3}$,563.10		$\bar{1}$,756.4	2,079.6				5,302.8									
	σ_{P1}	2,633.75		4,296.0	21,543.9				30,984.2		18		30	150				215
	σ_{P2}	9,789.95		11,950.4	1,962.5				0		68		83	14				0
	$\tan 2\phi_{P1}$	0		1.936,005	$\bar{4}$.600,548				$\bar{2}$.745,035									
	ϕ_{P1}	0		31°20′	−38°52′				−35°00′									
400	$\frac{1}{2}(\sigma_z+\sigma_y)$	7,767.39		9,906.3	13,957.8	18,117.3			21,890.5									
	$\frac{1}{2}(\sigma_z-\sigma_y)$	$\bar{5}$,602.61		$\bar{3}$,626.9	550.90	4,620.7			7,492.7									
	σ_{P1}	2,164.78		4,924.3	23,892.4	34,758.7			43,781.0		15		34	166	241			304
	σ_{P2}	13,370.0		14,888.3	4,023.2	1,475.9			0		93		103	28	10			0
	$\tan 2\phi_{P1}$	0		.941,713	$\overline{18}$.005,627	$\bar{3}$.459,865			$\bar{2}$.745,105									
	ϕ_{P1}	0		21°38′	−43°25′	−36°56′			−35°00′									
350	$\frac{1}{2}(\sigma_z+\sigma_y)$	9,966.71		12,611.7	15,556.6	18,843.8	22,916.9		26,375.9									
	$\frac{1}{2}(\sigma_z-\sigma_y)$	$\bar{5}$,941.05		$\bar{2}$,561.7	1,378.4	4,976.2	7,788.0		9,028.0									
	σ_{P1}	3,073.98		5,985.6	25,608.4	34,378.0	44,741.3		52,751.8		21		42	178	239	311		366
	σ_{P2}	16,859.44		19,237.8	5,504.8	3,309.6	1,092.5		0		117		134	38	23	8		0
	$\tan 2\phi_{P1}$	.588,246		2.385,486	$\bar{7}$.223,447	$\bar{2}$.957,196	$\bar{2}$.617,809		$\bar{2}$.745,092									
	ϕ_{P1}	15°14′		33°38′	−41°04′	−38°39′	−34°33′		−35°00′									
315	$\frac{1}{2}(\sigma_z+\sigma_y)$	12,071.82	12,466.20	15,142.8	17,712.4	20,497.9	23,822.2	28,008.1	29,454.7									
	$\frac{1}{2}(\sigma_z-\sigma_y)$	$\bar{6}$,279.22	$\bar{5}$,819.50	$\bar{2}$,284.6	1,357.2	4,783.2	7,670.4	9,696.0	10,081.8									
	σ_{P1}	4,790.41	5,383.20	7,788.1	28,446.3	36,249.9	45,310.1	55,607.1	58,909.8		33	37	54	198	252	315	386	409
	σ_{P2}	19,353.23	19,549.20	22,497.5	6,978.5	4,745.9	2,334.3	409.1	0		134	136	156	48	33	16	3	0
	$\tan 2\phi_{P1}$	.587,097	.693,805	3.060,011	$\bar{7}$.845,417	$\bar{3}$.137,690	$\bar{2}$.616,839	$\bar{2}$.664,986	$\bar{2}$.745,096									
	ϕ_{P1}	15°14′	17°23′	35°57′	−41°22′	−36°10′	−34°33′	−34°43′	−35°00′									
	$\frac{1}{2}(\sigma_z+\sigma_y)$																	
	$\frac{1}{2}(\sigma_z-\sigma_y)$																	
	σ_{P1}																	
	σ_{P2}																	
	$\tan 2\phi_{P1}$																	
	ϕ_{P1}																	

Figure A-9. Friant Dam study-principal stresses.—DS2-2(12)

	Upstream face: Reservoir empty			Upstream face: Normal res. W.S. El. 578				Horizontal section	Horizontal section: Normal res. W.S. El. 578		Downstream face: Normal res. W.S. El. 578				Downstream face: Reservoir empty		
Elevation	Vertical stress	Stress parallel to face	Horiz. shear stress	Water pressure	Vertical stress	Stress parallel to face	Horiz. shear stress	Sliding factor	Shear friction factor	Max. horiz. shear stress	Water pressure	Vertical stress	Stress parallel to face	Horiz. shear stress	Vertical stress	Stress parallel to face	Horiz. shear stress
	Pounds per square inch										Pounds per square inch						
MAXIMUM NON-OVERFLOW SECTION																	
EL. 550	43	43	0	12	30	30	0	.286	74.48	15	0	22	33	15	9	14	6
EL. 500	99	99	0	34	46	46	0	.565	22.38	36	0	52	78	36	-1	-1	-0.7
EL. 450	149	149	0	56	53	53	0	.659	13.30	66	0	95	141	66	-2	-2	-1
EL. 400	201	201	0	77	61	61	0	.704	9.58	97	0	138	206	97	-1	-2	-1
EL. 350	201	219	-60	99	74	71	8	.677	8.23	123	0	175	261	123	33	50	23
EL. 315	213	232	-64	114	87	84	8	.658	7.50	140	0	200	297	140	51	75	35
MAXIMUM SPILLWAY SECTION																	
EL. 500	69	69	0	34	38	38	0	.581	26.10	33	0	46	69	33	10	15	7
EL. 450	130	130	0	56	58	58	0	.611	14.77	61	0	87	129	61	11	17	8
EL. 400	188	188	0	77	73	73	0	.643	10.39	89	0	127	190	89	10	15	7
EL. 350	197	215	-59	99	85	84	4	.629	8.74	116	0	166	248	116	40	60	28
EL. 300	219	239	-66	121	107	105	4	.613	7.59	139	2	200	297	139	63	94	44



MAXIMUM NON-OVERFLOW SECTION



UPSTREAM FACE — MAXIMUM SPILLWAY SECTION — HORIZONTAL SECTION — DOWNSTREAM FACE

↓ Resultant-concrete weight only. ↘ Resultant-water pressure and weight. ↘ Resultant-water pressure, weight and uplift.

Sliding factor = $\frac{\text{Horizontal Force}}{\text{Weight - Uplift}}$

Shear-friction Factor = $\frac{(\text{Weight - Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 450 pounds per square inch.
Coefficient of internal friction = 0.65

Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontal section.

Total load carried by vertical cantilever

All normal stresses are compressive except those preceded by a negative sign, which are tensile.

Positive shear stresses are caused by shear forces acting thus ⇌ Negative shear stresses are caused by shear forces acting thus ⇌.

Weight of gate included in analysis of spillway section

STUDY No. 3

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA
KENNETT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES
MAXIMUM NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
NORMAL CONDITIONS

DRAWN J.T.R. SUBMITTED O. F. W. Kern
TRACED J.J.S. F.L.D. RECOMMENDED Frank E. Houk
CHECKED H.P.M. APPROVED J. L. Savage

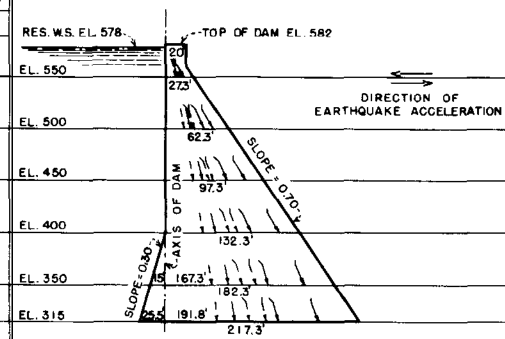
DENVER, COLORADO — FEB. 1, 1940
SHEET 1 OF 5
214-D-4158

Figure A-IO. Friant Dam study-gravity analyses for normal conditions.

MAXIMUM NON-OVERFLOW SECTION

UPSTREAM FACE (Pounds per Square Inch)

	RESERVOIR EMPTY			NORMAL RES. W. S. EL. 578			
	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
EL. 550	*34	34	0	15	16	16	0
	**51	51	0	9	44	44	0
EL. 500	84	84	0	39	18	18	0
	114	114	0	29	74	74	0
EL. 450	128	128	0	62	13	13	0
	171	171	0	49	94	94	0
EL. 400	172	172	0	85	9	9	0
	230	230	0	69	113	113	0
EL. 350	171	186	-51	108	21	13	26
	232	253	-70	90	127	130	-11
EL. 315	180	197	-57	124	32	23	28
	246	268	-74	105	142	145	-11



MAXIMUM NON-OVERFLOW SECTION

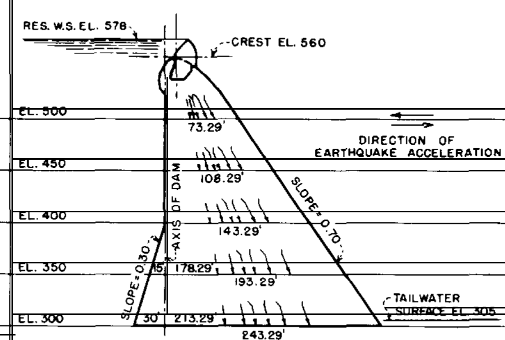
HORIZONTAL SECTION / DOWNSTREAM FACE

		NORMAL RES. W. S. EL. 578						RESERVOIR EMPTY		
	SLIDING FACTOR	SHEAR-FRICTION FACTOR	MAX. HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
			Pounds per Square Inch							
EL. 550	.502	42.39	25	0	36	53	25	18	27	12
	.069	306.80	6	0	8	12	6	1	1	0.3
EL. 500	.811	15.60	56	0	80	119	56	14	21	10
	.319	39.63	17	0	24	36	17	-16	-24	-11
EL. 450	.898	9.76	94	0	135	201	94	20	30	14
	.420	20.86	38	0	54	81	38	-23	-35	-16
EL. 400	.934	7.22	133	0	190	283	133	28	41	19
	.474	14.24	61	0	87	129	61	-31	-45	-21
EL. 350	.886	6.29	160	0	228	340	160	64	95	45
	.467	11.92	86	0	122	182	86	3	4	2
EL. 315	.857	5.76	178	0	255	379	178	83	124	58
	.459	10.76	101	0	145	215	101	18	27	13

MAXIMUM SPILLWAY SECTION

UPSTREAM FACE

	RESERVOIR EMPTY			NORMAL RES. W. S. EL. 578			
	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
EL. 500	*62	62	0	39	21	21	0
	**76	76	0	28	55	55	0
EL. 450	113	113	0	62	26	26	0
	146	146	0	49	90	90	0
EL. 400	163	163	0	85	28	28	0
	213	213	0	69	118	118	0
EL. 350	169	184	-51	108	36	30	22
	226	246	-68	90	134	138	-13
EL. 300	187	204	-56	131	53	46	23
	252	274	-76	111	160	166	-15



MAXIMUM SPILLWAY SECTION

HORIZONTAL SECTION / DOWNSTREAM FACE

		NORMAL RES. W. S. EL. 578						RESERVOIR EMPTY		
	SLIDING FACTOR	SHEAR-FRICTION FACTOR	MAX. HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
EL. 500	.832	18.24	44	0	63	95	44	17	26	12
	.331	45.87	21	0	29	44	21	3	4	2
EL. 450	.843	10.71	83	0	118	177	83	28	41	19
	.379	23.81	38	0	55	82	38	-5	-8	-4
EL. 400	.866	7.72	121	0	173	257	121	35	52	25
	.421	15.89	58	0	82	123	58	-15	-22	-10
EL. 350	.835	6.59	151	0	215	321	151	69	103	48
	.423	12.99	82	0	117	175	82	12	18	8
EL. 300	.807	5.77	176	2	254	377	176	95	142	67
	.419	11.11	101	2	147	217	101	30	45	21

Resultant-concrete weight and earthquake (horizontal upstream). Resultant - concrete weight and earthquake (horizontal downstream)
Resultant - water pressure, weight, and earthquake (horizontal upstream) Resultant-water pressure, weight, and earthquake (horizontal downstream).
Resultant - water pressure, weight, uplift, and earthquake (horizontal upstream). Resultant - water pressure, weight, uplift, and earthquake (horizontal downstream).

Sliding factor = $\frac{\text{Horizontal Force}}{\text{Weigh-Uplift}}$ Shear-friction Factor = $\frac{(\text{Weight - Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 450 pounds per square inch. Coefficient of internal friction = 0.65
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting Over two-thuds the area of the horizontal section; assumed to be unaffected by earthquake
Horizontal earthquake acceleration = 0.1 gravity, period: 1 second.
Total load carried by vertical cantilever
All normal stresses ore compressive except those preceded by a negative sign, which ore tensile.
Positive shear stresses are caused by shear forces acting thus ⇌. Negative shear stresses are caused by shear forces acting thus ⇌.
* Including earthquake acceleration (horizontal upstream). ** Including earthquake acceleration (horizontal downstream).
Weight of gate included in analysis of spillway section.

STUDY No. 3

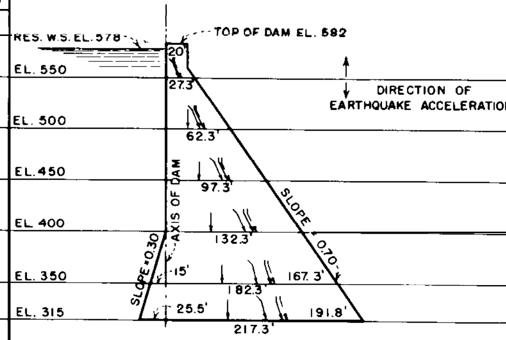
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIFORNIA
KENNETT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES
MAXIMUM NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZONTAL EARTHQUAKE ACCELERATION

DRAWN... J.T.R. .... SUBMITTED ....
TRACED... J.J.S. N.H. .... RECOMMENDED ....
CHECKED.. H.F.W. .... APPROVED ....
DENVER, COLORADO — FEB. 1, 1940 214-D-4159
SHEET 2 OF 4

Figure A-1 1. Friant Dam study-gravity analyses with horizontal earthquake acceleration.

Maximum Non-Overflow Section — Upstream Face

RESERVOIR EMPTY			NORMAL RES. W. S. EL. 578			
VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
Pounds Per Square Inch						
*47	47	0	13	33	33	0
**38	38	0	11	27	27	0
108	108	0	37	50	50	0
89	89	0	31	41	41	0
164	164	0	61	59	59	0
135	135	0	50	48	48	0
221	221	0	85	67	67	0
181	181	0	70	55	55	0
221	241	-66	109	81	78	8
181	197	-54	89	66	64	7
234	256	-70	126	95	93	9
192	209	-58	103	78	76	7



MAXIMUM NON-OVERFLOW SECTION

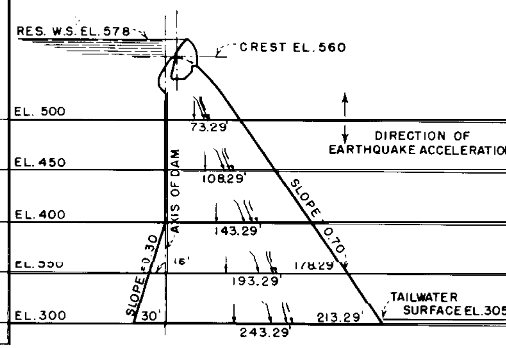
Maximum Non-Overflow Section — Horizontal Section and Downstream Face

SLIDING FACTOR	SHEAR FRICTION FACTOR	MAX. HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
	NORMAL RES. W.S. EL. 576						RESERVOIR EMPTY		
		Pounds Per Square Inch							
.280	67.96	17	0	24	36	17	10	15	7
.292	82.46	14	0	20	29	14	8	12	6
.550	20.48	40	0	57	85	40	-1	-2	-1
.585	24.71	33	0	47	70	33	-1	-1	-1
.640	12.21	73	0	104	155	73	-2	-2	-1
.685	14.63	60	0	85	127	60	-1	-2	-1
.682	8.82	107	0	152	227	107	-2	-2	-1
.732	10.51	87	0	125	186	87	-1	-2	-1
.655	7.60	135	0	193	287	135	37	55	26
.705	9.00	110	0	158	235	110	30	45	21
.637	6.94	154	0	220	327	154	56	83	39
.685	8.19	126	0	180	268	126	46	[illegible]	[illegible]

Maximum Spillway Section — Upstream Face

VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
*76	76	0	37	42	42	0
**62	62	0	30	34	34	0
143	143	0	61	64	64	0
117	117	0	50	52	52	0
207	207	0	85	81	81	0
169	169	0	70	66	66	0
217	230	[illegible]	109	94	95	4
177	193	-53	89	77	76	4
241	263	-72	133	117	116	5
197	215	-59	109	96	95	4

UPSTREAM FACE



MAXIMUM SPILLWAY SECTION

Maximum Spillway Section — Horizontal Section and Downstream Face

SLIDING FACTOR	SHEAR FRICTION FACTOR	MAX. HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
.563	23.86	36	0	51	76	36	11	16	8
.606	28.83	29	0	42	62	29	9	13	6
.592	13.56	67	0	95	142	67	12	18	9
.635	16.25	55	0	78	116	55	10	15	7
.624	9.57	98	0	140	209	98	11	17	8
.669	11.39	80	0	115	171	80	9	14	6
.609	8.08	128	0	183	272	128	44	66	31
.655	9.56	105	0	150	223	105	36	54	25
.593	7.03	154	2	220	327	154	69	103	48
.638	8.27	126	2	180	267	126	57	84	40

HORIZONTAL SECTION — DOWNSTREAM FACE

Resultant - concrete weight and earthquake (vertical upward), coincides with Resultant - concrete weight and earthquake (vertical downward).
Resultant - water pressure, weight and earthquake (vertical upward), coincides with Resultant - water pressure, weight and earthquake (vertical downward).
Resultant - water pressure, weight, uplift, and earthquake (vertical upward). Resultant - water pressure, weight, uplift, and earthquake (vertical downward)

Sliding factor = $\frac{\text{Horizontal Force}}{\text{Weight - Uplift}}$ Shear-friction Factor = $\frac{(\text{Weight - Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 450 pounds per square inch. Coefficient of internal friction = 0 65.
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure ot downstream face acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake.
Vertical earthquake acceleration = 0 1 g , Period = 1 second
Total load carried by vertical cantilever
All normal stresses are compressive except those preceded byo negative sign, which are tensile
Positive shear stresses ore caused by shear forces acting thus ⇌ Negative shear stresses ore caused by shear forces acting thus ⇌
* Including eorthquoke acceleration (vertical upward). xx Including earthquake acceleration (vertical downward)
Weight of gote included in analysis of spillway section.

STUDY No. 3

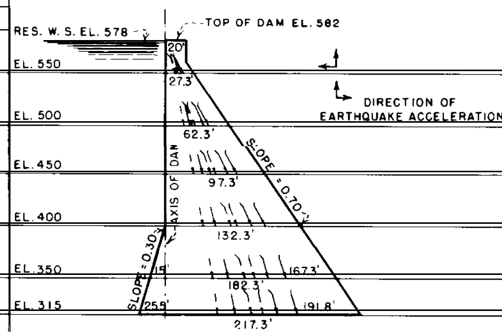
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIFORNIA
KENNETT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES
MAXIMUM NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH VERTICAL EARTHQUAKE ACCELERATION

DRAWN J.T.R. SUBMITTED F. D. Kirn
TRACED J.J.S. O.R.S. RECOMMENDED Jeremy E. Houk
CHECKED H.P.W. APPROVED J. L. Savage

DENVER, COLORADO — FEB. 1, 1940
SHEET 3 OF 5
214-D-4160

Figure A-12. **Friant Dam study-gravity analyses with vertical earthquake acceleration.**

RESERVOIR EMPTY			NORMAL RES. W. S. EL.578			
VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
Pounds Per Square Inch						
*38	38	0	16	19	19	0
**56	56	0	10	47	47	0
93	93	0	42	22	22	0
123	123	0	32	78	78	0
143	143	0	68	18	18	0
186	186	0	54	99	99	0
192	192	0	93	15	15	0
250	250	0	77	119	119	0
191	208	-57	118	28	20	27
252	275	-76	100	134	137	-10
202	220	-61	135	40	32	28
267	291	-80	116	150	153	-10

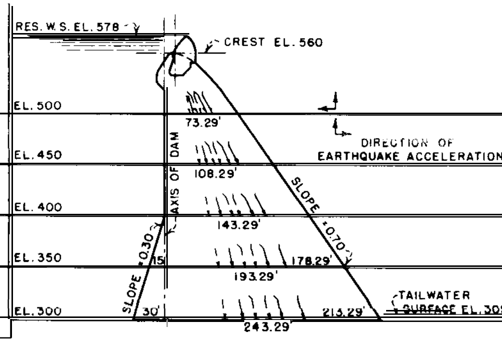


MAXIMUM NON-OVERFLOW SECTION

NORMAL RES. W. S. EL.578							RESERVOIR EMPTY		
SLIDING FACTOR	SHEAR FRICTION FACTOR	MAX. HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS
		Pounds Per Square Inch							
.474	40.25	27	0	38	57	27	19	28	13
.088	218.08	7	0	10	15	7	1	2	0.9
.768	14.68	60	0	85	127	60	14	21	10
.332	33.89	21	0	29	44	20	-16	-24	-11
.851	9.19	102	0	144	215	102	20	30	14
.429	18.21	45	0	64	95	45	-24	-35	-16
.885	6.81	143	0	204	304	143	27	41	19
.479	12.56	70	0	100	150	70	-31	-46	-21
.839	5.93	172	0	246	366	172	67	100	47
.471	10.58	98	0	140	209	98	6	9	4
.812	5.45	192	0	275	409	192	88	132	62
[illegible]	[illegible]	110	0	157	235	110	23	34	16

*69	69	0	43	25	25	0
**83	83	0	32	59	59	0
126	126	0	68	32	32	0
159	159	0	54	95	95	0
182	182	0	93	35	35	0
232	232	0	77	126	126	0
188	205	-56	118	45	38	22
245	267	-74	100	143	147	-13
209	228	-63	143	64	57	24
274	298	-82	123	171	175	-14

UPSTREAM FACE



MAXIMUM SPILLWAY SECTION

.783	17.15	48	0	68	102	48	18	27	13
.342	39.25	24	0	34	51	24	4	6	3
.797	10.08	89	0	127	189	89	29	43	20
.388	20.70	45	0	64	95	45	-4	-6	-3
.820	7.28	130	0	185	276	130	36	54	25
.428	13.96	67	0	95	142	67	-14	-21	-10
.790	6.23	162	0	232	345	162	73	109	51
.428	11.49	94	0	134	199	94	16	24	11
.764	5.46	190	2	274	407	190	101	151	71
.422	9.88	115	3	167	247	115	37	25	26

HORIZONTAL SECTION — DOWNSTREAM FACE

ALL RESULTANT FORCES INCLUDE VERTICAL EARTHQUAKE ACCELERATION UPWARD.
Resultant-concrete weight and earthquake (horizontal upstream). Resultant-concrete weight and earthquake (horizontal downstream)
Resultant-water pressure, weight, and earthquake (horizontal upstream). Resultant-water pressure, weight and earthquake (horizontal downstream).
Resultant-water pressure, weight, uplift, and earthquake (horizontal upstream). Resultant-water pressure, weight, uplift, and earthquake (horizontal downstream).

Sliding factor $= \frac{\text{Horizontal Force}}{\text{Weight - Uplift}}$ Shear-friction Factor $= \frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 450 pounds per square inch. Coefficient of internal friction = 0.65.
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake.
Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1 gravity, period = 1 second.
Total load carried by vertical cantilever.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses are caused by shear forces acting thus ⇌ Negative shear stresses are caused by shear forces acting thus ⇋
* Including earthquake acceleration (horizontal upstream and vertical upward). ** Including earthquake acceleration (horizontal downstream and vertical upward).
Weight of gate included in analysis of spillway section.

STUDY No. 3

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIFORNIA
KENNETT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES
MAXIMUM NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION UPWARD

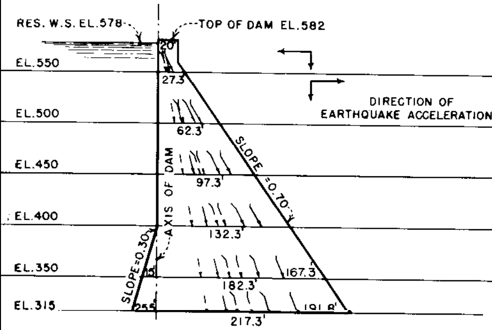
DRAWN J.T.R SUBMITTED F. D. Kirn
TRACED J.J.S. D.W.S. RECOMMENDED
CHECKED H.F.W. APPROVED
DENVER, COLORADO — FEB. 1, 1940
SHEET 4 OF 5
214-D-4161

Figure A-13. **Friant Dam study-gravity analyses with horizontal and vertical earthquake effects, vertical acceleration upward.**

MAXIMUM NON-OVERFLOW SECTION

UPSTREAM FACE (Pounds Per Square Inch)

Elevation	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress	Normal Res. W.S. El. 578: Water Pressure	Normal Res. W.S. El. 578: Vertical Stress	Normal Res. W.S. El. 578: Stress Parallel to Face	Normal Res. W.S. El. 578: Horiz. Shear Stress
EL.550	*30	30	0	14	13	13	0
	**47	47	0	8	41	41	0
EL.500	74	74	0	36	13	13	0
	104	104	0	25	69	69	0
EL.450	113	113	0	57	8	8	0
	156	156	0	43	88	88	0
EL.400	152	152	0	77	3	3	0
	210	210	0	62	106	106	0
EL.350	150	164	-45	98	13	6	25
	212	231	-64	80	119	123	-12
EL.315	159	173	-48	112	23	15	27
	224	245	-67	93	133	137	-12



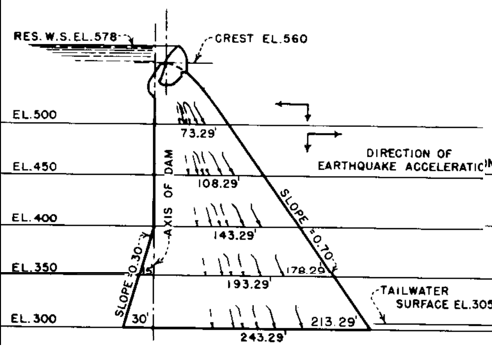
HORIZONTAL SECTION / DOWNSTREAM FACE (Pounds Per Square Inch)

Elevation	Sliding Factor	Shear Friction Factor	Normal Res.: Max. Horiz. Shear Stress	Normal Res.: Water Pressure	Normal Res.: Vertical Stress	Normal Res.: Stress Parallel to Face	Normal Res.: Horiz. Shear Stress	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress
EL.550	.537	44.78	24	0	34	50	24	17	25	12
	.046	519.78	4	0	6	9	4	-1	-1	-0.4
EL.500	.867	16.65	52	0	75	112	52	14	21	10
	.302	47.83	14	0	19	28	13	-16	-24	-11
EL.450	.960	10.43	88	0	125	187	88	20	31	14
	.409	24.50	31	0	45	67	31	-23	-35	-16
EL.400	.999	7.71	123	0	176	263	123	28	41	19
	.466	16.51	51	0	73	108	51	-30	-45	-21
EL.350	.947	6.70	148	0	211	314	148	61	90	42
	.463	13.72	73	0	105	156	73	-1	-1	-0.5
EL.315	[illegible]	6.13	164	0	233	350	164	78	117	55
	.455	12.33	87	0	125	186	87	13	19	9

MAXIMUM SPILLWAY SECTION

UPSTREAM FACE (Pounds Per Square Inch)

Elevation	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress	Normal Res. W.S. El. 578: Water Pressure	Normal Res. W.S. El. 578: Vertical Stress	Normal Res. W.S. El. 578: Stress Parallel to Face	Normal Res. W.S. El. 578: Horiz. Shear Stress
EL.500	*55	55	0	36	17	17	0
	**70	70	0	25	51	51	0
EL.450	100	100	0	57	20	20	0
	133	133	0	43	84	84	0
EL.400	144	144	0	78	21	21	0
	194	194	0	61	111	111	0
EL.350	149	162	-45	98	28	22	21
	206	224	-62	80	126	130	-14
EL.300	165	180	-49	119	42	36	23
	230	250	-69	98	150	154	-15



HORIZONTAL SECTION / DOWNSTREAM FACE (Pounds Per Square Inch)

Elevation	Sliding Factor	Shear Friction Factor	Normal Res.: Max. Horiz. Shear Stress	Normal Res.: Water Pressure	Normal Res.: Vertical Stress	Normal Res.: Stress Parallel to Face	Normal Res.: Horiz. Shear Stress	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress
EL.500	.896	19.49	41	0	59	88	41	16	24	11
	.316	55.33	17	0	25	37	17	2	3	1
EL.450	.903	11.43	77	0	110	164	77	26	39	18
	.367	28.11	32	0	46	69	32	-6	-10	-4
EL.400	.926	8.23	112	0	160	238	112	34	51	24
	.412	18.51	49	0	70	104	49	-16	-24	-11
EL.350	.893	7.01	139	0	196	296	139	65	97	45
	.417	15.02	70	0	101	150	70	8	12	5
EL.300	.863	6.12	162	2	234	348	162	89	133	62
	.414	12.77	89	2	127	188	89	24	36	17

ALL RESULTANT FORCES INCLUDE VERTICAL EARTHQUAKE ACCELERATION DOWNWARD.

Resultant-concrete weight and earthquake (horizontal upstream). Resultant-concrete weight and earthquake (horizontal downstream).
Resultant-water pressure, weight, and earthquake (horizontal upstream). Resultant-water pressure, weight, and earthquake (horizontal downstream)
Resultant-water pressure, weight, uplift, and earthquake (horizontal upstream). Resultant-water pressure, weight, uplift, and earthquake (horizontal downstream).

Sliding factor = $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$ Shear-friction Factor = $\frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

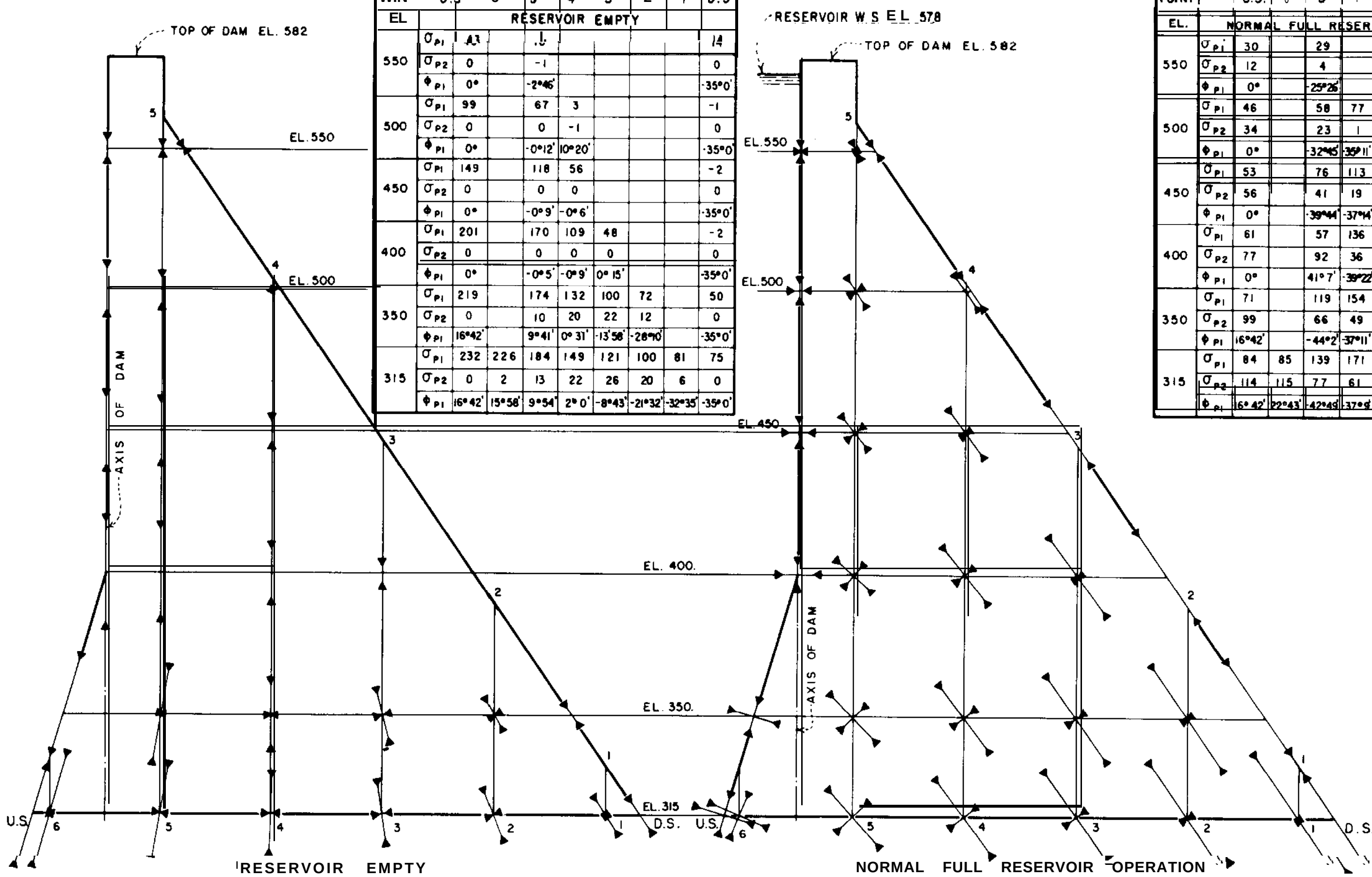
Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 450 pounds per square inch. Coefficient of internal friction = 0.65.
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake.
Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1 gravity, period = 1 second.
Total load carried by vertical cantilever.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses are caused by shear forces acting thus ⇌. Negative shear stresses are caused by shear forces acting thus ⇌.
*-Including earthquake acceleration (horizontal upstream and vertical downward). **Including earthquake acceleration (horizontal downstream and vertical downward).
Weight of gate included in analysis of spillway section.

STUDY No. 3

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIFORNIA
KENNETT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES
MAXIMUM NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION DOWNWARD

DRAWN... J.T.R. .... SUBMITTED ... F. D. Kirn
TRACED ... J.J.S. F.L.D. .... RECOMMENDED ... Ivan E. Houk
CHECKED ... H.P.K. .... APPROVED ... J. L. Savage
DENVER, COLORADO — FEB. 1, 1940 — SHEET 6 OF 6 — 214-D-4106

Figure A-14. **Friant Dam study-gravity analyses with horizontal and vertical earthquake effects, vertical acceleration downward.**



STRESSES IN POUNDS PER SQ IN.

WIN		U.S	6	5	4	3	2	1	D.S
EL	RESERVOIR EMPTY								
550	σ_{P1}	43		10					14
	σ_{P2}	0		-1					0
	ϕ_{P1}	0°		-2°46'					-35°0'
500	σ_{P1}	99		67	3				-1
	σ_{P2}	0		0	-1				0
	ϕ_{P1}	0°		-0°12'	10°20'				-35°0'
450	σ_{P1}	149		118	56				-2
	σ_{P2}	0		0	0				0
	ϕ_{P1}	0°		-0°9'	-0°6'				-35°0'
400	σ_{P1}	201		170	109	48			-2
	σ_{P2}	0		0	0	0			0
	ϕ_{P1}	0°		-0°5'	-0°9'	0°15'			-35°0'
350	σ_{P1}	219		174	132	100	72		50
	σ_{P2}	0		10	20	22	12		0
	ϕ_{P1}	16°42'		9°41'	0°31'	-13°58'	-28°10'		-35°0'
315	σ_{P1}	232	226	184	149	121	100	81	75
	σ_{P2}	0	2	13	22	26	20	6	0
	ϕ_{P1}	16°42'	15°58'	9°54'	2°0'	-8°43'	-21°32'	-32°35'	-35°0'

STRESSES IN POUNDS PER SQ IN.

POINT		U.S.	6	5	4	3	2	1	D.S.
EL.	NORMAL FULL RESERVOIR OPERATION								
550	σ_{P1}	30		29					33
	σ_{P2}	12		4					0
	ϕ_{P1}	0°		-25°26'					-35°0'
500	σ_{P1}	46		58	77				78
	σ_{P2}	34		23	1				0
	ϕ_{P1}	0°		-32°45'	-35°11'				-35°0'
450	σ_{P1}	53		76	113				141
	σ_{P2}	56		41	19				0
	ϕ_{P1}	0°		-39°44'	-37°14'				-35°0'
400	σ_{P1}	61		57	136	177			206
	σ_{P2}	77		92	36	16			0
	ϕ_{P1}	0°		41°7'	-39°22'	-36°39'			-35°0'
350	σ_{P1}	71		119	154	192	232		261
	σ_{P2}	99		66	49	32	13		0
	ϕ_{P1}	16°42'		-44°2'	-37°11'	-35°28'	-35°1'		-35°0'
315	σ_{P1}	84	85	139	171	207	244	285	297
	σ_{P2}	114	115	77	61	44	25	6	0
	ϕ_{P1}	16°42'	22°43'	-42°49'	-37°9'	-35°24'	-34°54'	-34°56'	-35°0'

Unit weight of concrete =150 pounds per cubic foot.
Effect of uplift neglected.
Total load carried by vertical cantilever.
- Denotes compression.
- Denotes tension.
U.S. denotes upstream face of section.
D.S. denotes downstream face of section
Numbers denote vertical planes.

STUDY No. 3

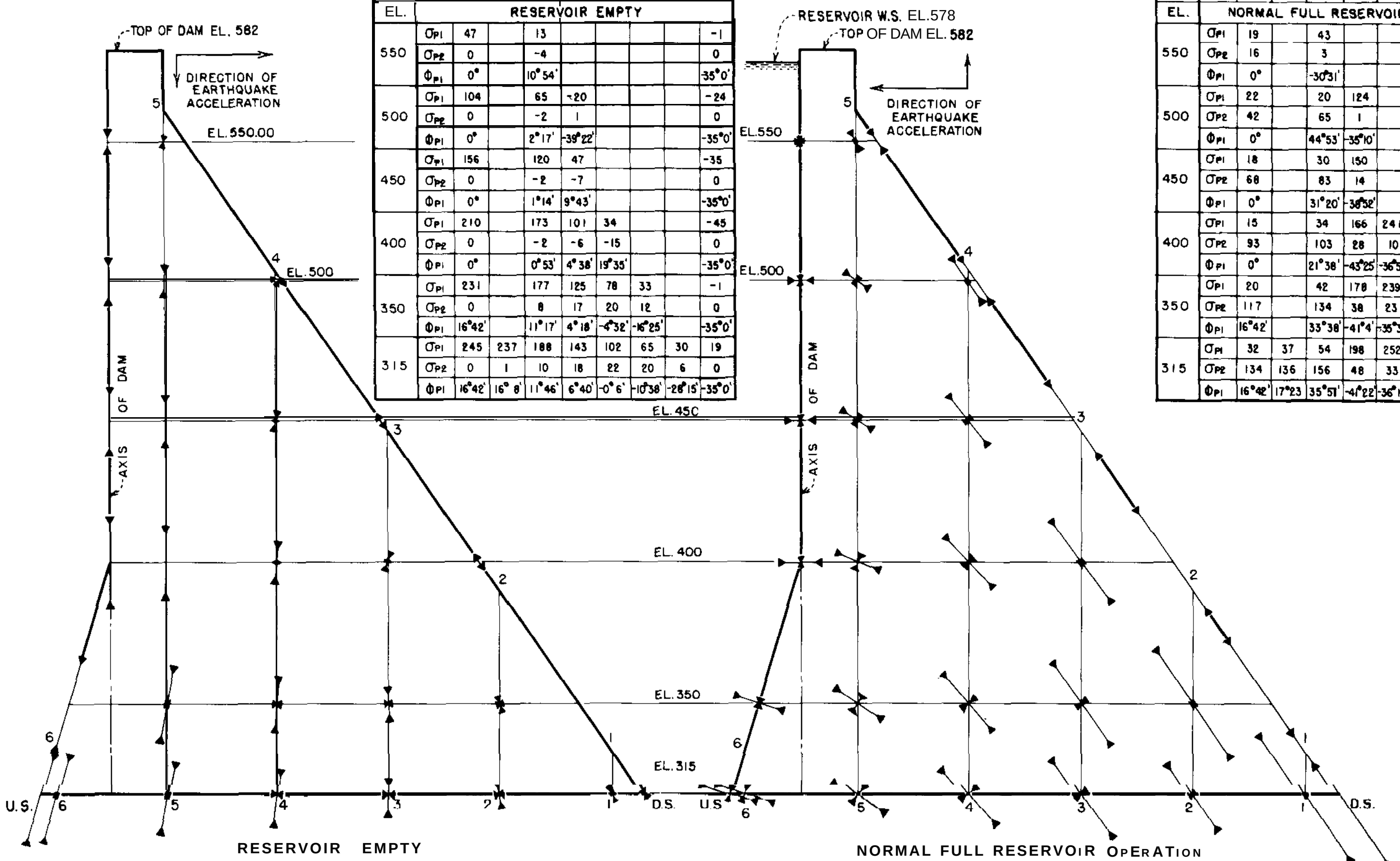
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA
FRIANT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES OF MAXIMUM NONOVERFLOW SECTION
PRINCIPAL STRESSES
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION

DRAWN J.T.R. SUBMITTED F. D. Kerr
TRACED F.A.J.-H.S. RECOMMENDED
CHECKED RSM APPROVED J. L. Savage
DENVER, COLORADO AUG. 12, 1940 SHEET 5 OF 12 214-D-4760

Figure **A-1.5. Friant Dam study-principal stresses on the maximum nonoverflow section, normal conditions.**

STRESSES IN POUNDS PER SQ. IN.									
POINT		U.S.	6	5	4	3	2	1	D.S.
EL.	RESERVOIR EMPTY								
550	σ_{p1}	47		13					-1
	σ_{p2}	0		-4					0
	ϕ_{p1}	0°		10°54'					-35°0'
500	σ_{p1}	104		65	-20				-24
	σ_{p2}	0		-2	1				0
	ϕ_{p1}	0°		2°17'	-39°22'				-35°0'
450	σ_{p1}	156		120	47				-35
	σ_{p2}	0		-2	-7				0
	ϕ_{p1}	0°		1°14'	9°43'				-35°0'
400	σ_{p1}	210		173	101	34			-45
	σ_{p2}	0		-2	-6	-15			0
	ϕ_{p1}	0°		0°53'	4°38'	19°35'			-35°0'
350	σ_{p1}	231		177	125	78	33		-1
	σ_{p2}	0		8	17	20	12		0
	ϕ_{p1}	16°42'		11°17'	4°18'	-4°32'	-16°25'		-35°0'
315	σ_{p1}	245	237	188	143	102	65	30	19
	σ_{p2}	0	1	10	18	22	20	6	0
	ϕ_{p1}	16°42'	16°8'	11°46'	6°40'	-0°6'	-10°38'	-28°15'	-35°0'

STRESSES IN POUNDS PER SQ. IN.									
POINT		U.S.	6	5	4	3	2	1	D.S.
EL.	NORMAL FULL RESERVOIR OPERATION								
550	σ_{p1}	19		43					57
	σ_{p2}	16		3					0
	ϕ_{p1}	0°		-30°31'					-35°0'
500	σ_{p1}	22		20	124				127
	σ_{p2}	42		65	1				0
	ϕ_{p1}	0°		44°53'	-35°10'				-35°0'
450	σ_{p1}	18		30	150				215
	σ_{p2}	68		83	14				0
	ϕ_{p1}	0°		31°20'	-38°52'				-35°0'
400	σ_{p1}	15		34	166	241			304
	σ_{p2}	93		103	28	10			0
	ϕ_{p1}	0°		21°38'	-43°25'	-36°56'			-35°0'
350	σ_{p1}	20		42	178	239	311		366
	σ_{p2}	117		134	38	23	8		0
	ϕ_{p1}	16°42'		33°38'	-41°4'	-35°39'	-34°33'		-35°0'
315	σ_{p1}	32	37	54	198	252	315	386	409
	σ_{p2}	134	136	156	48	33	16	3	0
	ϕ_{p1}	16°42'	17°23	35°51'	-41°22'	-36°10'	-34°33'	-34°43'	-35°0'



STUDY No.3

Unit weight of concrete: 150 pounds per cubic foot
Effect of uplift neglected.
Total load carried by vertical cantilever.
Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1g, period = one second
▶—◀ Denotes compression.
◀—▶ Denotes tension
U.S denotes upstream face of section
D. S. denotes downstream face of section.
Numbers denote vertical planes

0 400 80
SCALE OF STRESS
POUNDS PER SQUARE INCH

0 30 60
SCALE OF FEET

UNITED STATES
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CENTRAL VALLEY PROJECT-CALIFORNIA
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FRIANT DAM
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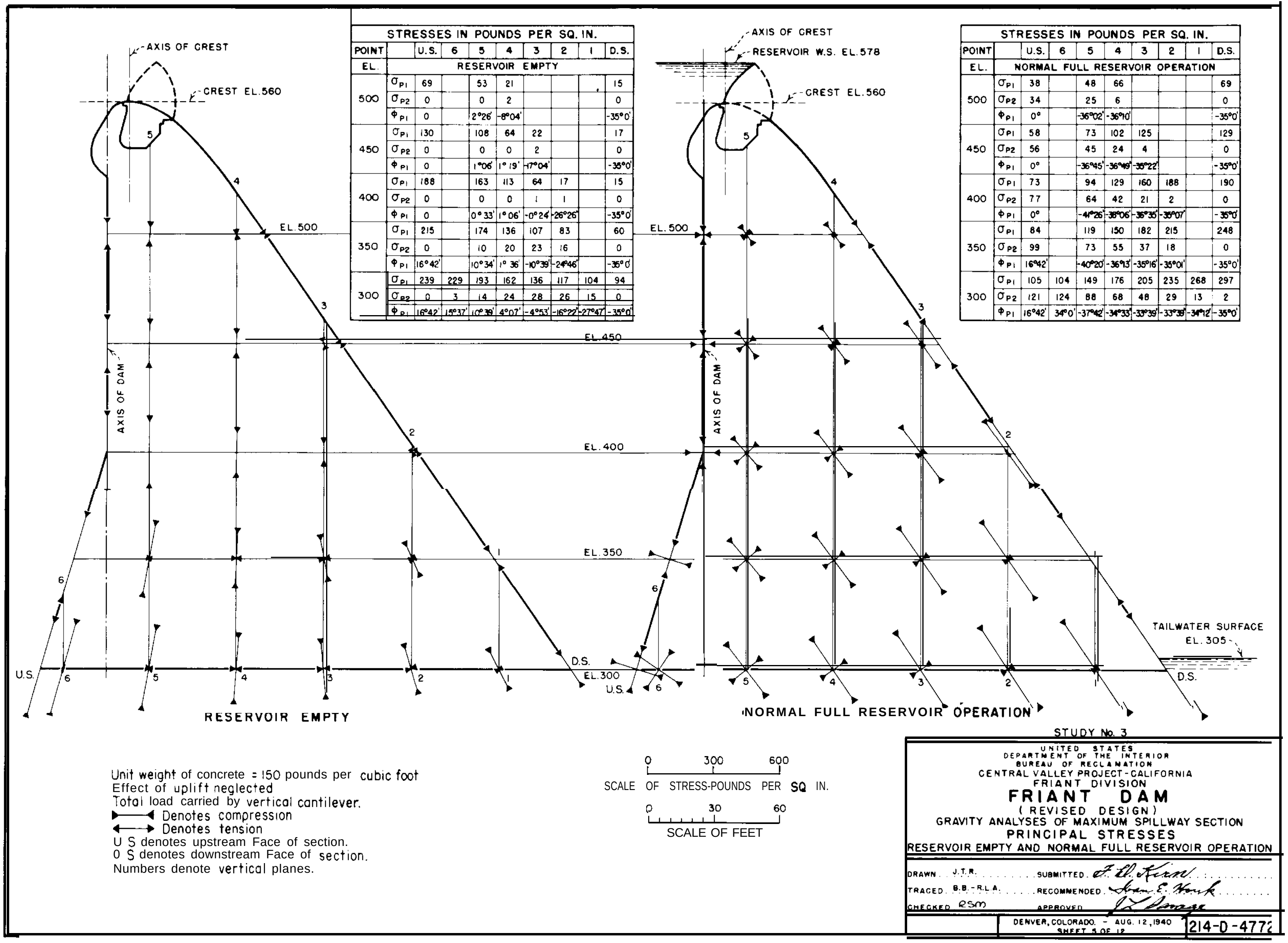
GRAVITY ANALYSES OF MAXIMUM NONOVERFLOW SECTION
PRINCIPAL STRESSES
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION WITH HORIZ AND VERT. EARTHQUAKE ACCELERATIONS INCLUDED

DRAWN. J.T.R. SUBMITTED. F. D. Kirn
TRACED. R.L.H.-N.H. RECOMMENDED. Kenneth E. Houk
CHECKED. RSM APPROVED. J. L. Savage

DENVER COLORADO - AUG. 12, 1940
SHEET 11 OF 12
214-D-4766

Figure A-16. Friant Dam study-principal stresses on the maximum nonoverflow section, horizontal and vertical earthquake accelerations included.



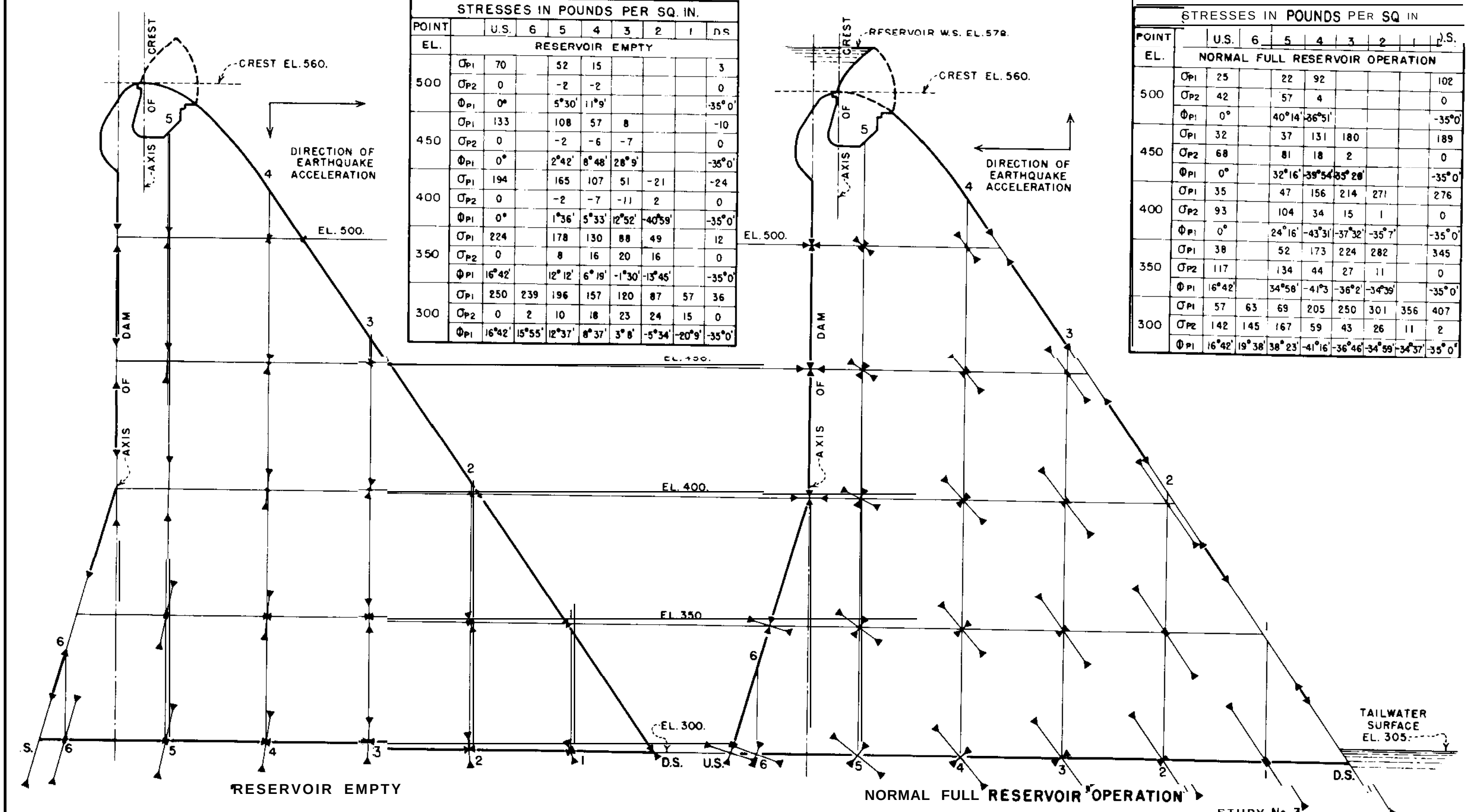
STRESSES IN POUNDS PER SQ. IN. — RESERVOIR EMPTY

POINT / EL.		U.S.	6	5	4	3	2	1	D.S.
500	σ_{P1}	69		53	21				15
	σ_{P2}	0		0	2				0
	ϕ_{P1}	0		2°26'	-8°04'				-35°0'
450	σ_{P1}	130		108	64	22			17
	σ_{P2}	0		0	0	2			0
	ϕ_{P1}	0		1°06'	1°19'	-17°04'			-35°0'
400	σ_{P1}	188		163	113	64	17		15
	σ_{P2}	0		0	0	1	1		0
	ϕ_{P1}	0		0°33'	1°06'	-0°24'	-26°26'		-35°0'
350	σ_{P1}	215		174	136	107	83		60
	σ_{P2}	0		10	20	23	16		0
	ϕ_{P1}	16°42'		10°34'	1°36'	-10°39'	-24°46'		-35°0'
300	σ_{P1}	239	229	193	162	136	117	104	94
	σ_{P2}	0	3	14	24	28	26	15	0
	ϕ_{P1}	16°42'	15°37'	10°39'	4°07'	-4°53'	-16°22'	-27°47'	-35°0'

STRESSES IN POUNDS PER SQ. IN. — NORMAL FULL RESERVOIR OPERATION

POINT / EL.		U.S.	6	5	4	3	2	1	D.S.
500	σ_{P1}	38		48	66				69
	σ_{P2}	34		25	6				0
	ϕ_{P1}	0°		-36°02'	-36°10'				-35°0'
450	σ_{P1}	58		73	102	125			129
	σ_{P2}	56		45	24	4			0
	ϕ_{P1}	0°		-36°45'	-36°49'	-35°22'			-35°0'
400	σ_{P1}	73		94	129	160	188		190
	σ_{P2}	77		64	42	21	2		0
	ϕ_{P1}	0°		-41°26'	-38°06'	-36°35'	-35°07'		-35°0'
350	σ_{P1}	84		119	150	182	215		248
	σ_{P2}	99		73	55	37	18		0
	ϕ_{P1}	16°42'		-40°20'	-36°13'	-35°16'	-35°01'		-35°0'
300	σ_{P1}	105	104	149	176	205	235	268	297
	σ_{P2}	121	124	88	68	48	29	13	2
	ϕ_{P1}	16°42'	34°0'	-37°42'	-34°33'	-33°39'	-33°39'	-34°12'	-35°0'

Figure A-1 7. Friant Dam study-principal stresses on the spillway section for normal conditions.



POINT		U.S.	6	5	4	3	2	1	DS
EL.	RESERVOIR EMPTY								
500	σ_{P1}	70		52	15				3
	σ_{P2}	0		-2	-2				0
	Φ_{P1}	0°		5°30'	11°9'				-35°0'
450	σ_{P1}	133		108	57	8			-10
	σ_{P2}	0		-2	-6	-7			0
	Φ_{P1}	0°		2°42'	8°48'	28°9'			-35°0'
400	σ_{P1}	194		165	107	51	-21		-24
	σ_{P2}	0		-2	-7	-11	2		0
	Φ_{P1}	0°		1°36'	5°33'	12°52'	-40°59'		-35°0'
350	σ_{P1}	224		178	130	88	49		12
	σ_{P2}	0		8	16	20	16		0
	Φ_{P1}	16°42'		12°12'	6°19'	-1°30'	-13°45'		-35°0'
300	σ_{P1}	250	239	196	157	120	87	57	36
	σ_{P2}	0	2	10	18	23	24	15	0
	Φ_{P1}	16°42'	15°55'	12°37'	8°37'	3°8'	-5°34'	-20°9'	-35°0'

STRESSES IN POUNDS PER SQ IN

POINT		U.S.	6	5	4	3	2	1	D.S.
EL.	NORMAL FULL RESERVOIR OPERATION								
500	σ_{P1}	25		22	92				102
	σ_{P2}	42		57	4				0
	Φ_{P1}	0°		40°14'	-36°51'				-35°0'
450	σ_{P1}	32		37	131	180			189
	σ_{P2}	68		81	18	2			0
	Φ_{P1}	0°		32°16'	-39°54'	-35°28'			-35°0'
400	σ_{P1}	35		47	156	214	271		276
	σ_{P2}	93		104	34	15	1		0
	Φ_{P1}	0°		24°16'	-43°31'	-37°32'	-35°7'		-35°0'
350	σ_{P1}	38		52	173	224	282		345
	σ_{P2}	117		134	44	27	11		0
	Φ_{P1}	16°42'		34°58'	-41°3	-36°2'	-34°39'		-35°0'
300	σ_{P1}	57	63	69	205	250	301	356	407
	σ_{P2}	142	145	167	59	43	26	11	2
	Φ_{P1}	16°42'	19°38'	38°23'	-41°16'	-36°46'	-34°59'	-34°37'	-35°0'

Unit weight of concrete = 150 pounds per cubic foot.
Effect of uplift neglected.
Total load carried by vertical cantilever.
Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1g, period = one second.
◄——► Denotes compression.
◄——► Denotes tension.
U.S. denotes upstream face of section.
D.S. denotes downstream face of section.
Numbers denote vertical planes.

0 400 800
SCALE OF STRESS
POUNDS PER SQUARE INCH

0 30 60
SCALE OF FEET

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIFORNIA
FRIANT DIVISION
FRIANT DAM
(REVISED DESIGN)
GRAVITY ANALYSES OF MAXIMUM SPILLWAY SECTION
PRINCIPAL STRESSES
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZ AND VERT. EARTHQUAKE ACCELERATIONS INCLUDED

DRAWN J.T.R. SUBMITTED F. D. Kern
TRACED B.B.-H.H. RECOMMENDED Ivan E. Houk
CHECKED E.S.(?) APPROVED J. L. Savage
DENVER, COLORADO — AUG. 12, 1940 SHEET 11 OF 18 214-D-4778

Figure A-18. **Friant Dam study-principal stresses on the spillway section, horizontal and vertical earthquake accelerations included.**

Table A-1.—*Friant Dam, nonoverflow and spillway sections (revised design)-maximum stresses, sliding factors, and minimum shear-friction factors. DS2-2(22)*

Loading conditions	Nonoverflow section					Spillway section				
	Stress, lbs. per sq. in.			Max. sliding factor	Min. shear-friction factor	Stress, lbs. per sq. in.			Max. sliding factor	Min. shear-friction factor
	Direct		Max. shear			Direct		Max. shear		
	Compr.	Tens.				Compr.	Tens.			
A. Normal conditions:										
1. Reservoir empty	232	2	64	—	—	239	none	66	—	—
2. Normal full reservoir operation	297	none	140	0.704	7.50	297	none	139	0.643	7.59
B. Including earthquake effect:										
1. Reservoir empty	291	46	80	—	—	298	24	82	—	—
2. Normal full reservoir operation	409	none	192	0.999	545	407	none	190	0.926	546

Trial-load Twist Analysis-Joints Grouted

B-l. ***Example of Twist Analysis, Joints Grouted—Canyon Ferry Dam.*** -Illustrations from a trial-load twist analysis, joints grouted, of a gravity dam are given on the following pages. The dam selected is Canyon Ferry Dam, and the plan, elevation, and selected elements are shown on figure B-l.

B-2. ***Design Data.*** -The following design data and assumption are presented for Canyon Ferry Dam:

(1) Elevation top of dam, 3808.5.

(2) Elevation of spillway crest, 3766.0.

(3) Maximum and normal reservoir water surface, elevation 3800.0.

(4) Minimum tailwater surface with gates closed, elevation 3633.0.

(5) Concentrated ice load of 7 tons per linear foot at elevation 3798.75. Provision is to be made so that no ice will form against the radial gates.

(6) Sustained modulus of elasticity of concrete in tension and compression, 3,000,000 pounds per square inch.

(7) Sustained modulus of elasticity of foundation and abutment rock, 3,000,000 pounds per square inch.

(8) Maximum horizontal earthquake assumed to have an acceleration of 0.1 gravity, a period of vibration of 1 second, and a direction of vibration normal to the axis of the dam.

(9) Maximum vertical earthquake assumed to have an acceleration of 0.1 gravity, a period of vibration of 1 second, and a direction that gives maximum stress conditions in the dam.

Note. Figure A-2 is a graph showing values of the coefficient K_E, which was used to determine hydrodynamic effects for the example given. However, this procedure is not consistent with current practice. A discussion of the coefficient C_m, which is presently used to determine hydrodynamic pressures, is given in section 4-34.

(10) Poisson's ratio for concrete and foundation rock, 0.20.

(11) Unit weight of water, 62.5 pounds per cubic foot.

(12) Unit weight of concrete, 150 pounds per cubic foot.

(13) Weight of radial gates, 3,000 pounds per linear foot.

(14) Weight of bridge, 5,500 pounds per linear foot.

(15) Unit shear resistance of concrete or concrete on rock, 400 pounds per square inch.

(16) Coefficient of internal friction of concrete on rock, 0.65.

(17) Uplift pressure on the base or horizontal sections above the base varies from full-reservoir water pressure at the upstream face to zero or tailwater pressure at the downstream face and acts over two-thirds the area of the base or horizontal sections.

Note. This uplift assumption is no longer used by the Bureau of Reclamation. See section 3-9 for uplift assumptions now in use.

(18) Effects of spillway bucket are included in the analyses.

(19) Effects of increased horizontal thickness of beams in spillway section are included.

B-3. ***Abutment Constants.*** -The method of determining abutment constants for elements of a concrete dam is shown in section 4-14.

B-4. ***Deflections and Slopes Due to Unit Loads.***-Certain data pertaining to unit loads

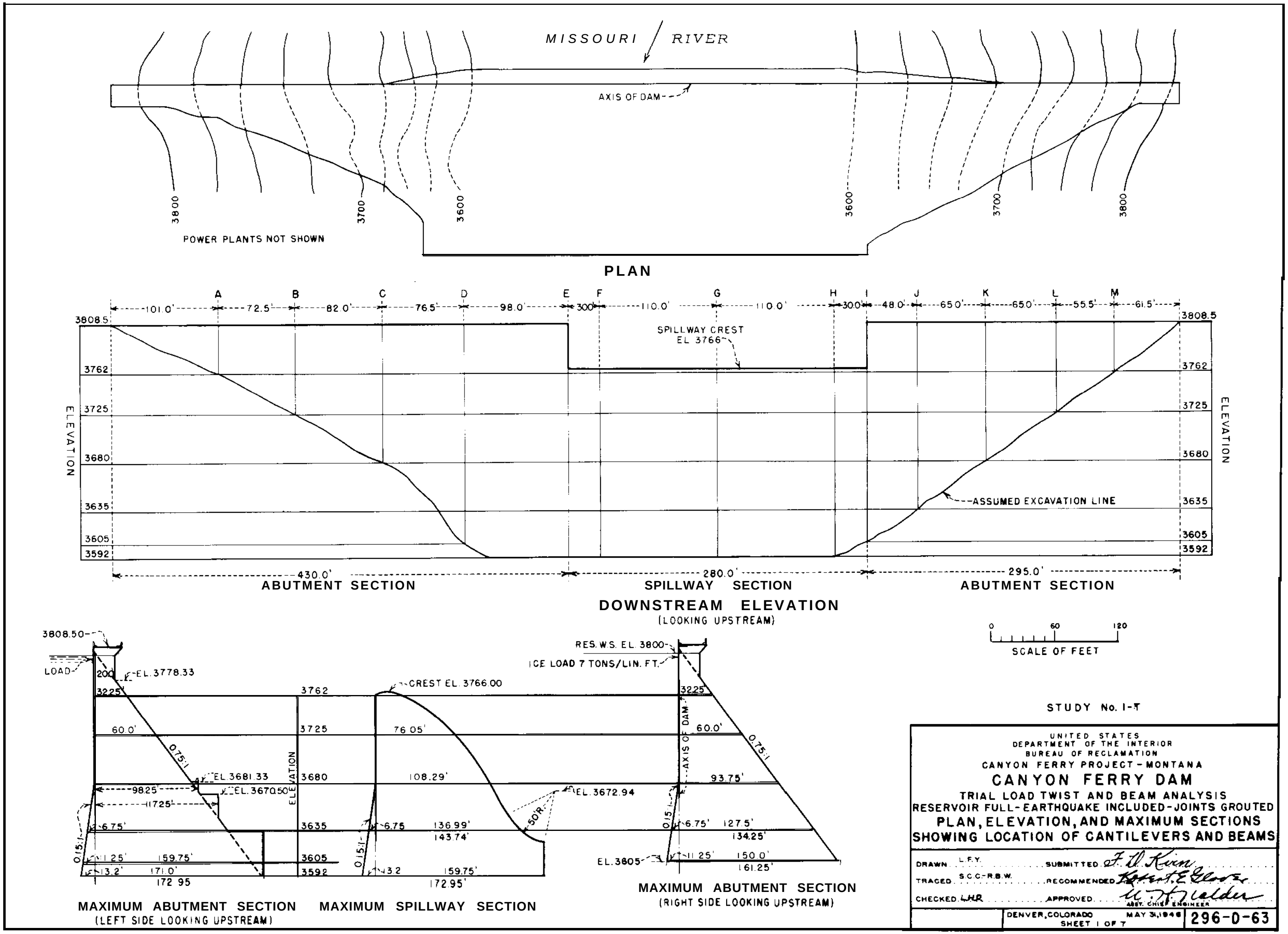


Figure B-1. **Canyon Ferry Dam study-plan, elevation, and maximum sections.**

are required prior to starting an adjustment. These include beam deflections for each unit triangular load, uniform load, and concentrated load and moment at the dividing plane; the slope of the beam at the abutment and at the dividing plane, due to unit loads; shears and twisted-structure deflections due to unit triangular, uniform, and concentrated shear loads on horizontal elements of the twisted structure; deflections of the vertical elements of the twisted structure due to unit triangular loads; cantilever deflections due to unit triangular normal loads; and shears and rotations of vertical elements of the twisted structure due to unit loads. Typical tabulations of these values are shown on figures B-2 through B-7. Calculations were by equations given in sections 4-29, 4-17, and 4-19. For identification of the cantilevers and beams in these drawings, see figure B-l. In the beam symbols, L means the left portion of the beam and R the right. A ΔG load is a triangular load with a value of 1,000 pounds per square foot at the abutment and zero at G, and so on for other loads. Cantilever loads are designated by the elevation at which the load is peaked.

B-5. ***Deflections of Cantilevers due to Initial Loads.*** -Cantilever deflections due to initial loads must be calculated prior to making a deflection adjustment. These deflections represent the position from which deflections of the cantilevers are measured when subjected to trial loads. Figure B-8 shows a tabulation of deflections due to initial loads on the cantilevers. These were computed by means of equation (17) in section 4-1 7. The initial loads are not shown but include loads of the type discussed in the latter part of section 4-16.

B-6. ***Trial-Load Distribution.-The*** total horizontal waterload is divided by trial between the three structures. However, it must be remembered that the twisted-structure load is split in half (see sec. 4-25), one-half to be placed on the horizontal elements and one-half on the vertical elements. In order to accomplish the trial-load distribution, the horizontal load ordinates must be determined at locations of the vertical elements, as illustrated on figure B-9. By multiplying these ordinates by loads on the horizontal elements, the equivalent loads on the vertical elements are obtained. The first trial-load distribution on elements of the left half of the dam is given on figure B-10, and the sixth and final trial-load distribution for these elements is shown on figure B-l 1.

The total waterload at any point must equal the cantilever load plus the loads on the horizontal and vertical twisted elements (or twice the load on the horizontal twisted element) plus the beam load. Accordingly, at elevation 3680 for cantilever G, the total waterload in kips is equal to 7.269 plus (1.9 x **2** x 0) plus (0.8 x 2) plus 0.2, or 9.069.

The values for P and M for beam loads are required to provide slope and deflection agreement at the dividing plane. These may be established by trial, or more easily by calculation by assuming approximate values of deflection components from previous trials, and computing the P and M necessary to give the same slope (not equal to zero) and deflection of left and right portions of the beam at the crown. Two equations involving V_c and M_c are obtained from the conditions that the slope and deflection of the two halves of the beam must be in agreement at the dividing plane. The simultaneous solution of these two equations gives the amount of shear V_c (or P) and moment M_c necessary at the crown of the beam to restore continuity in the beam structure.

B-7. ***Cantilever Deflections.*** -Cantilever deflections due to final trial loads are shown on figure B-l 2 for the left half of the dam. On the upper half of the sheet are deflections due to normal loads. These are obtained by multiplying loads given in the upper right-hand section of figure B-l 1 by corresponding deflections for unit normal loads. On the lower half of the figure are deflections due to shear loads on vertical elements of the twisted structure. These loads are given in the lower right of figure B-l 1. The loads are multiplied by cantilever deflections due to unit shear loads (see fig. B-4) to obtain the values shown. At the bottom of figure B-12 are inserted the values for abutment movements due to beam and twisted-structure elements which have common abutments with the cantilever

. . CANYON. FERRY DAM. .... SECTION. STUDY NO. 1 ....

.... PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS— TRIAL-LOAD TWIST ....

DEFLECTION OF BEAM DUE TO UNIT NORMAL LOADS -LEFT SIDE

BEAM 3725L .... By L.R.S. -Dote...???!..

POINT / LOAD		Δy							4				
		B	C	D	E	F	G		B	G			
Δ G		-.0^{3}562 79	-.015,269	-.039,130	-.075,211	-.086,608	-.128,414		-.0^{3}048,408	-.0^{3}379,78			
Δ F		-.0^{3}380,22	-.007,927,0	-.018,741	-.033,597	-.038,074	-.054,736		-.0^{3}025,474	-.0^{3}151 ,48			
Δ E		-.0^{3}333,95	-.006,335,5	-.014,511	-.025,413	-.028.741	-.040.941		-.0^{3}020,486	-.0^{3}1 10,9 1			
Δ D		-.0^{3}193,33	-.002,368,3	-.004,647,4	-.007,519,9	-.008,399,3	-.011,624		-.0^{3}007,975,0	-.0^{3}029,311			
Δ C		-.0^{3}094,75	-.0^{3}601,4 7	-.0^{3}998,3 9	-.001,506,9	-.001,662,5	.002,233,3		-.0^{3}002 234,1	-.0^{3}005,188,6			
Unif.		-.001,257.5	-.046,000	-.126,530	-.258.534	-.301,761	-.461,466		-.0^{3}144,2 3	-.001,450,34			
Conc. P		-.0^{3}004,169,4	-.0^{3}232,51	-.0^{3}692,17	-.001,534,8	-.001,828,4	-.002,945,3		-.0^{6}722,4 6	-.0^{3}010,190			
Conc. M		-.0^{6}005,033,9	-.0^{6}585,76	-.0^{3}001,907,2	-.0^{3}004,699,6	-.0^{3}005,771,9	-.0^{3}010,190		-.0^{6}001,809,4	-.0^{6}043,637			

Figure B-2. Canyon Ferry Dam study-deflection of a beam due to unit normal loads. –DS2-2(32)

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS—TRIAL-LOAD TWIST

DEFLECTION OF HORIZONTAL ELEMENT* DUE TO UNIT SHEAR LOADS —— LEFT SIDE

ELEMENT 3725 L

By L. R. S. Date 3-2-46

POINT / LOAD		Δy B	Δy C	Δy D	Δy E	Δy F	Δy G
ΔG		$.0^3$430,89	.001,950,1	.002,807,7	.003,330,0	.003,384,3	.003,435,4
ΔF		$.0^3$311,35	.001,318,9	.001,753,5	.001,892,9	.001,894,3	.001,894,3
ΔE		$.0^3$278,75	.001,148,3	.001,477,1	.001,547,9	→	.001,547,9
ΔD		$.0^3$172,25	$.0^3$602,37	$.0^3$656,86	→	→	$.0^3$656,86
ΔC		$.0^3$089,11	$.0^3$218,82	→	→	→	$.0^3$218,82
Unif.		$.0^3$861,78	.004,235,7	.006,753,7	.008,825,5	.009,167,9	.009,720,4
Conc. P		$.0^3$002,173,5	$.0^3$011,664	$.0^3$020,518	$.0^3$031,860	$.0^3$034,600	$.0^3$044,644

* Beam or twisted-structure element

Figure B-3. Canyon Ferry Dam study-deflection of a horizontal element due to unit shear loads.–DS2-2(33)

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER-STRESS ANALYSIS — TRIAL-LOAD TWIST

DEFLECTION OF CANTILEVER DUE TO UNIT SHEAR LOADS

CANTILEVER D By C.W.J. Date 3-2-46

ELEV. / LOAD	3808.5	3762	3725	3680	3635	3605
3808.5	$-.0^{3}549,3$	$-.0^{3}432,9$	$-.0^{3}290,5$	$-.0^{3}191,2$	$-.0^{3}125,4$	$-.0^{3}093,2$
3762	$-.0^{3}820,1$	$-.0^{3}703,7$	$-.0^{3}521,7$	$-.0^{3}343,4$	$-.0^{3}225,2$	$-.0^{3}167,4$
3725	$-.0^{3}493,3$	$-.0^{3}493,3$	$-.0^{3}453,8$	$-.0^{3}337,2$	$-.0^{3}221,2$	$-.0^{3}164,4$
3680	$-.0^{3}370,1$	$-.0^{3}370,1$	$-.0^{3}370,1$	$-.0^{3}332,6$	$-.0^{3}242,7$	$-.0^{3}180,4$
3635	$-.0^{3}216,8$	$-.0^{3}216,8$	$-.0^{3}216,8$	$-.0^{3}216,8$	$-.0^{3}190,6$	$-.0^{3}150,3$
3605	$-.0^{3}069,3$	$-.0^{3}069,3$	$-.0^{3}069,3$	$-.0^{3}069,3$	$-.0^{3}069,3$	$-.0^{3}060,1$

Figure B-4. Canyon Ferry Dam study-deflection of a cantilever due to unit shear loads. —DS2-2(34)

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS — TRIAL-LOAD TWIST

DEFLECTION OF CANTILEVER DUE TO UNIT NORMAL LOADS

CANTILEVER D

By C.W.J. Date 3-2-46

ELEV. / LOAD		3808.5		3762		3725		3680		3635		3605	
38085		-.003,552,6	-.001,853,5		$-.0^{3}982,$	3	$-.0^{3}477,$	6	$-.0^{3}220,$	3	$-.0^{3}119,$	5	
3762		-.003,904,7	-.002,444,7		-.001,	497,	9		$-.0^{3}773,3$	$-.0^{3}370,9$		$-.0^{3}207,6$	
3725		-.001,716,5	-.001,352,0		-.001,032,5		$-.0^{3}632,$	1	$-.0^{3}327,$	2	$-.0^{3}193,$	3	
3680		$-.0^{3}967,9$		$-.0^{3}818,7$		$-.0^{3}700,0$		$-.0^{3}523,9$		$-.0^{3}318,7$		$-.0^{3}200,7$	
3635		$-.0^{3}416,0$		$-.0^{3}370,7$		$-.0^{3}334,6$		$-.0^{3}290,7$		$-.0^{3}222,6$		$-.0^{3}158,2$	
3605		$-.0^{3}102,4$		$-.0^{3}095,0$		$-.0^{3}089,1$		$-.0^{3}082,0$		$-.0^{3}074,8$		$-.0^{3}061,0$	

Figure B-5. Canyon Ferry Dam study-deflection of a cantilever due to unit normal loads. —DS2-2(35)

CANYON FERRY DAM SECTION. STUDY NO. I

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS—TRIAL-LOAD TWIST

SHEARS IN TWISTED STRUCTURE (V_T) DUE TO UNIT LOADS—LEFT SIDE

By L.R.S. Date 2-4-46

POINT / LOAD		B	C	3635L	D	E	F	G		L'			
				Element 3725L									
ΔG		-198,250	-124,729	-97,108	-71,430	-24,716	-15,259	0		396.5			
ΔF		-143,250	-72,985	-48,964	-28,593	-1,571	0			286.5			
ΔE		-128,250	-59,357	-36,854	-18,721	0				256.5			
ΔD		-79,250	-18,461	-4,922	0					158.5			
ΔC		-41,000	0							82.0			
Unif.		-396,500	-314,500	-277,500	-238,000	-140,000	-110,000	0					
Conc. P		-1,000.	→					-1,000					
				-Element 3680 L									
ΔG			-157.250	-122.426	-90.054	-331.161	-19.237	0		319 5			
ΔF			-102,250	-68,597	-40,059	-2,200	0			204.5			
ΔE			-87,250	-54,173	-27,519	0				174.5			
ΔD			-38 250	-10.198	0					76.5			
Unif.			-314.500	-277.500	-238.000	-140.000	-110.000	0					
Conc. P			-1,000.	→				-1,000					
					Element 3635 L								
ΔG				-138,750	-102,061	-35,315	-21,802	0		277.5			
ΔF				-83,750	-48,907	-2,687	0			167.5			
ΔE				-68,750	-34,924	0				137.5			
ΔD				-19,750	0					39.5			
Unif.				-277,500	-238,000	-140,000	-110,000	0					
Conc. P				-1,000.	→			-1000.					

Figure B-6. Canyon Ferry Dam study-shears in twisted structure due to unit loads. –DS2-2(36)

..CANYON . FERRY ...DAM.2 .... SECTION. STUDY NO. ...1...............

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS— TRIAL-LOAD TWIST

ROTATIONS OF VERTICAL TWISTED-STRUCTURE ELEMENTS DUE TO UNIT COUPLE LOADS

CANTILEVERS C, D, AND 3635 L

By C.W.J. Date 3-2-46

ELEV. / LOAD	Cantilever C 3808.5	3762	3725	3680		
3808.5	$-.0^{3}001,161,6$	$-.0^{6}624,37$	$-.0^{6}130,55$	$-.0^{6}028,664$		
3762	$-.0^{3}001,318,3$	$-.0^{6}781,08$	$-.0^{6}234,44$	$-.0^{6}051,473$		
3725	$-.0^{6}204,92$	$-.0^{6}204,92$	$-.0^{6}152,10$	$-.0^{6}050,548$		
3680	$-.0^{6}048,220$	⟷	$-.0^{6}048,220$	$-.0^{6}027,740$		

ELEV. / LOAD	3808.5	3762	Cantilever D 3725	3680	3635	3605
3808.5	$-.0^{3}001,177,9$	$-.0^{6}640,72$	$-.0^{6}146,90$	$-.0^{6}045,008$	$-.0^{6}016,639$	$-.0^{6}009,510$
3762	$-.0^{3}001,347,6$	$-.0^{6}810,43$	$-.0^{6}263,79$	$-.0^{6}080,822$	$-.0^{6}029,879$	$-.0^{6}017,077$
3725	$-.0^{6}233,74$	$-.0^{6}233,74$	$-.0^{6}180,92$	$-.0^{6}079,369$	$-.0^{6}029,342$	$-.0^{6}016,770$
3680	$-.0^{6}087,113$	⟷	$-.0^{6}087,113$	$-.0^{6}066,633$	$-.0^{6}032,205$	$-.0^{6}018,406$
3635	$-.0^{6}030,712$	←		$-.0^{6}030,712$	$-.0^{6}023,738$	$-.0^{6}015,339$
3605	$-.0^{6}007,635$	←			$-.0^{6}007,635$	$-.0^{6}006,135$

Base of Cantilever 3635 L

ELEV. / LOAD	3635
3808.5	$-.0^{6}010,331$
3762	$-.0^{6}018,551$
3725	$-.0^{6}018,218$
3680	$-.0^{6}019,995$
3635	$-.0^{6}009,997,7$

Figure B-7. Canyon Ferry Dam study-rotations of vertical twisted-structure elements due to unit couple loads. –DS2-2(37)

CANYON FERRY DAM SECTION. STUDY NO.
PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS— TRIAL-LOAD TWIST
DEFLECTION OF CANTILEVERS DUE TO INITIAL LOADS
By L.R.S. Date 3-5-46

Elev.	A	B	C	D	E	F	G	H	I	J	K	L	M
						Cantilevers							
3808.5	-$.0^3$675,3	-.001,746,3	-.003,056,8	-.004,850,5	-.005,762,9	← Spil	lway	→	-.005,002,1	-.004,167,2	-.003,086,7	-.001,698,1	-$.0^3$658,8
3762	-$.0^3$096,8	-$.0^3$738,9	-.001,774,7	-.003,492,5	-.004,311,3	-.006,625,6	→	→	-.003,602,5	-.002,804,0	-.001,798,2	-$.0^3$713,9	-$.0^3$086,1
3725		-$.0^3$205,7	-.001,015,0	-.002,672,2	-.003,416,7	-.004,850,6	→	→	-.002,749,1	-.001,979,7	-.001,033,3	-$.0^3$191,1	
3680			-$.0^3$388,1	-.001,972,0	-.002,626,4	-.003,665,9	→	→	-.002,008,4	-.001,274,3	-$.0^3$400,2		
3635				-.001,394,4	-.001,958,8	-.002,627,7	→	→	-.001,392,7	-$.0^3$693,8			
3605				-.001,024,6					-$.0^3$997,7				
3592					-.001,353,8	-.001,703,5	→	→					
		Base of 3635L											
		-$.0^3$517,1											

Figure B-8. **Canyon Ferry Dam study-deflections of cantilevers due to initial loads.** —DS2-2(38)

CANYON FERRY DAM. SECTION. STUDY NO. I

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS — TRIAL-LOAD TWIST

LOAD ORDINATES AT CANTILEVER POINTS — LEFT SIDE

BEAM OR TWISTED- STRUCTURE LOADS

By L.R.S. Date 3-5-46

Beam	Load	Abt.	A	B	C	3635L	D	E	F	G			
3805	ΔE	1.0	765,12	.596,51	.405,81	.319,77	.227,91	0					
	ΔD	1.0	.695,78	.477,41	.230,42	.118,98	0						
	AC	1.0	604,70	.320,94	0								
	AB	1.0	.417,87	0									
3762	AG		1.0	.845,42	670,58	.591,68	.507,46	.298,51	.234,54	0			
	AL		1.0	.798,05	.569,64	.466,57	.356,55	.083,57	0				
	ΔE		1.0	.779,63	.530,40	.417,93	.297,87	0					
	ΔD		1.0	.686,15	.331,17	.171,00	0						
	ΔC		1.0	.530,74	0								
	AB		1.0	G									
375	AG			1.0	.793,19	.699,87	.600,25	353,09	277,43	0			
	AF			1.0	.713,79	.584,64	.446,77	,104,71	0				
	ΔE			1.0	.680,31	.536,06	.382,07	0					
	ΔD			1.0	.482,65	.249,21	0						
	AC			1.0	0								
368	AG				1.0	.882,35	.756,76	.445,15	.349,76	0			
	AF				1.0	.819,07	.625,92	.146,70	0				
	AE				1.0	.787,97	.561,60	0					
	AD				1.0	.516,34	0						
3635	AG					1.0	.857,66	.504,50	.396,40	0			
	AF					1.0	.764,18	.179,10	0				
	AE					1.0	.712,73	0					
	AD					1.0	0						

Figure B-9. Canyon Ferry Dam study-load ordinates at cantilever points. **–DS2-2(39)**

CANYON FERRY DAM . . . . . SECTION. STUDY NO.

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS—TRIAL-LOAD

TRIAL-LOAD DISTRIBUTION—LEFT SIDE

(TRIAL NO. 1)

	Horizontal twisted-structure loads												Normal cant/lever	
	Unif.	ΔB	ΔC	ΔD	ΔE	ΔF	ΔG		Conc.	A	B	C	3635L	D
38085														
3762						+ 1.0				+.973	1.397	1.877	2.093	2.32
3 725						+2.4					+.203	1.806	2.529	3.30
3680						+3.5						+.069	1.697	t3.436
3635						+4.0							+.251	+3.08
3 605														i-3.0
3592														

	Beam loads									Vertical twisted-structure or can				
	Unif.	ΔB	ΔC	ΔD	ΔE	ΔF	ΔG	Conc. P	Conc. M	A	B	C	3635L	D
38085	-.001				+.015			—	—					
3 762						+.1			-565.	+1.0	+.798	+.570	+.467	+.35
3725						+.8			-3,645		t2.4	+1.713	'1.403	+1.07
3680						+2.0			-6,060			+3.5	+2.867	+2.19
3635						+4.0			-9,478				+4.0	+3 05
3 605														+3.5
3592														

Figure **B-IO. Canyon Ferry Dam study-trial-load distribution (trial** *No.* **1). —DS2-2(40)**

CANYON FERRY DAM SECTION. STUDY NO. I

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS— TRIAL-LOAD TWIST

TRIAL-LOAD DISTRIBUTION—LEFT SIDE

(FINAL) By L. R. S. Date 4-6-46

	Horizontal twisted -structure loads								Normal Cantilever loads							
	ΔG	ΔF	ΔE	ΔD	ΔC	ΔB	Unif.	Conc.	A	B	c	3635L	D	E	F	G
3808.5				+.15			—		—.161	—.135	—.105	—.066	—.024	+.04	—	—
3762	—	+4.0	-4.5	+.2	+.25		+0.05	-5.	3.033	3.027	3.020	2.897	2.766	2.296	2.973	2.973
3725	+.3	+3.0	-2.0	-2.5	+1.0		+ .45	-30.		5.053	5.060	4.161	3.200	3.910	4.595	+ 4.803
3680	+1.9	t3.5	-3.0	— .5			+ .8	-50.			+ 1.469	i-1.789	+ 2.130	+ 4.257	+ 5.940	+ 7.269
~~3635~~		~~+4.0~~	~~-2.5~~				~~+ 1.7~~	~~--1/0.~~				~~+1151~~	+ 2.662	+ 6.412	f8.651	+ 8.651
3605													+3.000			
3592												Estimated-		+15.288	+12.469	t12.469

	Beam loads									Vertical twisted-structure or cantilever shear loads							
	ΔG	ΔF	ΔE	ΔD	ΔC	ΔB	Unif.	Conc. P	Conc. M	A	B	C	3635L	D	E	F	G
3808.5	—	—	+.28	-.163	-.179		-.04	—	—	+.104	+.072	+ .035	+ .018	0	0	—	—
3762		+.10		-.06				t.596	-570.8	0	+.004	+ .008	+ ,070	+ .136	+ .384	+ .050	+ .050
3725	+.15						+.1	—.136	-5,324.3		+ .250	t .262	+ .719	+1.206	+ .870	+ .533	+ .450
3680		+2.0					+.2	t2.474	-12,793			+2.700	+ 2.721	+2.744	+2.159	t 1.465	t .800
3635		+4.5					+.2	0	-17,953				+3.200	+2.975	t-2.416	+ 1.700	+ 1.700
3605														+3.500			
35921													Estimated-.		0	0	0

Figure B-II. Canyon Ferry Dam study-trial-load distribution (final) –DS2-2(41)

CANYON FERRY DAM SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS — TRIAL-LOAD TWIST

CANTILEVER DEFLECTION COMPONENTS - LEFT SIDE

(FINAL) By .... Date ....

			Cantilever Δy due to normal loads					
	A	B	C	3635L	D	E	F	G
3808.5	-.001,686	-306,309	-.013,045	—	-.019,684	.029,482	—	—
3762	-.0^{3}312	-.003,106	-.008,112	—	-.014,059	-.023,258	-.021,650	-.023,238
3725		-.0^{3}993	-.004,634	—	-.010,073	-.018,391	-.018,326	-.019,678
3680			-.001,531	—	-.006,286	-.013,292	-.014,028	-.015,051
3635				-.001,263	-.003,563	-.009,038	-.009,815	-.010,477
3605					-.002,222			
3592						-.005,471	-.005,978	-.006,345
			Cantilever Δy due to shear loads					
	A	B	C	3635L	D	E	F	G
3808.5	.0^{3}022	-.0^{3}056,2	-.0^{3}427,1	—	-.002,609,5	-.002,431,9	—	—
3762	-.0^{3}010	-.0^{3}047,3	-.0^{3}422,1	—	-.002,593,7	-.002,387,2	-.001,369,2	-.001,056,5
3725		-.0^{3}026,4	-.0^{3}405,3	—	-.002,521,3	-.002,282,9	-.001,351,0	-.001,040,9
3680			-.0^{3}268,6	—	-.002,253,6	-.002,032,0	-.001,248,4	-.0^{3}967,9
3635				-.0^{3}647,8	-.001,772,9	-.001,628,6	-.001,030,0	-.0^{3}812,3
3605					-.001,373,5			
3592						-.001,181,2	-.0^{3}756,9	-.0^{3}602,6
	Abutment Δy due to beam and twisted-structure loads.							
	-.0^{3}119,61	-.0^{3}669,7	-.002,423,2	-.003,569,6	-.002,382			

Note: These deflections are due to load on vertical twisted-structure element.

Figure B-12. **Canyon Ferry Dam study-cantilever deflection components (final). --DS2-2(42)** structure. The three component deflections given on figure B-12 represent the deflections due to trial loads on the structure which must be added algebraically to the deflections due to initial loads (see fig. B-8) to obtain the total deflection of the cantilever structure. These values are shown on Figure B-l 3. It should be noted at this point that the abutment movements of each structure are equal.

B-8. ***Twisted-Structure Deflections.*** -Shears due to loads on the horizontal elements of the twisted structure and angular rotations of vertical elements due to these shears are shown on figure B-14. Loads on horizontal twisted elements in the upper left of figure B-l 1 operate on unit shears to give the shear at each point in the horizontal twisted-structure element. The shear is divided by negative 1,000 to get units of twist load to operate on the unit rotations given on figure B-7 because the maximum ordinate for a unit twist load was assumed to be minus 1,000 foot-pounds per square foot. At each point where the vertical element and beam have a common base and abutment, it is desirable to note the value of abutment rotation of the vertical element due to load on the beam. These values are obtained for each element from figure B-l 6 and are indicated by asterisks (*) on figure B-14. At the base of element ***D*** there is no beam and a value is estimated.

In the upper half of figure B-l 5, rotations of vertical elements are integrated from the abutment to the crown using values calculated in figure B-14. Here the abutment rotations of the beams have been included. These are deflections of the horizontal elements due to rotation of vertical elements and abutment rotation of the beams. In the lower half of the figure are given the shear detrusions of horizontal elements due to loads on the beams (see the lower left-hand section of figure B-l 1). Detrusions are obtained by using deflections due to unit shear loads on horizontal elements as shown on figure B-3.

The lower half of figure B-16 shows values of shear detrusions due to twisted-structure loads. These are calculated by using deflections due to unit shear loads on horizontal elements, from figure B-3. Not only are these values components of the twisted-structure deflections, but they are also components of deflections of the beam structure, as will be shown later.

At the base of the deflection columns for cantilevers A to ***D,*** inclusive, on the lower half of figure B-16, the abutment movements of the cantilever and of the beam due to moment only, $M\alpha_2$, are entered for inclusion in the total twisted-structure deflection. Thus, the abutment movement at the base of cantilever A is equal to $-.0^3$,023 (fig. B-15) plus $-.0^3$,086, plus $-.0^3$,430 (fig. B-16) or equal to $-.0^3$,539. This is equal to the abutment movement at the base of the cantilever structure at ***A*** (see fig. B-13). Final twisted-structure deflections are given on figure B-13. These are compared with beam and cantilever deflections given on this same sheet.

B - ***9. Beam-Structure Deflections.*** – Deflections of beams due to bending are calculated in the upper half of figure B-16. These are determined by means of beam loads given on figure B-l 1 and unit deflections given on figure B-2. Slopes at the abutment and at the crown are also shown. Slopes at the crown include rotation of the common abutment due to twist loads on the vertical elements, but the slope shown at the abutment is only the rotation due to beam loads. Immediately above each deflection due to bending, the deflection of the beam due to rotation of the vertical element at the abutment is entered. Deflections are calculated by multiplying the slope at the abutment by the horizontal distance to each cantilever. At the abutment of each beam there are also additional movements due to initial, trial normal, and trial shear loads on the cantilevers which are entered at the bottom of figure B-16. Another component of the total beam deflection is due to shear detrusion for twisted-structure loads on horizontal elements. These values were previously calculated for the twisted structure and are shown in the lower half of figure B-16. Total deflections of beams may now be calculated by adding deflections due to bending, rotation, shear detrusion, and abutment movement. For example, the total deflection at the abutment of beam 3762, which coincides with the base of cantilever ***A,***

....CANYON FERRY DAM. ....SECTION. STUDY NO.

. . ....PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS- TRIAL-LOAD

TOTAL DEFLECTIONS — LEFT SIDE

(FINAL)

	Beam deflection									
	Abt	A	B	C	D	E	F	G		A
3808.5	-.0³034	-.002,949	-.010,225	7020,639	-029,166	-037,843	—	—		-.002,5
3762		-.0³539	-.005,198	7013,444	~021,169	-.028,365	7029,707	-.031,175		-.0³53
3725			-.001,895	-.009,121	-.016,821	-.023,437	-.024,414	-.025,152		
3680				-.004,611	-.012,097	'7017,796	-.019,123	-.019,594		
3635	-.005,998				-.008,789	-.013,173	-.013,799	-.014,110		

	Twisted-structure deflection									
	Abt.	A	B	C	D	E	F	G.		D
3808.5	-.0³034	-.003,049	-.008,411	-.017,789	-.027,889	-.035,712	—	—		-.029,5
3762		-.0³539	-.004,549	-.012,732	-.022,003	-.029,494	-.030,064	-.030,076		-.022,5
3725			-.001,895	-.009,084	-.017,499	-.024,421	-.025,143	-.025,147		-.017,6
3680				-.004,546	-.012,201	-.018,356	-.019,038	-.019,162		-.012,8
3635	-.005,998				-.008,873	-.013,141	-.013,616	-.013,563		-.009,1
3605										-.007,0
3592									Estimated	

Figure B-13. Canyon Ferry Dam study-total deflections (final). –DS2-2(43)

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS— TRIAL-LOAD TWIST

SHEARS (V) IN HORIZONTAL ELEMENTS AND ROTATIONS (Δφ) OF VERTICAL ELEMENTS

DUE TO TWISTED-STRUCTURE LOAD — LEFT SIDE (FINAL) By G.F.B. Date 4-9-46

		C		3635L		D		E		F		G	
		V	Δφ	V	Δφ	V	Δφ	V	Δφ	V	Δφ	V	Δφ
3808.5		-1,322	-.0^{3}121,07	-352.5	—	0	-.0^{3}126,09	0	-.0^{3}030,18	← Spillway →			
3762		-37,992	-.0^{3}099,95	-36,556	—	-32,494	-.0^{3}108,63	-7,012	-.0^{3}030,18	- 500.	-.0^{3}012,36	+ 5,000.	+.0^{3}013,09
3725		-203,032	-.0^{3}067,802	-184,886	—	-146,866	-.0^{3}083,11	-45,128	-.0^{3}027,43	-24,078	-.0^{3}011,72	+ 30,000	+.0^{3}012,18
3680		-577,375	-.0^{3}028,273	-477,081	—	-369,152	-.0^{3}054,69	-128,906	-.0^{3}019,64	-74,550	-.0^{3}009,37	+ 50,000	+.0^{3}009,63
3635				-524,875	-.0^{3}018,84	-402,918	-.0^{3}030,17	-138,748	-.0^{3}011,82	-77,000	-.0^{3}006,25	+110,000	+.0^{3}006,66
3605						-450,000	—						
3592		←				Not	required					→	
		*	-.0^{3}012,407	*	-.0^{3}008,877,3		*-.0^{3}004	Estimated					

	3808.5L	A		B	
		V	Δφ	V	Δφ
3808.5		-12,054	-.0^{3}041,44	-5,675	-.0^{3}079,407
3762		-38,613	-.0^{3}014,22	-38,479	-.0^{3}055,688
3725				-224,025	-.0^{3}020,019
	*+.0^{4}25	*	-.0^{3}007,800	*	-.0^{3}011,952

Notes: At any point in structure — 1,000 pounds shear represents one unit of twist load.
*Rotation of abutment due to beam loads.

Figure B-14. Canyon Ferry Dam study-shears in horizontal elements and rotations of vertical elements due to twisted-structure load (final). —DS2-2(44)

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS — TRIAL-LOAD TWIST

TWISTED-STRUCTURE DEFLECTION DUE TO ROTATIONS OF VERTICAL ELEMENT—LEFT SIDE

TWISTED-STRUCTURE DEFLECTION DUE TO BEAM LOADS (FINAL) By G.F.B. Date 4-10-46

	Elev.	3808.5		3762		3725		3680			3635		
	$\Delta \frac{x}{2}$	$\Delta\phi$ *	$\int\Delta\phi$	$\Delta\phi$ *	$\int\Delta\phi$	$\Delta\phi$ *	J-09	$\Delta\phi$ *	$\int\Delta\phi$	$\Delta \frac{x}{2}$	$\Delta\phi$ *	$\int\Delta\phi$	
A bt.		+.0$_6$25	0	.					*Note: $\Delta\phi$ includes rotations due to beam abutment forces				
	50.5												
A		-.0$_3$049,240	-.002,474	-.0$_3$022,02	0								
	36.25												
B		-.0$_3$091 359	-.007,571	-.0$_3$067,640	-.003,250	-.0$_3$031,971	0						
	41.0												
C		-.0$_3$133,48	-.016,189	-.0$_3$112,36	-.010,630	-.0$_3$080,209	-.004,599	-.0$_3$040,68	0	ABT	-.0$_3$027,72	0	
	38.25									19.75			
D		-.0$_3$130,09	-.026,870	-.0$_3$112,63	-.019,236	-.0$_3$087,11	-.010,999	-.0$_3$058,69	-.003,801		-.0$_3$034,17	-.001,222	
	49.0									49.0			
E		-.0$_3$030,18	-.034,724	-.0$_3$030,18	-.026,234	-.0$_3$027,43	-.016,612	-.0$_3$019,64	-.007,639,6		-.0$_3$011,82	-.003,476	
	15.0									15.0			
F		—	—	-.0$_3$012,36	-.026,871	-.0$_3$011,72	-.017,199	-.0$_3$009,37	-.008,074		-.0$_3$006,25	-.003,747	
	55.0									55.0			
G		—	—	+.0$_3$013,09	-.026,832	+.0$_3$012,18	-.017,174	+.0$_3$009,63	-.008,060		+.0$_3$006,66	-.003,724	

Twisted-structure fly for beam loads

		Abt	A	B	C	D	E	F	G
3808.5		+.0$_3$013	+.0$_3$108	+.0$_3$061	-.0$_3$007	-.0$_3$014	+.0$_3$017	—	—
3762			-.0$_3$023	-0,982	-.0$_3$309	-.0$_3$380	-.0$_3$413	-.0$_3$417	-.0$_3$431
3725				-.0$_3$151	-.0$_3$714	-.001,094	-.001,378	-.001,420	-.001,481
3680					-.0$_3$752	-.001,858	-.002,401	-.002,456	-.002,551
3635					-.001,577	-.002,285	-.002,957	-.002,999	-.003,057

Figure B-1.5. **Canyon Ferry Dam study-twistedstructure deflection due to rotations of vertical element, and twisted-structure deflection due to beam loads (final). —DS2-2(45)**

. CANYON FERRY. DAM • SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS — TRIAL-LOAD TWIST

BEAM DEFLECTION DUE TO BEAM LOADS AND ABUTMENT ROTATIONS — LEFT SIDE (FINAL)

DEFLECTION OF HORIZONTAL ELEMENTS DUE TO TWISTED-STRUCTURE LOAD By G.F.B. Date 4-10-46

	Rotation due to twist	Beam Δy due to bending and shear / Beam Δy due to rotation of abutment caused by twist on vertical elements.									Rotation due to beam loads	Total slope of beam at crown	
		Abf.	A	B	C	D	E	F	G		ø Abt.	ø Cr.	
3808.5	0	$+.0^{3}014$	-002,216	009, 324	-019,646	-028,161	-036,838	—	—		$+.0^{6}$ 25	$-.0^{3}085$	
ø x L			0	001, 031	-002,197	-003,285	-004,678	-005, 105	-006,669				
3762	$-.0^{3}014$		$-.0^{3}034$	003, 061	-009,465	-015,508	-020,851	-021, 747	-021,708		$-.0^{3}008$	$+.0^{3}011$	
ø x L				0	'001, 642	-003,173	7005, 134	005,735	-007,938				
3725	$-.0^{3}020$			$-.0^{3}183$	-003,740	-008,274	-011,904	-012,187	-010,754		$-.0^{3}012$	$+.0^{3}012$	
ø x L					0	-002,163	-004,934	-005,782	-008,892				
3680	$-.0^{3}028$				$-.0^{3}815$	~003, 455	-004,609	-004,896	-002,214		$-.0^{3}012$	——	
ø x L					ABT.	$-.0^{3}744$	-002,591	-003,156	-005,228				
3635	$-.0^{3}019$				-001,655	-002,756	-003,943	-003,850	-002,177		$-.0^{3}009$	——	
	Beam Δy, also twisted-structure Δy (shear detrusion) due to twisted-structure loads.												
		Abf	A	B	C	D	E	F	G				
3808.5		$-.0^{3}048$	$-.0^{3}683$	$-.0^{3}901$	$-.0^{3}993$	-001,005	~001, 005	——	——				
3762			$-.0^{3}086$	$-.0^{3}687$	-001,363	-001,957	-002,417	1002, 436	~002, 383				
3725				$-.0^{3}487$	-002, 514	-004,149	-005,174	-005,267	-005,235				
3680					301,609	-004,292	-006,066	-066,258	7006, 301				
3635					-001,915	-002,861	-004,211	-004,365	-004,277				
		Abutment movements of beam due to loads on other elements											
			(37621	(3725)	(3680)	(3635)							
			$-.0^{3}419$	-001, 225	-002,187	-002,428							
		Abutment movements of twisted structure due to loads on other elements.											
			$-.0^{3}430$	-001,257	-002, 250	-002, 505							

Figure B-16. **Canyon Ferry Dam study-beam deflection due to beam loads and abutment rotations, and deflection of horizontal elements due to twisted-structure loads (final).–DS2-2(46)** is equal to $-.0^3,033,974$, plus $-.0^3,419$, plus $-.0^3,085,64$ (fig. B-16), or $-.0^3,539$. Inspection of figure B-13 shows that this agrees with the cantilever and twisted-structure deflections at the same point.

B-10. ***Total*** Deflections. -Total deflections for the right side of the dam are given on figure B-17. Note that at the crown point, G, the deflections agree closely with those computed for G for the left side of the dam (see fig. B-13).

B-l 1. ***Moment and Shear due to Trial Loads on*** Beams.-Total bending moments for each beam are calculated by multiplying final beam loads by bending moments in beams due to unit loads. The total shear is obtained by adding the beam load and the twisted-structure load on the horizontal element, and multiplying the result by the shear due to unit load. These moments and shears are tabulated for the left side of the dam on figure B- 18.

B-12. ***Beam*** Stresses.-Stresses at the faces of beams due to pure bending are calculated from the well-known formula, $\sigma_x = \pm Mc/I$. No weight is carried by the beams, since it has been assumed that weight is assigned to the cantilevers. Beam stresses are calculated in pounds per square foot, but are tabulated in pounds per square inch. These calculations are not shown due to their simplicity.

B- 13. ***Cantilever Stresses.*** -Vertical cantilever stresses at the faces are calculated by means of the usual formula, $W/A \pm Mc/I$. The inclined cantilever stress parallel to either face of the dam at any point is calculated by dividing the corresponding vertical cantilever stress by the square of the cosine of the angle, ϕ, between the face and a vertical line, and subtracting from this quotient the product of the net normal water pressure and the square of the tangent of the angle ϕ. (See the lower part of figure 4-2 for equation and method of allowing for earthquake effect.)

In the example given here, an upward vertical earthquake acceleration was assumed. Consequently, the effective weight of the dam is found by multiplying by 1.1. The total moment is found by adding algebraically the moments due to weight, horizontal earthquake, vertical waterload, vertical earthquake, ice load, and trial load on the cantilever. Stresses at the faces are then calculated, using the formulas mentioned in the preceding paragraph. Principal stresses are calculated by means of equations given on figure 4-3.

Stability factors on horizontal planes are computed by formulas previously given in section 4-10. In computing the stability factors on inclined abutment planes, the equivalent horizontal force is the total shearing force due to the sum of the shears from the cantilever element and the abutting horizontal element of the twisted structure.

Assuming a unit area on the sloping surface, the total inclined abutment shear is computed by the equation,

$$\Sigma V = V_c \sin \psi + V_T \cos \psi$$

where :

ΣV = total inclined abutment shear on unit area,
V_c = shear in horizontal plane at base of cantilever,
V_T = shear in vertical plane at abutment of horizontal element, and
ψ = angle between vertical and inclined plane of contact.

The total force normal to the inclined abutment plane is equal to the resultant of the total vertical force and horizontal thrust transferred from the vertical cantilever and horizontal element, respectively. This force (see fig. B-l 9) is equal to

$$F_N = \frac{(W+U)\sin\psi}{\sin\psi} = W + U$$

where :

F_N = total force normal to inclined abutment plane,
U = uplift force, and
ψ = angle between the vertical and the inclined abutment plane.

After the above values have been obtained, the sliding factor is computed by dividing the total inclined abutment shear by the normal

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS—TRIAL-LOAD TWIST

TOTAL BEAM AND TWISTED-STRUCTURE DEFLECTIONS — RIGHT SIDE

(FINAL) By L.R.S. Date 4-10-46

	Beam deflection									Cantilever deflection			
	Abt.	M	L	K	J	I	H	G		M	L	K	J
3808.5	-.0^{3}006	7002,408	-.008,162	-.017,065	-.025,711	1031,664	—	—		-.002,915	-.009,264	7016,625	7024,270
3762		-.0^{3}649	7004,945	-.012,494	-.020,041	7024,672	7026,983	-.031,495		-.0^{3}649	7004,820	-.011, 121	-.018,237
3725			-.002,168	-.009,215	-.016,377	-.020,372	~022,163	-.025,882			-.002,168	-.007,379	-.013,896
3680				-.004,018	-.010,172	-.014,003	-.015,831	-.019,603				-.004,018	-.009,547
3635					-.006,033	-.009,327	-.010,912	-.014,019					-.006,033
	Twisted -structure deflection												
	Abt.	M	L	K	J	I	H	G		I	H	G	
3808.5	-.0^{3}006	-.001,993	-.007,178	-.016,576	-.026,167	-.031,239	—	—		-.031,092	—	—	
3762		-.0^{3}649	-.004,670	-.012,365	-.020,548	~025,360	~027,278	~030,503		-.024,027	-.026,447	-.030,919	
3725			7002,168	-.008,698	-.015,580	-.020,008	-.021,963	-.025,401		-.018,920	-.021,739	-.025,570	
3680				7004,018	7009,820	-.013,695	-.015,571	-.018,667		-.013,642	-.016,673	-.019,685	
3635					7006,033	-.009,644	-.011,393	-.014,419		-.009,255	-.011,856	-.013,917	
3605										7006,758			
3592											-.007,434	~008,652	

Figure B-17. **Canyon Ferry Dam study-total beam and twisted-structure deflections (final). —DS2-2(47)**

CANYON FERRY DAM. SECTION. STUDY NO. 1

PARALLEL-SIDE CANTILEVER--STRESS ANALYSIS — TRIAL-LOAD TWIST

BENDING MOMENTS (ΣM) IN BEAM DUE TO TRIAL LOADS — LEFT SIDE (FINAL)

TOTAL SHEAR (ΣV) IN HORIZONTAL ELEMENTS DUE TO TRIAL LOADS By G.F.B. Date 4-20-46

	Abt.	A	B	C	3635L	D	E	F	G
					Beam 3808.5				
ΣM	+11,268	-260,706	-125,422	+68,972		+89,935	Spillway		
ΣV	-17,975	-12,675	-8,314	-2,819		+793			
					Beam 3762				
	ΣM	-1,323,134	-584,898	+5,699		+331,591	+486,106	505,240	570,800
	ΣV	-50,229	-47,244	-43,652		-35,372	-7,733	-1,096	+4,404
					Beam 3725				
		ΣM	-6,412,700	-1,539,820		+1,674,450	4,190,326	4,650,338	5,324,300
		ΣV	-293,276	-253,056		-181,244	-62,699	-37,231	+30,136
					Beam 3680				
			ΣM	-1,816,170		+3,121,450	+10,442,630	+11,310,860	+12,793,000
			ΣV	-847,249		-499,344	-163,780	-99,024	+47,526
					Beam 3635				
				ΣM	-10,789,800	+2,898,360	+15,872,100	+16,743,000	+17,953,000
				ΣV	-957,250	-670,600	-178,840	-99,000	+110,000

Note:

ΣM = **Beam** trial loads times unit **moment** (M_B) in beam due to unit loads.

ΣV = Beam plus twisted-structure trial loads times unit **shear** (V_T) in horizontal element due to unit loads.

Figure B-18. Canyon Ferry Dam study-bending moments in beam due to trial loads (final), and total shear in horizontal elements due to trial loads (final). —DS2-2(48)

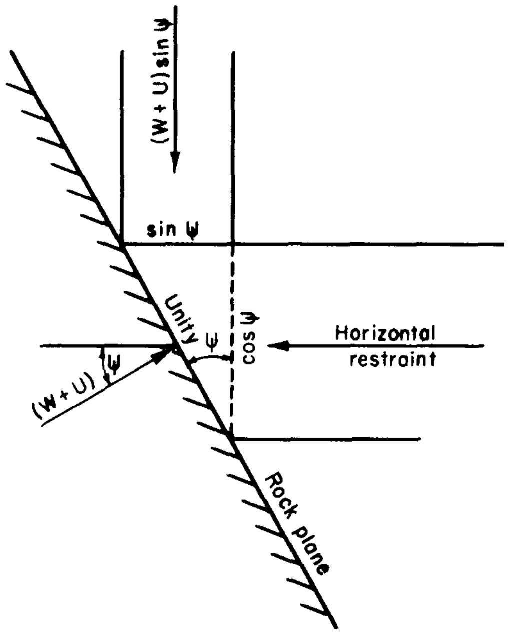


Figure B-l 9. **Force normal to an inclined abutment plane. –DS2-2(49)**

resisting force. The shear-friction factor is also computed. See section 4-10 for equations and a discussion of these factors. If the computed factors are not within the allowable values, the dam must be reproportioned to correct this condition.

B-14. ***Final*** Results. -Final results of the trial-load twist analysis of Canyon Ferry Dam are given on figures B-20 to B-25, inclusive. These show load distribution and adjustment on horizontal and vertical elements; stresses in horizontal beams and cantilevers; principal stresses at the faces of the dam; and stability factors for both the twist analysis and the gravity analysis.

The following conclusions were made from the twist analysis:

(1) Results determined from the trial-load twist analysis show that the maximum compressive principal stress is 263 pounds per square inch and occurs at elevation 3680 at the downstream face of cantilever C.

(2) The maximum tensile principal stress occurs at the upstream face of the right abutment of the beam at elevation 3725 and amounts to 146 pounds per square inch.

(3) The maximum rock-plane shearing stress occurs at the base of cantilever G, elevation 3592, and also at the left abutment of the beam at elevation 3635, and amounts to 101 pounds per square inch.

(4) The maximum sliding factor on horizontal planes is 0.812 and occurs at elevation 3725 in cantilever G. The maximum sliding factor on inclined abutment planes occurs at the base of cantilever ***L*** and is 1.197.

(5) The minimum shear-friction factor of safety on horizontal planes is 6.78 and occurs at the base elevation of cantilever G. The minimum shear-friction factor on inclined abutment planes is 6.32 and occurs at the base of cantilever C.

(6) Tensile principal stresses which occur at the left and right abutments of the dam at practically all elevations at the upstream face indicate that some diagonal cracking may occur in the concrete in these regions.

(7) In order to reduce the extent of diagonal cracking, it is recommended that the concrete in the dam be subcooled 8° F. or more, if possible, below mean annual temperature prior to grouting the contraction joints.

(8) Maximum compressive stresses in the beams and cantilevers, principal compressive stresses, and rock-plane shear stresses are conservative and well within allowable design limits for good concrete.

(9) The maximum sliding factor of 1.197 that occurs at the inclined base of cantilever ***L*** indicates that somewhat unsatisfactory stability conditions may be considered to exist at higher elevations along the abutments of the dam if sliding factors are used as the criterion for judging whether or not the dam is safe against failure by sliding. However, if shear-friction factors are used as the criterion instead of the sliding factors, stability conditions in the dam can be considered as being satisfactory. The minimum value for the shear-friction factor calculated from the trial-load twist analysis was 6.32.

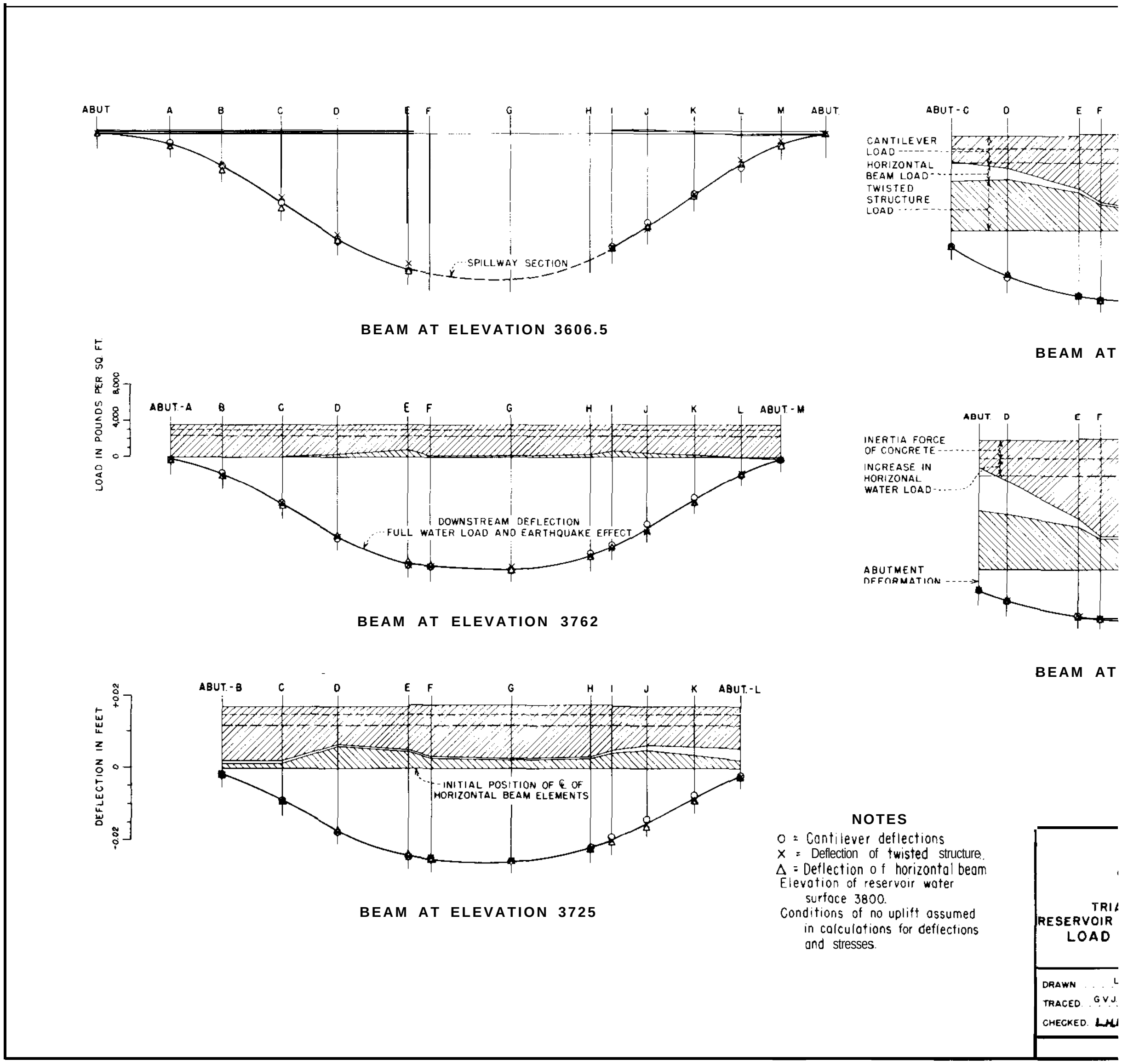


Figure B-20. **Canyon Ferry Dam study-load distribution and adjustment on horizontal elem**

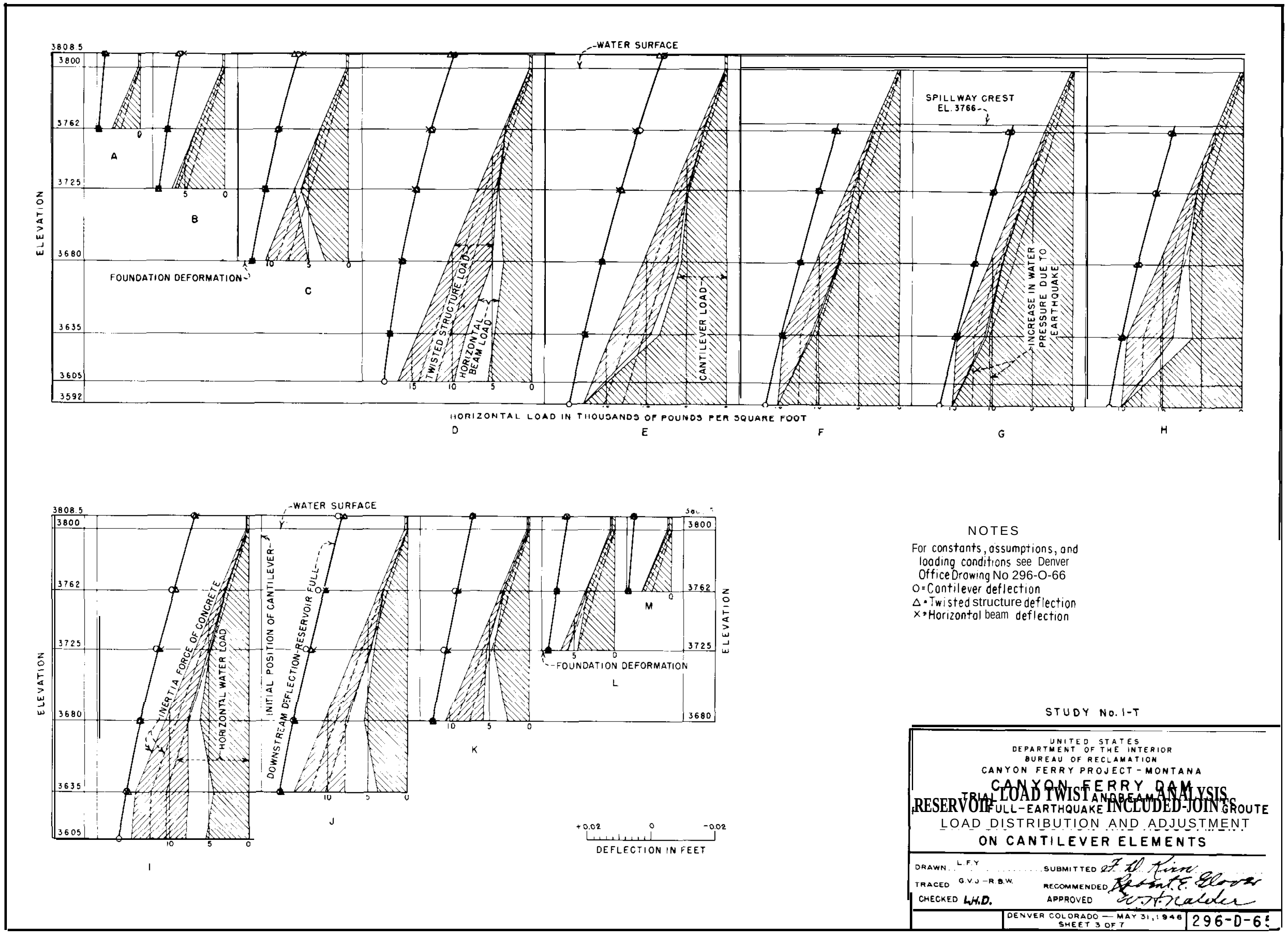
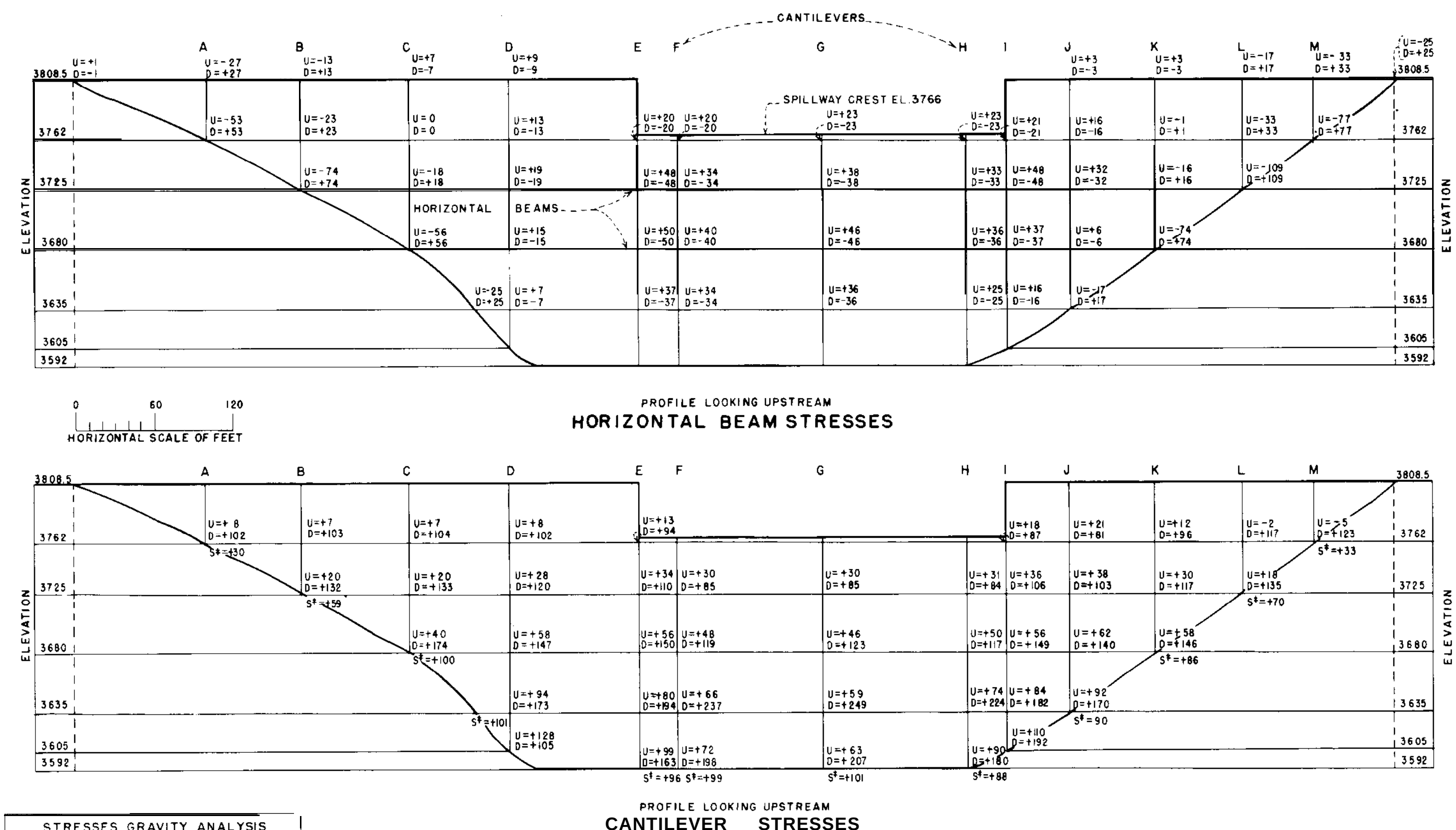


Figure B-21. **Canyon Ferry Dam study-load distribution and adjustment on cantilever elements.**



STRESSES GRAVITY ANALYSIS			
ELEVATION	LEFT ABUTMENT SECTION	SPILLWAY SECTION	RIGHT ABUTMENT SECTION
3762	U= + 3 D= + 110		U=+3 D=+110
3725	U=+15 D=+140	U=+28 D=+89	U=+15 D=+140
3680	U=+20 D=+206	U=+37 D=+134	U=+20 D=+206
3635	U=+28 D=+274	U=+42 D=+278	U=+30 D=+265
3605	U=+53 D=+179		U=+39 D=+307
3592	U=+35 D=+226	U=+35 D=+234	

NOTES

U=Stress at upstream face.
D=Stress at downstream face.
+ Indicates compression
— Indicates tension
All stresses are in pounds per. square inch
Cantilever stresses are acting in inclined directions parallel to the edges of the cantilevers.
S[†]=Maximum shear stress at rock abutment, (+) indicates downstream shear.
Ice load 7 tons per lineal foot concentrated at elevation 3798.75.
Water surface at elevation 3800.00.

EARTHQUAKE ASSUMPTIONS

Horizontal acceleration upstream
Vertical acceleration upward
Increased water pressure acts equally on all cantilevers
Period of vibration = 1.0 sec. Acceleration = 0.1g.
Modulus of elasticity for concrete = 3,000,000 pounds per square inch.
Poisson's ratio for concrete = 0.2
Joints assumed grouted so that dam can act as monolith

STUDY No. 1-T

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CANYON FERRY PROJECT – MONTANA
CANYON FERRY DAM
TRIAL LOAD TWIST AND BEAM ANALYSIS
RESERVOIR FULL–EARTHQUAKE INCLUDED–JOINTS GROUTED
STRESSES IN HORIZONTAL BEAM ELEMENTS AND IN CANTILEVER ELEMENTS

DRAWN L.F.Y. SUBMITTED F. D. Kirn
TRACED E.C.S. RECOMMENDED [illegible]
CHECKED L.H.D. APPROVED W. H. Nalder
ASST. CHIEF ENGINEER

DENVER, COLORADO MAY 31, 1946
SHEET 4 OF 7
296-D-66

Figure B-22. **Canyon Ferry Dam study-stresses in horizontal beam elements and in cantilever elements.**

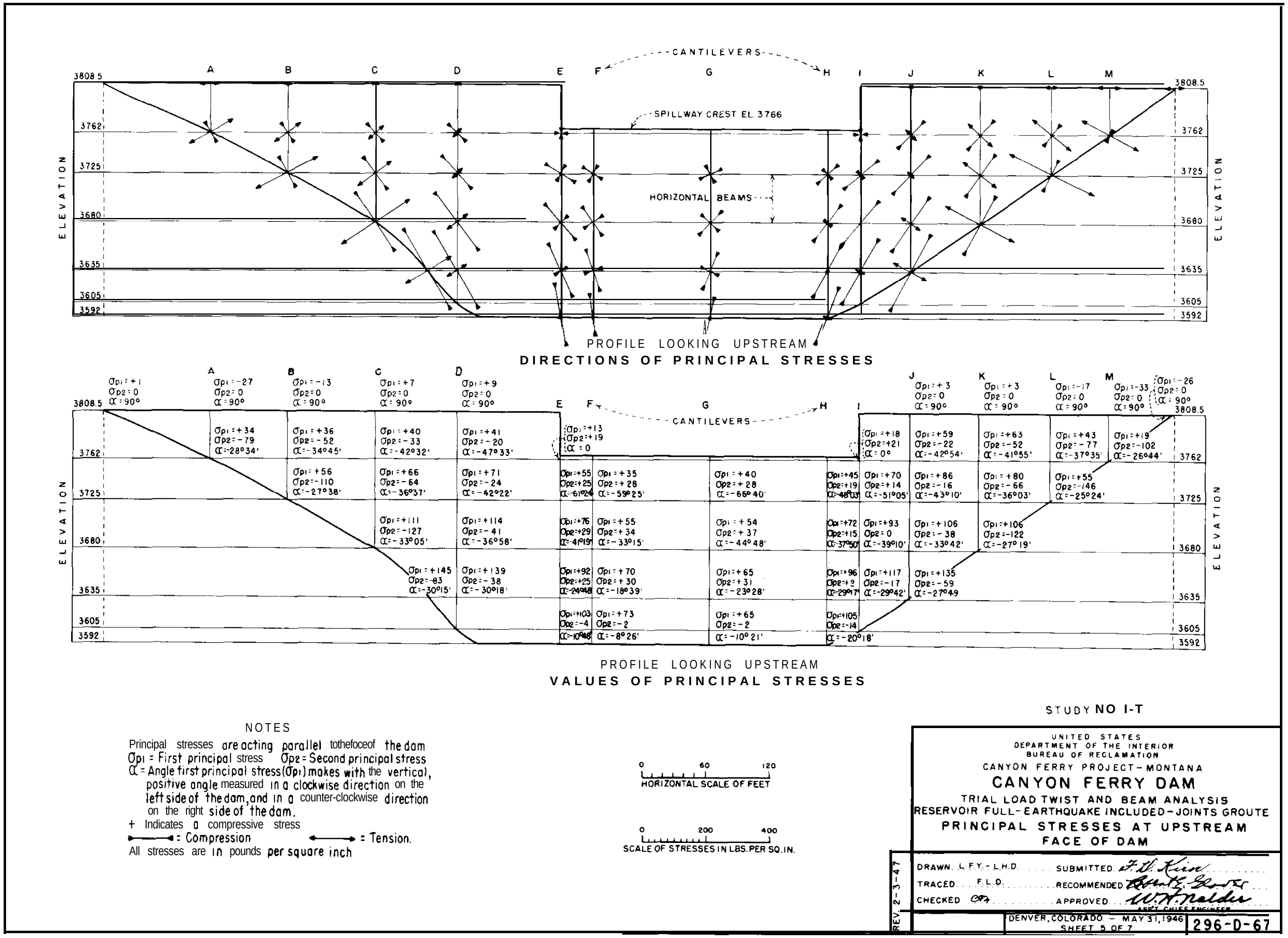


Figure B-23. **Canyon Ferry Dam study-principal stresses at upstream face of dam.**

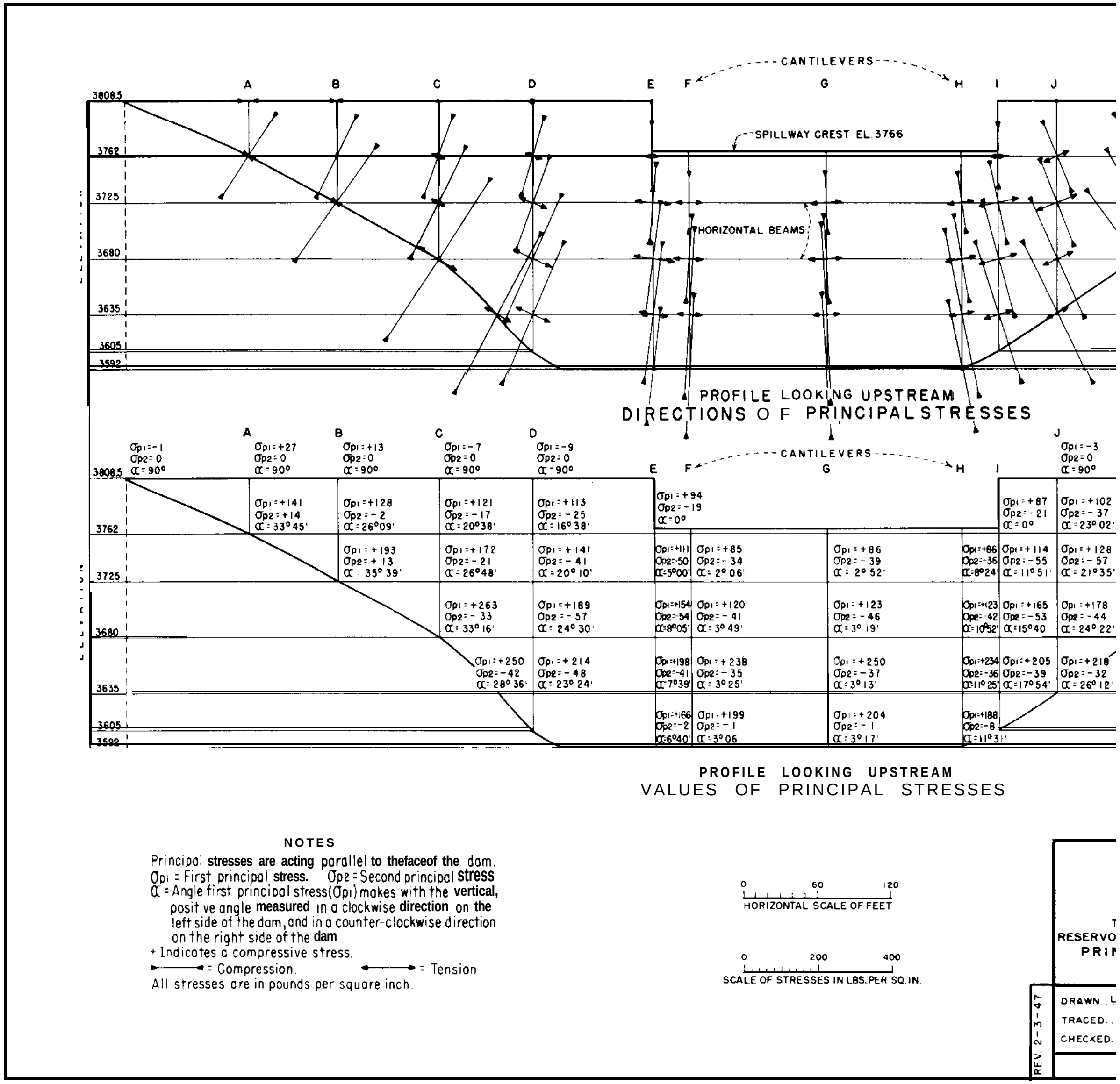


***Figure* B-24. Canyon Ferry Dam study-principal stresses at downstream face of dam**

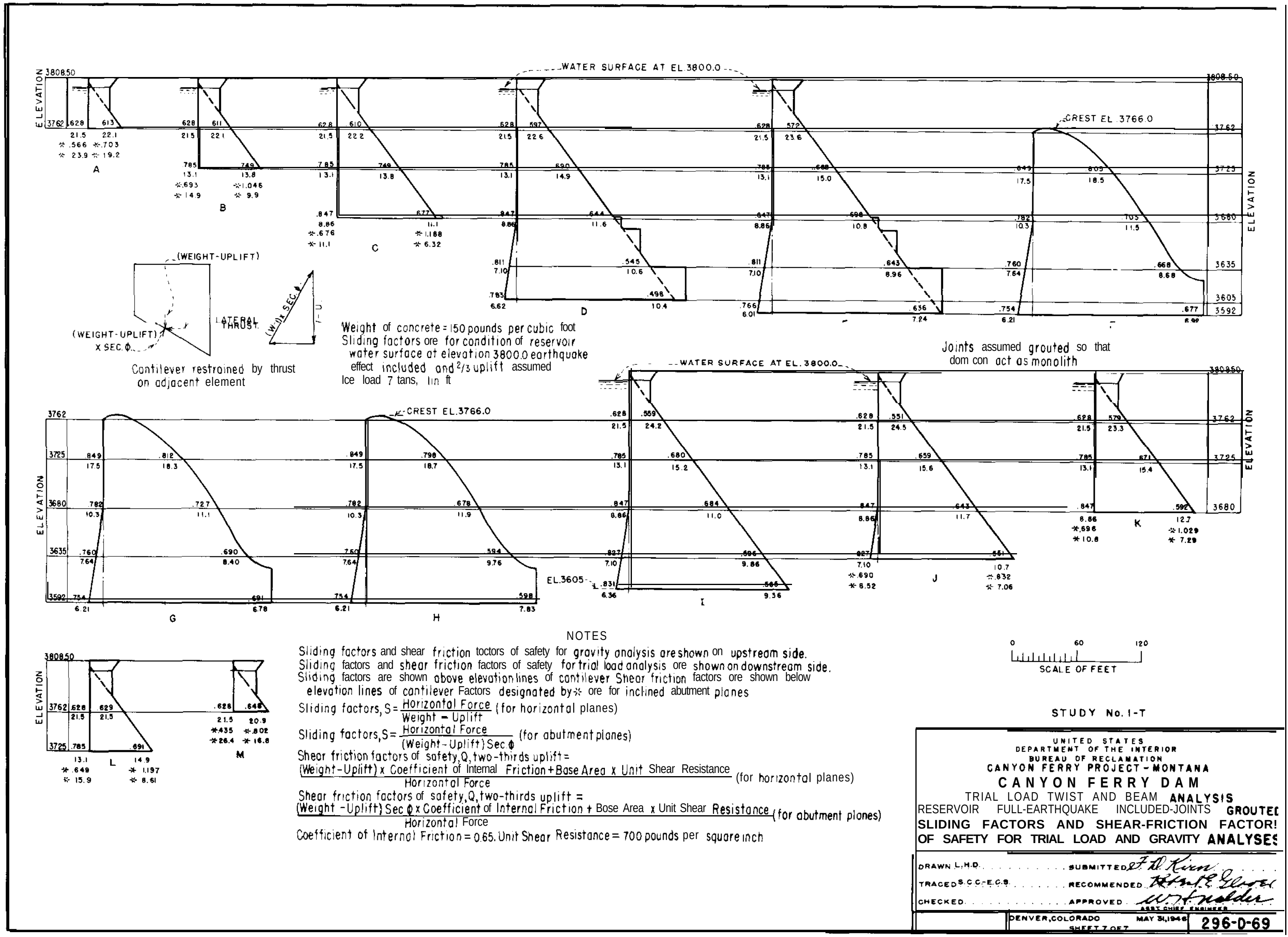


Figure B-25. **Canyon Ferry Dam study-sliding factors and shear-friction factors of safety for trial-load and gravity analyses.**

Finite Element Method of Analysis

A. TWO-DIMENSIONAL FINITE ELEMENT ANALYSIS

C-* 1. *Introduction. -The two-dimensional finite element analysis, discussed in sections 4-36 through 4-44, is illustrated by the following foundation study of the Grand Coulee Forebay Dam. Figure C-l shows a partial grid of section DG through the dam, reservoir, and foundation.

C-2. Description of Problem. -Foundation rock under Grand Coulee Forebay Dam and reservoir has a wide range of deformation moduli, with several faults or planes of weakness. One fault area, because of its low modulus, causes the concrete in the dam immediately above it to bridge over the fault causing horizontal tensions. By treating this fault (replacing part of the low-modulus fault material with concrete) these stresses in the dam will be minimized. This study was made to determine the depth of treatment necessary to obtain satisfactory stress.

C-3. Grid and Numbering System. -Figure C-l shows a portion of the grid used in this study. The nodes are numbered starting in the upper right corner and from left to right at each elevation. The entire grid has 551 nodal points. The elements are designated by a number in a circle. The numbering starts in the upper right corner and proceeds from left to right in horizontal rows. The entire grid has 517 elements. Numbers in squares designate the material numbers. The boundaries for each material are defined by elements. There are 23 materials assumed in this study.

C-4. Input.-Printouts of portions of the input are shown on figures C-2, C-3, and C-4. Figure C-2 shows the number of nodal points, the number of elements, and the number of different materials as indicated above. An acceleration of – 1.0 in the Y-direction is a means of including the weight of the materials. Each material is defined for mass density, moduli of elasticity in compression and tension, and Poisson's ratio. Figure C-3 is a listing of the nodal points showing type of restraint (if any), X and Y coordinates, load or displacement in the X or Y direction, and temperature. As an example node 19 is free to move in either direction; it is 653.0 feet to the right of the X reference line and 799.0 feet upward from the Y reference line; and a horizontal load of 27.0 kips is acting on the node in a direction to the left. There is no load in the Y direction and no temperature change.

Figure C-4 is a listing of the nodes enclosing an element and the element material. As an example, element 45 is bounded by nodes 53, 52, 63, 64 and is composed of material number *6.*

C-5. Output.-The results of an analysis are given as the displacements of the nodes in the X and Y directions and the stresses in the elements.

A printout of displacements for nodes 51 through 100 for the condition of no treatment of the foundation is shown on figure C-5. A similar printout for a loading condition where the foundation is treated for 25 feet is shown on figure C-6. Without treatment, node 69 is displaced 0.007,05 foot in the X direction to the left and 0.037,6 foot downward. After the

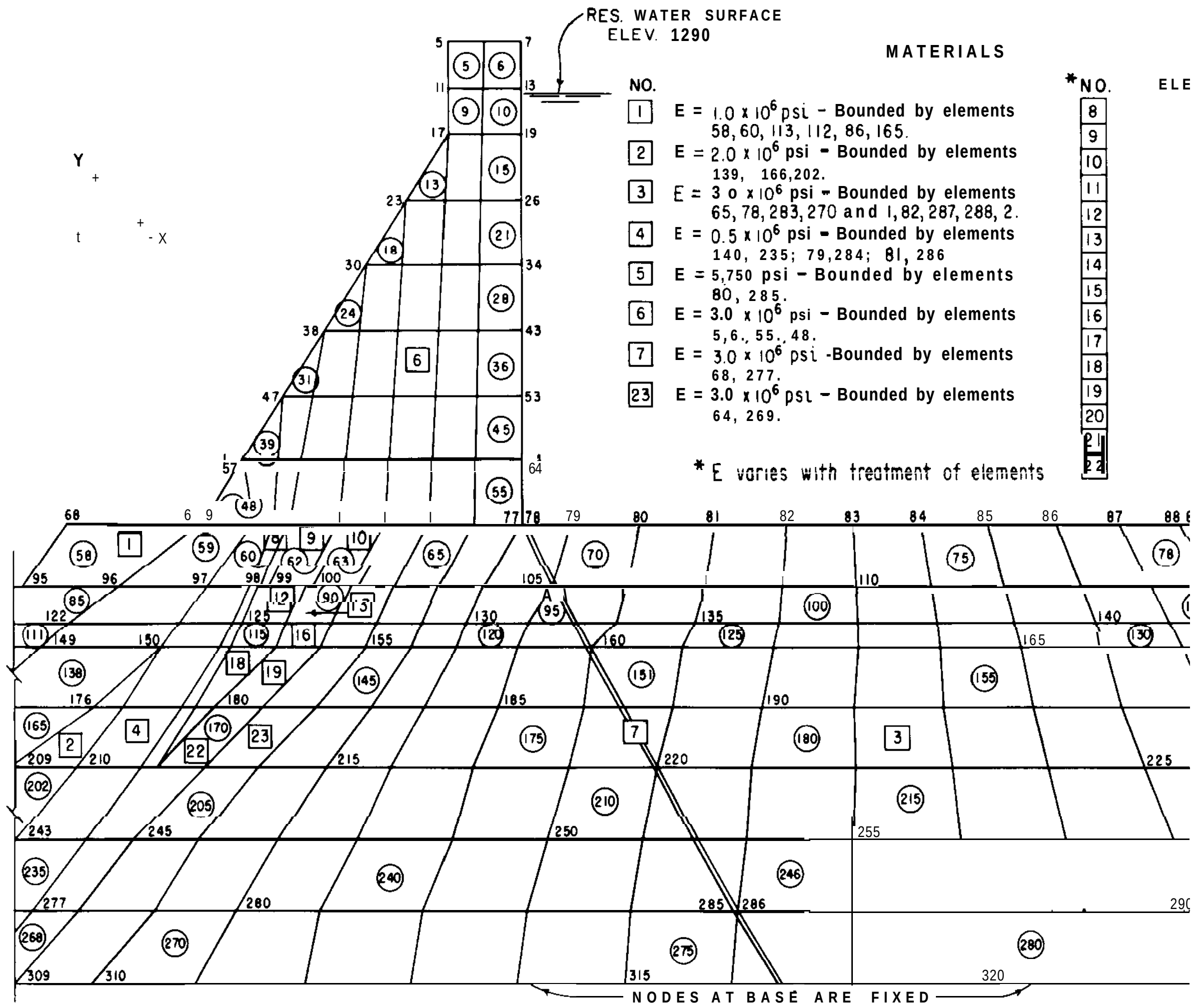


Figure C-1. Grid layout for section *DG* of Grand Coulee Forebay Dam, including excavated cut slope along

ANALYSIS OF PLANE PROBLEMS

PAGE NUMBER 1
DATE 05/27/70

COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, NO TREATMENT

********** INPUT DATA **********

NOTE-- INPUT UNITS MATCH OUTPUT UNITS UNLESS SPECIFIED

DATA PREPARED BY----

DATA CHECKED BY-----

COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, NO TREATMENT

NUMBER OF NODAL POINTS------551

NUMBER OF ELEMENTS----------517

NUMBER OF DIFF. MATERIALS--- 23

NUMBER OF PRESSURE CARDS---- -3

X-ACCELERATION-------------- -0.0000+000

Y-ACCELERATION-------------- -1.0000+000

REFERENCE TEMPERATURE------- -O.OOOO+OOO

NUMBER OF APPROXIMATIONS---- 1

MATERIAL NUMBER = 1, NUMBER OF TEMPERATURE CARDS = 1, MASS DENSITY = -O.OOOO+000

TEMPERATURE	E(C)	NU	E(T)	G/H2	ALPHA	X-STRESS	Y-STRESS
-0.000	144000.0000000	0.1300000	144000.0000000	-0.0000000	-0.0000000	-0.0000000	-0.0000000

MATERIAL NUMBER = 2, NUMBER OF TEMPERATURE CARDS = 1, MASS DENSITY = [illegible]

TEMPERATURE	E(C)	NU	E(T)	G/H2	ALPHA	X-STRESS	Y-STRESS
-0.000	288000.0000000	0.1300000	288000.0000000	-0.0000000	-0.0000000	-0.0000000	-0.0000000

MATERIAL NUMBER = 3, NUMBER OF TEMPERATURE CARDS = 1, MASS DENSITY = -0.0000+000

TEMPERATURE	E(C)	NU	E(T)	G/H2	ALPHA	X-STRESS	Y-STRESS
-0.000	432000.0000000	0.1300000	432000.0000000	-0.0000000	-0.0000000	-0.0000000	-0.0000000

MATERIAL NUMBER = 4, NUMBER OF TEMPERATURE CARDS = 1, MASS DENSITY = -0.0000+000

TEMPERATURE	E(C)	NU	E(T)	G/H2	ALPHA	X-STRESS	Y-STRESS
-0.000	72000.0000000	0.1300000	72000.0000000	-0.0000000	-0.0000000	-0.0000000	-0.0000000

MATERIAL NUMBER = 5, NUMBER OF TEMPERATURE CARDS = 1, MASS DENSITY = [illegible]

TEMPERATURE	E(C)	NU	E(T)	G/H2	ALPHA	X-STRESS	Y-STRESS
-0.000	828.0000000	0.2500000	828.0000000	-0.0000000	-0.0000000	-0.0000000	-0.0000000

Figure **C-2. Two-dimensional input data-control data and material properties. -288-D-3160**

ANALYSIS OF PLANE PROBLEMS

PAGE NUMBER 4
DATE 05/27/70

COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, NO TREATMENT

NODAL POINT	TYPE	X-ORDINATE (FT)	Y-ORDINATE (FT)	X LOAD OR DISPLACEMENT (KIPS) (FT)	Y LOAD OR DISPLACEMENT (KIPS) (FT)	TEMPERATURE (DEG F)
1	1.00	1081 .DOD	880.000	-O.DOOOOOO+DOO	-D.DODDODD+OOO	-O.DOD
2	0.00	1072 .OOO	859 .DOO	-O.OOODOOO+OOO	-O.DDOOOOO+OOO	-0.000
3	0.00	1076.500	859 .ooo	0.0000000+000	D.DDOODDO+OOO	0.000
4	1.00	1081 .OOD	859 .ooo	-O.OOOOOOO+OOD	-0 .OOOOOOO+ODO	-0.000
5	0.00	623.000	836.000	-O.OOOODOO+DOO	-O.DOOOODO+OOO	-0.000
6	0.00	637.500	836.500	D .OOOOOOO+OOO	D.OOOOOOO+OOD	0.000
7	0.00	652.000	837 .ODO	-O.OOOOODO+OOO	-O.OOOODOO+OOO	-0.000
a	0.00	1060.000	837 .OOO	-O .DOOODO+DOO	-O.OOOOODD+ODD	-0.000
9	0.00	1070.000	837.000	D .OOODOOO+ODO	0.0000000+000	0.000
10	1.00	1080.000	837.000	-D.OOOODOO+DOD	-0.0000000+000	-0.000
11	0.00	623.000	819.000	-0.0000000+000	-O.OOODOOO+OOO	-0.000
12	0.00	638.000	819 .ooo	0 .OOOODOO+OOD	O.ODDDODD+OOO	0.000
13	0.00	653 .DDO	819.000	-1.1250000+000	-D.DOODDOO+DOO	-0.000
14	0.00	1051 .DDO	819 .ODO	1.1250000+000	-5.0600000-001	-0.000
15	0.00	1065.500	819 .DOO	O.DODODDD+OOO	O.OOOOOOO+ODO	0.000
16	1.00	1080.000	819.000	-O.OODODOD+OOO	-O.OOOOOOD+OOD	-0.000
17	0.00	623 .OOO	799 .ooo	-O.OOOOOOD+OOO	-O.DODDODO+OOD	-0.000
18	0.00	638.000	799 .ooo	0.0000000+000	O.OOOOOOO+OOD	0.000
19	0.00	653.000	799.000	-2.7000000+001	-0.0000000+000	-0.000
20	0.00	1042 .ODD	799 .ooo	2.7000000+001	-1.2938000+001	-0.000
21	0.00	1061.500	799.000	O.OOOOOOO+DOO	O.OOOODDO+OOD	0.000
22	1.00	1081 .DOO	799.000	-O .OODDOOO+ODO	-O.OOODODO+OOO	-0.000
23	0.00	606.000	771 .ooo	-D.OODOOOO+DDD	-O.OOOOOOD+ODO	-0.000
24	0.00	621.667	771.000	O.OOODDOO+OOO	O.OOOOOOO+ODO	0.000
25	0.00	637.333	771.000	O.DOODODO+OOO	O.OOOOOOO+DOO	0.000
26	0.00	653.000	771.000	-7.7000000+001	-O.OOOODDD+DDD	-0.000
27	0.00	1028 .ooo	771 .ooo	7.7000000+001	-3.8500000+001	-0 .ooo
28	0.00	1054.500	771.000	D.OOOOOOO+OOO	O.OOOOOOO+ODD	0.000
29	1.00	1081 .DOO	771.000	-O.DOODOOO+OOO	-O.OODOOOO+DOD	-0.000
30	0.00	587.000	743 .ooo	-0.0000000+000	-0.0000000+000	-0.000
31	0.00	603.500	743 .ooo	O.OODOOOO+OOO	D.DOOODDO+OOO	0.000
32	0.00	620.000	743.000	0 .DODODOO+OOO	0.0000000+000	0.000
33	0.00	636.500	743 .DOO	O.OODDOOO+DOO	0.0000000+000	0.000
34	0.00	653 .OOO	743 .ODD	-1.2867800+002	-0. DOODDOO+OOO	-0.000
35	0.00	1014.000	743.000	1.2867800+002	-6.3210000+001	-0 .ooo
36	0.00	1047.500	743.000	0.0000000+000	D.OOOODDO+ODO	0.000
37	1.00	1081 .000	743 .OOD	-D.OOOOOOO+DOD	-O.DOOOODD+OOO	-0.000
38	0.00	570.000	714.000	-O.DDOOOOO+ODO	-O.OOOOOOO+ODO	-0.000
39	0.00	586.600	714.000	D.DOOOODD+OOO	O.OOOOOOD+OOO	0.000
40	0.00	603.200	714.000	D.DODDOOD+OOO	O.OODOOOD+OOO	0.000
41	0.00	619.800	714.000	D.ODDDOOD+OOO	0.0000000+000	0.000
42	0.00	636.400	714.000	0.0000000+000	D.OOODOOO+DDO	0.000
43	0.00	653 .000	714.000	-1.7586800+002	-O.OOOOOOO+OOD	-0.000
44	0.00	1000.000	714.000	1.7900900+002	-8.7934000+001	-0.000
45	0.00	1040.500	714.000	0 .DDOOOOO+DOO	O.DOOOOOO+OOO	0.000
46	1.00	1081 .DOO	714.000	-O.ODOODOO+OOO	-O.OOOOOOO+DDO	-0.000
47	0.00	552 .DDO	687.000	-D.ODOOODD+OOO	-O.OODOOOO+OOO	-0.000
48	0.00	565.000	687 .OOO	-D.OODDOOO+OOO	-O.OOOOOOO+ODD	-0.000
49	0.00	582.600	687 .OOO	0 .0000000+000	0.0000000+000	0.000
50	0.00	600.200	687 .OOO	D.OOODDOO+OOO	O.OOOOOOO+ODO	0.000

Figure C-3. **Two-dimensional input data-loading and description of section by nodal points. -288-D-3161**

ANALYSIS OF PLANE PROBLEMS

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COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, NO TREATMENT

ELEMENT NO	I	J	K	L	MATERIAL
1	1	2	3	3	3
2	1	3	4	4	3
3	3	2	a	9	3
4	4	3	9	10	3
5	6	5	11	12	6
6	7	6	12	13	6
7	9	a	14	15	3
a	10	9	15	16	3
9	12	11	17	18	6
10	13	12	la	19	6
11	15	14	20	21	3
12	16	15	21	22	3
13	17	23	24	24	6
14	la	17	24	25	6
15	19	la	25	26	6
16	21	20	27	28	3
17	22	21	28	29	3
18	23	30	31	31	6
19	24	23	31	32	6
20	25	24	32	33	6
21	26	25	33	34	6
22	28	27	35	36	3
23	29	28	36	37	3
24	30	38	39	39	6
25	31	30	39	40	6
26	32	31	40	41	6
27	33	32	41	42	6
28	34	33	42	43	6
29	36	35	44	45	3
30	37	36	45	46	3
31	38	47	48	48	6
32	39	38	48	49	6
33	40	39	49	50	6
34	41	40	50	51	6
35	42	41	51	52	6
36	43	42	52	53	6
37	45	44	54	55	3
38	46	45	55	56	3
39	47	57	58	58	6
40	48	47	58	59	6
41	49	48	59	60	6
42	50	49	60	61	6
43	51	SO	61	62	6
44	52	51	62	63	6
45	53	52	63	64	6
46	55	54	65	66	3
47	56	55	66	67	3
48	57	69	70	70	6
49	58	57	70	71	6
SO	59	58	71	72	6

Figure C-4. Two-dimensional input data-elements defined by nodal points with material. -288-D-3162

ANALYSIS OF PLANE PROBLEMS

PAGE NUMBER 28
DATE 05/27/70

COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, NO TREATMENT

NODAL POINT	DISPLACEMENT--X (FT)	DISPLACEMENT--Y (FT)
51	-1.9811064-002	-2.0667878-002
52	-1.9941570-002	-1.6607736-002
53	-2.0141816-002	-1.2279052-002
54	1.2091843-003	-2.4071188-002
55	5.5973499-004	-2.3599376-002
56	0.0000000+000	-2.3429267-002
57	-1.2884864-002	-3.7561138-002
58	-1.3239801-002	-3.5288567-002
59	-1.3559575-002	-3.2812637-002
60	-1.3979795-002	-2.8402294-002
61	-1.3880246-002	-2.4046112-002
62	-1.3516598-002	-1.9974260-002
63	-1.3437772-002	-1.6478346-002
64	-1.3871664-002	-1.2207339-002
65	1.2985815-003	-2.4044677-002
66	5.8831362-004	-2.3523886-002
67	0.0000000+000	-2.3303277-002
68	-2.9159215-003	-3.2254766-002
69	-7.0522162-003	-3.7597583-002
70	-7.6117066-003	-3.5823494-002
71	-7.5138281-003	-3.4742080-002
72	-7.3896957-003	-3.3570870-002
73	-7.0165442-003	-2.8818410-002
74	-6.7566923-003	-2.1328360-002
75	-6.9195435-003	-1.8176320-002
76	-6.8467254-003	-1.5494703-002
77	-6.2642486-003	-1.3235448-002
78	-6.1773772-003	-1.3248088-002
79	-5.0546562-003	-1.3404921-002
80	-3.9716033-003	-1.3389674-002
81	-3.5272857-003	-1.3428153-002
82	-3.2858108-003	-1.3391417-002
83	-3.1882931-003	-1.3314550-002
84	-3.1574305-003	-1.3196625-002
85	-3.1735731-003	-1.2980730-002
86	-3.2264351-003	-1.2680855-002
87	-3.2859588-003	-1.2212955-002
88	-3.2372784-003	-1.1813686-002
89	-2.8409063-003	-1.2637024-002
90	1.1806029-003	-2.5426777-002
91	1.0295768-003	-2.4358245-002
92	1.0241282-003	-2.4091306-002
93	5.8904893-004	-2.3338998-002
94	0 .0000000+000	-2.3113335-002
95	-4.0600306-003	-3.1606605-002
96	-4.5308027-003	-3.3783533-002
97	-5.0386242-003	-3.5799620-002
98	-5.0921594-003	-3.5052235-002
99	-3.9714688-003	-3.2638101-002
100	-3.9967861-003	-3.1880739-002

Figure C-5. Nodal **point displacements (no treatment). -288-D-3163**

ANALYSIS OF PLANE PROBLEMS

PAGE NUMBER 3
DATE 05/27/70

COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, 25 FT TREATMENT

NODAL POINT	DISPLACEMENT--X (FT)	DISPLACEMENT--Y (FT)
51	-1.8724543-002	-2.0415804-002
52	-1.8937366-002	-1.7466395-002
53	-1.9147395-002	-1.4287073-002
54	1.2031721-003	-2.4049716-002
55	5.5581550-004	-2.3582274-002
56	0.0000000+000	-2.3412686-002
57	-1.3924929-002	-3.1812965-002
58	-1.4099223-002	-3.0061762-002
59	-1.4272226-002	-2.8128268-002
60	-1.4290854-002	-2.5340594-002
61	-1.4190080-002	-2.2586042-002
62	-1.4060031-002	-1.9924903-002
63	-1.4032529-002	-1.7248892-002
64	-1.4441423-002	-1.3770922-002
65	1.2863362-003	-2.4022650-002
66	5.8064917-004	-2.3506172-002
67	0.0000000+000	-2.3285657-002
68	-4.8971975-003	-2.9753268-002
69	-8.6758414-003	-3.2517384-002
70	-9.3259331-003	-3.0508820-002
71	-9.5034153-003	-2.9319722-002
72	-9.5773006-003	-2.8390606-002
73	-9.7625006-003	-2.4257549-002
74	-9.2909087-003	-2.1070602-002
75	-8.8710655-003	-1.8504728-002
76	-8.4077281-003	-1.6204246-002
77	-7.6451459-003	-1.4086363-002
78	-7.5582207-003	-1.4071215-002
79	-6.2934332-003	-1.3983721-002
80	-5.0034774-003	-1.3774928-002
81	-4.4302627-003	-1.3719672-002
82	-4.0935786-003	-1.3606782-002
83	-3.9344774-003	-1.3477032-002
84	-3.8588070-003	-1.3316513-002
85	-3.8335854-003	-1.3053523-002
86	-3.8570314-003	-1.2710341-002
87	-3.8942547-003	-1.2192559-002
88	-3.8333803-003	-1.1747112-002
89	-3.4139745-003	-1.2555023-002
90	1.1148291-003	-2.5411830-002
91	9.9502980-004	-2.4338743-002
92	9.9658980-004	-2.4071306-002
93	5.7744601-004	-2.3319313-002
94	0.0000000+000	-2.3094178-002
95	-5.1226583-003	-2.9567080-002
96	-5.5237841-003	-3.0422893-002
97	-5.9118189-003	-3.1153132-002
98	-5.6404379-003	-3.0135814-002
99	-5.5543256-003	-2.9776455-002
100	-5.1357266-003	-2.6788933-002

Figure C-6. **Nodal point displacements (25-foot treatment). -288-D-3164**

25-foot treatment, node 69 is displaced 0.008,67 foot to the left and 0.032,5 foot downward.

Printouts of stresses for the analysis with the no-treatment condition are shown on figure C-7. This listing gives the element number, the location of the stresses in X and Y ordinates, stresses in the X and Y planes, the shear stress in the XY plane, and the principal stresses with the angle from the horizontal to the maximum stress. In this case, a shear stress along a specified plane and a stress normal to that plane were found. A similar printout for the condition with the foundation treated for 25 feet is shown on figure C-8. Stresses in element 51 are the key to this foundation problem. By treating the foundation, the compressive stresses in element 51 are increased from 8 to 33 pounds per square inch in the horizontal direction, and from 26 to 120 pounds per square inch in the vertical direction.

Microfilm plots of the grid and stresses are also provided by the computer as part of the regular output. Principal stresses in the dam for the no-treatment condition are shown on figure C-9. Principal stresses shown on figure C- 10 are for the condition where the foundation is treated for 25 feet. These latter principal stresses are derived from the vertical stresses shown on figure C-l 1, the horizontal stresses shown on figure C-12, and the shear stresses shown on figure C- 13.

Occasionally the finite element mesh is so fine that sufficient detail cannot be portrayed on the microfilm. In order to gain greater detail of a particular area and its stresses, the area can be plotted to an enlarged scale and more accurate stresses thus obtained.

B. THREE-DIMENSIONAL FINITE ELEMENT ANALYSIS

C-6. Introduction. -The analysis of the Grand Coulee Forebay Dam demonstrates the capabilities of the three-dimensional finite element system of stress analysis, discussed in sections 4-45 through 4-48. Distribution of stresses around the penstock is of special interest because of the large size of the opening in relation to the size of the block.

C-7. Layout and Numbering System.-A three-dimensional drawing of half of a block with the opening for a penstock is shown on figure C-14. To clarify the penstock area, vertical sections normal to the penstock are also shown. Although no foundation is shown, a treated foundation was assumed in the analysis. The block is divided into hexahedron elements. Nodal points are numbered consecutively from left to right starting at the top. There are 588 nodes in the example problem. The elements are numbered starting at the top and follow the general pattern set up for the nodes. There are 374 elements in this example.

C-8. Input. -Examples of the required input data are shown on the printouts in figures C-15 through C-l 8. Figure C-l 5 shows the numbers of elements, nodes, boundary nodes, loaded nodes, and different materials. Also shown is the maximum band width expected. Data given for each of the materials are modulus of elasticity, Poisson's ratio, and the mass density. The nodal points are described using ordinates in the X, Y, and Z directions as shown on figure C-l 6. For example, node 45 is 14.0 feet from the centerline of the block in the X direction, 19.58 feet from the upstream face in the Y direction, and at 273.0 (elevation 1273) in the Z direction. The nodal points that enclose the elements, the element material, and the integration rule are shown on figure C-17. Element 41 is bounded by nodal points 49, 55, 103, 97, 50, 56, 104, and 98. It contains material number 1 and is to be integrated by rule 2.

Forces or loads are applied at the nodal points. In this problem the loads are due to weight of the concrete, the hydraulic pressure on the upstream face, the uplift pressure at the base of the dam, and the internal pressure in the penstock and gate shaft. An example of

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COULEE 3R0 **FOUNDATION** SEC. DG, GRID 9, HYDRO LOAD, NO TREATMENT

EL.NO.	X (FT)	Y (FT)	X-STRESS (PSI)	Y-STRESS (PSI)	XY-STRESS (PSI)	MAX-STRESS (PSI)	MIN-STRESS (PSI)	ANGLE (DEG)	SHEAR-PLANE (PSI)	NORMAL-PLANE (PSI)
51	569.25	649.50	-82.9413-001	-26.5010+000	-10.1157+000	-37.8874-001	-31.0064+000	-24.01	-13.3122+000	-20.2235+000
52	588.25	649.25	-18.2965+000	-18.0532+001	81.2799+000	15.4183+000	-21.4246+001	22.53	29.8317+000	-21.0304+001
53	606.75	649.00	-22.4601+000	-25.5836+001	-65.2115+000	-54.7449-001	-27.2822+001	-14.60	-11.4819+001	-20.7597+001
54	625.25	649.00	-92.6967-001	-14.1428+001	-89.3897+000	35.8131+000	-18.6511+001	-26.76	-11.0453+001	-87.8800+000
55	643.75	649.00	16.2583+000	33.5292+000	-10.0265+001	12.5530+001	-75.7425+000	-47.46	-82.5144+000	82.5048+000
56	995.13	648.75	-33.7595+000	-26.2658+000	27.2580+000	-24.9834-001	-57.5269+000	48.91	25.4795+000	-40.3968+000
57	1051.38	648.25	-34.9132+000	-27.1232+000	68.5384-001	-23.1349+000	-38.9015+000	59.80	78.8312-001	-31.0719+000
58	480.00	623.00	-47.9419+000	-14.3210+000	-24.0107+000	-18.2099-001	-60.4419+000	-62.50	-12.3886+000	-45.6785-001
59	519.00	623.00	-29.7315+000	-72.1687+000	-32.0523+000	-12.5108+000	-89.3894+000	-28.25	-38.3674+000	-53.2998+000
60	540.00	623.00	-35.8968-001	-33.0459+000	-93.5017-001	-87.2364-002	-35.7632+000	-16.20	-15.4615+000	-26.3976+000
61	548.75	623.00	56.7466-002	-86.0317-002	54.4996-002	75.1718-002	-10.4457-001	18.68	11.5035-002	-10.3717-001
62	560.90	623.00	28.7067-001	-14.7545+000	19.5284-001	30.8445-001	-14.9683+000	6.25	-27.1509-001	-14.5503+000
63	580.20	622.75	-48.5870-002	-21.8737-001	10.5970-001	22.3277-003	-26.9556-001	25.62	49.2352-002	-26.0324-001
64	599.00	622.50	-76.1447+000	-43.0465+001	-65.8382+000	-64.3064+000	-44.2303+001	-10.19	-14.5598+001	-37.3811+001
65	617.80	622.50	-30.8871+000	-21.0267+001	-11.7020+000	-30.1269+000	-21.1027+001	-3.72	-54.9792+000	-19.2400+001
66	636.60	622.50	21.3056+000	-85.4769+000	-59.5000-001	21.6361+000	-85.8074+000	-3.18	-31.8485+000	-75.3488+000
67	653.50	622.50	14.2442+000	37.3463+000	-44.3987+000	71.6720+000	-20.0815+000	-52.29	-32.6749+000	57.9981+000
68	661.00	622.50	11.4532+001	-18.7765+000	-73.9562+000	14.7438+001	-51.6827+000	-23.99	-97.3751+000	27.1316+000
69	665.67	626.67	17.2429+001	-37.2502+000	-77.6679+000	19.8064+001	-62.8853+000	-18.27	-11.9682+001	15.6296+000
70	685.78	622.50	66.0723+000	-63.1979+000	-33.8962+000	74.4211+000	-71.5467+000	-13.84	-61.6725+000	-37.5903+000
71	715.08	622.50	32.9545+000	-72.6528+000	-12.5610+000	34.4280+000	-74.1262+000	-6.69	-37.2800+000	-59.2979+000
72	744.64	622.50	13.9267+000	-76.8051+000	-66.6803-001	14.4141+000	-77.2925+000	-4.18	-28.4576+000	-67.3932+000
73	773.69	622.50	26.2043-001	-77.4449+000	-42.0971-001	28.4116-001	-77.6657+000	-3.00	-23.6621+000	-69.9767+000
74	801.25	622.50	-43.5231-001	-77.9217+000	-28.6859-001	-42.4063-001	-78.0334+000	-2.23	-20.8766+000	-71.5592+000
75	829.56	622.50	-93.4967-001	-77.7625+000	-19.7601-001	-92.9265-001	-77.8196+000	-1.65	-18.8145+000	-72.1917+000
76	858.86	622.50	-12.3082+000	-77.7409+000	-68.1714-002	-12.3011+000	-77.7480+000	-0.60	-16.9486+000	-73.0169+000
77	888.67	622.50	-12.6495+000	-75.7173+000	12.2243-001	-12.6258+000	-75.7409+000	1.11	-14.7083+000	-72.1037+000
78	917.72	622.50	-90.0447-001	-82.2540+000	82.2306-001	-80.9269-001	-83.1658+000	6.33	-11.1910+000	-81.4587+000
79	936.00	622.50	54.4767-001	-42.5930+000	-81.2811-001	67.8562-001	-43.9310+000	-9.35	-19.0493+000	-35.3109+000
80	941.75	622.50	46.3713-001	-17.2536+000	-93.3747-001	80.7889-001	-20.6953+000	-20.23	-13.5592+000	-11.1185+000
81	947.75	622.50	-63.9990-001	-20.3811+000	81.5241-001	-26.5131-001	-24.1297+000	24.69	35.6488-001	-23.5208+000
82	971.00	622.50	-23.7352+000	-56.2670+000	23.2021+000	-11.6653+000	-68.3369+000	27.48	11.9607+000	-65.6888+000
83	999.50	626.67	-26.3419+000	-36.6750+000	22.7138+000	-82.1449-001	-54.8025+000	38.59	17.0874+000	-47.3397+000
84	1049.88	622.50	-30.7179+000	-33.7637+000	72.9640-001	-24.7872+000	-39.6944+000	39.11	55.5741-001	-37.2079+000
85	458.50	602.50	-14.0753+000	-15.5933+000	-23.8834+000	90.6109-001	-38.7298+000	-44.09	-21.0631+000	-35.4997-001
86	497.50	602.50	-17.5592+000	-44.1041+000	-30.1450+000	21.0587-001	-63.7692+000	-33.12	-32.7426+000	-27.2534+000
87	528.00	602.50	-67.8666-001	-41.5736+000	-15.1161+000	-11.3608-001	-47.2241+000	-20.50	-21.7876+000	-31.6852+000
88	539.75	602.50	88.7687-002	-23.1891-001	15.6864-001	15.2742-001	-29.5864-001	22.19	55.6837-002	-28.8843-001
89	550.80	602.50	-13.1075-001	-14.0082+000	11.1145-002	-13.0978-001	-14.0092+000	0.50	-30.7811-001	-13.2132+000
90	569.40	602.50	-49.3957-002	-26.5319-001	18.4298-001	56.2345-002	-37.0949-001	29.82	10.5626-001	-34.3004-001
91	588.00	602.50	-50.9808+000	-27.7548+001	-10.2420+001	-11.5456+000	-31.6983+001	-21.06	-14.5340+001	-21.1161+001
92	606.60	602.50	-41.9202+000	-27.4568+001	-71.6463-001	-41.6998+000	-27.4788+001	-1.76	-64.3666+000	-25.5401+001
93	625.20	602.50	-41.7499+000	-14.6202+001	18.9042+000	-38.4338+000	-14.9518+001	9.95	-97.4159-001	-14.8657+001
94	648.25	602.50	-20.0766+000	-54.3948+000	-63.2339-001	-18.9485+000	-55.5229+000	-10.11	-14.0558+000	-48.9342+000
95	666.67	600.00	-16.8373+000	87.1260-002	-27.6822+000	21.0808+000	-37.0468+000	-53.87	-19.5464+000	13.5261+000
96	672.00	602.50	-78.4602-001	-14.0913+000	-38.3291+000	27.4874+000	-49.4247+000	-42.67	-34.7553+000	54.9163-001
97	684.28	602.50	21.0208+000	-44.9724+000	-39.9128+000	39.8104-000	-63.7620+000	-25.21	-51.0638+000	-20.5953+000
98	711.52	602.50	19.3361+000	-62.8762+000	-26.3745+000	27.0697+000	-70.6098+000	-16.34	-43.3940+000	-44.1818+000
99	742.45	602.50	11.4645+000	-73.2695+000	-15.3108+000	14.1461+000	-75.9512+000	-9.93	-34.4430+000	-59.9380+000
100	773.38	602.50	35.1875-001	-75.7009+000	-96.2477-001	46.7134-001	-76.8535+000	-6.83	-28.1402+000	-65.5818+000

Figure C- 7. ***Stresses*** **in elements (no treatment h-288-D-3165**

ANALYSIS OF PLANE PROBLEMS

COULEE 3RD **FOUNDATION** SEC. DG, GRID 9, HYDRD LOAD, 25 FT TREATMENT

EL.NO.	X (FT)	Y (FT)	X-STRESS (PSI)	Y-STRESS <PSI>	XY-STRESS (PSI)	MAX-STRESS (PSI)	MIN-STRESS (PSI)	ANGLE (DEG)
51	569.25	649.50	-32.7145+000	-12.0034+001	94.2840-001	-31.7080+000	-12.1041+001	6.09
52	588.25	649.25	74.9617-001	-16.6625+001	-16.8468+000	91.1118-001	-16.8240+001	-5.48
53	606.75	649.00	20.2207+000	-16.1694+001	-51.0114+000	33.5486+000	-17.5022+001	-14.64
54	625.25	649.00	21.3625+000	-12.1475+001	-76.0907+000	54.3011+000	-15.4413+001	-23.41
55	643.75	649.00	26.5266+000	-10.3843+000	-91.3180+000	10.1235+001	-85.0932+000	-39.29
56	995.13	648.75	-33.1891+000	-26.1342+000	27.5627+000	-18.7407-001	-57.4492+000	48.65
57	1051.38	648.25	-34.4154+000	-27.2529+000	69.1634-001	-23.0456+000	-38.6227+000	58.69
58	480.00	623.00	-43.1646+000	-12.0455+000	-21.7198+000	-88.7067-002	-54.3230+000	-62.81
59	519.00	623.00	-26.6169+000	-52.2818+000	-34.0554+000	-30.5646-001	-75.8422+000	-34.68
60	540.00	623.00	-39.8110-002	-17.9328+000	-22.0893+000	14.6001+000	-32.9311+000	-34.18
61	548.75	623.00	-21.4494-001	-45.9332+000	-13.3666+000	16.1279-001	-49.6909+000	-15.70
62	560.90	623.00	11.5484+000	-46.6719+000	12.8563+000	14.2610+000	-49.3844+000	11.91
63	580.20	622.75	15.2437+000	-24.6529+001	57.3429+000	27.2540+000	-25.8539+001	11.83
64	599.00	622.50	-20.0826+000	-24.6460+001	-42.8305+000	-12.2501+000	-25.4292+001	-10.36
65	617.80	622.50	-65.3651-002	-16.5254+001	-48.8801+000	12.7676+000	-17.8675+001	-15.35
66	636.60	622.50	43.4393+000	-85.5618+000	-29.8111+000	49.9952+000	-92.1177+000	-12.40
67	653.50	622.50	30.4139+000	15.7429+000	-54.1453+000	77.7183+000	-31.5615+000	-41.14
68	661.00	622.50	12.4017+001	-21.8999+000	-70.6748+000	15.2636+001	-50.5182+000	-22.04
69	665.67	626.67	19.4144+001	-42.7663+000	-76.2355+000	21.6555+001	-65.1781+000	-16.38
70	685.78	622.50	81.0329+000	-63.9932+000	-37.0082+000	89.9309+000	-72.8911+000	-13.52
71	715.08	622.50	44.1654+000	-71.8777+000	-14.5382+000	45.9591+000	-73.6713+000	-7.03
72	744.64	622.50	22.3087+000	-76.6165+000	-78.5996-001	22.9293+000	-77.2372+000	-4.51
73	773.69	622.50	89.9168-001	-77.3023+000	-50.2472-001	92.8327-001	-77.5939+000	-3.32
74	801.25	622.50	63.2754-002	-77.8593+000	-34.8766-001	78.7417-002	-78.0140+000	-2.54
75	829.56	622.50	-54.5643-001	-77.7359+000	-24.6500-001	-53.7246-001	-77.8199+000	-1.95
76	858.86	622.50	-93.7148-001	-77.7381+000	-10.8770-001	-93.5418-001	-77.7554+000	-0.91
77	888.67	622.50	-10.5079+000	-75.6816+000	88.8498-002	-10.4958+000	-75.6937+000	0.78
78	917.72	622.50	-74.7314-001	-82.0137+000	80.6889-001	-66.0970-001	-82.8772+000	6.11
79	936.00	622.50	66.1293-001	-42.4652+000	-79.2355-001	78.6046-001	-43.7128+000	-8.95
80	941.75	622.50	58.1203-001	-17.0752+000	-91.3237-001	90.0933-001	-20.2725+000	-19.30
81	947.75	622.50	-51.6067-001	-20.2284+000	84.1768-001	-13.9780-001	-23.9913+000	24.09
82	971.00	622.50	-22.8074+000	-56.1539+000	23.3454+000	-10.7926+000	-68.1687+000	27.23
83	999.50	626.67	-25.5592+000	-36.7323+000	22.8377+000	-76.3471-001	-54.6568+000	38.13
84	1049.88	622.50	-30.1020+000	-33.9169+000	73.6312-001	-24.4033+000	-39.6156+000	37.74
85	458.50	602.50	-12.4712+000	-15.1605+000	-21.4961+000	77.2229-001	-35.3540+000	-43.21
86	497.50	602.50	-13.6288+000	-35.8360+000	-25.7387+000	32.9915-001	-52.7640+000	-33.33
87	528.00	602.50	29.1846-001	-30.7301+000	-10.9243+000	61.5399-001	-33.9657+000	-16.50
88	539.75	602.50	68.9677-002	-75.2481-002	31.5427-002	75.5649-002	-81.8453-002	11.81
89	550.80	602.50	45.0945-001	-13.2139+000	17 [illegible]		-13.3777+000	5.47
90	569.40	602.50	-22.6987-002	-15.5969-001	77.7242+000	46.7330-001 13.0444-002	-19.1712-001	24.70
91	588.00	602.50	-87.3317+000	-47.8188+001	-37 7765+000	-83.7141+000	-48.1806+001	-5.47
92	606.60	602.50	-69.8602+000	-19.5830+001	-10.5696+000	-68.9795+000	-19.6711+001	-4.76
93	625.20	602.50	-41.3323+000	-12.2190+001	-98 4612-001	-40.1506+000	-12.3372+001	-6.84
94	648.25	602.50	-83.1921-001	-54.0970+000	-21 2528+000	26.1804-003	-62.4424+000	-21. [illegible]
95	666.67	600.00	-85.5344-001	-41.6698-001	-33 9891+000	27.6996+000	-40.4200+000	-46.85
96	672.00	602.50	31.5872-001	-17.2564+000	-42.5064+000	36.6660+000	-50.7637+000	-38.25
97	684.28	602.50	28.6650+000	-46.0280+000	-43.3399+000	48.5297+000	-65.8926+000	-24.62
98	711.52	602.50	26.3567+000	-61.4328+000	-29.6761+000	35.4470+000	-70.5231+000	-17.03
99	742.45	602.50	17.7289+000	-72.5470+000	-17.8019+000	21.1125+000	-75.9306+000	-10.76
100	773.38	602.50	86.6945-001	-75.3200+000	-11.3828+000	10.1848+000	-76.8353+000	-7.58

Figure C-8. Stresses in elements (25-foot treatment). -288-D-3166

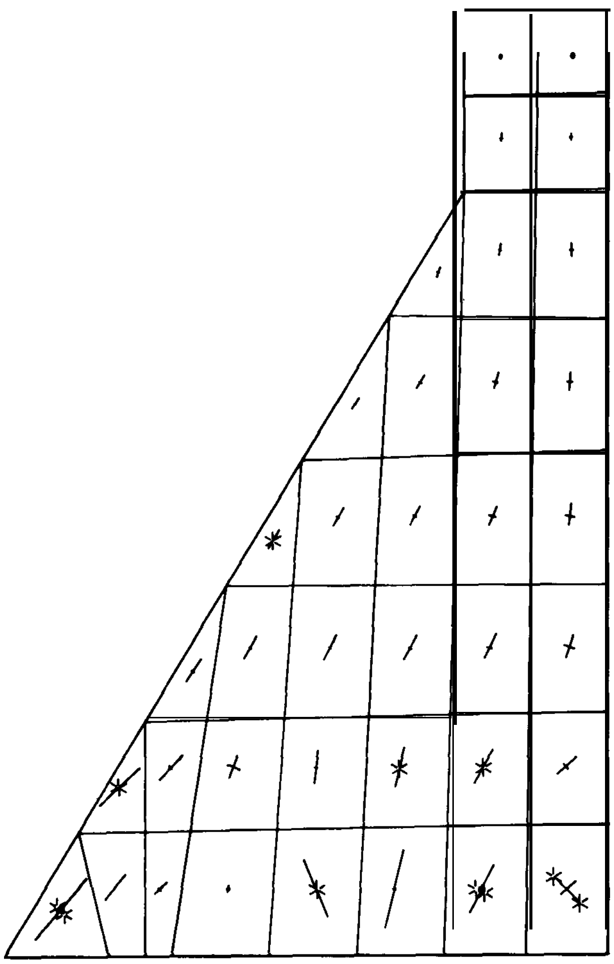


Figure **C-9. Grand Coulee Forebay Dam foundation study-microfilm printout showing principal stresses (no treatment). -288-D-3167**

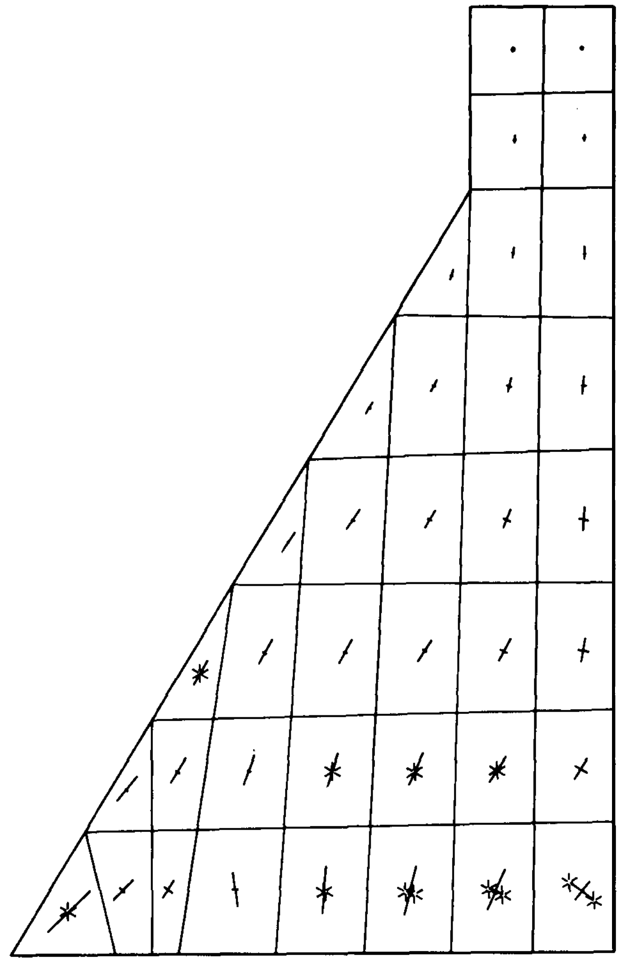


Figure C-IO. **Grand Coulee Forebay Dam study-microfilm printout showing principal stresses (25-foot treatment). -288-D-3168**

load vectors is shown on figure C-18. Nodal point 10 has a load of 4,105 pounds in the positive X direction, 2,711 pounds in the positive Y direction, and 143,590 pounds in the negative Z direction.

C-9. Output. -Displacements of the nodes are given in X, Y, and Z directions. Shear stresses and stresses normal to each of the three planes are computed at each node.

Some of the stresses of interest at the base of the dam and around the penstock are shown on figure C-19. The maximum compressive stress is about 255 pounds per square inch and the maximum tensile stress, 98 pounds per square inch.

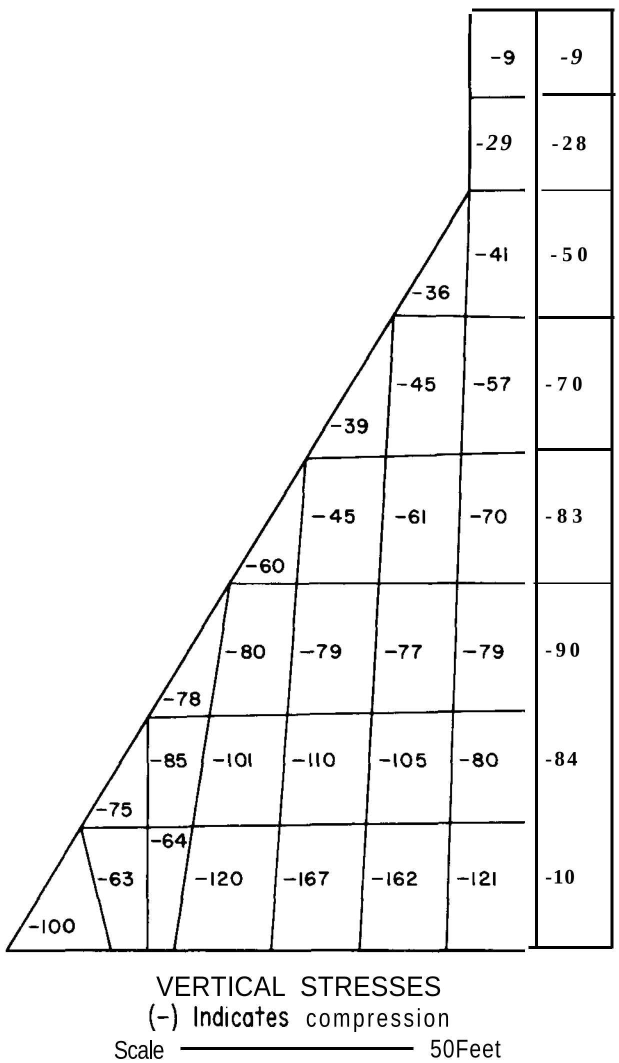


Figure C-11. **Grand Coulee Forebay Dam study-microfilm printout showing vertical stresses (25-foot treatment). -288-D-3169**

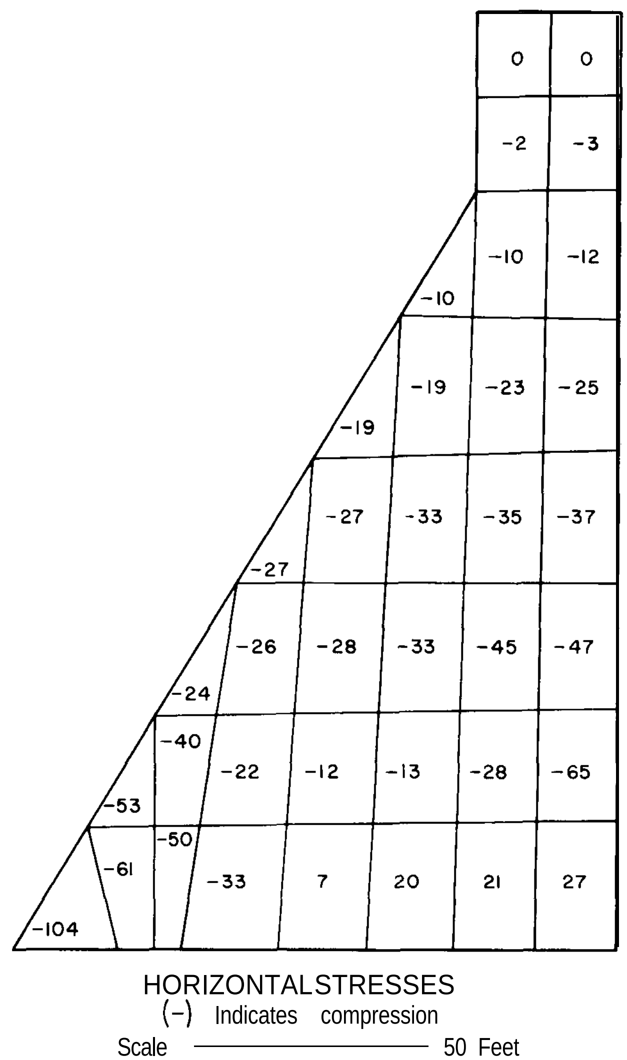


Figure C-12. **Grand Coulee Forebay Dam study-microfilm printout showing horizontal stresses (25-foot treatment). -288-D-3170**

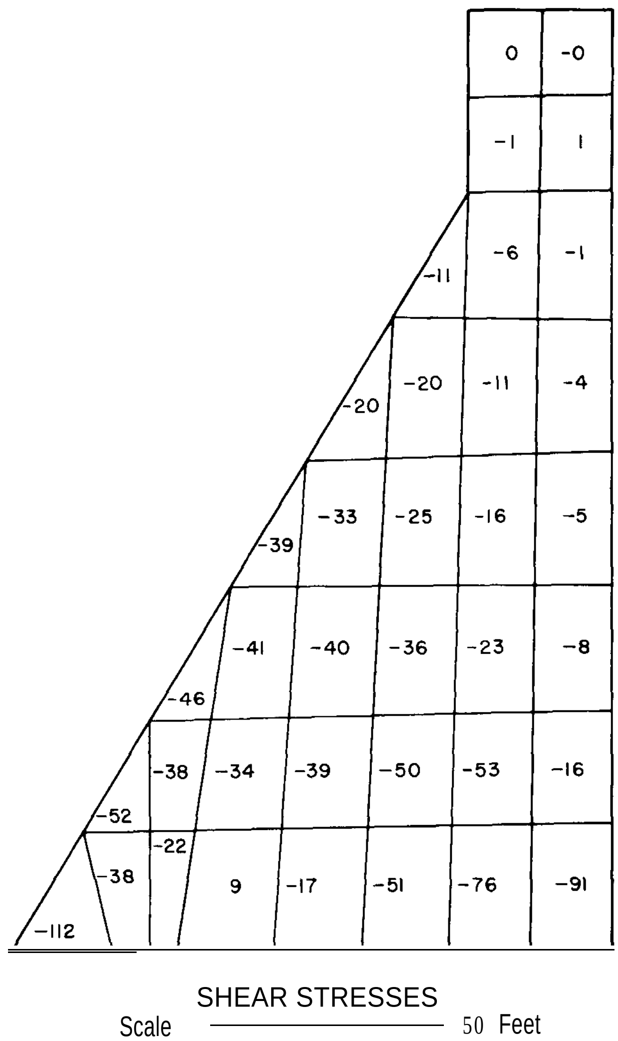


Figure **C-13. Grand Coulee Forebay Dam study-microfilm printout showing shear stresses (25-foot treatment). -288-D-3171**

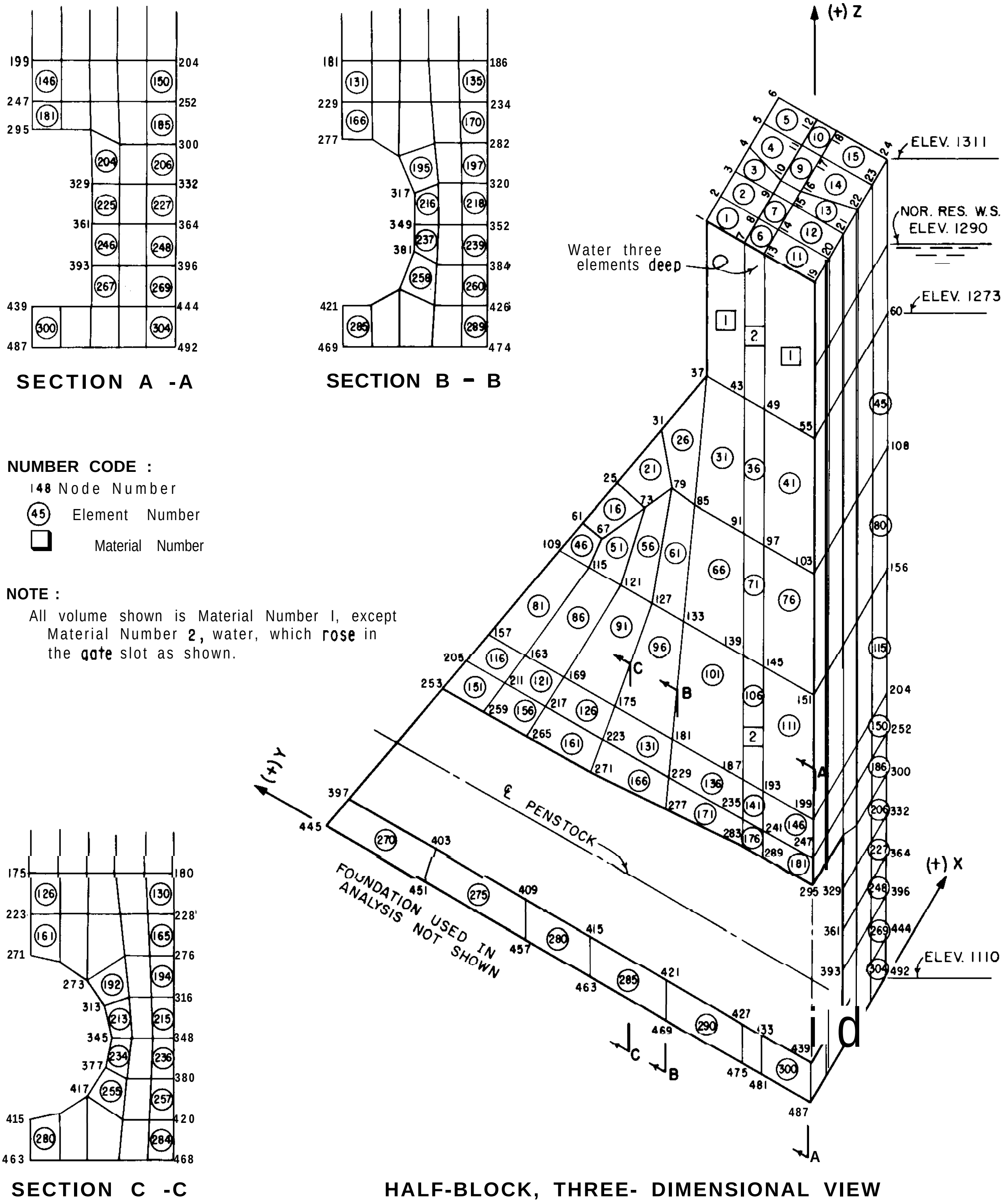


Figure **C-14. Grand Coulee Forebay Dam study-threedimensional finite element grid. -288-D-3172**

GRAND COULEE FOREBAY--FAULT U/S OF HEEL--LOADS DUE TO GRAVITY, HYDROSTATIC, UPLIFT

NUMBER OF ELEMENTS----------- 374
NUMBER OF NODES------------- 588
NUMBER OF BOUNDARY NODES----- 194
MAXIMUM BAND WIDTH----------- 168
NUMBER OF MATERIALS---------- 5
NUMBER OF LOADED NODES------- -0

MATERIAL NUMBER	MODULUS	POISSON	DENSITY
1	4.320+008	0.15	150.00
3	0.000+000	0.00	0.00
4	3.880+008	0.13 0.13	0.00 0.00
5	1.728+008	0.13	0.00

Figure C-15. **Three-dimensional input data-control data and material properties. -288-D-3173**

NODE	XORD	YORD	ZORD
1	0 .oooo	30.0000	311 .oooo
2	7.0000	30.0000	311 .oooo
3	14.0000	30.0000	311 .oooo
4	27.0000	30.0000	311.0000
5	28.0000	30.0000	311 .oooo
6	35.0000	30.0000	311.0000
7	0.0000	19.5800	311.0000
a	7.0000	19.5800	311 .oooo
9	14.0000	19.5800	311 .oooo
10	17.5000	19.5800	311 .oooo
11	28 .oooo	19.5800	311.0000
12	35.0000	19.5800	311 .oooo
13	0.0000	14.2800	311 .oooo
14	7.0000	14.2800	311.0000
15	14.0000	14.2800	311 .oooo
16	17.5000	14.2800	311.0000
17	28 .oooo	14.2800	311.0000
la	35.0000	14.2800	311.0000
19	0 .0000	0.0000	311.0000
20	7.0000	0.0000	311.0000
21	14.0000	0.0000	311.0000
22	21 .oooo	0.0000	311.0000
23	28 .oooo	0.0000	311.0000
24	35.0000	0 .0000	311.0000
25	0.0000	57.9500	230.0000
26	7.0000	57.9500	230.0000
27	14.0000	57.9500	230.0000
28	21.0000	57.9500	230.0000
29	28.0000	57.9500	230.0000
30	35.0000	57.9500	230.0000
31	0 .0000	44.9500	250.0000
32	7.0000	44.9500	250.0000
33	14.0000	44.9500	250.0000
34	21.0000	44.9500	250.0000
35	28.0000	44.9500	250.0000
36	35.0000	44.9500	250.0000
37	0 .0000	30.0000	273.0000
38	7.0000	30.0000	273.0000
39	14.0000	30.0000	273.0000
40	21.0000	30.0000	273.0000
41	28.0000	30.0000	273.0000
42	35.0000	30.0000	273.0000
43	0 .0000	19.5800	273.0000
44	7.0000	19.5800	273.0000
45	14.0000	19.5800	273.0000
46	17.5000	19.5800	273.0000
47	28.0000	19.5800	273.0000
48	35.0000	19.5800	273.0000
49	0 .0000	14.2800	273.0000
50	7.0000	14.2800	273.0000
51	14.0000	14.2800	273.0000
52	17.5000	14.2800	273.0000
53	28 .oooo	14.2800	273.0000
54	35.0000	14.2800	273.0000
55	0 .0000	0.0000	273.0000
56	7.0000	0 .oooo	273.0000
57	14.0000	0.0000	273.0000

***Figure* C-16. Threedimensional input data-description of section by nodal points. -288-D-3174**

ELEMENT	CONNECTED NODES								MATERIAL	INT. RULE
1	1	7	43	37	2	8	44	38	1	2
2	2	8	44	38	3	9	45	39	1	2
3	3	9	45	39	4	10	46	40	1	3
4	4	10	46	40	5	11	47	41	1	3
5	5	11	47	41	6	12	48	42	1	2
6	7	13	49	43	8	14	50	44	2	2
7	8	14	50	44	9	15	51	45	2	2
8	9	15	51	45	10	16	52	46	2	2
9	10	16	52	46	11	17	53	47	1	2
10	11	17	53	47	12	18	54	48	1	2
11	13	19	55	49	14	20	56	50	1	2
12	14	20	56	50	15	21	57	51	1	2
13	15	21	57	51	16	22	58	52	1	3
14	16	22	58	52	17	23	59	53	1	3
15	17	23	59	53	18	24	60	54	1	3
16	61	25	73	67	62	26	74	68	1	3
17	62	26	74	68	63	27	75	69	1	3
18	63	27	75	69	64	28	76	70	1	3
19	64	28	76	70	65	29	77	71	1	3
20	65	29	77	71	66	30	78	72	1	3
21	25	31	79	73	26	32	80	74	1	4
22	26	32	80	74	27	33	81	75	1	4
23	27	33	81	75	28	34	82	76	1	4
24	28	34	82	76	29	35	83	77	1	4
25	29	35	83	77	30	36	84	78	1	4
26	31	37	85	79	32	38	86	80	1	
27	32	38	86	80	33	39	87	81	1	4
28	33	39	87	81	34	40	88	82	1	4
29	34	40	88	82	35	41	89	83	1	4
30	35	41	89	83	36	42	90	84	1	4
31	37	43	91	85	38	44	92	86	1	3
32	38	44	92	86	39	45	93	87	1	3
33	39	45	93	87	40	46	94	88	1	3
34	40	46	94	88	41	47	95	89	1	3
35	41	47	95	89	42	48	96	90	1	
36	43	49	97	91	44	50	98	92	2	2
37	44	50	98	92	45	51	99	93	2	2
38	45	51	99	93	46	52	100	94	2	2
39	46	52	100	94	47	53	101	95	1	2
40	47	53	101	95	48	54	102	96	1	2
41	49	55	103	97	50	56	104	98	1	2
42	50	56	104	98	51	57	105	99	1	2
43	51	57	105	99	52	58	106	100	1	3
44	52	58	106	100	53	59	107	101	1	3
45	53	59	107	101	54	60	108	102	1	2
46	61	67	115	109	62	68	116	110	1	3
47	62	68	116	110	63	69	117	111	1	3
48	63	69	117	111	64	70	118	112	1	3
49	64	70	118	112	65	71	119	113	1	3
50	65	71	119	113	66	72	120	114	1	3
51	67	73	121	115	68	74	122	116	1	4
52	68	74	122	116	69	75	123	117	1	4
53	69	75	123	117	70	76	124	118	1	4
54	70	76	124	118	71	77	125	119	1	4
55	71	77	125	119	72	78	126	120	1	4
56	73	79	127	121	74	80	128	122	1	4
57	74	80	128	122	75	81	129	123	1	4

Figure C-17. **Three-dimensional input data-elements defined by nodal points with material.-288-D-3 175**

LOAD VECTOR

NODE	X-LOAD	Y-LOAD	Z-LOAD
1	0.0000+000	0.0000+000	-5.1970+004
2	0.0000+000	0.0000+000	-1.0394+005
3	0.0000+000	0.0000+000	-1.1508+005
4	0.0000+000	0 .0000+000	-1.0394+005
5	0.0000+000	0.0000+000	-9.2803+004
6	0.0000+000	0.0000+000	-5.1970+004
7	-2.6494-007	5.4220+003	-5.1970+004
8	-5.2988-007	1.0844+004	-1.0394+005
9	-5.2988-007	8.1331+003	-9.6515+004
10	4.1053+003	2.7110+003	-1.4359+005
11	0.0000+000	0.0000+000	-1.7745+005
12	0.0000+000	0.0000+000	-7.8403+004
13	0.0000+000	-5.4220+003	-7.1221+004
14	0.0000+000	-1.0844+004	-1.4244+005
15	0.0000+000	-8.1331+003	-1.1870+005
16	4.1053+003	-2.7110+003	-1.8209+005
17	0.0000+000	0.0000+000	-2.3227+005
18	0.0000+000	0 .0000+000	-9.7655+004
19	0.0000+000	5.4220+003	-7.1221+004
20	0.0000+000	1.0844+004	-1.4244+005
21	0.0000+000	1.0844+004	-1.3057+005
22	0.0000+000	1.0844+004	-1.4244+005
23	0.0000+000	1.0844+004	-1.5431+005
24	0.0000+000	5.4220+003	-7.1222+004
25	0.0000+000	0.0000+000	-3.5017+004
26	0.0000+000	0.0000+000	-7.0035+004
27	0.0000+000	0.0000+000	-7.0035+004
28	0.0000+000	0.0000+000	-7.0035+004
29	0.0000+000	0.0000+000	-7.0035+004
30	0.0000+000	0.0000+000	-3.5017+004
31	0.0000+000	0.0000+000	-5.6622+004
32	0.0000+000	0.0000+000	-1.1324+005
33	0.0000+000	0.0000+000	-1.1324+005
34	0.0000+000	0 .0000+000	-1.1324+005
35	0.0000+000	0.0000+000	-1.1324+005
36	0.0000+000	0 .0000+000	-5.6622+004
37	0.0000+000	0. 0000+000	-1.4149+005
38	0.0000+000	0.0000+000	-2.8299+005
39	0.0000+000	0.0000+000	-2.7593+005
40	0.0000+000	0.0000+000	-2.8299+005
41	0.0000+000	0.0000+000	-2.9005+005
42	0.0000+000	0 .0000+000	-1.4149+005
43	-4.5776-006	1.2828+005	-1.0176+005
44	-9.1553-006	2.5656+005	-2.0353+005
45	-9.1553-006	1.9242+005	-1.7456+005
46	9.7124+004	6.4139+004	-2.7761+005
47	0.0000+000	0.0000+000	-3.5597+005
48	0.0000+000	0.0000+000	-1.5115+005
49	-9.3561-008	-1.2828+005	-1.3307+005
50	-1.8712-007	-2.5656+005	-2.6614+005
51	-1.8712-007	-1.9242+005	-2.2179+005
52	9.7124+004	-6.4139+004	-3.4023+005
53	0. 0000+000	0 .0000+000	-4.3397+005
54	0.0000+000	0.0000+000	-1.8246+005
55	0.0000+000	1.2828+005	-1.3307+005

Figure C-18. **Threedimensional input data-load vectors. — 288-D-3176**

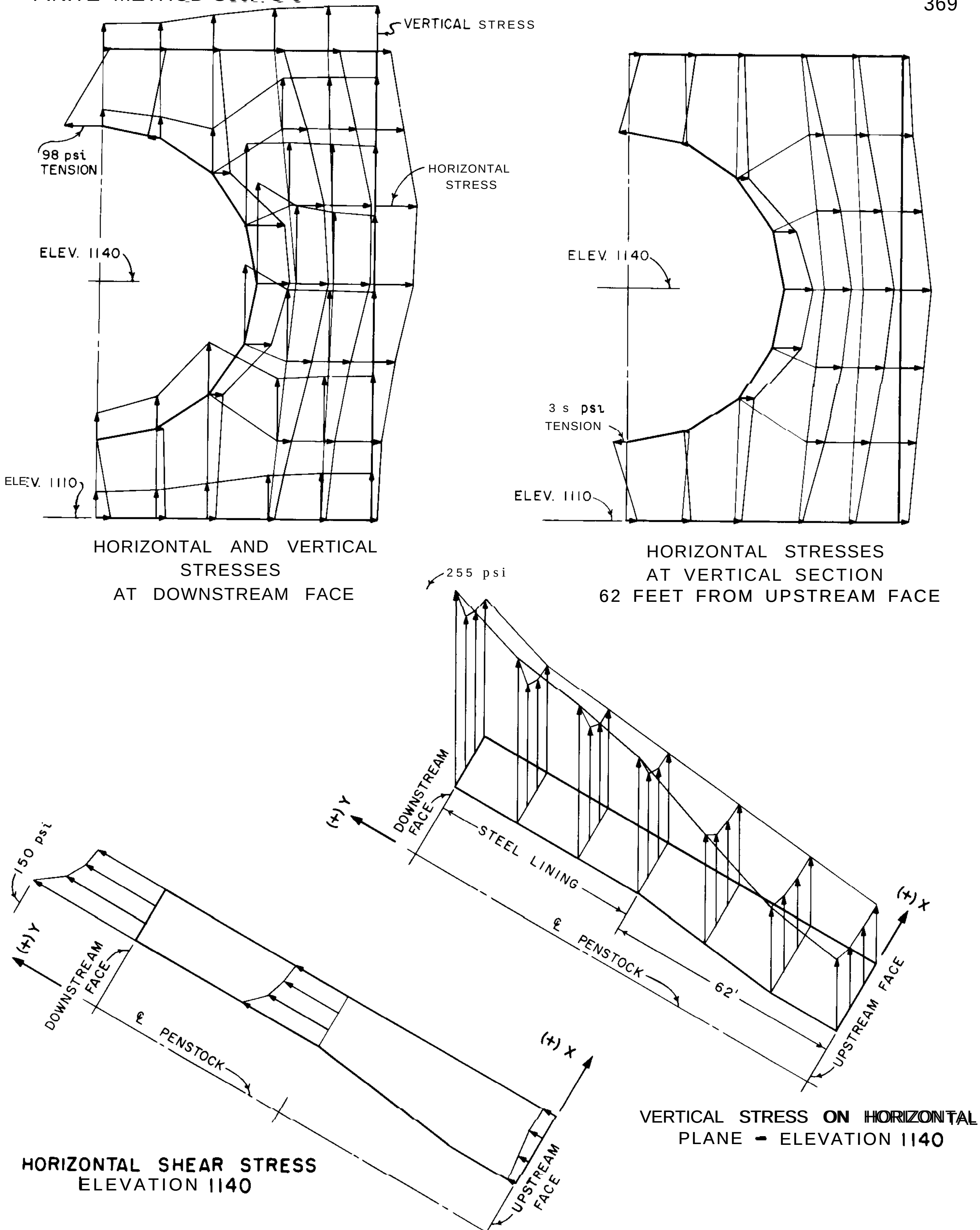


Figure C-19. Grand Coulee Forebay Dam study-stresses at nodal points. -288-D-3177

Special Methods of Nonlinear Stress Analysis

D- 1. Introduction. -The systems for determining nonlinear stresses presented here are the "Slab Analogy Method," "Lattice Analogy Method," "Experimental Models," and "Photoelastic Models." None of these methods, except photoelastic models, are used presently in the Bureau of Reclamation because of their complexity and the time consumed in performing the analyses. These methods are included in the discussion since they were used in some of the examples shown in this manual.

Modern two-dimensional and three-dimensional finite element methods provide more sophisticated and more economical analyses for the determination of nonlinear stress distribution than the methods mentioned above. The finite element methods are discussed in subchapter E of chapter IV and in appendix C.

D-2. ***Slab Analogy Method.*** -Although the exact law of nonuniform stress distribution is unknown, an approach towards a determination of true stresses can be made by means of the theory of elasticity. The "Slab Analogy Method" was developed as a result of a suggestion by H. M. Westergaard in 1930, in connection with the design of Hoover (formerly Boulder) Dam. This method is described in detail in one of the Boulder Canyon Project Final Reports.' Consequently, the method will be only briefly described here. It is a lengthy, laborious method and is justified only for unusually high and important dams. The analysis is based upon an analogy between an Airy's surface, which defines the stresses in a two-dimensional elastic structure, and the deflections of an unloaded slab bent by forces and couples applied around its edges. The slab has the same shape as a cantilever section including a large block of the foundation. The edges of the slab are bent into a form which corresponds to the stresses at the surface of the structure. The analysis is made by dividing the analogous slab into horizontal and vertical beams which are brought into slope and deflection agreement by trial loads. The curvatures in the slab are then proportional to the shears in the structure, and consequently the moments in the horizontal and vertical beams are proportional to the stresses in the vertical and horizontal directions, respectively.

Nonlinear stress investigations by the slab analogy method have been made for three large dams: Hoover, Grand Coulee, and Shasta. Conclusions drawn from several studies of maximum cantilever sections are that stresses in the vicinity of the upstream and downstream edges of the base are greater than those found by the gravity method and warrant special consideration in design. These studies also indicated that nonlinear effects are important within approximately one-third the height of the cantilever, and reach a maximum at the base.

The maximum nonlinear effects which were found in the vicinity of the bases of Hoover,

"'Stress Studies for Boulder Dam," Bulletin 4 of Part V, Boulder Canyon Project Final Reports, Bureau of Reclamation, 1939.

Grand Coulee, and Shasta Dams are shown in table D-l. The table also shows a comparison between stresses based on linear and nonlinear distribution for the vertical, horizontal, and shear stresses in the regions of the upstream and downstream toes. Since the nonlinear (slab analogy) method bears out the proof by the theory of elasticity that the theoretical maximum shear stresses are infinite at the reentrant corners of the base, the values given are for the maximum computed shear stresses at conjugate beam points nearest the corners. The vertical stresses were the ones which showed the greatest changes when computed by the nonlinear method. The maximum increase in vertical upstream stress was 18 percent, and occurred for Hoover Dam; while the maximum increase in vertical stress at the downstream toe was 64 percent and occurred for the maximum nonoverflow section of Grand Coulee Dam.

The studies of Shasta Dam showed the least departure of stresses from the linear law of any nonlinear studies completed to date. The upstream vertical stresses were decreased by approximately 12 percent and the downstream stresses were increased by corresponding amounts. This close agreement of linear and nonlinear stresses was believed to be due to the fact that the batter of the upstream face at the base of the cantilever was 0.5 to 1, which allowed for a better introduction of stresses from the dam into the foundation than would a sharper reentrant.

Table D-l shows that horizontal stresses as computed by the nonlinear method may be over twice the values computed by the ordinary linear assumption. This is an important consideration in the design of gallery and drainage systems, outlet works, power penstocks, elevator shafts, and other openings in the dam. The studies show that shear stresses computed by the nonlinear method follow rather closely the parabolic distribution obtained by an ordinary gravity analysis, except of course, at the reentrant corners.

D-3. ***Lattice Analogy Method.*** -Many of the two-dimensional problems encountered in engineering are difficult or impossible of solution when treated mathematically. Necessity has fostered the approximate "Lattice Analogy Method" of dealing with such problems. This section will describe the method and some of its applications rather than the derivation of formulas involved in its use. As far as practical engineering problems are concerned, the field of application is restricted only by two limitations: (1) The shape of the section must be such that it can be built up, exactly or to a satisfactory approximation, from a limited number of square elements; and (2) the value of Poisson's ratio must be equal to one-third. The limitation upon Poisson's ratio is usually unimportant. In many cases, stress distribution is independent of the values of the elastic constants, and in cases where these constants affect the results, the value of one-third will ordinarily be close enough to the true value that only small differences will exist in stresses or displacements.

As in the usual treatment of two-dimensional problems in elasticity, a section of the structure to be analyzed is considered as though it were a slice or plate of unit thickness, in accordance with the generalized theory of plane stress. The plate is simulated in size and shape by a lattice network composed of interconnected elemental square frames, each diagonally connected at the corners. When the plate has irregular boundaries, its outline may be approximated to any desired degree of accuracy depending on the number of frames chosen. As the number is increased, however, the solution becomes more involved so that for any problem a practical decision must be made as to the refinement desired. The validity of the simulation may be shown by demonstrating that in the limit, as the dimensions of the square frames approach zero, the differential equations of equilibrium and compatibility become identical for the lattice and the plate, and the boundary conditions become expressible in the same form. Thus the two solutions become identical and for obvious reasons the lattice is referred to as analogous to the plate.

(a) *Conditions to be Satisfied.-In* the analogy between the lattice and the plate, three

TABLE D-1 .-*Maximum nonlinear stress effects in sections of various dams.*–DS2-2(T1)

Dam	BOULDER	GRAND COULEE		SHASTA		
Cantilever Section	Crown	Maximum Non-Overflow	Maximum Spillwoy With Bucket	Maximum Non-Overflow	Maximum Spillway	Maximum Non-Overflow
Loading Condition	Dead Load plus Trial Load† Water Load	Dead Load plus Full Water Load	Dead Load plus Full Water Load	Dead Load plus Full Water Load	Dead Load plus Full Water Load	Dead Load plus Full Water Load plus Earthquake
Region neor Upstream Edge of Base						
Maximum Change, vertical normal stress	555 to 654	261 to 302	245 to 260	227 to 204	239 to 172	155 to III
Maximum Chonge, horizontal normal stress	230 to 405	221 to 72	198 to 194	200 to 120	219 to 48	198 to 54
Maximum Change, shear stress*	68 to 32	0 to 160	5 to 95	-15 to -48	-9 to -72	36 to 73
Region near Downstream Edge of Base						
Maximum Change, vertical normal stress	271 to 377	332 to 546	289 to 196	248 to 282	339 to 371	356 to 397
Maximum Change, horizontal normal stress	139 to 299	226 to 406	184 to 369	179 to 256	199 to 310	222 to 317
Maximum Change, shear stress*	140 to 120	190 to 216	195 to 240	213 to 297	240 to 109	271 to 318

Notes:
† Based on trial-load arch dam analysis.
* Theoretical maximum shear is infinite at reentrant corner; therefore value given is the maximum computed stress in vicinity of corner.
Figures to left based on gravity stress analysis; figures to right based on nonlinear stress analysis. (Slab Analogy Method)

fundamental conditions must be satisfied in order that an assemblage of elemental lattices may constitute a plate. These conditions are:

(1) The normal and tangential stresses must be distributed throughout the plate in such a way that the forces acting upon each element are in equilibrium with respect to translation and rotation of the element.

(2) The extensions and detrusions of the elements resulting from these stresses constitute a single-valued system of displacements.

(3) Any special conditions of stress or displacement which are specified at the boundaries must be satisfied.

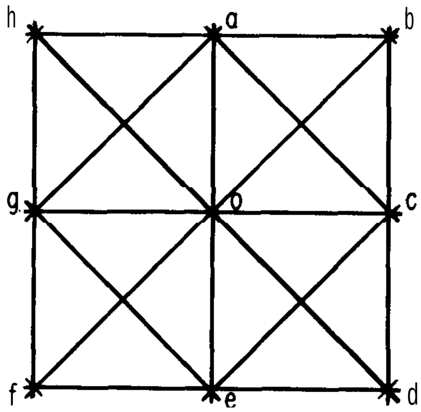
(b) *Solution.* -Having replaced the plate prototype by a lattice framework, a solution may be devised for the lattice and applied to the plate. The essential concept involved in this solution is a systematic relaxation of restraints at the joints. A description of a relaxing process to aid in an understanding of the adjustment will be given subsequently. After the adjustment of the lattice to remove restraint has been completed, the strains are deduced from relative displacements between successive joints and from these the stresses may be computed.

The fundamental device employed in the lattice analogy is the elemental square frame which is composed of six elastic members, two of which are horizontal, two vertical, and two diagonal. The length of the sides is considered unity in the derivation of the lattice formulas. The six members are assumed to be connected at the corners by frictionless joints. The elastic properties of the frame members are so determined that the behavior of the frame under given boundary conditions will correspond exactly to the square element of the plate section with respect to axial elongation, lateral contraction, and shear detrusion. In a lattice network composed of many frames, the amount of work involved in a conventional solution would be tremendous. However, by using a relaxation method, one may deal with a small region in which equilibrium is easily established and the method can consequently be applied to intricate lattice systems. Adjustment of equilibrium in a second region disturbs the first, but still leaves it approaching final equilibrium. The operations of adjustment are easily applied when each step is confined to a single joint.

To illustrate the method, consider the simplest case where the boundary conditions are given in terms of displacements. The procedure of adjustment may be visualized as follows: Consider a lattice actually constructed to a given scale, with elastic members coming together at the corners to form frictionless hinged joints. Lay this lattice out on a horizontal board, and before applying any displacements, completely restrain all joints by nailing them to the board. Next, displace and secure again, the various boundary joints through distances corresponding to their assigned displacements. Then, working in a line of joints adjacent to a displaced boundary, free one joint and allow it to move to a new position of equilibrium and resecure it. Repeat the process at successive joints until all have been adjusted (keeping only the boundary joints fixed in conformity with the given displacements) as many times as is required to give a satisfactory approach to the condition of complete transfer of forces from the interior nails to the members of the lattice. Simple relationships then exist between displacements and stresses.

(c) Equations. -Lattice equations for the displacement of an interior joint, an exterior corner joint, a reentrant corner joint, and a boundary joint have been developed in terms of loads at the joint and in terms of displacement of the surrounding joints. These equations are shown on figure D-l.

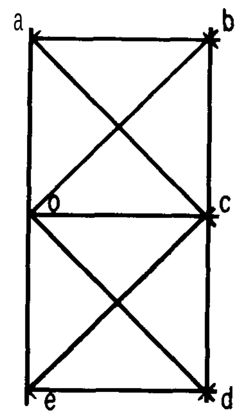
(d) *Boundary Conditions.* -Boundary conditions for the problem can be given either in terms of loadings or displacements. For the design of structures, estimated or computed loads would probably comprise the boundary conditions, but for structures already built it is more likely that boundary conditions would be given in terms of measured displacements. In either case, the loads or displacements for the plate must be expressed in terms of loads or displacements for the boundary joints of the lattice. However, the adjustment of the lattice



INTERIOR- JOINT

$$h_o=\frac{4}{9E}H_o+\frac{1}{12}(h_b+v_b+4h_c+h_d-v_d+h_f+v_f+4h_g+h_h-v_h)$$

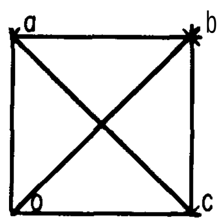
$$V_o=\frac{4}{9E}V_o+\frac{1}{12}(4v_a+h_b+v_b-h_d+v_d+4v_e-h_f+v_f-h_h+v_h)$$



BOUNDARY JOINT

$$h_o=\frac{8}{9E}H_o+\frac{1}{6}(h_b+v_b+4h_c+h_d-v_d)$$

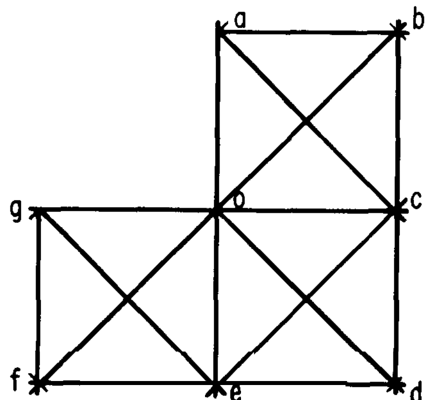
$$V_o=\frac{8}{9E}V_o+\frac{1}{6}(2v_a+h_b+v_b-ha+v_d+2v_e)$$



EXTERIOR CORNER

$$h_o=\frac{2}{3E}(3H_o-V_o)+\frac{1}{4}(-v_a+h_b+v_b+3h_c)$$

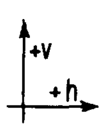
$$V_o=\frac{2}{3E}(3v_o-H_o)+\frac{1}{4}(3v_a+h_b+v_b-h_c)$$



REENTRANT CORNER

$$h_o=\frac{3}{5E}(H_o-\frac{1}{9}V_o)+\frac{1}{40}(-v_a+4h_b+4v_b+18h_c+5h_d-5v_d-2v_e+4h_f+4v_f+9h_g$$

$$V_o=\frac{3}{5E}(V_o-\frac{1}{9}H_o)+\frac{1}{40}(9v_a+4h_b+4v_b-2h_c-5h_d+5v_d+18v_e+4h_f+4v_f-h_g)$$



h,v indicate displacements of joint 0.

H,V indicate forces per unit thickness at 0 representing body forces at 0 in plane of lattice.

E is elastic modulus of prototype material.

Figure D-I. **Lattice analogy-equations for displacement of joint 0.-103-D-274**

is always made by adjusting displacements at the interior joints to remove restraints.

(e) *Stresses.*—Normally, the purpose of computing a lattice would be to determine stresses in the prototype. The adjustment of the lattice to remove restraint having been completed, the resulting displacements may be applied to the plate. The difference in displacements between successive lattice joints will yield strains, and stresses may be computed from the conventional stress-strain relationship.

(1) *Restraining* forces.-At any time during adjustment of a lattice, the restraining forces at the joints may be computed. For an exact solution, these forces will reduce to zero, and they are, therefore, a measure of the accuracy of the adjustment at any stage. Ordinarily, the computation of the restraining forces involves considerable work so that other methods are used to judge the end of an adjustment. The easiest way is to overadjust the displacements so that reversal occurs in their direction because of passing the end point.

(2) *Body forces.* -The equations previously mentioned concerning displacement at certain joints due to loads at these joints, will apply to the body forces of the structure. Such loads can be introduced into the lattice adjustment by computing the horizontal and vertical components, computing the displacements, and adding these displacements to those produced by displacements of the surrounding joints. Certain limited types of body forces, including gravity forces, may also be handled by Biot's method of applying fictitious boundary pressures.

(3) Thermal stresses. -A system has been devised in which displacements due to temperature change are computed by the application of fictitious body and boundary forces. The determination of the fictitious forces is somewhat involved and will not be given here, and the application of body and boundary forces to a lattice system has already been discussed.

(f) *Applications and Limitations.-The* lattice analogy method is used for solving two-dimensional nonlinear stress problems in engineering and has many applications that are involved in the design of masonry dams. The method is adaptable to the computation of stresses in a gravity dam. A section from a gravity dam is normally computed of unit thickness and its outline could be approximated by a lattice network made up of squares. As has been pointed out, boundary forces (waterloads), body forces (dead loads), and thermal forces cause no particular difficulty in adjusting lattice displacements. The principal limitations placed on application to gravity dam design or other purposes are the time and labor involved in the calculations. The method has been found useful in determining the stress distribution in a body composed of two or more different materials. This represents a problem of great practical importance, especially in the design of reinforced concrete structures. Another problem which is fundamental in the study of concrete structures is that of uniform shrinkage of a two-dimensional section on a rigid foundation. This problem has been analyzed successfully, using the lattice analogy.

D-4. ***Experimental Models.*** -The use of models is a very valuable addition to the analytical methods used in the design of dams and similar structures. Models are necessary for any careful design development and can be used for checking of theory. All models come under one of two major classifications: (1) similar models, or those that resemble the prototype; and (2) dissimilar models. Principal among the former group are the two-dimensional and three-dimensional types of elastic displacement models; photoelastic models; and models used in studies employing the slab analogy. In the dissimilar group are those employing such analogies as the membrane, electric, and sand-heap analogies. These last-mentioned types, while of considerable value to stress studies of special problems, do not concern us here, and it is only those model types which have proved adaptable to experimental studies of masonry dam structures that will be discussed.

(a) *Three-Dimensional Models.*– Three-dimensional displacement models are those constructed of elastic materials to proportionate size and loading of the prototype so that deformation, structural action, and stress conditions of the latter can be predicted by measurement of displacements of the model.

The following conditions must be fulfilled, in order to obtain similarity between a model and its prototype, while at the same time satisfying theoretical considerations and the requirements of practical laboratory procedure:

(1) The model must be a true scalar representation of the prototype.

(2) The loading of the model must be proportional to the loading of the prototype.

(3) Upon application of load, resulting strains and deflections must be susceptible of measurement with available laboratory equipment. Because of reduced scale this condition ordinarily requires a higher specific gravity and greatly reduced stiffness in the model compared with the prototype.

(4) Because of influence of volume strains on the stress distribution, Poisson's ratio should be the same for both model and prototype.

(5) The model material must be homogeneous, isotropic, and obey Hooke's law within the working-stress limits, since these conditions are assumed to exist in a monolithic structure such as a concrete dam.

(6) Foundations and abutments must be sufficiently extensive to allow freedom for the model to deform in a manner similar to the prototype.

(7) If effects of both live load and gravity forces are to be investigated, the ratio of dead weight to live load should be the same in both model and prototype. If the effects of live load only are to be investigated, the results are not affected by the specific gravity of the model, providing Hooke's law is obeyed and no cracking occurs.

If all requirements of similarity are fulfilled, the relations between model and prototype may be expressed in simple mathematical terms of ratios. Overall compliance with this restriction is not always possible in model tests of masonry dams, but since the purpose of many tests made on dam models, such as the Hoover Dam model tests, is to obtain data for verifying analytical methods, some variation from true similarity does not detract greatly from the value of the test. Complete details of model tests for Hoover Dam are given in the Boulder Canyon Project Final Reports.[1]

(b) *Two-Dimensional Displacement Models.*-Two-dimensional displacement models are often referred to as cross-sectional or slab models. Acting under two-dimensional stress such a model can be compared directly only to a similar slice through the prototype acting as a separate stressed member, since in the actual structure all interior points are under three-dimensional stress. The model slab, having no forces applied normal to the section, is considered to be in a state of plane stress. A cross-sectional element or cantilever acting as an integral part of a masonry dam is stressed by a more complex system of forces, and is under neither plane stress nor plane strain. A state of plane strain is closely approached, however, in the central cantilever element in a long, straight gravity dam and also in a vertical slice through the foundation under the crown cantilever of an arch dam. Assuming a state of plane strain can be realized, similarity of stress and strain can be had if Poisson's ratio is the same for model and prototype. For fairly reliable results in the evaluation of stress distribution in the cantilever section of a dam, the usefulness of the two-dimensional model is limited to the straight gravity type of dam, and then only when applied to the central cantilever element. This usefulness is further limited in its application to arch dam cantilevers, to the immediate neighborhood of the base of the crown cantilever, and to that part of the foundation slab contiguous with it. Two-dimensional arch models, while usually failing to give stress and deformation values which can be taken as representative of those occurring in the prototype, have furnished valuable information in connection with the evaluation of abutment rotation and deformation for use in analytical studies.

D-S. *Photoelastic* Models.-Photoelastic models are used extensively by the Bureau for

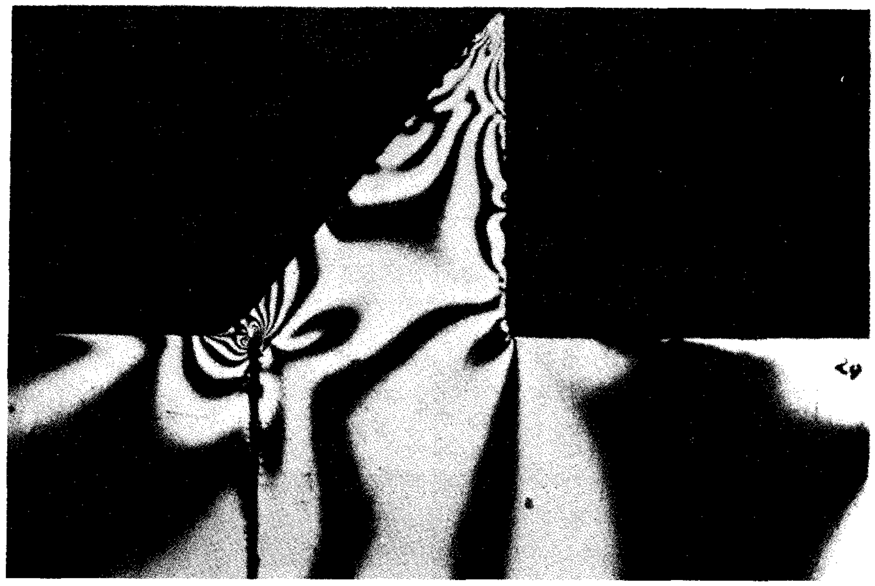
[1] Bulletin 2, "Slab Analogy Experiments," Bulletin 3, "Model Tests of Boulder Dam," and Bulletin 6, "Model Tests of Arch and Cantilever Elements," Part V, Technical Investigations, Boulder Canyon Project Final Reports, 1938-40.

design and analysis of localized portions of masonry dams and their appurtenant works. Stresses in photoelastic models are determined by means of the visible optical effects which are produced by passing polarized light through the model while it is under load. The model material must be elastic, transparent, isotropic, and free from initial or residual stresses. Bakelite, celluloid, gelatin, and glass have been successfully used. Studies employing photoelastic models are usually limited to conditions of plane stress or strain, and may be said to have their most important application in the determination of regions of stress concentrations.

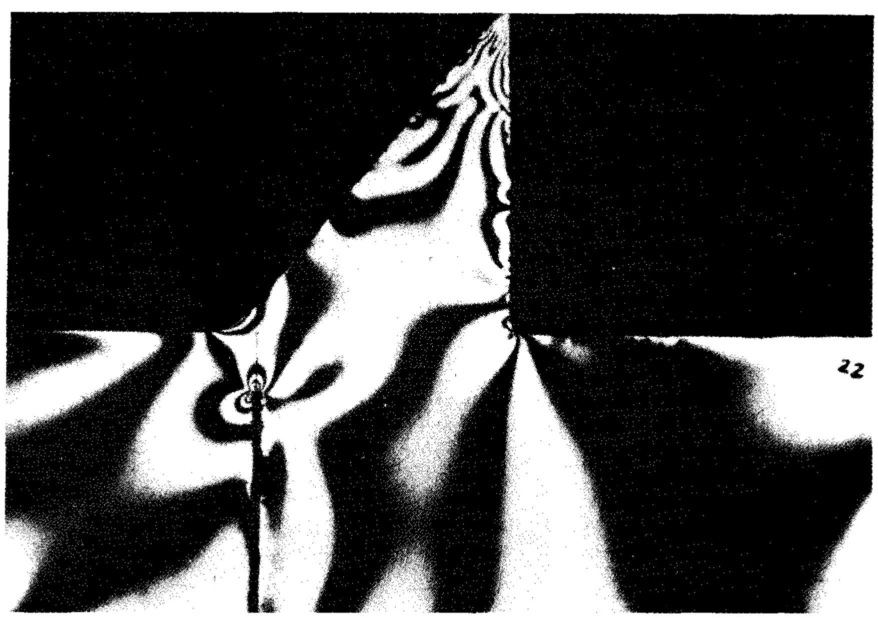
Effects of stress in a photoelastic model are made visible by means of an optical instrument known as the photoelastic polariscope. Through a system of Polaroids, the polariscope directs a beam of light through the model so polarized than when the material of the model becomes doubly refractive under stress, the familiar photoelastic pattern is projected to the observer on a screen or photographic plate. The alternate color bands of the pattern, or fringes as they are called, furnish a means of measuring the stress quantity, by a known relation between principal stresses and their retardative effect on polarized light-waves passing through the stressed model. This "unit" of measure, called *material fringe value,* is readily evaluated in the laboratory. For bakelite, the most extensively used material, the value is 87 pounds per square inch per inch of thickness, and represents the stress corresponding to one fringe. Values for any number of fringes, or fringe order, are found by direct proportion; and by applying a suitable factor of proportionality, corresponding values of the stress quantity in the prototype structure may be determined. This stress quantity is the difference in principal stresses at any point (twice the maximum shear stress), and has particular significance along free boundaries, where one of the principal stresses is zero.

Where it is desirable to know the magnitude and direction of the individual principal stresses acting at a point within the model, optical instruments such as the photoelastic interferometer or the Babinet compensator are used. The determination of stress from photoelastic models and the techniques used in this type of investigation are subjects too complex to properly come within the scope of this appendix.

Much valuable information has been obtained through photoelastic studies in connection with stress distribution and magnitude in dam and foundation structures. The photoelastic studies made on Shasta Dam furnish a good example of the application of the method. These studies were made to determine what effects would be produced on dam and foundation stresses by several weak-rock conditions which had been exposed in the foundation during the excavation for construction. A 5-foot clay-filled fault seam was discovered lying in a direction making an approximate 60° angle with the proposed axis of the dam. It was desired to determine the depth, if any, to which the seam in question should be excavated and backfilled with concrete in order to keep stresses within allowable limits. Because of the direction of the seam with respect to the dam, three possible locations of the seam were assumed for the tests. Photographs of the photoelastic stress patterns were taken of models constructed and loaded to represent the critical cantilever section under full reservoir load with the fault seam at the three alternate locations and at varying depths under the cantilever. These stress patterns were studied with regard to the effect of the various depths of seam repair on the stress at the downstream-toe fillet; where the most critical stress condition existed. Figure D-2 shows two photographs of the photoelastic model under stress. Figure D-3 gives curves showing the relation between the values of downstream-toe fillet stresses obtained from the photoelastic stress patterns, and the depth of the 5-foot clay-filled fault seam.

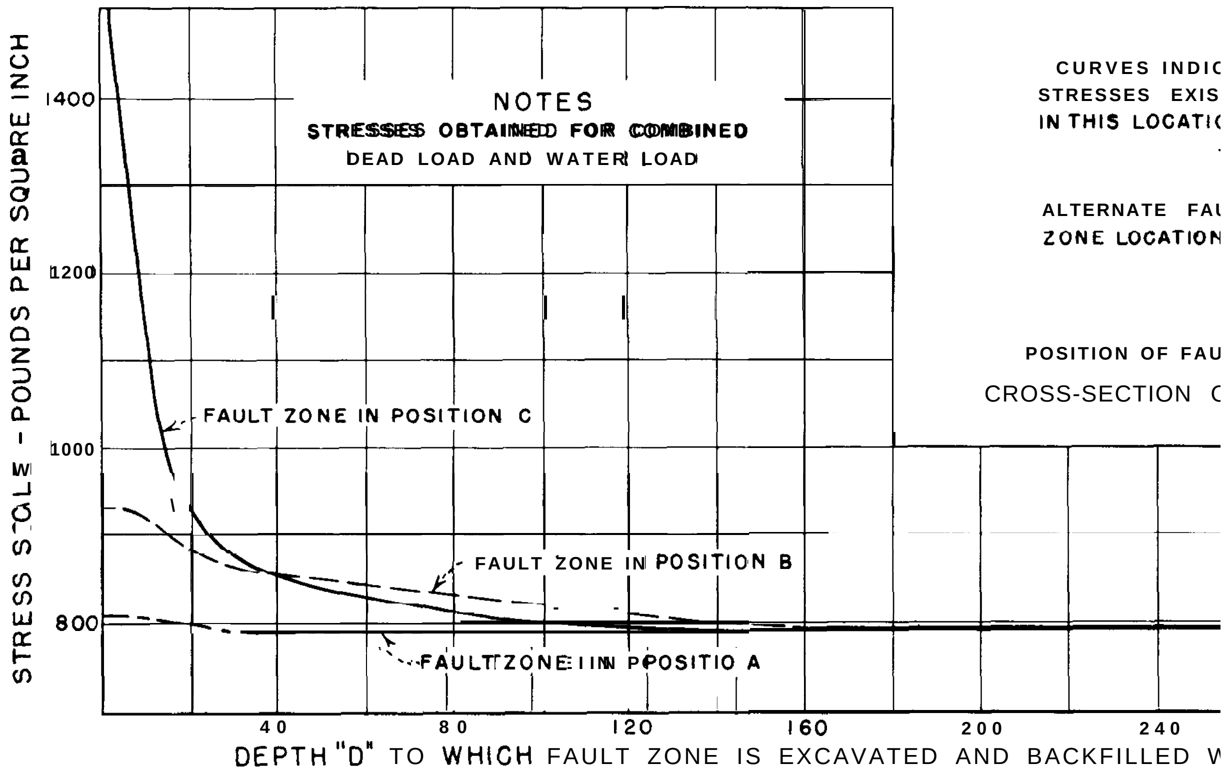


(a) FAULT SEAM UNDISTURBED



(b) FAULT SEAM EXCAVATED, BACKFILLED 52 FEET

Figure D-2. Photoelastic study of foundation fault seam near downstream face of Shasta Dam-reservoir full.-PX-D-74424



SHASTA DAM

PHOTOELASTIC STUDY OF FOUNDATION

Figure D-3. **Relation of stress at toe of dam to depth and location of fault zone.–DS2-2**

Comparison of Results by Gravity and Trial-load Methods

E- 1. ***Stresses and Stability Factors. -Stresses*** and stability factors for normal and maximum loading conditions for 12 gravity dams are given on figures E-l to E-29, inclusive. All of these dams were analyzed by the "Gravity Method," and, in addition, three were analyzed by the "Trial-Load Twist Method" and one by the "Trial-Load Arch and Cantilever Method." These are Grand Coulee, Kortes, and Angostura Dams; and East Park Dam, respectively. For these four dams, stresses obtained by the gravity analysis are shown on the same sheet with stresses obtained by the trial-load analysis. The same arrangement is used for showing stability factors. This facilitates comparison of results obtained by the two methods.

E-2. ***Structural Characteristics of Dams and Maximum Stresses Calculated by the Gravity and Trial-Load Methods.*** -A tabulation of structural characteristics, maximum stresses, and maximum stability factors for the 12 gravity dams mentioned in the preceding section is shown in table E-l. The 12 dams are divided into four groups in accordance with their relative heights. Structural characteristics are given in the upper half of the sheet. The ratios of crest-length to height, base to top width, and base to height of the crown cantilever define the relative characteristics of each dam. The cantilever profiles are shown for which the maximum stresses are tabulated in the lower half of the figure. The cross-canyon profile is shown for those dams for which a trial-load analysis was made.

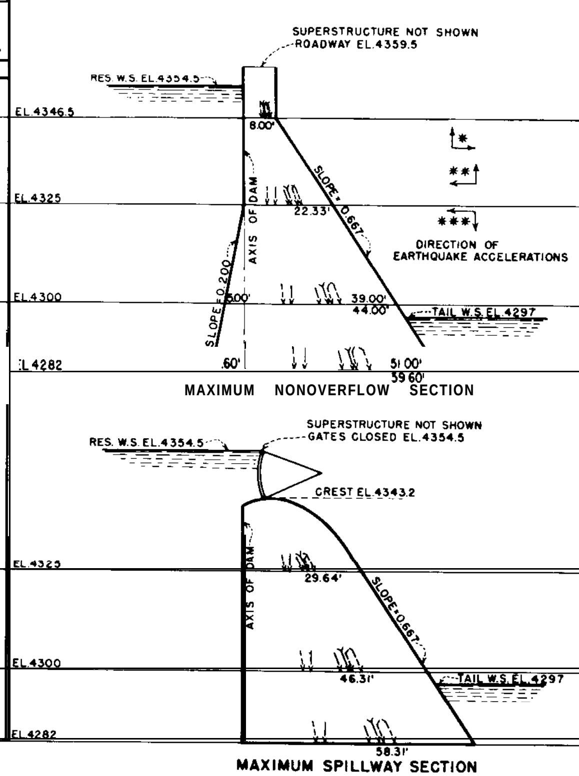
In the lower half of table E-l is shown the critical stress at the upstream face of each dam. This critical stress is considered to be that stress at the upstream face which is less than water pressure at the same point. In most cases this stress occurs at the base of the crown cantilever. These critical stresses are tabulated for normal loading conditions and maximum loading conditions. The water pressure at the same point is also shown. Examination of critical cantilever stresses at the upstream face for maximum loading conditions reveals that in all cases the water pressure exceeds the stress shown for the designated loading. Tensile stresses are indicated at the upstream face for three dams; namely, Black Canyon, East Park, and Keswick. However, it is felt that this is an exceptional condition with little likelihood of occurrence. The criterion to be used, therefore, is the normal loading condition, for which in no case is the stress at the upstream face below a value of about 40 percent of the water pressure at the same point.

Maximum stresses parallel to the downstream face for normal operating reservoir load and for maximum loading are also shown in table E-l. Maximum stresses computed by the trial-load analysis are given for comparison with gravity stresses. Generally, the two methods show very little stress disagreement in the central section of the dam, but usually show significant differences in stress and stability factors in the region of the abutments.

Maximum sliding factors and minimum shear-friction factors are also shown in table E-l for the 12 dams as computed by the gravity and trial-load analyses. These factors are for maximum loading conditions. For

UPSTREAM FACE

	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Normal Res. W.S. EL 4354.5: Water Pressure	Normal Res. W.S. EL 4354.5: Vertical Stress	Normal Res. W.S. EL 4354.5: Stress Parallel to Face	Normal Res. W.S. EL 4354.5: Horiz. Shear Stress
	Pounds Per Square Inch					
	*	*	**	**	**	**
Maximum nonoverflow section						
EL 4346.5	12 / 0	12 / 0	5 / 3	-17 / -3	-17 / -3	0 / 0
EL 4325	56 / 43	56 / 43	16 / 13	10 / 21	10 / 21	0 / 0
EL 4300	66 / 52	68 / 54	28 / 24	11 / 23	10 / 23	3.5 / 0.1
EL 4282	80 / 63	83 / 66	37 / 31	14 / 28	13 / 27	4.7 / 0.8
Maximum spillway section						
EL 4325	30 / 25	30 / 25	16 / 13	11 / 15	11 / 15	0 / 0
EL 4300	69 / 56	69 / 56	28 / 24	15 / 25	15 / 25	0 / 0
EL 4282	95 / 77	95 / 77	37 / 31	15 / 29	15 / 29	0 / 0



HORIZONTAL SECTION and **DOWNSTREAM FACE**

	Sliding Factor	Shear-Friction Factor	Normal Res. W.S. EL. 4354.5: Water Pressure	Normal Res. W.S. EL. 4354.5: Vertical Stress	Normal Res. W.S. EL. 4354.5: Stress Parallel to Face	Normal Res. W.S. EL. 4354.5: Horiz. Shear Stress	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face
			Pounds Per Square Inch					
	***	**	**	**	**	**	*	*
Maximum nonoverflow section								
EL 4346.5	.265 / .108	98.5 / 236.4		54 / 37	54 / 37	0 / 0	26 / 34	26 / 34
EL 4325	.700 / .467	32.1 / 48.7		37 / 22	54 / 31	25 / 15	-9 / -1	-14 / -1
EL 4300	.774 / .551	20.0 / 28.5		61 / 42	87 / 61	40 / 28	1 / 9	2 / 13
EL 4282	.783 / .555	16.2 / 23.0	7 / 7	80 / 57	112 / 80	49 / 34	6 / 14	8 / 21
Maximum spillway section								
EL 4325	.780 / .524	42.8 / 64.0		23 / 16	34 / 23	16 / 11	2 / 5	3 / 7
EL 4300	.797 / .551	20.8 / 29.9		54 / 38	78 / 55	36 / 25	1 / 6	-2 / 8
EL 4282	.816 / .573	15.7 / 22.5	7 / 7	76 / 55	109 / 77	47 / 32	-5 / 5	-7 / 8

Resultant-concrete weight only. Resultant-water pressure and weight. Resultant-water pressure, weight and uplift.
Resultant-concrete weight and earthquake (vertical upward and horizontal downstream). Resultant-water pressure, weight and earthquake (vertical upward and horizontal upstream). Resultant-water pressure, weight, uplift and earthquake (vertical upward and horizontal upstream)
Uplift pressure varies as a straight line from reservoir pressure at upstream face to tailwater pressure or zero at downstream face, acting over one-half the area of the horizontal section
Unit weight of concrete = 150 pounds per cubic foot
All normal stresses are compressive except those preceded by a negative sign, which are tensile
Positive shear stresses are caused by shear forces acting thus ⇌, negative shear stresses are caused by shear forces acting thus ⇋.
Total load carried by vertical cantilever.

$$\text{Sliding Factor} = \frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$$

$$\text{Shear-friction Factor} = \frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$$

Coefficient of internal friction = 0.65. Unit shear resistance = 400 pounds per square inch
Weight of bridge and gates included in analysis
* Figures above elevation lines include earthquake accelerations (vertical upward and horizontal downstream)
** Figures above elevation lines include earthquake accelerations (vertical upward and horizontal upstream)
*** Figures above elevation lines include earthquake accelerations (vertical downward and horizontal upstream)
figures below elevation lines are for normal conditions.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
[illegible] PROJECT - IDAHO
AMERICAN FALLS DAM
GRAVITY ANALYSIS - NONOVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
INCLUDING COMBINED EARTHQUAKE ACCELERATIONS

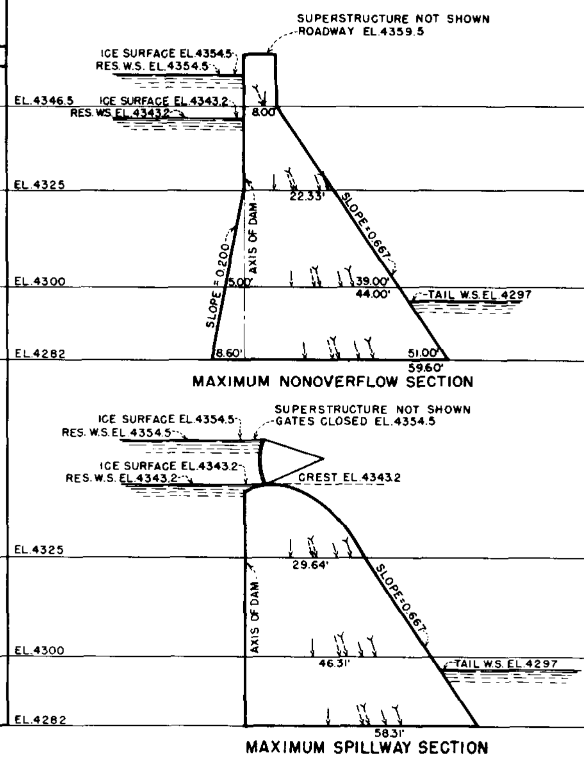
DRAWN H.P.W. SUBMITTED [illegible]
TRACED E.F.J.-L.E.S. RECOMMENDED [illegible]
CHECKED S.D.L. APPROVED [illegible]
DENVER, COLORADO, SEPT. 8, 1949
SHEET 1 OF 3
17-D-1169

Figure E-I. **American Falls Dan-gravity analyses of nonoverflow and spillway sections including effects of earthquake accelerations**

Upstream face (Pounds Per Square Inch; figures above / below elevation lines)

Section	Elevation	Reservoir empty: Vertical stress	Reservoir empty: Stress parallel to face	Normal res. W.S. El. 4354.5: Water pressure	Normal res.: Vertical stress	Normal res.: Stress parallel to face	Normal res.: Horiz. shear stress
Maximum nonoverflow section	EL. 4346.5	0 / 0	0 / 0	3 / 0	-80 / 0	-80 / 0	0 / 0
	EL. 4325	43 / 43	43 / 43	13 / 6	-24 / 14	-24 / 14	0 / 0
	EL. 4300	52 / 52	54 / 54	24 / 17	1 / 30	0 / 30	4.7 / −2.6
	EL. 4282	63 / 63	66 / 66	31 / 25	11 / 40	11 / 40	4.0 / -3.0
Maximum spillway section	EL. 4325	25 / 25	25 / 25	13 / 6	-11 / 8	-11 / 8	0 / 0
	EL. 4300	56 / 56	56 / 56	24 / 17	5 / 27	5 / 27	0 / 0
	EL. 4282	77 / 77	77 / 77	31 / 25	12 / 38	12 / 38	0 / 0

UPSTREAM FACE



Horizontal section and downstream face (figures above / below elevation lines)

Section	Elevation	Sliding factor	Shear-friction factor	Water pressure	Vertical stress	Stress parallel to face	Horiz. shear stress	Reservoir empty: Vertical stress	Reservoir empty: Stress parallel to face
Maximum nonoverflow section	EL. 4346.5	1.150 / 0	22.0 / ∞		114 / 34	114 / 34	0 / 0	34 / 34	34 / 34
	EL. 4325	.802 / .414	28.4 / 50.5		67 / 29	96 / 42	45 / 19	-1 / -1	-1 / -1
	EL. 4300	.666 / .374	23.5 / 39.0		64 / 36	92 / 52	43 / 24	9 / 9	13 / 13
	EL. 4282	.624 / .380	20.5 / 31.5	7 / 7	74 / 47	103 / 64	45 / 27	14 / 14	21 / 21
Maximum spillway section	EL. 4325	.899 / .469	37.3 / 66.3		42 / 21	60 / 31	28 / 14	5 / 5	7 / 7
	EL. 4300	.666 / .385	24.7 / 40.9		58 / 34	83 / 49	38 / 23	6 / 6	8 / 8
	EL. 4282	.645 / .404	20.0 / 30.8	7 / 7	72 / 45	101 / 63	44 / 26	5 / 5	8 / 8

HORIZONTAL SECTION — DOWNSTREAM FACE

Resultant-concrete weight only. Resultant-water pressure, weight, and ice at elevation 4354.5. Resultant-water pressure, weight, uplift and ice at elevation 4354.5.
Resultant-water pressure, weight and ice at elevation 4343.2. Resultant-water pressure, weight, uplift and ice at elevation 4343.2.
Uplift pressure varies as a straight line from reservoir pressure at upstream face to tailwater pressure or zero at downstream face, acting over one-half the area of the horizontal section.
Unit weight of concrete = 150 pounds per cubic foot.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses are caused by shear forces acting thus ⇌; negative shear stresses are caused by shear forces acting thus ⇋.
Total load carried by vertical cantilever.

$$\text{Sliding Factor} = \frac{\text{Horizontal Force}}{\text{Weight - Uplift}}$$

$$\text{Shear-friction Factor} = \frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$$

Coefficient of internal friction = 0.65. Unit shear resistance = 400 pounds per square inch.
Weight of bridge and gates included in analysis.
Ice load, four feet thick, exerts ten tons per linear foot.
Figures above elevation lines are for ice load at elevation 4354.5.
Figures below elevation lines are for ice load at elevation 4343.2.

STUDY No. C - ICE LOAD AT EL. 4354.5
STUDY No. D - ICE LOAD AT EL. 4343.2

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MINIDOKA PROJECT - IDAHO
AMERICAN FALLS DAM
GRAVITY ANALYSIS - NONOVERFLOW AND SPILLWAY SECTIONS
NORMAL FULL RESERVOIR OPERATION
WITH ICE LOAD

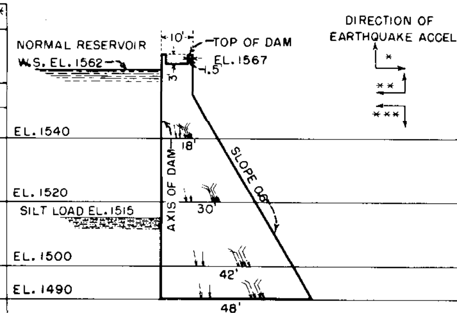
DRAWN H.P.W. SUBMITTED F. D. [signature]
TRACED E.F.J.-L.E.S. RECOMMENDED [signature]
CHECKED [signature] APPROVED J. L. Savage
DENVER, COLORADO, SEPT. 6, 1940 SHEET 2 OF 3 17-D-117

Figure **E-2. American Falls Dam-gravity analyses of nonoverflow and spillway sections, normal conditions with ice load.**

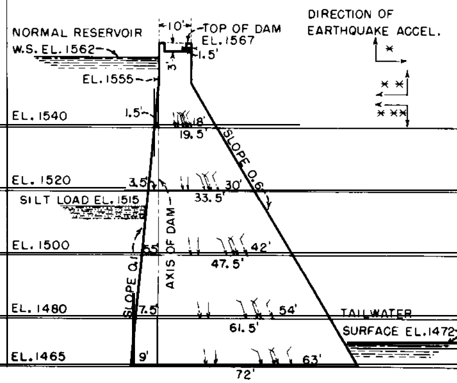
RESERVOIR EMPTY*			NORMAL RES. W.S. EL. 1562**					NORMAL RES. W.S				
VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS		SLIDING FACTOR	SHEAR FRICTION FACTOR	MAX HORIZ SHEAR STRESS	WATER PRESSURE	VE ST
Pounds per Square Inch								***				
33	33	0	10	19	19	0	EL. 1540	.386	70.2	9.8	—	
43	43	0	12	18	18	0		.610	44.1	12	—	
55	55	0	18	19	19	0	EL. 1520	.565	32.5	21	—	
70	70	0	22	15	15	0	SILT LOAD EL. 1515	.806	22.4	27	—	
75	75	0	27	16	16	0	EL. 1500	.651	21.1	35	—	
95	95	0	32	11	11	0		.893	15.1	43	—	
85	85	0	31	15	15	0	EL. 1490	.678	18.0	42	—	
108	108	0	37	9	9	0		.920	13.1	51	—	
28	28	-2.8	10	17	17	-0.8	EL. 1540	.363	76.1	11		
36	36	-3.6	12	10	10	0.2		.591	46.5	17		
45	45	-4.5	18	20	21	-0.2	EL. 1520	.502	36.3	21		
58	58	-5.8	22	9	9	1.3	SILT LOAD EL. 1515	.738	24.4	31		
63	64	-6.3	27	24	24	0.3	EL. 1500	.560	23.9	31		
80	81	-8.0	32	8	8	2.4		.792	16.7	46		
80	81	-8.0	36	27	27	0.8	EL. 1480	.588	18.0	42		
102	103	-10	42	7	7	3.5		.814	12.8	61		
94	95	-9.1	42	30	30	1.2	EL. 1465	.607	15.2	49	3	
118	120	-12	50	7	7	4.2		.832	11.0	70	3	

UPSTREAM FACE

HORIZONTAL SECTION



MAXIMUM ABUTMENT SECTION



MAXIMUM NONOVERFLOW SECTION

Resultant-weight. Resultant-water pressure and weight. Resultant-water pressure, weight and uplift.
Resultant-weight and earthquake (vertical upward and horizontal downstream). Resultant-water pressure, weight and earthquake (vertical upward and horizontal upstream). Resultant-water pressure, weight, uplift and earthquake (vertical upward and horizontal upstream).

Uplift pressure varies as a straight line from reservoir pressure at upstream face to tailwater pressure or zero at downstream face, acting over one-half the area of the horizontal section.

Unit weight of concrete=150 lbs. per cu. ft.

All normal stresses ore compressive except those preceded by a negative sign, which ore tensile.

Positive shear stresses ore caused by shear forces acting thus ⇌; negative shear stresses are caused by shear forces acting thus ⇌.

Total load carried by vertical cantilever.

Sliding Factor= $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$. Shear-Friction Factor= $\frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Coefficient of Internal Friction=0.65. Unit Shear Resistance=400 lbs. per sq. in.

Silt load to elevation 1515. Unit weight of silt=120 lbs. per cu. ft.

Figures above elevation lines ore for normal conditions.

*Figures below elevation lines include earthquake acceleration (vertical upward and horizontal downstream).

**Figures below elevation lines include earthquake acceleration (vertical upward and horizontal upstream).

***Figures below elevation lines for sliding factor include earthquake acceleration (vertical downward and horizontal upstream).

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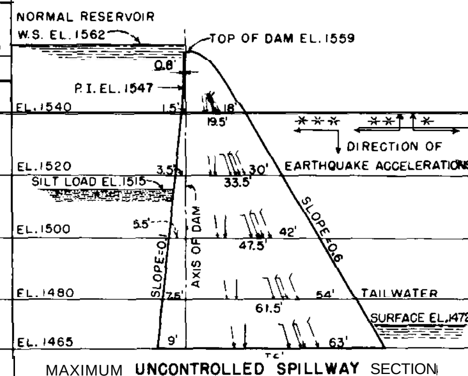
Figure **E-3. Altus Dan-gravity analyses of maximum abutment and nonoverflow sections.**

MAXIMUM UNCONTROLLED SPILLWAY SECTION (Pounds per Square Inch)

Elevation	Reservoir empty* — Vertical stress	Reservoir empty* — Stress parallel to face	Reservoir empty* — Horiz. shear stress	Normal res. W.S. El. 1562** — Water pressure	Normal res. — Vertical stress	Normal res. — Stress parallel to face	Normal res. — Horiz. shear stress	Sliding factor ***	Shear friction factor	Normal res. — Max. horiz. shear stress	Normal res. — Water pressure	Normal res. — Vertical stress	Normal res. — Stress parallel to face	Normal res. — Horiz. shear stress	Reservoir empty* — Vertical stress	Reservoir empty* — Stress parallel to face	Reservoir empty* — Horiz. shear stress
EL. 1540	29	30	-2.9	10	19	19	-0.9	.297	76.4	13	—	22	31	13	11	15	6.5
	38	39	-3.8	12	11	11	0.1	.503	45.0	20	—	34	46	20	6	8	3.6
EL. 1520	48	49	-4.8	18	24	24	-0.6	.463	36.4	21	—	34	47	21	8	11	4.7
	63	63	-6.3	22	12	12	1.1	.689	24.2	32	—	53	72	32	0	0	-0.2
EL. 1500	65	66	-6.5	27	26	26	0.1	.537	24.0	31		52	72	31	10	13	5.8
	84	85	-8.4	32	10	10	2.2	.764	16.6	46		77	104	46	-1	-1	-0.5
EL. 1480	83	84	-8.3	36	30	30	0.6	.574	18.0	42		71	96	42	13	16	7.5
	105	106	-11	42	9	9	3.3	.796	12.8	61	—	101	138	61	-1	-1	-0.5
EL. 1465	95	96	-10	42	32	32	1.0	.595	15.3	49	3	86	115	49	15	20	9.1
	122	123	12	50	9	8	4.1	.818	11.0	70	3	120	162	70	0	-1	-0.3

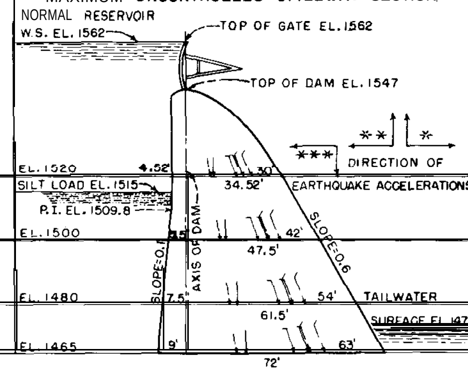


MAXIMUM UNCONTROLLED SPILLWAY SECTION

MAXIMUM CONTROLLED SPILLWAY SECTION (Pounds per Square Inch)

Elevation	Reservoir empty* — Vertical stress	Reservoir empty* — Stress parallel to face	Reservoir empty* — Horiz. shear stress	Normal res. W.S. El. 1562** — Water pressure	Normal res. — Vertical stress	Normal res. — Stress parallel to face	Normal res. — Horiz. shear stress	Sliding factor ***	Shear friction factor	Normal res. — Max. horiz. shear stress	Normal res. — Water pressure	Normal res. — Vertical stress	Normal res. — Stress parallel to face	Normal res. — Horiz. shear stress	Reservoir empty* — Vertical stress	Reservoir empty* — Stress parallel to face	Reservoir empty* — Horiz. shear stress
EL. 1520	35	35	0	16	11	11	0	.675	37.0	19		31	41	19	5	6	3.0
	43	43	-4.3	22	3	3	0	.966	25.7	26	—	43	59	26	2	2	0.8
EL. 1500	56	57	-5.6	27	15	15	1.2	.650	23.8	32	—	52	72	32	9	12	5.2
	70	71	-7.0	32	1	1	3.1	.908	16.8	44	—	73	100	44	2	2	0.9
EL. 1480	75	76	-7.5	36	20	20	1.5	.642	17.9	43	—	72	98	43	13	17	7.6
	94	95	-9.1	42	1	1	4.1	.882	12.8	60	—	100	136	60	3	4	1.5
EL. 1465	88	89	-8.8	42	23	23	1.9	.647	15.2	50	3	86	117	50	15	21	9.1
	111	112	-11	50	2	1	4.8	.882	11.0	70	3	119	161	70	3	5	1.8

UPSTREAM FACE — HORIZONTAL SECTION — DOWNSTREAM FACE



MAXIMUM CONTROLLED SPILLWAY SECTION

↓Resultant – weight. ↘ Resultant – water pressure and weight. ↙Resultant – water pressure, weight, and uplift.
↘Resultant – weight, and earthquake (vertical upward and horizontal downstream). ↘ Resultant – water pressure, weight, and earthquake (vertical upward and horizontal upstream). ↙Resultant – water pressure, weight, uplift, and earthquake (vertical upward and horizontal upstream).

Uplift pressure varies as a straight line from reservoir pressure at upstream face to tailwater pressure or zero at downstream face, acting over one-half the area of the horizontal section.

Unit weight of concrete = 150 lbs. per cu. ft

All normal stresses are compressive except those preceded by a negative sign, which are tensile.

Positive shear stresses are caused by shear forces acting thus, ⇄; negative shear stresses are caused by shear forces acting thus ⇄.

Total load carried by vertical cantilever.

Sliding Factor $= \dfrac{\text{Horizontal Force}}{\text{Weight} - \text{Uplift}}$; Shear-friction Factor $= \dfrac{(\text{Weight} - \text{Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Coefficient of Internal Friction = 0.65 Unit shear resistance = 400 lbs. per sq in.

Silt load to elevation 1515. Unit weight of silt = 120 lbs. per cu. ft.

Figures above elevation lines are for normal conditions.

* Figures below elevation lines include earthquake acceleration (vertical upward and horizontal downstream).

** Figures below elevation lines include earthquake acceleration (vertical upward and horizontal upstream).

*** Figures below elevation lines for sliding factor include earthquake acceleration (vertical downward and horizontal upstream).

STUDY No. 2G

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
ALTUS PROJECT – OKLAHOMA

ALTUS DAM

PROPOSED DESIGN FOR CONCRETE CONSTRUCTION
GRAVITY ANALYSES OF MAXIMUM UNCONTROLLED SPILLWAY ANOCONTROLLEO SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION WITH AND WITHOUT HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS

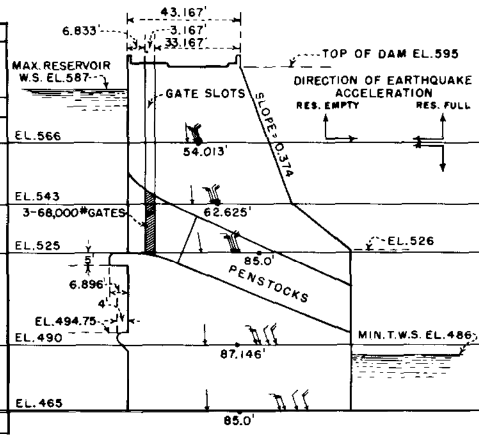
DRAWN	W E E	SUBMITTED	F. D. Kirn
TRACED	P.L.O.- P.V.L.	RECOMMENDED	Ivan E. Houk
CHECKED	S.D.L.	APPROVED	W. H. Nalder

DENVER, COLORADO – OCT. 8, 1941 SHEET 2 OF 4 — 258-D-104

Figure E-4. **Altus Dam-gravity analyses of spillway sections.**

UPSTREAM FACE

Elevation	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress	Normal Res. W.S. El. 587: Water Pressure	Normal Res. W.S. El. 587: Vertical Stress	Normal Res. W.S. El. 587: Stress Parallel to Face	Normal Res. W.S. El. 587: Horiz. Shear Stress
	Pounds per Square Inch						
EL.566	*43	43	0	‡ 12	33	33	0
				‡ 10	26	26	0
EL.543	124	124	0	24	71	71	0
				20	54	54	0
EL.525	214	214	0	33	138	138	0
				27	109	109	0
EL.490	149	297	-149	50	39	78	11
				42	26	52	16
EL.465	205	205	0	63	-12	-12	0
				52	-20	-20	0



65 FOOT PENSTOCK SECTION

HORIZONTAL SECTION OOWNSTREAH FACE

Elevation	Normal Res. W.S. El. 587: Sliding Factor	Normal Res. W.S. El. 587: Shear-Friction Factor	Normal Res. W.S. El. 587: Max. Horiz. Shear Stress	Normal Res. W.S. El. 587: Water Pressure	Normal Res. W.S. El. 587: Vertical Stress	Normal Res. W.S. El. 587: Stress Parallel to Face	Normal Res. W.S. El. 587: Horiz. Shear Stress	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress
			Pounds per Square Inch							
EL.566	‡ .192	80.58	10	0	27	31	10	* 17	20	6.4
	‡ .225	85.85	8.6	0	23	26	8.6			
EL.543	.316	28.29	18	0	44	50	16	3.2	3.6	1.2
	.358	31.05	16	0	39	44	14			
EL.525	.379	16.36	46	0	37	37	0	-12	-12	0
	.417	18.13	41	0	33	33	0			
EL.490	.535	12.32	51	0	124	124	0	12	12	0
	.597	13.86	43	0	107	107	0			
EL.465	.667	8.25	83	9.3	234	234	0	18	18	0
	.764	9.25	71	7.5	202	202	0			

Resultant-Concrete weight and earthquake (horizontal downstream and vertical upward). Resultant-Water pressure, weight, and earthquake (horizontal upstream and vertical upward). Resultant-Water pressure, weight, uplift, and earthquake (horizontal upstream and vertical upward). Resultant-Water pressure, weight, and earthquake (horizontal upstream and vertical downward). Resultant-Water pressure, weight, uplift, and earthquake (horizontal upstream and vertical downward).

Sliding Factor = $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$ Shear-friction Factor = $\frac{(\text{Weight-Uplift}) \times \text{Coefficient of internal friction} + \text{Horizontal area} \times \text{Unit shear resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 lb. per cu ft. Unit shear resistance = 400 lb. per sq. in. Coefficient of internal friction = 0.65.

Uplift pressure varies as a straight line from reservoir water pressure at the upstream face to zero at the downstream face, or tailwater pressure at the downstream face of the power plant, acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake.

Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1 gravity; Period = 1 second.

Total load carried by vertical cantilever. Analyses made with gates down and penstocks empty.

All normal stresses are compressive except those preceded by a negative sign which are tensile.

Positive shear stresses are caused by shear forces acting thus ⇌. Negative shear stresses are caused by shear forces acting thus ⇋.

* All figures given for reservoir empty include earthquake acceleration (horizontal downstream and vertical upward). ‡ All figures given above line for reservoir V.S.El.587 include earthquake acceleration (horizontal upstream and vertical upward). # All figures given below line for reservoir W.S. El.587 include earthquake acceleration (horizontal upstream and vertical downward).

STUDY No. A-R5

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA

KESWICK POWER PLANT DAM

GRAVITY ANALYSES- GATES DOWN
65 FOOT PENSTOCK SECTION
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION INCLUDING HORIZONTAL ANDVERTICAL EARTHQUAKE ACCELERATIONS CAUSING THE MOST CRITICAL CONDITIONS

DRAWN.... D.S.R. .... SUBMITTED.... F. W. Kinn
TRACED.... F.L.D. .... RECOMMENDED.... Jerry F. Houk
CHECKED.... H.P.W. .... APPROVED.... J. L. Savage

DENVER, COLORADO. – JUNE 25, 1941. SHEET 1 OF 1 | 214-D-6783

Figure E-5. **Keswick Powerplant Dam-gravity analyses of penstock section including effects of earthquake accelerations.**

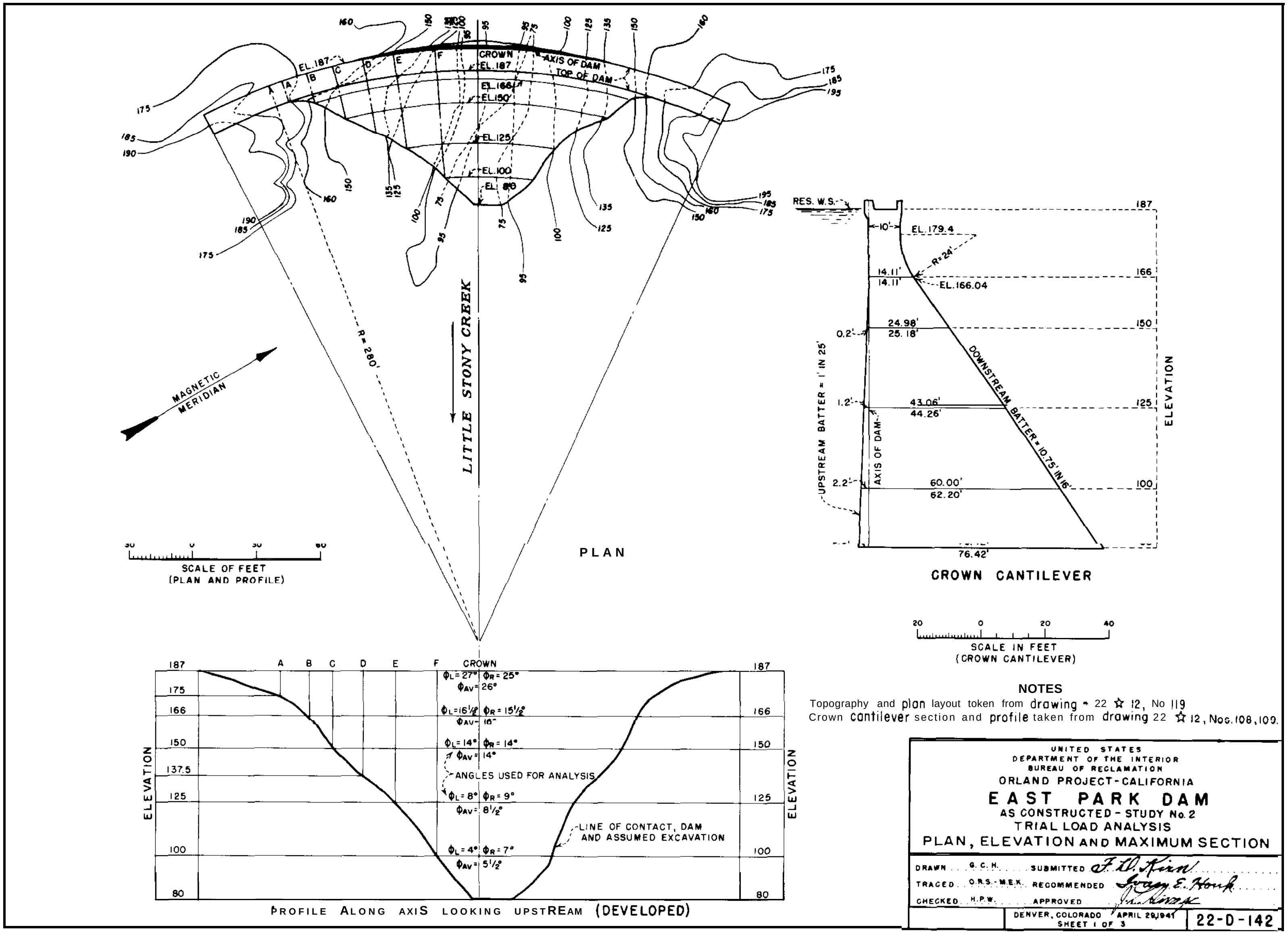
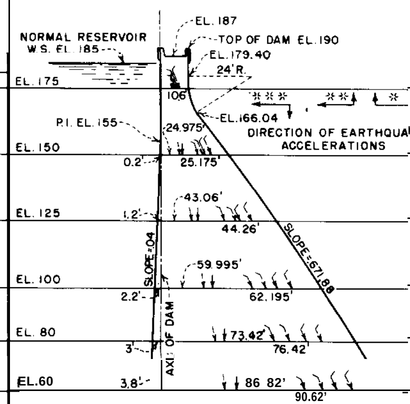


Figure E-6. **East Park Dam-plan, elevation, and maximum section.**

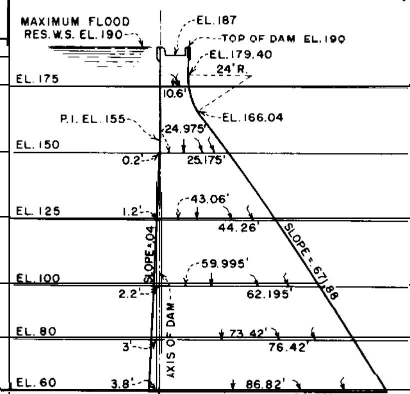
RESERVOIR EMPTY			NORMAL RES. W.S. EL. 185							
VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS	WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZ. SHEAR STRESS		SLIDING FACTOR	SHEAR-FRICTION FACTOR	MAX HORIZ. SHEAR STRESS
Pounds per Square Inch										
*15	*15	*0	**4	**12	**12	*0	EL. 175	***.185	**101.2	**12
22	22	0	5	6	6	0		.395	48.4	2.0
45	45	-1.8	15	16	16	0	EL. 150	.592	20.0	20
58	58	-2.3	17	2	2	0.6		.898	13.3	32
65	65	-2.6	26	20	20	0.3	EL. 125	.719	12.2	31
82	82	-3.3	29	2	2	1.1		1.033	8.6	48
87	87	-3.5	37	22	22	0.6	EL. 100	.755	8.8	47
109	109	-4.4	41	0	0	1.6		1.058	6.3	68
105	105	-4.2	46	24	24	0.9	EL. 80	.769	7.2	59
132	132	-5.3	50	-2	-2	2.1		1.065	5.3	84
124	124	-4.9	54	26	25	1.1	EL. 60	.780	6.2	72
155	155	-6.2	60	-4	-4	2.5		1.070	4.6	100

NORMAL RES. W.S. — WATER V… PRESSURE



MAXIMUM NONOVERFLOW SECTION

RESERVOIR EMPTY			MAX. FLOOD RES. W.S. EL. 190					MAXIMUM FLOOD RES		
15	15	0	7	2	2	0.2	EL. 175	.444	44.9	2
45	45	-1.8	17	2	2	0.6	EL. 150	.805	15.3	29
65	65	-2.6	28	7	7	0.9	EL. 125	.867	10.4	40
87	87	-3.5	39	9	9	1.2	EL. 100	.863	7.8	55
105	105	-4.2	48	11	11	1.5	EL. 80	.858	6.6	68
124	124	-4.9	56	13	13	1.7	EL. 60	.855	5.7	80



UPSTREAM FACE — MAXIMUM NONOVERFLOW SECTION — HORIZONTAL SECTION

Resultant-weight. Resultant-water pressure and weight. Resultant-water pressure, weight and up,,,,.
Resultant-weight and earthquake (vertical upward and horizontal downstream). Resultant-water pressure, weight and earthquake (vertical upward and horizontal upstream). Resultant-water pressure, weight, uplift and earthquake (vertical upward and horizontal upstream).

Uplift pressure varies as a straight line from reservoir pressure at upstream face to tailwater pressure or zero at downstream face, acting over two-thirds the area of the horizontal section

Unit weight of concrete = 145 lbs. per cu. ft.

All normal stresses are compressive except those preceded by a negative sign, which are tensile.

Positive shear stresses are caused by shear forces acting thus ⇌ ; negative shear stresses are caused by shear forces acting thus ⇋.

Total load carried by vertical cantilever.

$$\text{Sliding Factor} = \frac{\text{Horizontal Force}}{\text{Weight - Uplift}} \qquad \text{Shear-friction Factor} = \frac{(\text{Weight - Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$$

Coefficient of internal friction = 0 65. Unit Shear resistance = 200 lbs. per sq in

* Figures below elevation lines include earthquake acceleration (vertical upward and horizontal downstream)

** Figures below elevation lines include earthquake acceleration (vertical upward and horizontal upstream).

*** Figures below elevation lines for sliding factor include earthquake acceleration (vertical downward and horizontal upstream).

Figures above elevation lines do not include earthquake effects.

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Figure E-7. **East Park Dam-gravity analyses of maximum nonoverflow section.**

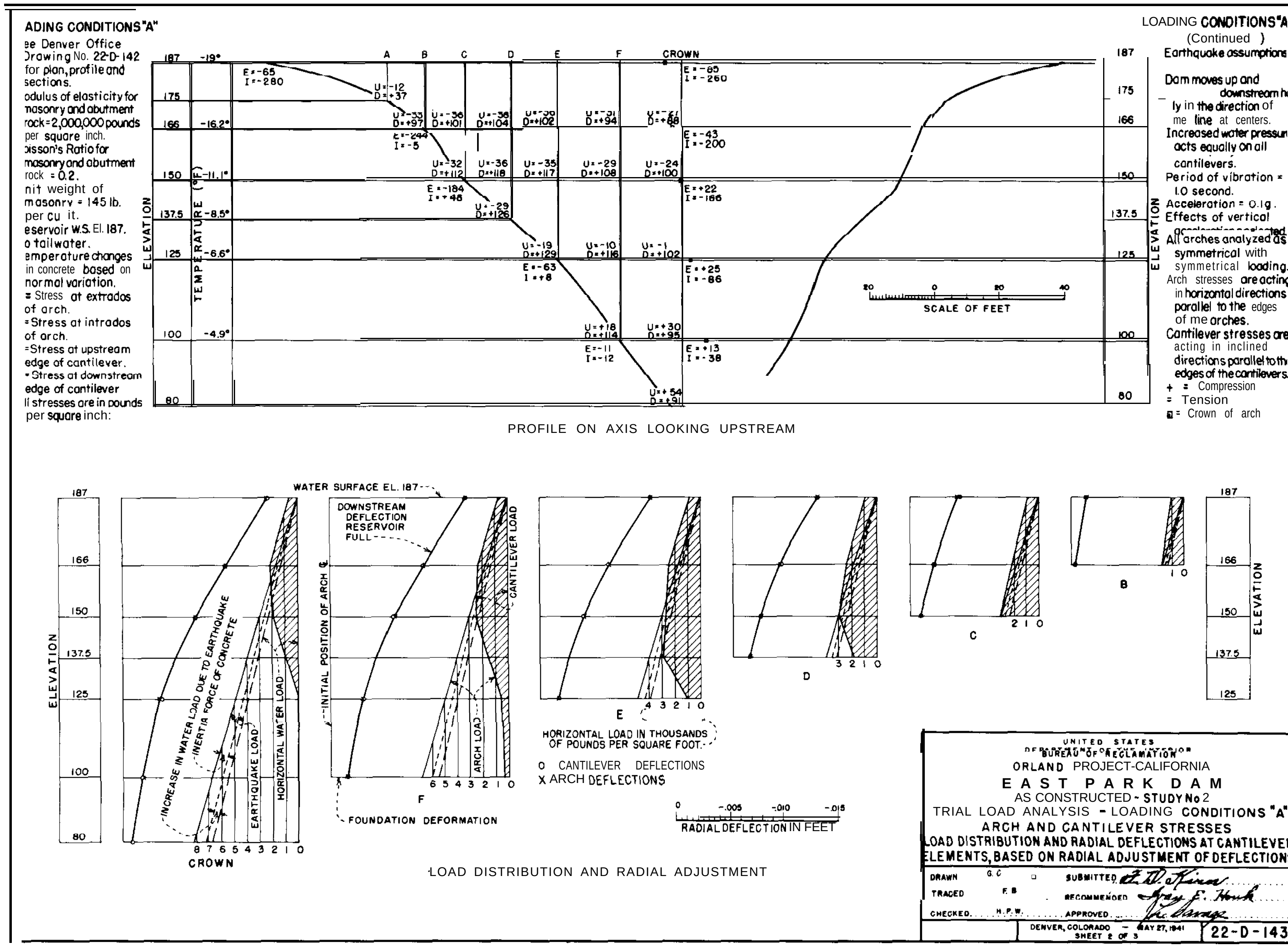


Figure E-8. **East Park Dam-stresses, load distribution, and radial deflections from trial-load analysis.**

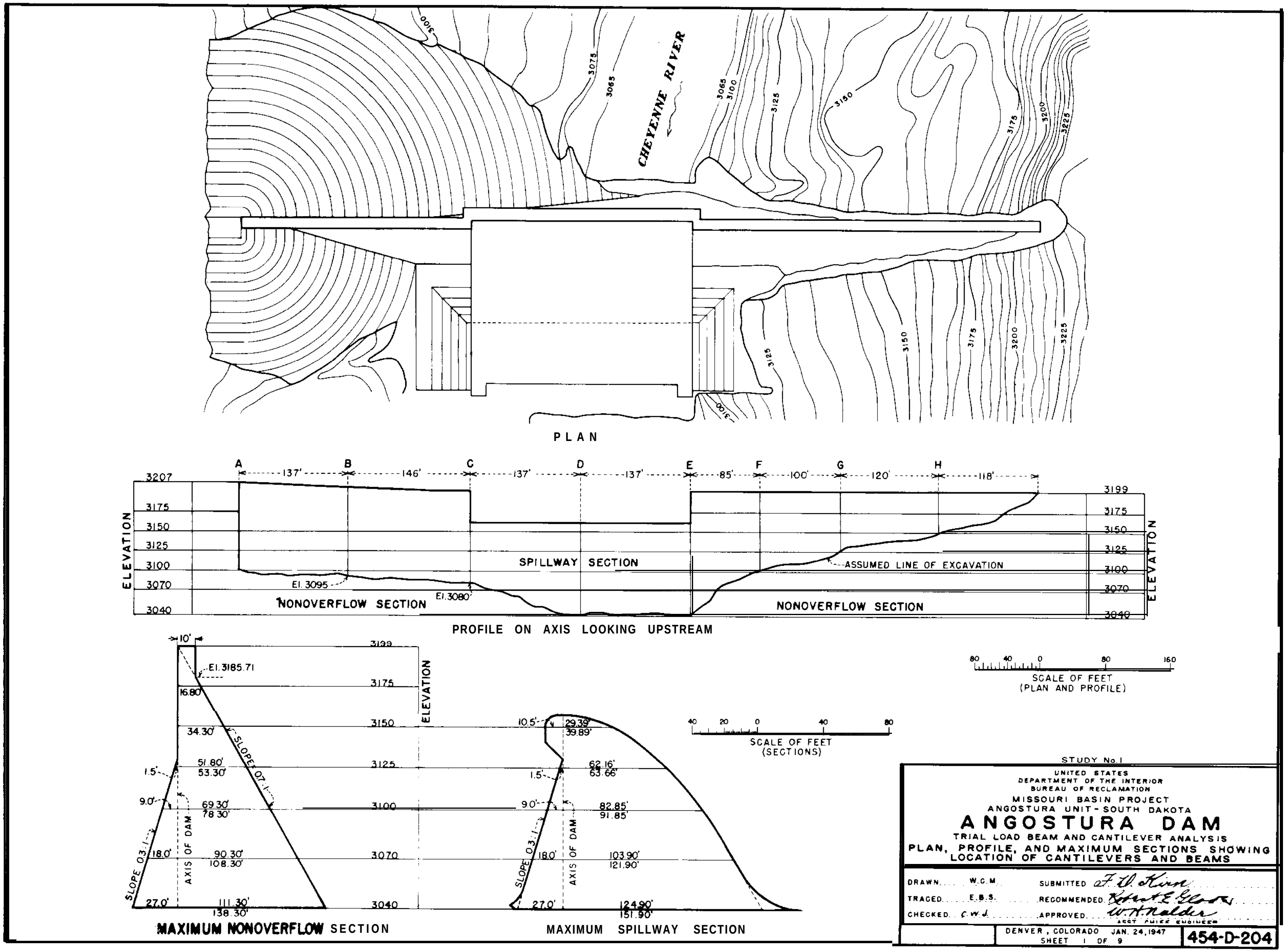
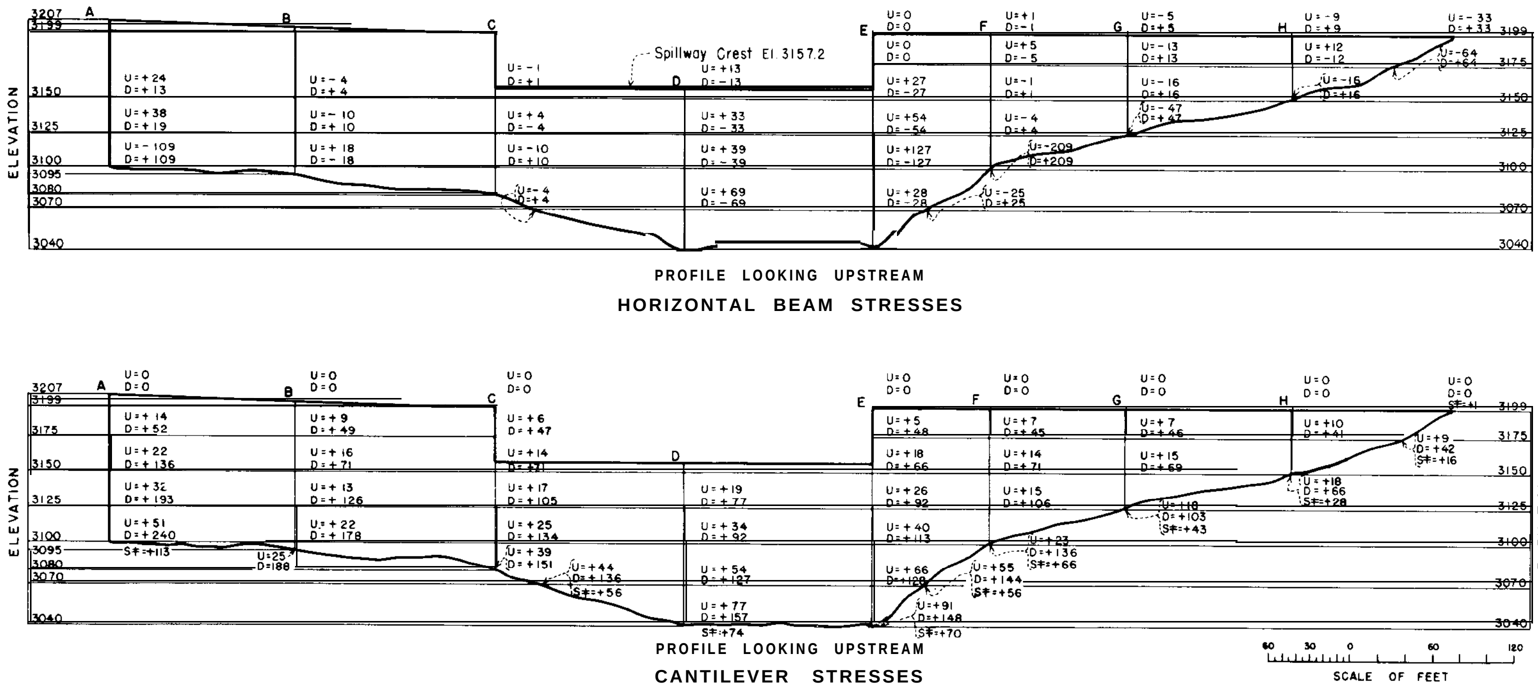


Figure E-9. Angostura Dam-plan, profile, and maximum section.



STRESSES-GRAVITY ANALYSIS			
ELEVATION	CANTILEVER A	CANTILEVER D	CANTILEVER E
3175	U=+14 D=+52		U=+17 D=+31
3150	U=+22 D=+136		U=+22 D=+59
3125	U=+31 D=+195	U=+21 D=+74	U=+22 D=+98
3100	U=+48 D=+243	U=+34 D=+92	U=+27 D=+130
3070		U=+43 D=+142	U=+38 D=+167
3040		U=+60 D=+184	U=+50 D=+204

LOADING CONDITIONS

Reservoir water surface at elevation 3187 2
Tailwater surface at elevation 3040
Unit weight of concrete, 150 pounds per cubic foot
Unit weight of water, 62 5 pounds per cubic foot
Modulus of elasticity for concrete: 3,000,000 lbs per sq inch, for rock 600,000 lbs per sq inch
Poisson's ratio for con crete, 0 2
Joints assumed grouted so dam can act as a monolith
All beams analyzed as non-symmetrical with non-symmetrical loading
Ice load of 5 tons per linear foot concentrated at elevation 3186.0.
Effect of earth embankment included.

EARTHQUAKE ASSUMPTIONS

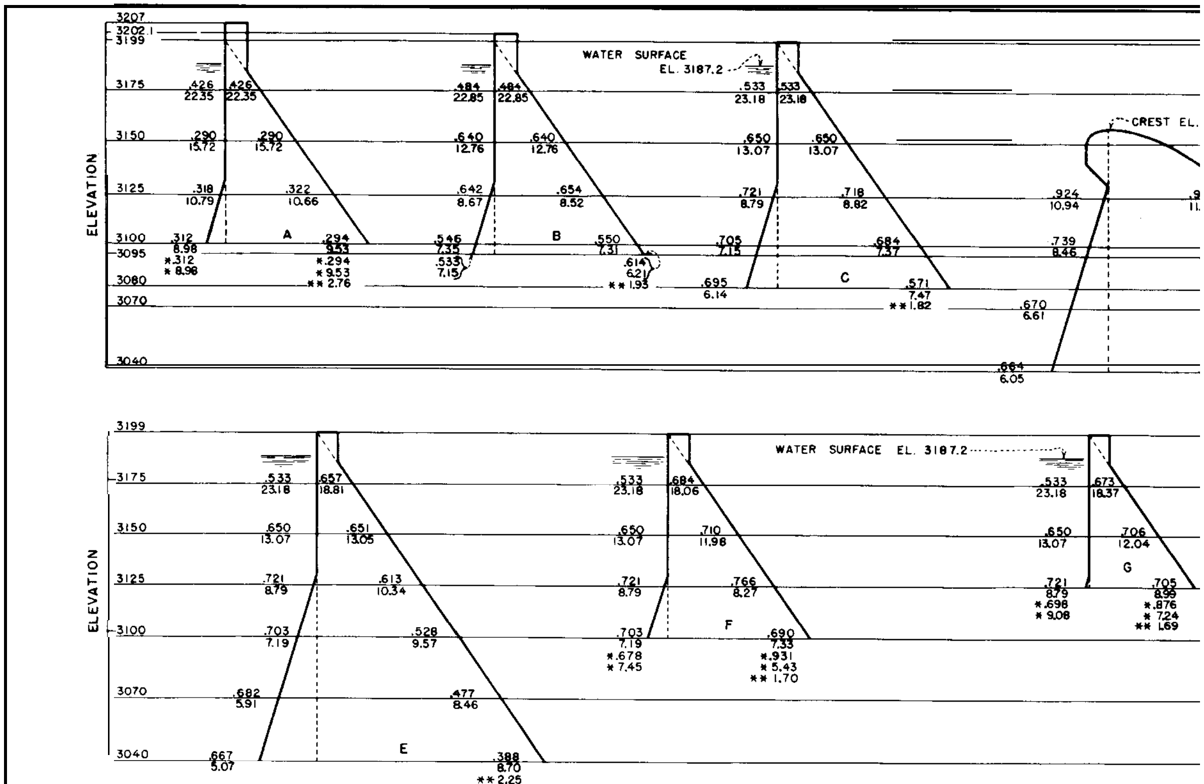
Horizontal acceleration upstream, 0 1 gravity
Vertical acceleration upward, 0 1 gravity
Period of vibration = 1 0 second
External pressure includes earthquake effect and acts equally on all cantilevers
U = Stress at upstream face
D = Stress at downstream face
\+ Indicates compression
\- Indicates tension
All stresses are in pounds per square inch
Cantilever stresses are acting in inclined directions parallel to the edges of cantilevers
S‡ = Maximum shear stress at rock abutment, (+) indicates downstream shear
For plan profile and cantilever sections see Denver Office Drawing 454-D-204

STUDY No. 1

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
ANGOSTURA UNIT - SOUTH DAKOTA
ANGOSTURA DAM
TRIAL LOAD BEAM AND CANTILEVER ANALYSIS
RESERVOIR FULL-EARTHQUAKE INCLUDED-JOINTS GROUTED
STRESSES IN HORIZONTAL BEAM ELEMENTS
AND IN CANTILEVER ELEMENTS

DRAWN W.C.M.-R.M.A. SUBMITTED F. W. Kinn
TRACED J H.W RECOMMENDED Robert E. Glover
CHECKED C.W.J. APPROVED W. H. Nalder
ASST. CHIEF ENGINEER
DENVER, COLORADO. JAN. 24, 1947
SHEET 6 OF 9
454-D-209

Figure E-10. **Angostura Dam-stresses from trial-load beam and cantilever analysis.**



For constants, assumptions, and loading conditions see Denver Office Drawing 454-D-209.
Sliding factors are computed for loading conditions as listed on Denver Office Drawing 454-O-209 and include effects of two-thirds uplift
Coefficient of Internal Friction = 0.65 Unit Shear Resistance = 200 pounds per square inch
Sliding factors and shear friction factors of safety for gravity analysis are shown on upstream side Sliding factors and shear friction factors of safety for trial load analysis are shown on downstream side Sliding factors are shown above elevation lines of cantilever Shear-friction factors are shown below elevation lines of cantilever
Factors designated by * are for inclined abutment planes
Factors designated by ** are for factor of safety against overturning

Sliding factors, $S = \dfrac{\text{Horizontal Force}}{\text{Weight - Uplift}}$ (for horizontal planes).

Sliding factors, $S = \dfrac{\text{Horizontal Force}}{\text{(Weight - Uplift) Sec } \phi}$ (for inclined abutment planes).

Shear friction factors of safety, Q, two-thirds uplift =
$\dfrac{\text{(Weight - Uplift) x Coefficient of Internal Friction + Base Area x Unit Shear Resistance}}{\text{Horizontal Force}}$ (for horizontal planes)

Shear friction factors of safety, Q, two-thirds uplift =
$\dfrac{\text{(Weight - Uplift) Sec } \phi \text{ x Coefficient of Internal Friction + Base Area x Unit Shear Resistance}}{\text{Horizontal Force}}$
(for inclined abutment planes)

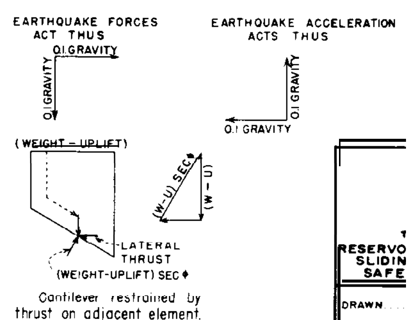


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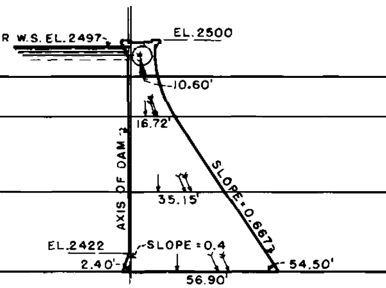
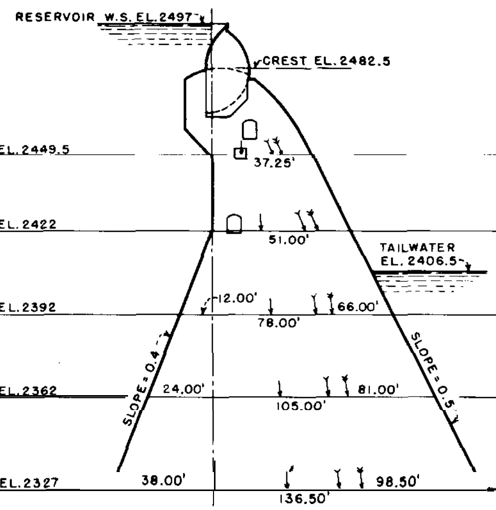
Figure E-II. **Angostura Dam-stability factors from trial-load beam and cantilever anal**

UPSTREAM FACE

Elevation	Res. Empty: Vertical Stress	Res. Empty: Stress Parallel to Face	Res. Empty: Horiz. Shear Stress	Reservoir Full: Water Pressure	Reservoir Full: Vertical Stress	Reservoir Full: Stress Parallel to Face	Reservoir Full: Horiz. Shear Stress
	POUNDS PER SQUARE INCH						
EL. 2487.5	12	12	0	4	9	9	0
EL. 2472.6	33	3 3	0	11	10	10	0
EL. 2445	61	61	0	23	12	12	0
EL. 2416	79	92	-32	35	14	10	9
EL. 2449.5	53	107	-53	21	7	-7	14
EL. 2422	80	80	0	33	0	0	0
EL. 2392	74	85	-29	46	8	2	15
EL. 2362	82	96	-33	59	18	12	16
EL. 2327	98	114	-39	74	35	28	16



MAXIMUM NON-OVERFLOW SECTION



MAXIMUM SPILLWAY SECTION

HORIZONTAL SECTION / DOWNSTREAM FACE

Elevation	Sliding Factor	Shear-friction Factor	Max. Horiz. Shear Stress	Reservoir Full: Water Pressure	Reservoir Full: Vertical Stress	Reservoir Full: Stress Parallel to Face	Reservoir Full: Horiz. Shear Stress	Res. Empty: Vertical Stress	Res. Empty: Stress Parallel to Face	Res. Empty: Horiz. Shear Stress
				POUNDS PER SQUARE INCH						
EL. 2487.5	.227	219.4	2	0	10	10	2	6	7	1
EL. 2472.6	.503	53.2	17	0	26	37	17	3	4	2
EL. 2445	.691	24.9	32	0	48	69	32	-2	-3	-1
EL. 2416	.716	16.9	49	0	74	107	49	5	8	3
EL. 2449.5	.819	31.2	18	0	36	45	18	-5	-7	-3
EL. 2422	.761	17.6	40	0	79	99	40	4	4	2
EL. 2392	.669	14.3	51	6	108	134	51	32	40	16
EL. 2362	.573	13.0	59	19	138	168	59	51	63	25
EL. 2327	.476	12.5	68	35	170	204	68	69	86	34

↓ Resultant-concrete weight only. ↓ Resultant-woter pressure and weight. ↓ Resultant-woter pressure, weight, and uplift.

$$\text{Sliding Factor} = \frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$$

$$\text{Shear-friction Factor} = \frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Thickness} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 400 pounds per square inch.
Coefficient of Internal Friction = 0.65.
Uplift pressure varies os a straight line from reservoir water pressure at upstream face to toilwoter pressure at downstreom face, acting over one-half area of horizontal section.
Total load carried by vertical cantilever.
All normal stresses ore compressive except those preceded by o negative sign which ore tensile.
Positive shear stresses are caused by shear forces acting thus ⇌; negative shear stresses ore caused by shear forces acting thus ⇌

STUDY NO. 1

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOISE PROJECT - IDAHO

BLACK CANYON DIVERSION DAM

GRAVITY ANALYSES
NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RES. OPERATION

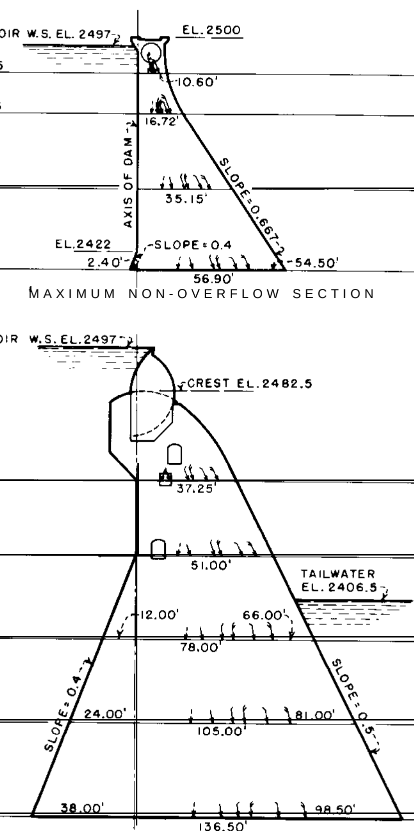
DRAWN T.W.L. SUBMITTED F. D. Kirn
TRACED C.F.M. N.H. RECOMMENDED Ivan E. Houk
CHECKED J.E.C. APPROVED W. H. Nalder

DENVER, COLO. MARCH 2, 1937 SHEET 1 OF 5 3-D-547

Figure E-12. **Black Canyon Diversion Dam-stresses for normal conditions from gravity analyses.**

RES. EMPTY			RESERVOIR FULL				
Vertical Stress	Stress Parallel to Face	Horiz. Shear Stress	Water Pressure	Vertical Stress	Stress Parallel to Face	Horiz. Shear Stress	
POUNDS PER SQUARE INCH							
*10	10	0	6	5	5	0	EL. 2487.5
**17	17	0	4	14	14	0	
29	29	0	13	-1	-1	0	EL. 2472.6
44	44	0	10	24	24	0	
57	57	0	27	-5	-5	0	EL. 2445
78	78	0	23	31	31	0	
75	86	-30	42	-7	-15	19	EL. 2416
100	116	-40	36	37	38	0	
54	108	-54	26	-6	-38	32	EL. 2449.5
64	127	-64	19	21	22	-1	
75	75	0	40	-26	-26	0	EL. 2422
100	100	0	32	26	26	0	
67	78	-27	55	-20	-32	30	EL. 2392
95	110	-38	45	36	35	4	
74	86	-30	70	-12	-25	33	EL. 2362
108	125	-43	59	52	51	3	
88	102	-35	87	1	-12	34	EL. 2327
128	148	-51	75	75	75	0	

UPSTREAM FACE



MAXIMUM NON-OVERFLOW SECTION

MAXIMUM SPILLWAY SECTION

		RESERVOIR FULL					RES. EMPTY		
Sliding Factor	Shear-friction Factor	Max Horiz. Shear Stress	Water Pressure	Vertical Stress	Stress Parallel to Face	Horiz. Shear Stress	Vertical Stress	Stress Parallel to Face	Horiz. Shear Stress
		POUNDS PER SQUARE INCH							
.391	114.5	5	0	15	16	3	10	11	2
.057	779.3	1	0	6	7	1	4	4	1
.690	34.8	27	0	41	59	27	11	16	7
.300	80.0	10	0	15	22	10	-4	-6	-3
.871	17.7	47	0	71	102	47	8	12	5
.483	31.9	23	0	34	49	23	-12	-17	-8
.877	12.3	69	0	103	149	69	19	27	12
.523	20.7	39	0	59	85	39	-7	-10	[illegible]
1.078	21.0	32	0	53	66	26	-1	-1	-1
.513	44.1	13	0	26	33	13	-11	-14	-5
.973	12.3	57	0	113	141	57	16	20	8
.514	23.2	31	0	61	76	31	-8	-10	-4
.844	10.1	67	6	140	174	67	50	62	25
.460	18.5	41	8	91	112	41	21	27	11
.736	9.1	82	19	183	225	82	73	91	36
.377	17.7	48	23	120	144	48	39	49	19
.630	8.4	95	35	224	272	95	96	120	48
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	70	28

HORIZONTAL SECTION

DOWNSTREAM FACE

Resultant-concrete weight and earthquake (horizontal upstream and vertical upward). Resultant-concrete weight, and earthquake (horizontal downstream and vertical upward). Resultant- water pressure, weight, and earthquake (horizontal upstream and vertical upward).
Resultant-water pressure, weight, and earthquake (horizontal downstream and vertical upward). Resultant-water pressure, weight, uplift and earthquake (horizontal upstream and vertical upward). Resultant-water pressure, weight, uplift, and earthquake (horizontal downstream and vertical upward).

Sliding Factor = $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$ Shear-friction Factor = $\frac{(\text{Weight}-\text{Uplift}) \times \text{Coefficient of Internal Friction} + \text{Thickness} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 400 pounds per square inch. Coefficient of internal friction = 0.65.
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to tailwater pressure of downstream face, acting over one-half area of horizontal section; assumed to be unaffected by earthquake.
Vertical earthquake acceleration and horizontal earthquake acceleration -0.1 gravity, period = 1 second.
Total load carried by vertical cantilever.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses are caused by shear forces acting thus ⇌; negative shear stresses are caused by shear forces acting thus ⇌.
* Including earthquake acceleration (horizontal upstream and vertical upward). ** Including earthquake acceleration (horizontal downstream and vertical upward).

STUDY NO. 1

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOISE PROJECT - IDAHO

BLACK CANYON DIVERSION DAM

GRAVITY ANALYSES-NONOVERFLOW AND SPILLWAY SECTIONS
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION UPWARD
RESERVOIR EMPTY AND NORMAL FULL RES. OPERATION

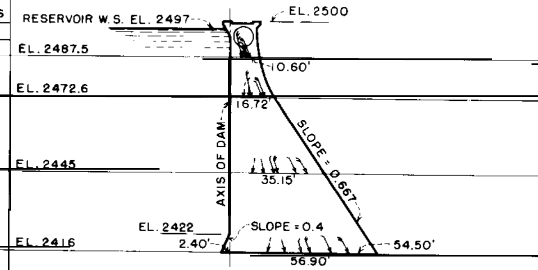
DRAWN T.W.L. SUBMITTED F. W. Kiene
TRACED C.F.M. N.H. RECOMMENDED Ivan E. Houk
CHECKED J.E.C. APPROVED W. H. Nalder

DENVER, COLO. FEB. 23, 1937 SHEET 4 OF 5 3-D-550

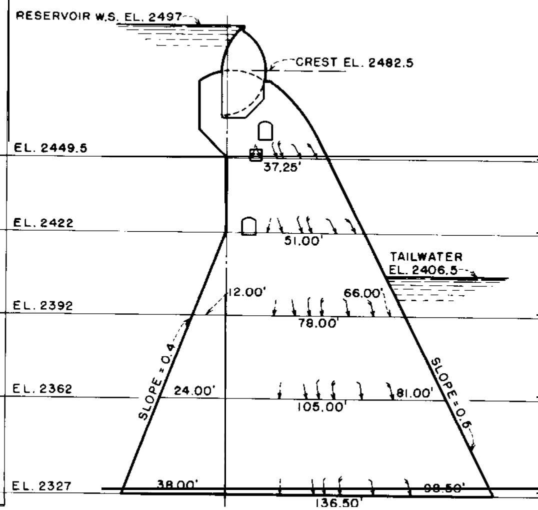
Figure E-13. **Black Canyon Diversion Dam-gravity analyses including effects of earthquake, vertical acceleration upward.**

RES. EMPTY Vertical Stress	RES. EMPTY Stress Parallel to Face	RES. EMPTY Horiz. Shear Stress	RESERVOIR FULL Water Pressure	RESERVOIR FULL Vertical Stress	RESERVOIR FULL Stress Parallel to Face	RESERVOIR FULL Horiz. Shear Stress	
POUNDS PER SQUARE INCH							
*8	8	0	5	3	3	0	EL. 2487.5
**14	14	0	3	12	12	0	
22	22	0	11	−3	−3	0	EL. 2472.6
37	37	0	8	22	22	0	
45	45	0	23	−7	−7	0	EL. 2445
65	65	0	18	29	29	0	
59	68	−23	35	−10	−17	18	EL. 2416
84	98	−34	29	35	35	−2	
43	86	−43	22	−7	−37	29	EL. 2449.5
53	106	−53	15	19	23	−4	
60	60	0	33	−26	−26	0	EL. 2422
84	84	0	25	26	26	0	
52	60	−21	46	−21	−32	27	EL. 2392
80	93	−32	36	35	35	0	
57	67	−23	58	−15	−27	29	EL. 2362
91	106	−36	47	48	48	0	
68	79	−27	72	−6	−18	31	EL. 2327
108	126	−43	60	[illegible]	[illegible]	[illegible]	

UPSTREAM FACE



MAXIMUM NON-OVERFLOW SECTION



MAXIMUM SPILLWAY SECTION

Elevation	Sliding Factor	Shear-friction Factor	RESERVOIR FULL Max. Horiz. Shear Stress	RESERVOIR FULL Water Pressure	RESERVOIR FULL Vertical Stress	RESERVOIR FULL Stress Parallel to Face	RESERVOIR FULL Horiz. Shear Stress	RES. EMPTY Vertical Stress	RES. EMPTY Stress Parallel to Face	RES. EMPTY Horiz. Shear Stress
			POUNDS PER SQUARE INCH							
EL. 2487.5	.439	127.5	4	0	13	14	3	9	9	2
	.021	2674.2	1	0	4	5	1	3	3	0
EL. 2472.6	.759	39.8	24	0	36	52	24	10	15	7
	.266	113.6	7	0	10	15	7	−5	−7	−3
EL. 2445	.958	20.4	41	0	61	88	41	9	12	6
	.462	42.3	16	0	25	36	16	−12	−17	−8
EL. 2416	.964	14.2	59	0	89	128	59	18	25	12
	.509	27.0	29	0	44	64	29	−8	−12	−5
EL. 2449.5	1.217	24.1	29	0	46	57	23	0	0	0
	.481	61.1	11	0	19	24	9	−10	−12	−5
EL. 2422	1.079	14.1	49	0	97	122	49	15	19	8
	.488	31.1	23	0	45	57	23	−9	−11	−4
EL. 2392	.939	11.6	61	4	126	156	61	43	54	22
	.443	24.5	31	7	69	85	31	15	19	7
EL. 2362	.829	10.3	70	15	156	191	70	62	78	31
	.361	23.6	36	19	92	111	36	29	36	14
EL. 2327	.718	9.5	81	28	190	[illegible]	41	[illegible]	103	41
	[illegible]	[illegible]	41	[illegible]	[illegible]	[illegible]	41	42	52	21

HORIZONTAL SECTION

DOWNSTREAM FACE

STUDY NO. 1

&Resultant-concrete weight and earthquake (horizontal upstream and vertical downward). ↙Resultant-concrete weight and earthquake (horizontal downstream and vertical downward). ↘Resultant-water pressure, weight, and earthquake (horizontal upstream and vertical downward). ⤹ Resultant-voter pressure, weight, and earthquake (horizontal downstream and vertical downward). ↘Resultant-water pressure, weight, uplift and earthquake (horizontal upstream and vertical downward). ⤹Resultant-water pressure, weight, uplift, and earthquake (horizontal downstream and vertical downward).

Sliding Factor = $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$. Shear-friction Factor = $\frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Thickness} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$.

Unit weight Of concrete = 150 pounds per cubic foot. Unit shear resistance = 400 pounds per square inch. Coefficient of internal friction = 0.65.

Uplift pressure varies as a straight tine from reservoir water pressure at upstream face to tailwater pressure at downstream face, acting over one-half area of horizontal section; assumed to be unaffected by earthquake.

Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1 gravity, period = 1 second.

Total load carried by vertical cantilever.

All normal stresses ore compressive except those preceded by a negative sign, which ore tensile.

Positive shear stresses are caused by shear forces acting thus ⇄; negative shear stresses are caused by shear forces acting thus ⇆.

* Including earthquake acceleration (horizontal upstream and vertical downward). ** Including earthquake acceleration (horizontal downstream and vertical downward).

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOISE PROJECT - IDAHO

BLACK CANYON DIVERSION DAM

GRAVITY ANALYSES-NON-OVERFLOW AND SPILLWAY SECTIONS
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION DOWNWARD
RESERVOIR EMPTY AND NORMAL FULL RES. OPERATION

DRAWN T.W.L.	SUBMITTED F. W. Kirn
TRACED C.F.M. F.B.	RECOMMENDED Ivan E. Houk
CHECKED J.C.C.	APPROVED W. H. Nalder

DENVER, COLO. FEB. 23, 1937 SHEET 5 OF 5 — 3-D-551

Figure *E-14.* Black Canyon Diversion Dam-gravity analyses including effects of earthquake, vertical acceleration downward.

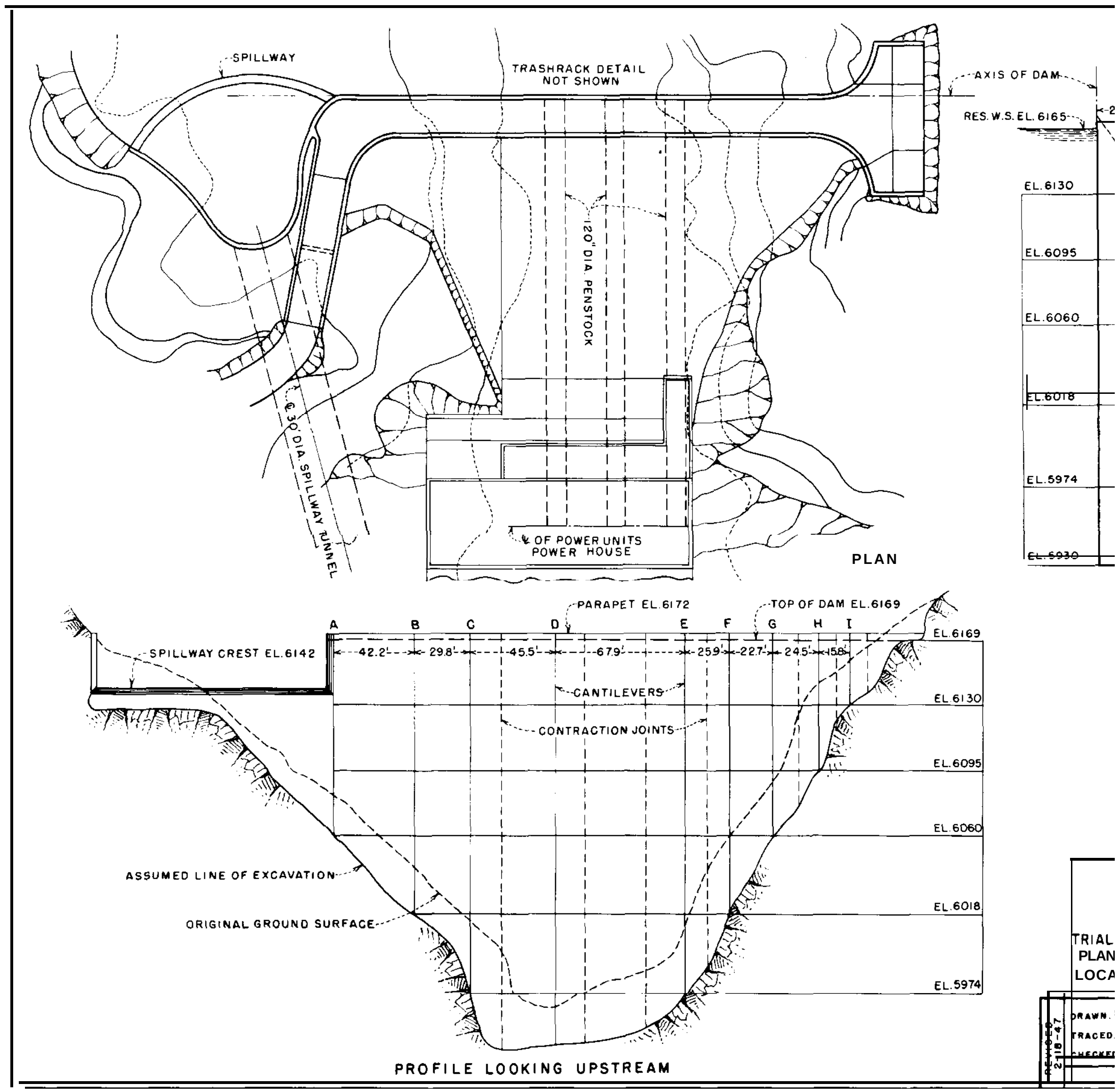


Figure E-15. **Kortes Dam-plan, elevation, and maximum section.**

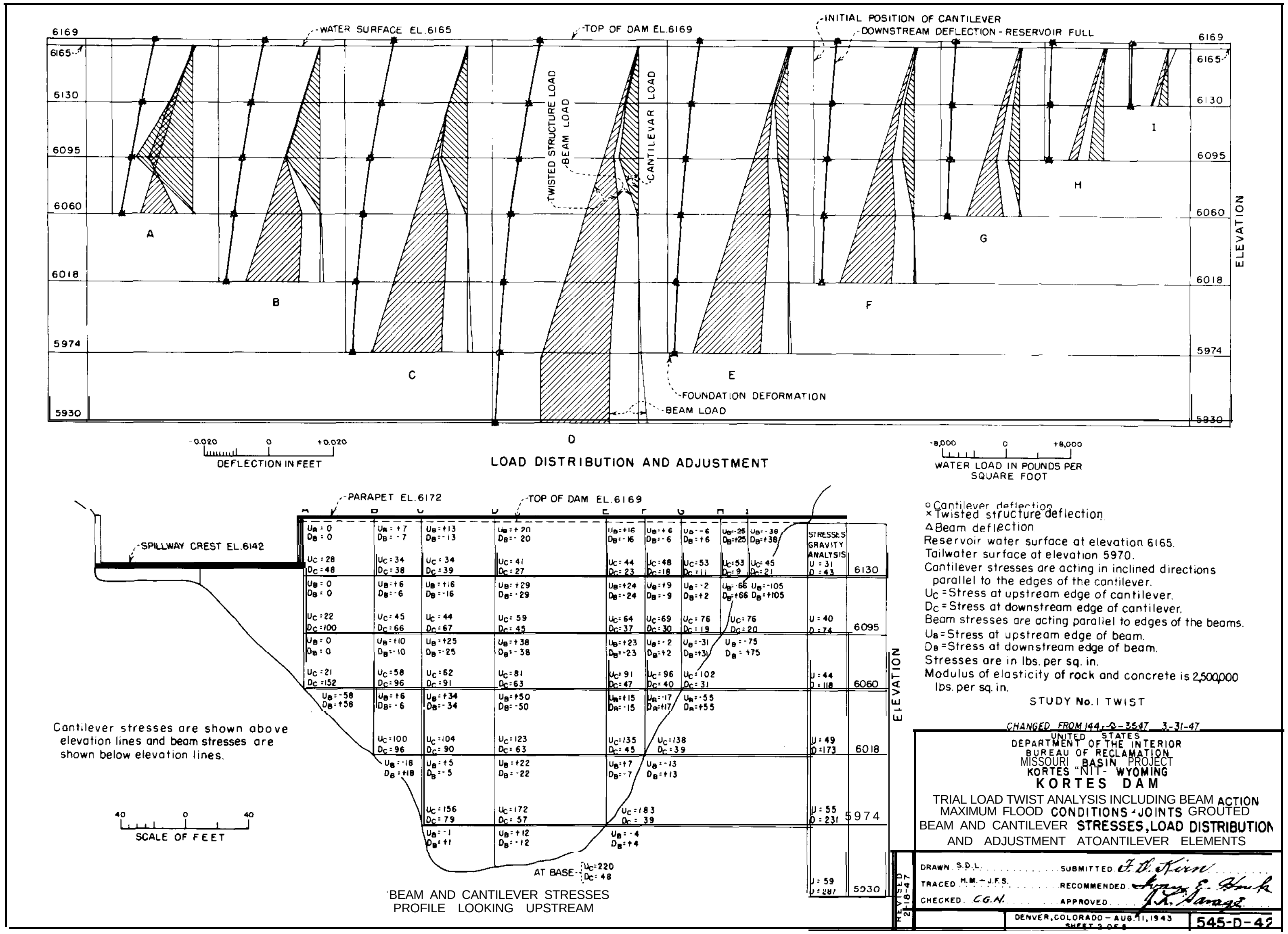
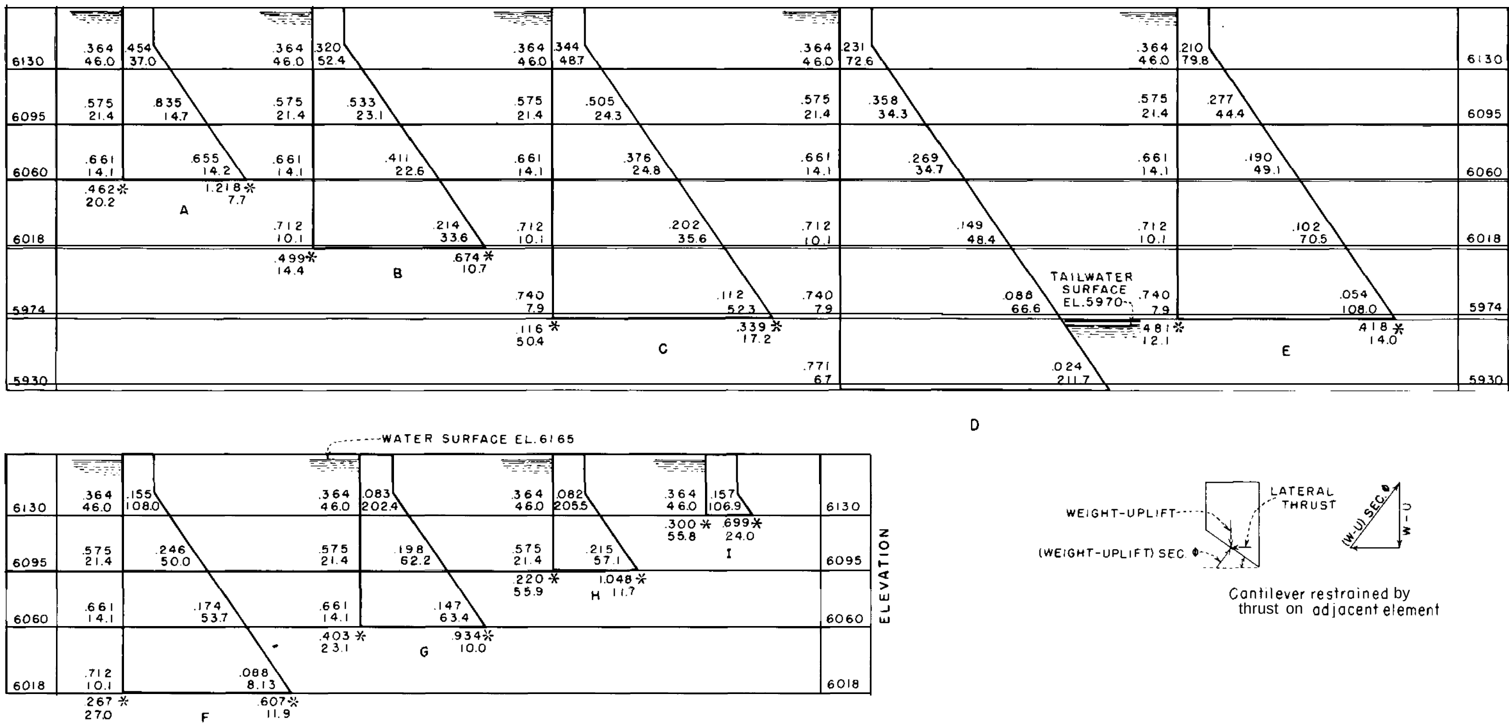


Figure E-16. **Kortes Dam-stresses and load distribution from trial-load twist analysis.**



Sliding factors and shear-friction factors of safety for gravity analysis are shown at upstream side
Sliding factors and shear-friction factors of safety for trial load analysis are shown at downstream side.
Sliding factors and shear-friction factors are shown above the elevation lines
* Stability factors are for inclined abutment planes

Sliding factor $= \dfrac{\text{Horizontal Force}}{\text{Weight-Uplift}}$ (for horizontal planes) Sliding factor $= \dfrac{\text{Horizontal Force}}{(\text{Weight-Uplift})\text{Sec}\,\phi}$ (for abutment planes)

Shear-friction factor $= \dfrac{(\text{Weight-Uplift}) \times \text{Coefficient of internal friction} + \text{Base area} \times \text{Unit shear resistance}}{\text{Horizontal Force}}$ (for horizontal planes)

Shear-friction factor $= \dfrac{(\text{Weight-Uplift})\text{Sec}\,\phi \times \text{Coefficient of internal friction} + \text{inclined base area} \times \text{Unit shear resistance}}{\text{Horizontal Force}}$ (for abutment planes)

Coefficient of internal friction = 0.65. Unit shear resistance = 400 pounds per square inch Unit weight of concrete = 150 lb per cu ft
Uplift pressure varies as a straight line from reservoir water pressure at the upstream face to zero or tailwater pressure at the downstream face, acting over two-thirds area of horizontal section

STUDY NO. I TWIST

CHANGED FROM 144-D-3548 J-3147

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI BASIN PROJECT
KORTES UNIT - WYOMING
KORTES DAM
TRIAL LOAD TWIST ANALYSIS INCLUDING BEAM ACTION
MAXIMUM FLOOD CONDITIONS - JOINTS GROUTED
SLIDING FACTORS AND SHEAR FRICTION FACTORS
OF SAFETY FOR TRIAL LOAD AND GRAVITY ANALYSES

DRAWN C.G. SUBMITTED F. D. Kirn
TRACED M.-E.W. RECOMMENDED [signature]
CHECKED MDC. APPROVED [signature]
REVISED 2-3-47
DENVER, COLORADO — AUG.-11-1943 SHEET 3 OF 5
545-D-43

Figure E-27. **Kortes Dam-stability factors from trial-load twist analysis.**

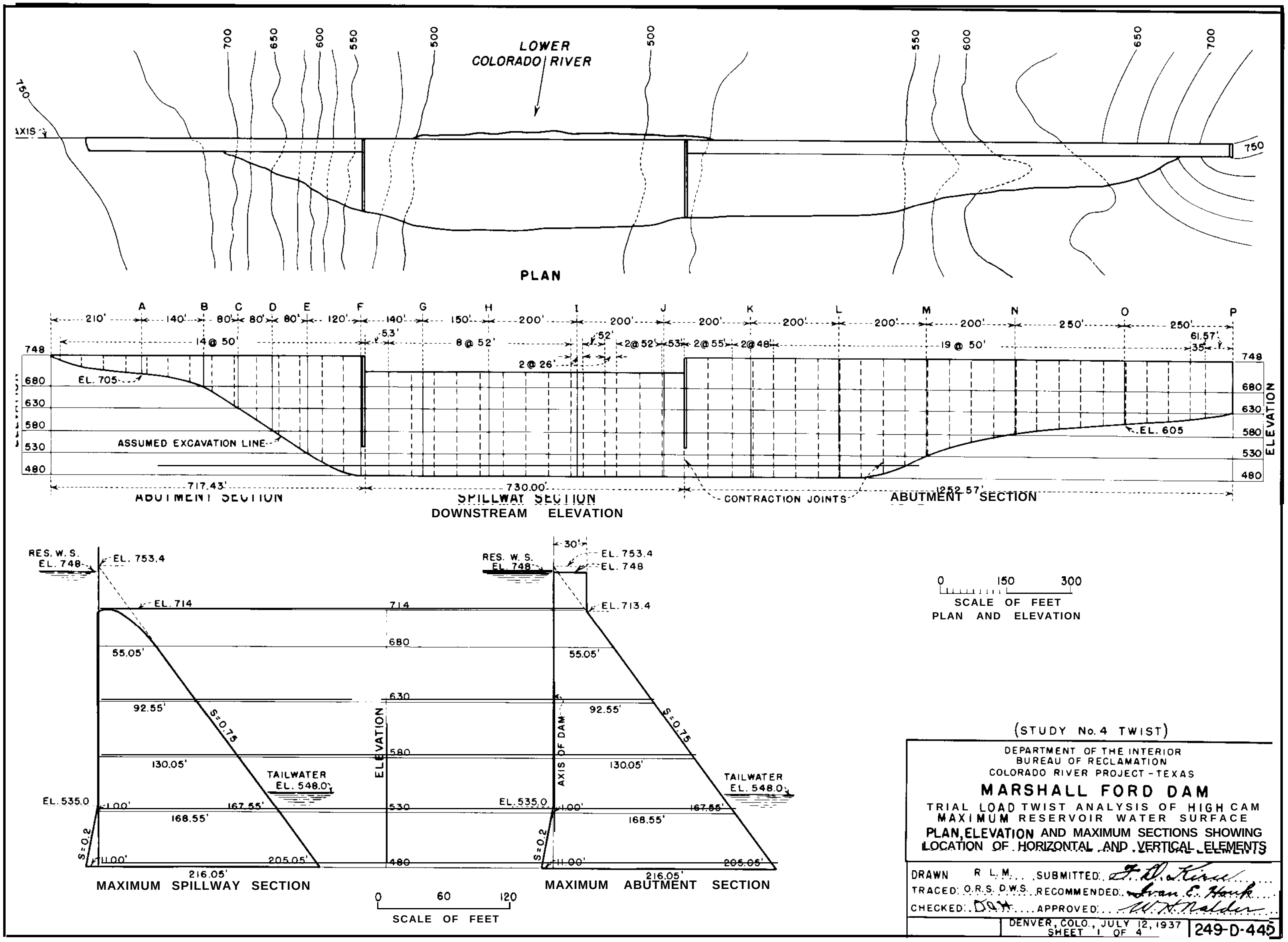
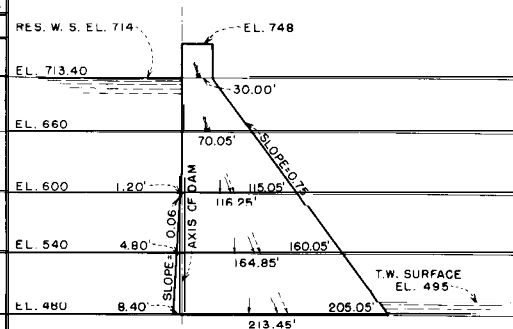


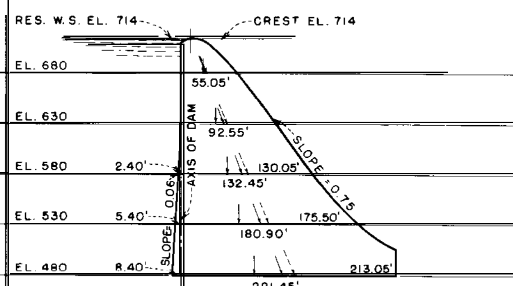
Figure E-18. **Marshall Ford Dam-plan, elevation, and maximum sections.**

Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress	Normal Res. W. S. El.: Water Pressure	Normal Res. W. S. El.: Vertical Stress	Normal Res. W. S. El.: Stress Parallel to Face	Normal Res. W. S. El.: Horiz. Shear Stress	
Pounds per square inch							
36	36	0	0.3	36	36	0	EL. 713.40
111	111	0	23	97	97	0	EL. 660
165	165	−9.9	49	119	119	−4.2	EL. 600
213	214	−13	76	136	136	−3.6	EL. 540
266	267	−16	102	155	155	−3.2	EL. 480
44	44	0	15	38	38	0	EL. 680
103	103	0	36	73	73	0	EL. 630
152	153	−9.1	58	96	96	−2.3	EL. 580
201	201	−12	80	125	125	−2.7	EL. 530
243	244	−15	102	141	141	−2.3	EL. 480

UPSTREAM FACE



MAXIMUM NON-OVERFLOW SECTION



MAXIMUM SPILLWAY SECTION

Sliding Factor	Shear-Friction Factor	Normal Res.: Max. Horiz. Shear Stress	Normal Res.: Water Pressu[re]
0	∞	0	
.191	36.6	12	
.365	14.1	36	
.458	9.0	67	
.517	6.7	94	7
.229	68.7	8.6	
.376	19.9	30	
.455	11.6	54	
.511	8.7	112	
.529	6.8	81	7

HORIZONTAL SECTION

↓ Resultant-concrete weight only. ↘ Resultant-water pressure and weight ↘ Resultant-water pressure, weight, and uplift.

Sliding Factor $= \dfrac{\text{Horizontal Force}}{\text{Weight - Uplift}}$

Shear-friction Factor $= \dfrac{(\text{Weight - Uplift}) \times \text{Coefficient of Internal Friction} + \text{Base Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 300 pounds per square inch.
Coefficient of internal friction = 0.65.
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontal section.
Total load carried by vertical cantilever.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses are caused by shear forces acting thus ⇌.
Negative shear stresses are caused by shear forces acting thus ⇋.

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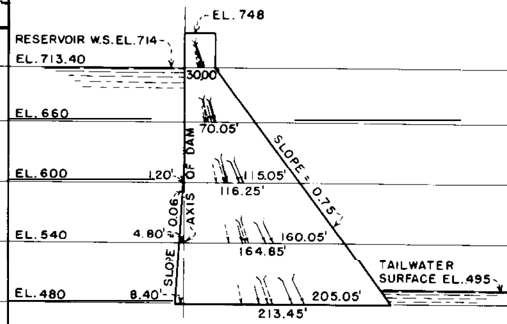
Figure E-19. Marshall Ford Dam-gravity analyses for normal conditions.

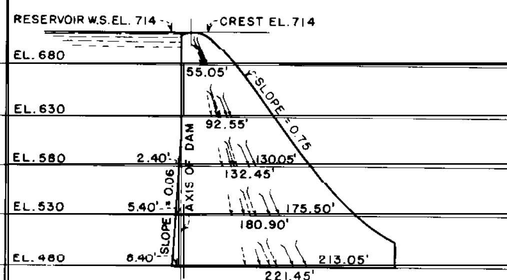
Pounds per square inch

Section / Elevation	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress	Normal Res. W.S. El.: Water Pressure	Normal Res. W.S. El.: Vertical Stress	Normal Res. W.S. El.: Stress Parallel to Face	Normal Res. W.S. El.: Horiz. Shear Stress
Non-overflow, EL. 713.40	*27	27	0	0.7	27	27	0
	**52	52	0	-0.1	52	52	0
Non-overflow, EL. 660	105	105	0	30	86	86	0
	139	139	0	22	128	128	0
Non-overflow, EL. 600	157	158	-9.4	60	98	98	-2.3
	205	205	-12	49	163	164	-6.9
Non-overflow, EL. 540	205	205	-12	90	106	106	-1.0
	264	265	-16	76	192	192	-7.0
Non-overflow, EL. 480	256	257	-15	120	117	117	0.2
	329	331	-20	103	223	224	-7.2
Spillway, EL. 680	45	45	0	19	36	36	0
	52	52	0	13	48	48	0
Spillway, EL. 630	101	101	0	45	61	61	0
	125	125	0	35	99	99	0
Spillway, EL. 580	148	148	-8.9	70	76	76	-0.3
	187	188	-11	58	136	136	-4.7
Spillway, EL. 530	197	197	-12	95	101	101	-0.4
	245	245	-15	80	174	174	-5.6
Spillway, EL. 480	237	238	-14	120	109	109	0.7
	298	299	-18	103	200	201	-5.8

UPSTREAM FACE



MAXIMUM NON-OVERFLOW SECTION



MAXIMUM SPILLWAY SECTION

Pounds per square inch

Section / Elevation	Sliding Factor	Shear-friction Factor	Max. Horiz. Shear Stress	Water Pressure	Normal Res. W.S. El.: Vertical Stress	Normal Res. W.S. El.: Stress Parallel to Face	Normal Res. W.S. El.: Horiz Shear Stress	Reservoir Empty: Vertical Stress	Reservoir Empty: Stress Parallel to Face	Reservoir Empty: Horiz. Shear Stress
Non-overflow, EL. 713.40	.091	90.2	5.4		52	52	0	52	52	0
	-.091	-90.3	-5.4		27	27	0	27	27	0
Non-overflow, EL. 660	.332	19.1	27		35	55	27	16	25	12
	.044	143.3	5.0		-7	-10	-5.0	-18	-28	-14
Non-overflow, EL. 600	.519	9.0	63		84	132	63	24	37	18
	.195	23.9	22		19	30	14	-23	-36	-17
Non-overflow, EL. 540	.611	6.1	105		140	219	105	38	60	29
	.282	13.2	42		55	86	41	-21	-33	-16
Non-overflow, EL. 480	.667	4.7	144	7	198	306	144	53	82	40
	.335	9.3	64	8	92	140	63	-21	-32	-15
Spillway, EL. 680	.393	35.8	14		18	28	14	10	16	7.5
	.055	225.1	5.2		7	11	5.2	3	4	2.0
Spillway, EL. 630	.541	12.4	47		62	98	47	23	35	17
	.192	34.8	18		25	38	18	-2	-3	-1.4
Spillway, EL. 580	.616	7.7	82		109	171	82	35	55	26
	.270	17.6	37		49	77	37	-4	-6	-3.0
Spillway, EL. 530	.668	6.0	169		132	349	169	33	86	42
	.323	12.3	76		60	157	76	-15	-40	-19
Spillway, EL. 480	.681	4.8	117	7	193	193	0	59	59	0
	.343	9.5	61	8	101	101	0	-3	-3	0

HORIZONTAL SECTION — DOWNSTREAM FACE

Resultant-concrete weight and earthquake (horizontal upstream and vertical upward). Resultant-concrete weight and earthquake (horizontal downstream and vertical upward) Resultant - water pressure, weight and earthquoke (horizontal upstream and vertical upward). Resultant-water pressure, weight, and earthquake (horizontal downstream and vertical upward). Resultant- water oressure, weight, uplift and earthquake (horizontal upstream and vertical upward). Resultant-water pressure, weight, uplift and earthquake (horizontal downstream ond vertical upward).

Sliding Factor $= \dfrac{\text{Horizontal Force}}{\text{Weight-Uplift}}$ Shear-friction Factor $= \dfrac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Base Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 150 pounds per cubic foot. Unit shear resistance = 300 pounds per square inch. Coefficient of Internal Friction: 0.65.

Uplift pressure varies as a stmight line from reservoir water pressure at upstream face to zero or toilwaterpressure atdownstream face, acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake.

Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1g., period = one second.

Total load carried by vertical cantilever.

All normalstresses are compressive except those preceded by a negative sign which are tensile.

Positive shearstresses are caused by shear force acting thus= ; negative shearstresses are caused by shear forces acting thus ⇌.

* Including earthquake acceleration (horizontal upstream and vertical upward).

** Including earthquake acceleration (horizontal downstream and vertical upward).

STUDY No. 2A

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS

MARSHALL FORD DAM

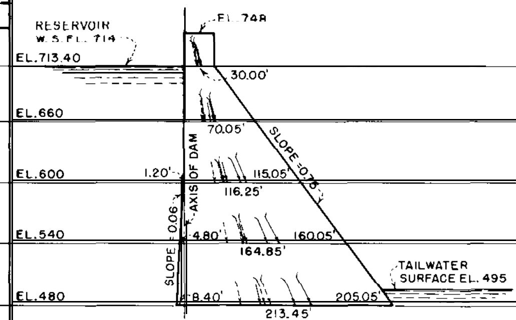
GRAVITY ANALYSES OF HIGH DAM
NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION UPWARD

DRAWN. H.J.B. SUBMITTED. J. D. Kien
TRACED. O.R.S.-N.H. RECOMMENDED. Ivan E. Houk
CHECKED. S.D.L. APPROVED. J. L. Savage

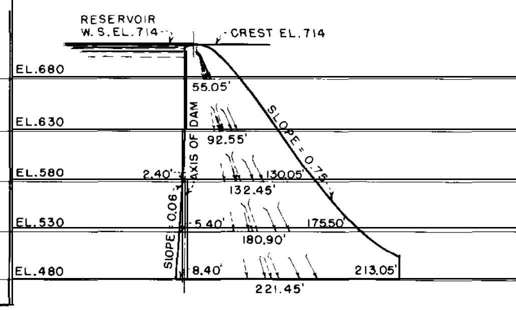
DENVER, COLO., DEC. 6, 1937 SHEET 4 OF 6 | 249-D-54

Figure E-20. **Marshall Ford Dam-gravity analyses including effects of earthquake, vertical acceleration upward.**

Upstream face: Reservoir empty – Vertical Stress	Upstream face: Reservoir empty – Stress Parallel to Face	Upstream face: Reservoir empty – Horiz. Shear Stress	Upstream face: Normal res. W. S. El. – Water Pressure	Upstream face: Normal res. W. S. El. – Vertical Stress	Upstream face: Normal res. W. S. El. – Stress Parallel to Face	Upstream face: Normal res. W. S. El. – Horiz Shear Stress	Elevation	Horizontal section: Sliding Factor	Horizontal section: Shear-Friction Factor	Horizontal section: Max Horiz Shear Stress	Horizontal section: Water Pressure	Downstream face: Normal res. W. S. El. – Vertical Stress	Downstream face: Normal res. W. S. El. – Stress Parallel to Face	Downstream face: Normal res. W. S. El. – Horiz. Shear Stress	Downstream face: Reservoir empty – Vertical Stress	Downstream face: Reservoir empty – Stress Parallel to Face	Downstream face: Reservoir empty – Horiz. Shear Stress
Pounds per square inch												Pounds per square inch					
Maximum non-overflow section																	
*20	20	0	0.7	20	20	0	EL.713.40	.112	88.9	5.4		45	45	0	45	45	0
**45	45	0	-0.2	45	45	0		-.112	-89.0	-5.4		20	20	0	20	20	0
83	83	0	25	66	66	0	EL.660	.376	20.8	25		33	51	25	16	26	12
117	117	0	17	109	109	0		.013	621.0	-7.0		-9.3	-15	-7.0	-18	-28	-13
125	125	-7.5	50	74	74	-1.4	EL.600	.584	10.0	56		75	117	56	24	37	18
172	172	-10	39	140	140	-6.1		.167	34.7	15		10	15	7.3	-23	-36	-17
162	163	-9.7	75	79	79	-0.2	EL.540	.687	6.7	92		123	192	92	37	58	28
222	222	-13	61	165	165	-6.3		.260	17.7	30		37	58	28	-23	-36	-17
203	204	-12	100	86	86	0.8	EL.480	.754	5.1	125	5	172	265	125	50	78	37
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-23	-37	-18
Maximum spillway section																	
36	36	0	17	29	29	0	EL.680	.452	39.5	12		16	25	12	9	14	6.7
43	43	0	10	40	40	0		.019	967.3	3.5		5	7	3.5	2	2	1.1
80	80	0	38	47	47	0	EL.630	.614	13.8	41		55	85	41	21	33	16
105	105	0	28	84	84	0		.162	52.2	12		17	26	12	-4	-6	-2.8
117	118	-7.0	59	56	56	0.1	EL.580	.695	8.6	71		95	148	71	32	50	24
156	157	-9.4	46	117	117	-4.2		.246	24.3	26		35	54	26	-7	-11	-5.1
157	157	-9.4	79	76	76	0.2	EL.530	.756	6.6	147		115	303	147	31	82	40
204	205	-12	64	149	149	-5.1		.305	16.4	54		42	111	54	-17	-44	-21
188	189	-11	100	81	81	1.2	EL.480	.772	5.3	101	5	166	166	0	54	54	0
250	251	-15	83	172	173	-5.4		.329	12.4	44	6	75	75	0	-8	-8	0



MAXIMUM NON-OVERFLOW SECTION



UPSTREAM FACE — MAXIMUM SPILLWAY SECTION — HORIZONTAL SECTION — DOWNSTREAM FACE

Resultant-concrete weight and earthquake (horizontal upstream and vertical downward). Resultant-concrete weight and earthquake (horizontal downstream and vertical downword). Resultant-water pressure, weight ond earthquake (horizontal upstream ond vertical downward). Resultant-water pressure, weight and earthquake (horizontal downstream ond vertical downward). Resultant-water pressure, weight, uplift and earthquake (horizontal upstream ond vertical downword). Resultant-water pressure, weight, uplift and earthquake (horizontal downstream and vertical downward).

Sliding Factor $= \frac{\text{Horizontal Force}}{\text{Weight - Uplift}}$, Shear-friction Factor $= \frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Base Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$,

Unit weight of concrete = 150 pounds per cubic foot. Unit Shear Resistance = 300 pounds per square inch. Coefficient of Internal Friction = 0.65

Uplift pressure varies as o straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontol section, assumed to be unaffected by earthquake.

Vertical earthquake acceleration ond horizontal earthquake acceleration = 0.1 g, period = one second.

Totol lood carried by vertical cantilever.

All normal stresses ore compressive except those preceded by o negative sign which are tensile.

Positive shear stresses ore caused by shear forces acting thus ⇌, negative shear stresses are caused by shear forces acting thus ⇋.

* Including earthquake occelerotion (horizontol upstream and vertical downward).

** Including eorthquoke acceleration (horizontal downstream and vertical downward).

STUDY No. 2A

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLORADO RIVER PROJECT - TEXAS
MARSHALL FORD DAM
GRAVITY ANALYSES OF HIGH DAM
NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION DOWNWARD

DRAWN H.J.B. SUBMITTED F. W. Kirn
TRACED O.R.S.-D.W.S. RECOMMENDED Ivan E. Houk
CHECKED S.D.L. APPROVED J. L. Savage

DENVER, COLO. DEC. 6, 1937 SHEET 5 OF 6 249-D-542

Figure E-21. **Marshall Ford Dam-gravity analyses including effects of earthquake, vertical acceleration downward.**

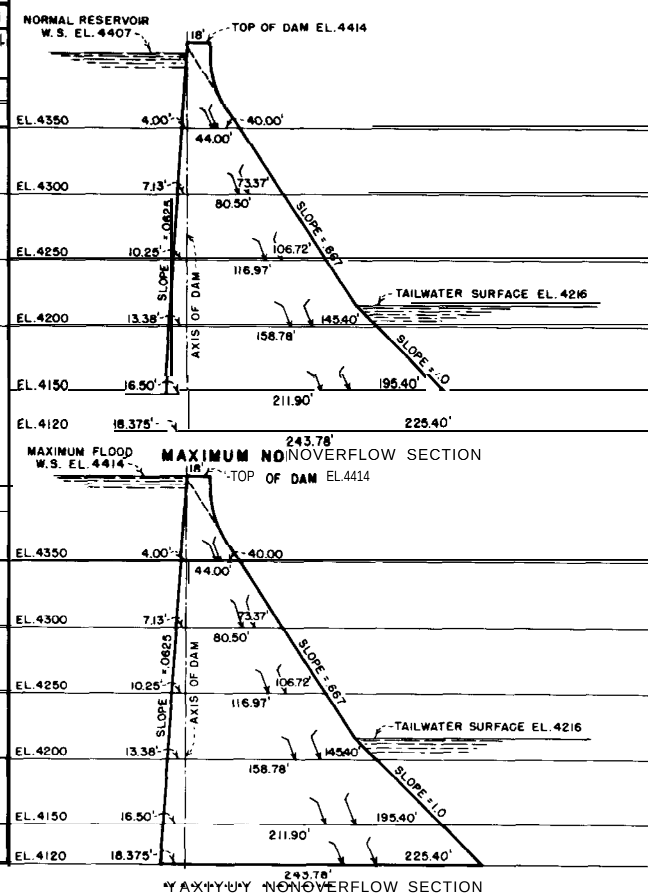
NORMAL RESERVOIR OPERATION

Elevation	Upstream face: Water pressure	Upstream face: Vertical stress	Upstream face: Stress parallel to face	Upstream face: Horizontal shear stress	Sliding factor	Shear-friction factor	Maximum horizontal shear stress	Downstream face: Water pressure	Downstream face: Vertical stress	Downstream face: Stress parallel to face	Downstream face: Horizontal shear stress
	Pounds per square inch							Pounds per square inch			
EL. 4350	25	29	30	-.2	.549	26.1	31	0	45	66	31
EL. 4300	46	32	32	.9	.715	13.9	57	0	85	123	57
EL. 4250	68	36	36	2.0	.771	9.6	85	0	128	185	85
EL. 4200	90	51	51	2.4	.822	7.7	148	7	155	304	148
EL. 4150	112	78	78	2.1	.812	7.1	143	29	171	314	143
EL. 4120	125	94	94	1.9	.769	7.0	145	42	187	331	145

MAXIMUM FLOOD OPERATION

Elevation	Upstream face: Water pressure	Upstream face: Vertical stress	Upstream face: Stress parallel to face	Upstream face: Horizontal shear stress	Sliding factor	Shear-friction factor	Maximum horizontal shear stress	Downstream face: Water pressure	Downstream face: Vertical stress	Downstream face: Stress parallel to face	Downstream face: Horizontal shear stress
EL. 4350	28	13	13	.9	.711	20.7	42	0	62	90	42
EL. 4300	49	16	16	2.1	.826	12.2	68	0	101	147	68
EL. 4250	71	20	20	3.2	.852	8.8	96	0	145	210	96
EL. 4200	93	36	36	3.5	.888	7.2	164	7	171	335	164
EL. 4150	115	65	65	3.1	.869	6.7	156	29	185	340	156
EL. 4120	128	82	82	2.9	.819	6.6	157	42	199	357	157

UPSTREAM FACE — HORIZONTAL SECTION — DOWNSTREAM FACE — STUDY No. A



Resultant - water pressure and weight. Resultant - water pressure, weight and uplift.

Sliding Factor = $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$ · Shear-friction Factor = $\frac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 137 pounds per cubic foot. Unit shear resistance = 400 pounds per square inch. Coefficient of internal friction = 0.65.
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontal section.
Total load carried by vertical cantilever.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses are caused by shear forces acting thus ⇌
Negative shear stresses are caused by shear forces acting thus ⇌.

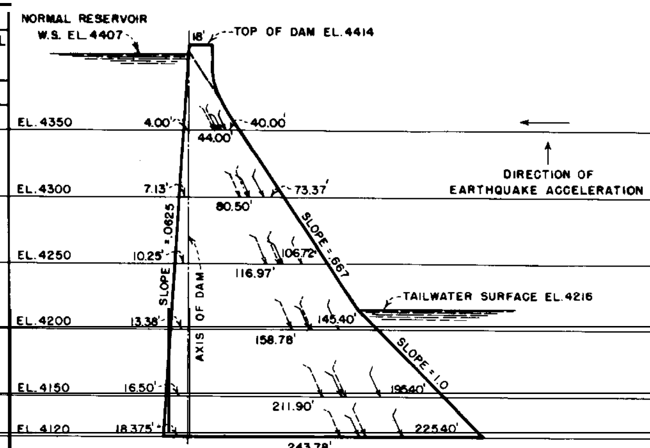
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
RIO GRANDE PROJECT - NEW MEXICO - TEXAS
ELEPHANT BUTTE DAM
GRAVITY ANALYSES OF MAXIMUM NONOVERFLOW SECTION
NORMAL FULL RESERVOIR OPERATION AND
MAXIMUM FLOOD OPERATION

DRAWN J.T.R.-H.P.W. ...... SUBMITTED F. O. Kirn
TRACED G.V.J.-R.L.A. ...... RECOMMENDED Ivan E. Houk
CHECKED S.D.L. ...... APPROVED J. L. Savage
DENVER, COLORADO, MAY 7, 1940 — 24-D-573

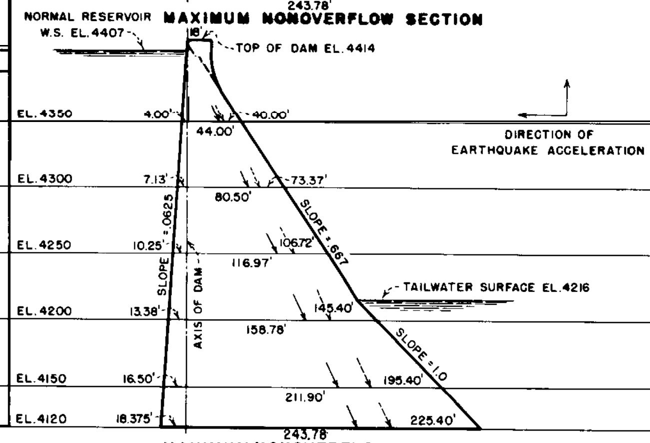
Figure E-22. Elephant Butte Dam-gravity analyses for maximum flood condition.

NORMAL RESERVOIR W.S. EL.			
WATER PRESSURE	VERTICAL STRESS	STRESS PARALLEL TO FACE	HORIZONTAL SHEAR STRESS
Pounds per square inch			
* 29	3	3	1.6
** 28	32	33	-.3
53	-3	-3	3.5
51	35	35	.4
76	-10	-10	5.4
75	40	39	2.2
99	0	-2	6.2
99	56	56	2.8
121	27	26	5.9
123	86	86	2.3
135	42	41	5.8
138	104	104	2.1
NORMAL RESERVOIR W.S. EL.			
*** 32	6	6	1.6
57	0	-1	3.6
83	-6	-6	5.6
108	5	4	6.4
133	35	34	6.1
147	51	50	6.1

UPSTREAM FACE



MAXIMUM NONOVERFLOW SECTION



MAXIMUM NONOVERFLOW SECTION

NORM		
SLIDING FACTOR	SHEAR-FRICTION FACTOR	MAXIM HORIZON SHEAR ST
*.812	17.7	48
**.535	23.9	34
.978	10.1	80
.693	12.7	63
1.021	7.3	116
.745	8.8	94
1.070	5.9	201
.789	7.1	163
1.056	5.5	196
.770	6.6	157
1.005	5.3	200
.725	6.5	160
NORM		
++		
.869	***16.7	51
1.049	9.5	86
1.096	6.8	125
1.156	5.5	217
1.161	5.2	211
1.112	5.1	215

HORIZONTAL SECTI

Resultant-water pressure, weight, and earthquake (horizontal upstream) Resultant-water pressure, weight, uplift and earthquake (horizontal upstream).
Resultant-water pressure, weight, and earthquake (vertical upward). Resultant-water pressure, weight, uplift and earthquake (vertical upward).
Resultant-water pressure, weight, and earthquake (horizontal upstream and vertical upward). Resultant-water pressure, weight, uplift and earthquake (horizontal upstream and vertical upward).

Sliding Factor = $\frac{\text{Horizontal Force}}{\text{Weight-Uplift}}$, Shear-friction Factor = $\frac{\text{(Weight-Uplift) x Coefficient of Internal Friction + Horizontal Area x Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 137 pounds per cubic foot. Unit shear resistance = 400 pounds per square inch. Coefficient of internal friction = 0.65
Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting Over two-thirds the Oreo of the horizontal section; assumed to be unaffected by earthquake.
Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1g, period = one second.
Total load carried by vertical contilever.
All normal stresses are compressive except those preceded by a negative sign, which are tensile.
Positive shear stresses ore caused by shear forces acting thus ⇌. Negative shear stresses are caused by shear forces acting thus ⇋
Earthquake acceleration • * Horizontal upstream. ** Vertical upward *** Horizontal upstream and Vertical upward.
++ Sliding factors computed for earthquake acceleration, horizontal upstream and vertical downward.

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CHECKED

Figure **E-23. Elephant Butte Dam-gravity analyses including effects of earthquake accele**

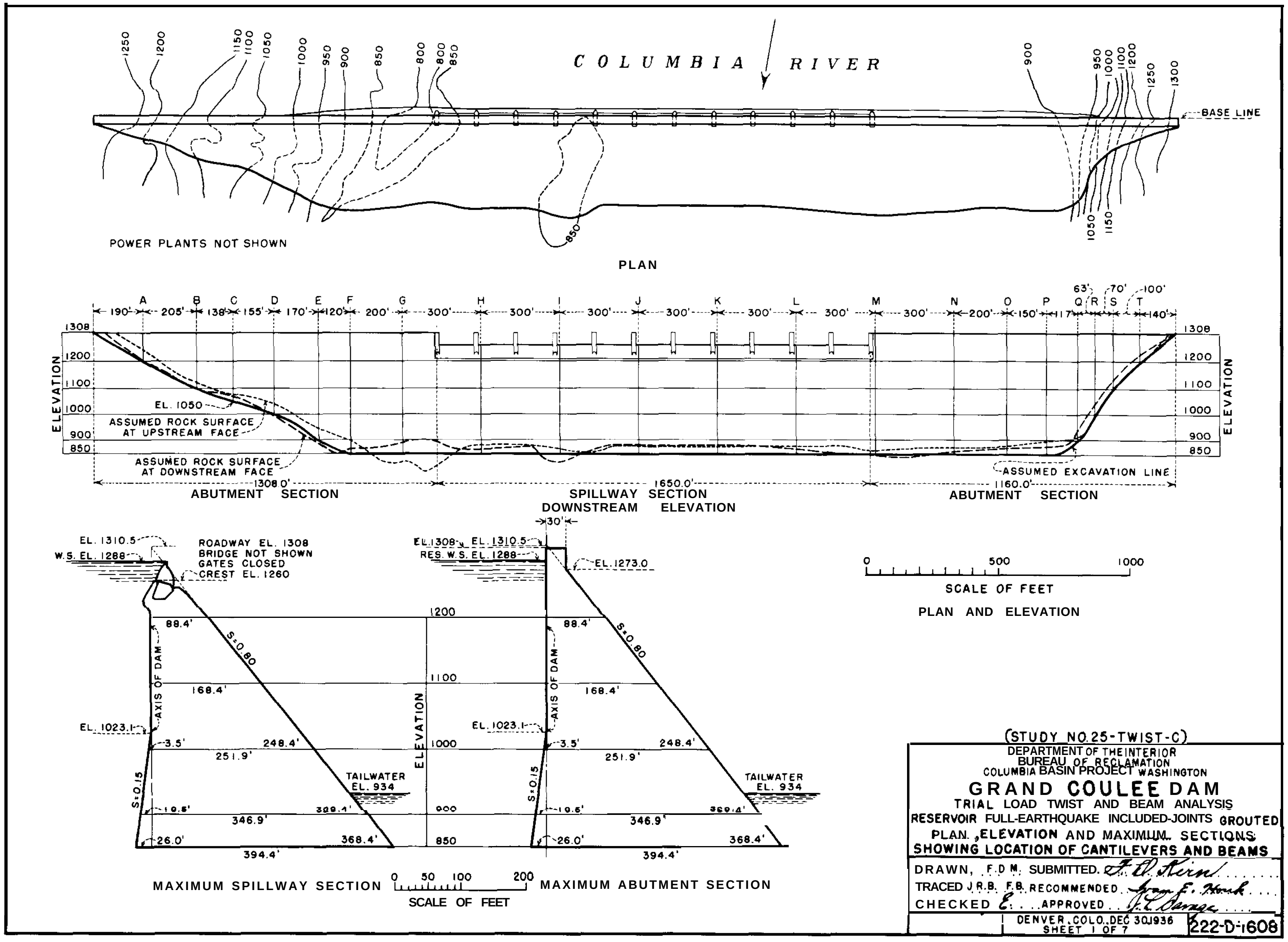


Figure E-24. **Grand Coulee Dam-plan, elevation, and maximum sections.**

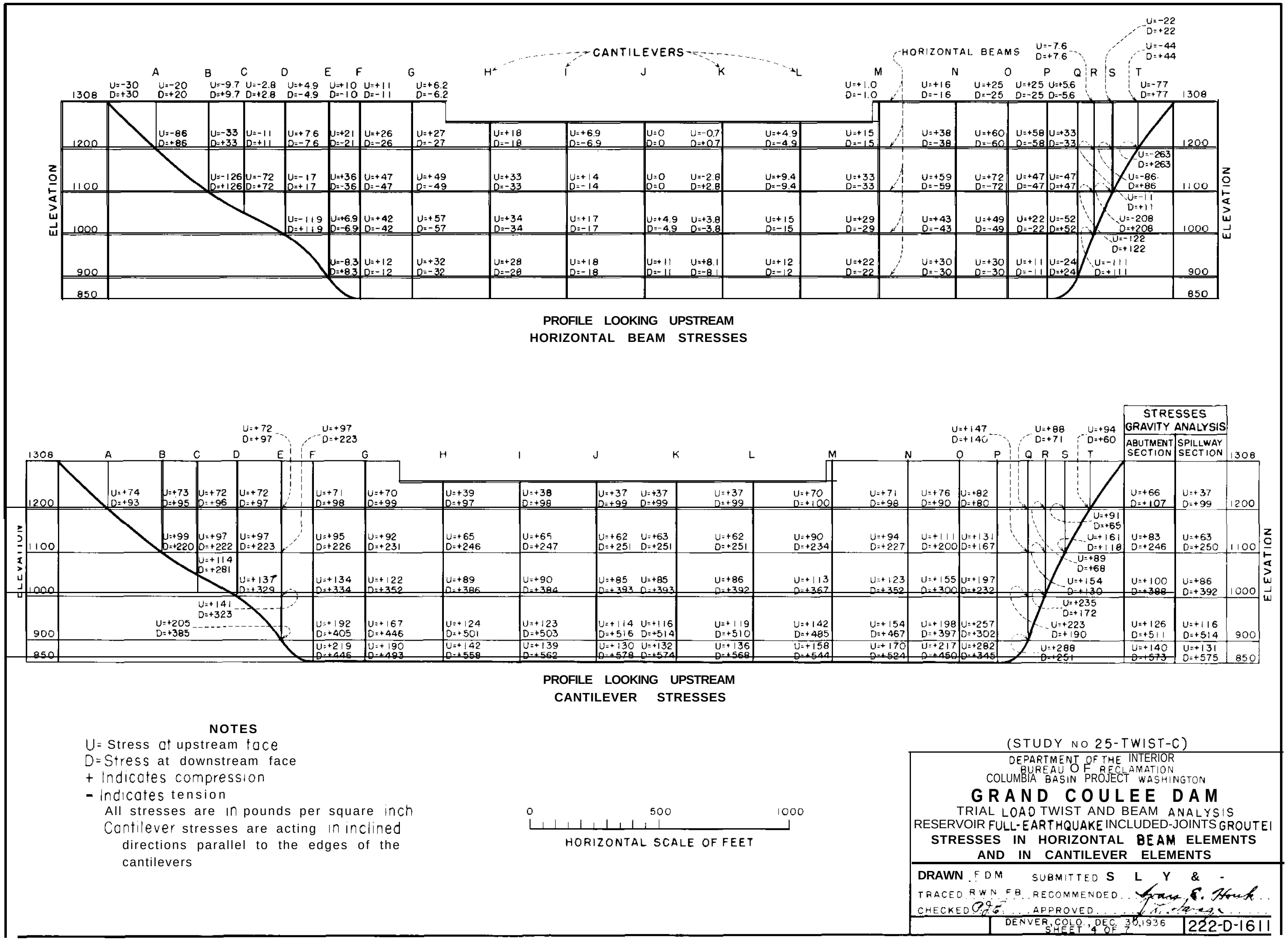


Figure **E-25. Grand Coulee Dam-stresses from trial-load twist and beam analysis.**

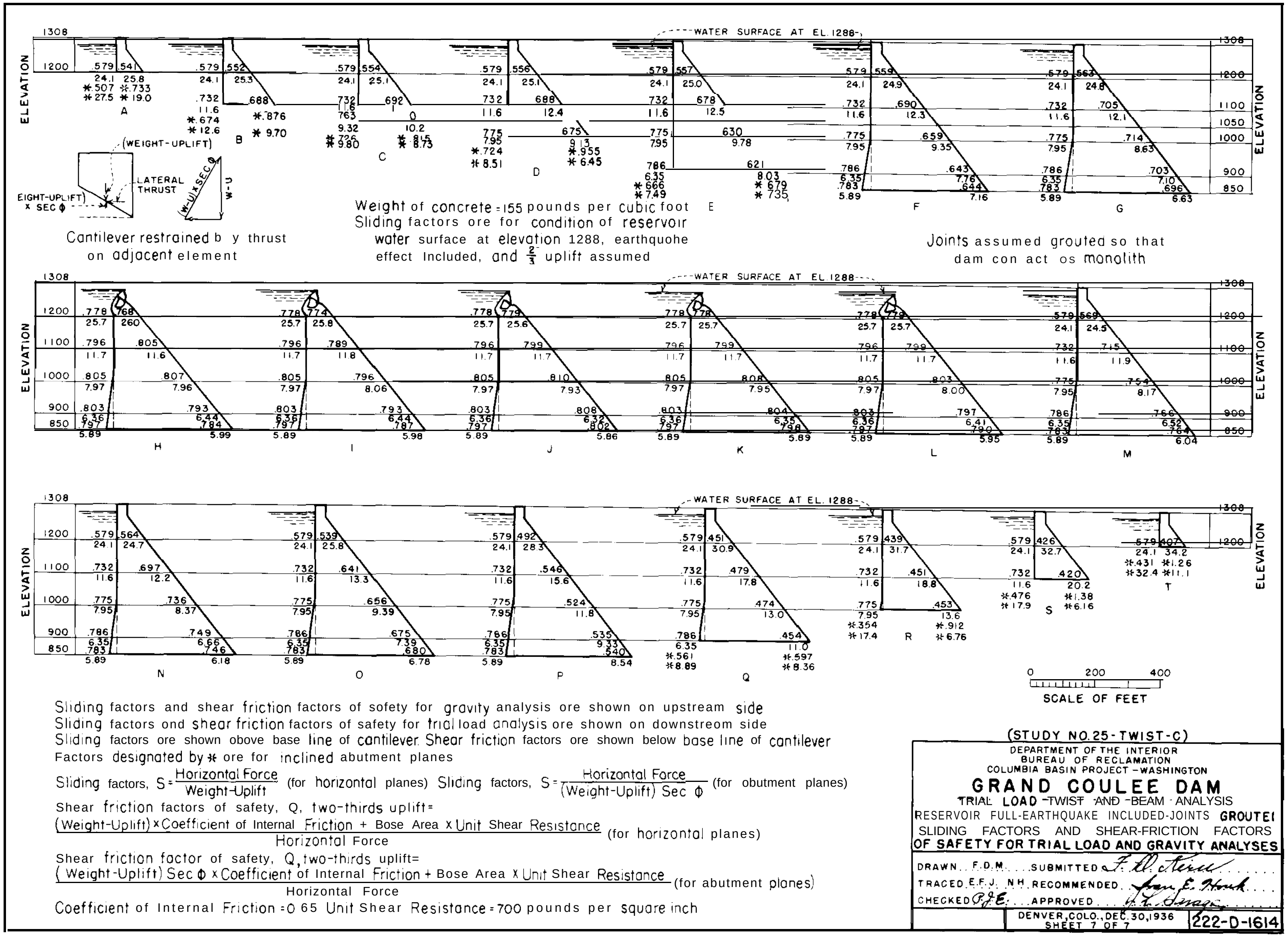
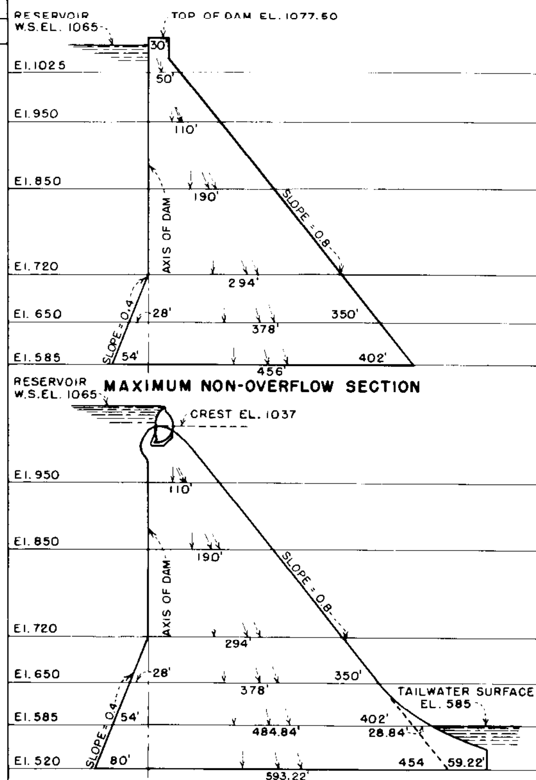


Figure E-26. Grand Coulee Dam-stability factors from trial-load twist and beam analysis.

	RESERVOIR EMPTY			NORMAL RES. W.S. EL.				NORMAL RES.			
	Vertical Stress	Stress Parallel to Face	Horiz Shear Stress	Water Pressure	Vertical Stress	Stress Parallel to Face	Horiz. Shear Stress	Sliding Factor	Shear friction Factor	Max Horiz Shear Stress	Water Pressure
	Pounds per Square Inch										
Maximum Non-Overflow Section											
El. 1025	72	72	0	17	61	61	0	.209	89.5	14	
El. 950	154	154	0	50	99	99	0	.439	24.5	42	
El. 850	259	259	0	93	139	139	0	.542	12.6	94	
El. 720	396	396	0	150	190	190	0	.595	7.9	164	
El. 650	382	443	-153	180	211	216	-12	.574	7.2	203	
El. 585	394	456	-157	208	240	245	-13	.554	6.6	235	
Maximum Spillway Section											
El. 950	112	112	0	50	69	69	0	.540	24.2	49	
El. 850	230	230	0	93	118	118	0	.581	12.5	101	
El. 720	377	377	0	150	176	176	0	.613	7.9	169	
El. 650	368	427	-147	180	197	200	-6.9	.588	7.2	207	
El. 585	383	444	-153	208	230	234	-8.7	●.572	7.0	238	
El. 520	409	474	-163	237	264	269	-11	●.558	6.8	249	28

UPSTREAM FACE



MAXIMUM SPILLWAY SECTION

HORIZONTAL SECTION

↓ Resultant- concrete weight only. ↘ Resultant-water pressure and weight ↘ Resultant-water pressure, weight, and uplift.

Sliding Factor $= \dfrac{\text{Horizontal Force}}{\text{Weight-Uplift}}$

Shear-friction Factor $= \dfrac{(\text{Weight-Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 154 pounds per cubic foot Unit shear resistance = 600 pounds per square inch Coefficient of Internal Friction = 0 65.

Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face acting over two-thirds the area of horizontal section.

Total load carried by vertical cantilever.

All normal stresses are compressive except those preceded by a negative sign, which are tensile

Positive shear stresses are caused by shear forces acting thus ⇌ Negative shear stresses are caused by shear forces acting thus ⇋

Weight of superstructure included in analysis of spillway section. ● Stability factors include effects of spillway apron.

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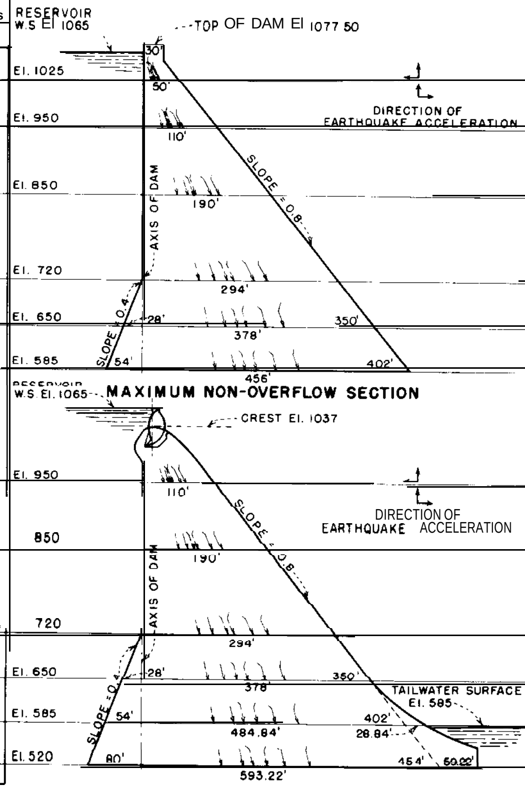
Figure **E-27. Shasta Dam-gravity analyses for normal conditions.**

Section	Elevation	Reservoir empty: Vertical Stress	Reservoir empty: Stress Parallel to Face	Reservoir empty: Horiz Shear stress	Normal Res. W S. El.: Water Pressure	Normal Res. W S. El.: Vertical Stress	Normal Res. W S. El.: Stress Parallel to Face	Normal Res. W S. El.: Horiz. Shear Stress	Normal Res. W. S. El.: Sliding Factor	Normal Res. W. S. El.: Shear-friction Factor	Normal Res. W. S. El.: Max. Horiz. Shear Stress	Normal Res. W. S. El.: Water Pressure	Normal Res. W. S. El.: Vertical Stress	Normal Res. W. S. El.: Stress Parallel to Face	Normal Res. W. S. El.: Horiz. Shear Stress	Reservoir empty: Vertical stress	Reservoir empty: Stress Parallel to Face	Reservoir empty: Horiz Shear Stress
		Pounds per Square Inch									Pounds per Square Inch							
Maximum non-overflow section	El. 1025	*68	68	0	25	50	50	0	395	42.5	29		36	59	29	18	29	14
		**90	90	0	13	84	84	0	.016	1,058	1.4		2	3	1.4	−4	−7	−3.5
	El. 950	149	149	0	65	72	72	0	.645	14.9	77		96	157	77	18	30	15
		189	189	0	45	147	147	0	.211	45.6	19		21	34	17	−22	−36	−17
	El. 850	252	252	0	116	92	92	0	.737	8.3	152		190	312	152	31	51	25
		317	317	0	89	214	214	0	.317	19.2	55		69	113	55	−35	−57	−28
	El. 720	386	386	0	182	121	121	0	.777	5.5	251		313	514	251	48	79	39
		486	486	0	147	297	297	0	.379	11.2	110		137	225	110	−51	−84	−41
	El. 650	369	428	−148	217	144	133	29	.740	5.0	293		367	601	293	115	189	92
		471	546	−188	179	319	342	−56	.373	10.0	154		192	315	154	14	23	11
	El. 585	380	441	−152	251	172	160	32	.716	4.6	333		416	682	333	162	265	129
		486	564	−194	207	357	380	−60	.359	9.3	185		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Maximum spillway section	El. 950	*112	112	0	65	46	46	0	766	15.1	78		97	159	78	23	37	16
		**136	136	0	45	107	107	0	.280	41.4	29		36	60	29	−1	−2	−1.0
	El. 850	226	226	0	116	74	74	0	781	8.3	156		194	319	156	38	62	30
		281	281	0	89	186	186	0	345	18.7	66		82	135	66	−17	−28	−14
	El. 720	368	368	0	182	108	108	0	796	5.5	254		317	520	254	54	88	43
		460	460	0	147	278	278	0	392	III	118		147	242	118	−38	−63	−31
	El. 650	357	414	−143	217	132	119	34	.755	5.0	206		370	607	296	118	194	95
		452	525	−181	179	302	322	−49	.383	9.9	160		200	328	160	22	36	18
	El. 585	370	429	−148	251	163	149	35	⊕.736	4.9	334		418	685	334	163	268	131
		472	547	−189	207	343	365	−54	⊕.370	9.7	190		237	389	190	61	101	49
	El. 520	395	458	−158	282	200	186	33	⊕.71	4.8	349	29	465	744	349	202	332	162
		504	585	−202	238	382	405	−57	BD.361	94	199	33	283	442	199	93	153	75

UPSTREAM FACE — HORIZONTAL SECTION — DOWNSTREAM FACE



ALL RESULTANT FORCES INCLUDE VERTICAL EARTHQUAKE ACCELERATION UPWARD.

Resultant-concrete weight and earthquake (horizontal upstream). Resultant-concrete weight and earthquake (horizontal downstream)

Resultant-water pressure, weight, and earthquake (horizontal upstream). Resultant-water pressure, weight, and earthquake (horizontal downstream).

Resultant-water Pressure weight, uplift, and earthquake (horizontal upstream). Resultant-water pressure, weight, uplift, and earthquake (horizontal downstream).

Sliding Factor = Horizontal Force / Weight-Uplift. Shear-friction Factor = (Weight-Uplift) x Coefficient of Internal Friction + Horizontal Area x Unit Shear Resistance / Horizontal Force.

Unit weight of concrete = 154 pounds per cubic foot. Unit shear resistance = 600 pounds per square inch. Coefficient of Internal friction = 0.65.

Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face, acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake.

Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1g, period = one second.

Total load carried by vertical cantilever.

All normal stresses are compressive except those preceded by a negative sign, which are tensile.

Positive shear stresses are caused by shear forces acting thus ⇌ Negative shear stresses are caused by shear forces acting thus ⇋

* Including earthquake acceleration (horizontal upstream and vertical upward) ** Including earthquake acceleration (horizontal downstream and vertical upward)

Weight of superstructure included in analysis of spillway section. ⊕ Stability factors include effects of spillway apron.

STUDY No 4

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT-CALIFORNIA
SHASTA DAM
GRAVITY ANALYSES
MAXIMUM NON-OVERFLOW AND SPILLWAY SECTIONS
RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATI
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECT
VERTICAL ACCELERATION UPWARD

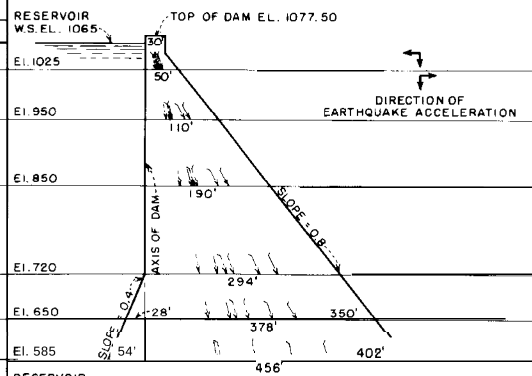
DRAWN	S.D.L.	SUBMITTED	F. W. Kiese
TRACED	F.B.	RECOMMENDED	Ivan E. Houk
CHECKED	L.K.	APPROVED	J. L. Savage

DENVER, COLORADO MAY 20, 1938 SHEET 4 OF 5 214-D-6

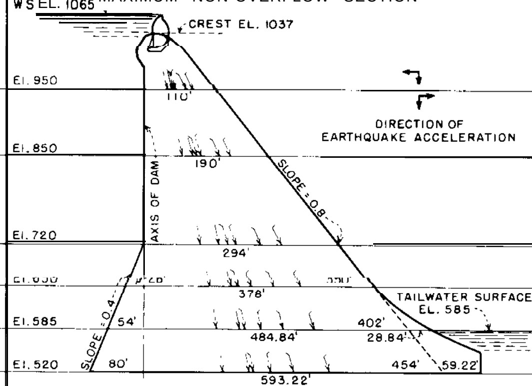
Figure **E-28. Shasta Dam-gravity analyses including effects of earthquake, vertical acceleration upward.**

UPSTREAM FACE

Section / Elevation	RESERVOIR EMPTY: Vertical Stress	RESERVOIR EMPTY: Stress Parallel to Face	RESERVOIR EMPTY: Horiz. Shear Stress	NORMAL RES. W.S. EL.: Water Pressure	NORMAL RES. W.S. EL.: Vertical Stress	NORMAL RES. W.S. EL.: Stress Parallel to Face	NORMAL RES. W.S. EL.: Horiz. Shear Stress
	Pounds per Square Inch						
Non-overflow El. 1025	*54	54	0	22	38	38	0
	**76	76	0	10	72	72	0
El. 950	118	118	0	55	52	52	0
	158	158	0	35	127	127	0
El. 850	200	200	0	98	65	65	0
	266	266	0	70	186	186	0
El. 720	307	307	0	152	83	83	0
	407	407	0	117	259	259	0
El. 650	293	340	-117	181	102	90	32
	394	457	-158	143	277	298	-54
El. 585	301	349	-120	210	124	110	34
	407	472	-163	165	308	331	-57
Spillway El. 950	*89	89	0	55	32	32	0
	**113	113	0	35	93	93	0
El. 850	180	180	0	98	50	50	0
	235	235	0	70	163	163	0
El. 720	293	293	0	152	73	73	0
	385	385	0	117	243	243	0
El. 650	283	328	-113	[illegible]	[illegible]	[illegible]	[illegible]
	379	440	-162	143	263	282	-48
El. 585	294	341	-117	210	117	102	37
	395	459	-158	165	297	318	-53
El. 520	313	363	-125	235	147	133	35
	422	490	-169	191	329	351	-55



MAXIMUM NON-OVERFLOW SECTION



MAXIMUM SPILLWAY SECTION

HORIZONTAL SECTION / DOWNSTREAM FACE

Section / Elevation	Sliding Factor	Shear-friction Factor	Max. Horiz. Shear Stress	NORMAL RES. W.S. EL.: Water Pressure	NORMAL RES. W.S. EL.: Vertical Stress	NORMAL RES. W.S. EL.: Stress Parallel to Face	NORMAL RES. W.S. EL.: Horiz. Shear Stress	RESERVOIR EMPTY: Vertical Stress	RESERVOIR EMPTY: Stress Parallel to Face	RESERVOIR EMPTY: Horiz. Shear Stress
			Pounds per Square Inch							
Non-overflow El. 1025	.453	46.6	26		33	54	26	17	27	13
	.027	774.9	-1.4		-2	-3	-1.4	-6	-9	-4.5
El. 950	.734	16.7	68		85	140	68	19	31	15
	.172	71.2	12		10	17	8.2	-21	-35	-17
El. 850	.836	9.3	133		167	273	133	31	51	25
	.288	26.9	36		45	74	36	-34	-56	-27
El. 720	.879	6.1	218		272	447	218	49	80	39
	.358	14.9	77		96	158	77	-51	-84	-41
El. 650	.838	5.6	253		316	518	253	103	170	83
	.356	13.1	113		141	231	113	2	3	1.6
El. 585	.811	5.1	285		357	585	285	142	233	113
	.343	12.1	138		173	283	138	36	58	28
Spillway El. 950	.880	17.0	68		85	140	68	21	34	17
	.243	61.8	19		24	40	19	-3	-5	-2.5
El. 850	.888	9.3	135		169	278	135	36	59	29
	.318	26.0	46		57	94	46	-19	-31	-15
El. 720	.902	6.1	220		275	451	220	53	86	42
	.373	14.7	84		105	172	84	-40	-65	-32
El. 650	.856	5.6	255		319	522	255	105	173	84
	.367	13.0	118		148	243	118	9	15	7.5
El. 585	⊛.840	5.4	287		358	587	287	143	234	114
	⊛.356	12.7	142		178	291	142	41	67	33
El. 520	⊛.824	5.2	299	23	397	636	299	175	288	140
	⊛.354	12.2	150	28	215	334	150	66	109	53

ALL RESULTANT FORCES INCLUDE VERTICAL EARTHQUAKE ACCELERATION DOWNWARD

- Resultant- concrete weight and earthquake (horizontal upstream) · Resultant - concrete weight and earthquake (horizontal downstream)
- Resultant-water pressure, weight, and earthquake (horizontal upstream) · Resultant- water pressure, weight, and earthquake (horizontal downstream)
- Resultant-water pressure, weight, uplift, and earthquake (horizontal upstream) · Resultant - water pressure, weight, uplift, and earthquake (horizontal downstream)

Sliding Factor = $\frac{\text{Horizontal Force}}{\text{Weight - Uplift}}$ Shear-friction Factor = $\frac{(\text{Weight - Uplift}) \times \text{Coefficient of Internal Friction} + \text{Horizontal Area} \times \text{Unit Shear Resistance}}{\text{Horizontal Force}}$

Unit weight of concrete = 154 pounds per cubic foot. Unit shear resistance = 600 pounds per square inch Coefficient of internal friction = 0.65

Uplift pressure varies as a straight line from reservoir water pressure at upstream face to zero or tailwater pressure at downstream face acting over two-thirds the area of the horizontal section; assumed to be unaffected by earthquake

Vertical earthquake acceleration and horizontal earthquake acceleration = 0.1 g., period = one second

Total load carried by vertical cantilever

All normal stresses are compressive except those preceded by a negative sign, which are tensile

Positive shear stresses are caused by shear forces acting thus ⇌ Negative shear stresses are caused by shear forces acting thus ⇌

* Including earthquake acceleration (horizontal upstream and vertical downward) ** Including earthquake acceleration (horizontal downstream and vertical downward)

Weight of superstructure included in analysis of spillway section. ⊛ Stability factors include effects of spillway apron

STUDY No. 4

DEPARTMENT OF THE INTERIOR
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RESERVOIR EMPTY AND NORMAL FULL RESERVOIR OPERATION
WITH HORIZONTAL AND VERTICAL EARTHQUAKE EFFECTS
VERTICAL ACCELERATION DOWNWARD

DRAWN S.O.L. SUBMITTED F. W. Kiene
TRACED N.H. RECOMMENDED Jay E. Houk
CHECKED L.K. APPROVED J. L. Savage

DENVER, COLORADO MAY 20, 1938 SHEET 5 OF 5 214-D-611

Figure **E-29. Shasta Dam-gravity analyses including effects of earthquake, vertical acceleration downward.**

TABLE E-l.-Comparison of stresses and stability factors for 12 dams. –DS2-2(T2)

	GROUP I				GROUP II			GROUP III			GROUP IV	
	AMERICAN FALLS DAM SNAKE RIVER, IDAHO	ALTUS DAM NORTH FORK, RED RIVER, OKLAHOMA	KESWICK SACRAMENTO RIVER, CALIFORNIA	EAST PARK LITTLE STONY CREEK, CALIFORNIA	ANGOSTURA CHEYENNE RIVER, SOUTH DAKOTA	BLACK CANYON PAYETTE RIVER, IDAHO	KORTES NORTH PLATTE RIVER, WYO.	FRIANT SAN JOAQUIN RIVER, CALIFORNIA	MARSHALL FORD COLORADO RIVER, TEXAS	ELEPHANT BUTTE RIO GRANDE RIVER, NEW MEXICO	GRAND COULEE COLUMBIA RIVER, WASHINGTON	SHASTA SACRAMENTO RIVER, CALIFORNIA
TYPE OF DAM	Straight Gravity	Straight Gravity	Straight Gravity	Curved Gravity	Straight Gravity	Bent Gravity	Straight Gravity	Straight Gravity	Straight Gravity	Straight Gravity	Straight Gravity	Curved Gravity
YEAR COMPLETED	1927	1945	U. C.	1910	U. C.	1925	U. C.	1942	1942	1916	1941	1944
MAXIMUM HEIGHT, FT. (ANALYZED SECTION)	77.5	102	130	130	159	170	239	267	268	294	460.5	492.5
CREST LENGTH, FT.	5,227	1,112	1,046	250	980	1,039	440	3,430	5,128	1,674	4,173	3,500
LENGTH-TO-HEIGHT RATIO	67.4	10.9	8.1	1.9	6.2	6.1	1.8	12.8	19.1	5.7	9.1	7.1
THICKNESS AT CROWN, FT. — TOP	8	10	43.2	10.6	10	10.6	20	20	30	18	30	30
THICKNESS AT CROWN, FT. — BASE	59.6	72	85	90.6	138.3	136.5	169.4	217.3	221.45	243.78	394.4	593.22
BASE-TO-TOP RATIO	7.5	7.2	2.0	8.5	13.8	12.9	8.5	10.9	7.4	13.5	13.1	19.8
BASE-TO-HEIGHT RATIO	0.8	0.7	0.7	0.7	0.9	0.8	0.7	0.8	0.8	0.8	1.2	1.2
VOLUME OF DAM (CU. YARDS OF CONCRETE)	166,000	70,000	192,000	12,200	224,000	79,000	130,000	2,030,000	1,770,000	605,000	9,790,000	6,440,000
CANTILEVER PROFILE												
CROSS CANYON PROFILE (TRIAL-LOAD ANALYSES)												
[®]CRITICAL CANTILEVER STRESS, UPSTREAM FACE — NORMAL LOADING	27 U 31 P 0.8 S	30 U 42 P 1.2 S	—	25 U 54 P 1.1 S	—	28 U 74 P 16 S	— —	84 U 114 P 8 S	141 U 102 P -2.3 S	94 U 125 P 1.9 S	—	269 U 237 P 11 S
[®]CRITICAL CANTILEVER STRESS, UPSTREAM FACE — MAXIMUM LOADING	13 U 37 P 4.7 S	7 U 50 P 4.2 S	-20 U 52 P 0 S	-4 U 60 P * -19 U 27 P	50 U 64 P ‡ 51 U 38 P	-18 U 72 P 31 S	59 U 102 P ‡220 U 102 P	32 U 135 P 28 S	81 U 100 P -7.2 S	104 U 138 P -10 U 76 P 6.4 S	131 U ‡130 U 190 P	186 U 282 P -57 S
MAXIMUM CANTILEVER STRESS, DOWNSTREAM FACE — NORMAL LOADING	80 D 34 S	115 D 49 S	—	155 D 72 S	-	204 D 68 S	—	297 D 140 S	230 D 112 S	331 U 149 S	—	539 249 S
MAXIMUM CANTILEVER STRESS, DOWNSTREAM FACE — MAXIMUM LOADING	112 D 49 S	162 D 70 S	234 D 83 S	217 D 100 S * 129 D * -200 I	243 D ‡240 D	272 D 95 S	287 D ‡155 D	409 D 192 S	349 D 169 S	471 D 215 S	575 D ‡578 D	744 D 349 S
MAXIMUM SLIDING FACTOR — GRAVITY ANALYSIS	0.783	0.832	0.764	1.070	0.924	1.217	0.771	0.999	0.772	1.161	0.805	0.902
MAXIMUM SLIDING FACTOR — TRIAL-LOAD ANALYSIS	—	—	—	—	†0.931	—	†1.218	—	—	—	†1.38	—
MINIMUM SHEAR-FRICTION FACTOR — GRAVITY ANALYSIS	16.2	11.0	8.25	4.6	5.07	84	67	5.45	4.8	5.1	5.89	4.8
MINIMUM SHEAR-FRICTION FACTOR — TRIAL-LOAD ANALYSIS	—	—		—	543	—	†7.7				5.86	
LOADING CONDITIONS, GRAV. ANAL. U.S. FACE — NORMAL LOADING	Res. full + T.W.	Res. full + silt + T.W.	— —	Res. full w/o T.W.	—	Res. full + T.W.	—	Res. full w/o T.W.	Res. full w/o T.W.	Res. full + T.W.		Res. full + T.W.
LOADING CONDITIONS, GRAV. ANAL. U.S. FACE — MAXIMUM LOADING	Normal + E or Ice, + E	Normal + E	Normal + E	Normal + E	Normal + E	Normal + E	Max. Flood	Normal + E	Normal + E	Normal + E	Normal + E w/o T.W.	Normal + E
LOADING CONDITIONS, GRAV. ANAL. D.S. FACE — NORMAL LOADING	Res. full + T.W.	Res. full + silt + T.W.	—	Res. full w/o T.W.	—	Res. full + T.W.	—	Res. full w/o T.W.	Res. full w/o T.W.	Res. full + T.W.	—	Res. full + T.W.
LOADING CONDITIONS, GRAV. ANAL. D.S. FACE — MAXIMUM LOADING	Normal + E or Ice	Normal + E	Normal + E	Normal + E	Normal + E	Normal + E	Max. Flood + T.W.	Normal + E	Normal + E	Normal + E	Normal + E w/o T.W.	Normal + E
MAXIMUM LOADING CONDITIONS, TRIAL-LOAD ANALYSIS	—	—	— E	Normal + Temp w/o T.W.	Normal + E + Ice + Earth	—	Max. Flood + T.W.	—	—	—	Normal + E	
REFERENCES	Unnumbered Memo Oct. 29, 1940	Unnumbered Memo Dec. 26, 1941	Unnumbered Memo July 28, 1941	Unnumbered Memo Aug. 25, 1941	Unnumbered Memo. Feb. 28, 1947	Tech. Mem. 549, Apr. 8, 1937	Unnumbered Memo. Sept. 3, 1943	Tech. Memo. 612, Sept. 21, 1940	Tech. Mem. 573, May 15, 1938	Unnumbered Memo. June 19, 1940	Tech. Memo. 546, Feb. 25, 1937	Tech. Mem. 575, May 15, 1938

® That stress which is lowest percentage of water pressure at the same point. Maximum compressive and tensile stresses parallel to the face are shown as well as water pressure at the point, if water pressure exceeds stress at face for any given loading condition.

* Results by Trial-Load Arch and Cantilever Analysis
‡ Results by Trial Load Beam and Cantilever Twist Analysis
† Near Abutment
P = Water Pressure
S = Horizontal Shear Stress
I = Intrados Arch Stress
D = Downstream Face
U = Upstream Face
E = Earthquake
T.W. = Tailwater
w/o = Without
U.C. = Under Construction

normal loading conditions sliding factors are considerably smaller and shear-friction factors larger (see figs. E-l through E-29, and also figs. A-l 0 through A-14 of app. A). The average maximum sliding factor for the gravity analyses for 12 dams is equal to 0.917 and the minimum shear-friction factor is equal to 7.19.

The maximum effects of twist action in seven gravity dams are shown in table E-2. The most noteworthy effects of twist action on stresses and stability factors obtained by trial-load analysis, as compared with those quantities obtained by gravity analysis, may be summarized briefly as follows:

(1) An increase in sliding factors along the steeper inclined rock planes which form the bases of the cantilevers in the abutment sections.

(2) A decrease in sliding factors in the longer cantilevers whose bases are located in the lower regions of the abutment slopes.

(3) A decrease in shear-friction factor of safety along the steeper inclined rock planes at the abutment cantilevers.

(4) An increase in shear-friction factor of safety at the high cantilevers near the lower ends of the abutment slopes.

(5) Relatively small changes in stresses and stability factors in the longer cantilevers near the central section of the dam where most of the external load is carried by the cantilevers.

(6) A decrease in inclined cantilever compressive stresses along the base of the dam at the downstream edges of the abutment sections and as far toward the center of the structure as appreciable portions of external load may be carried by twist action.

(7) An increase in inclined cantilever compressive stresses along the base of the dam at the upstream edges of the abutment sections and as far toward the center of the structure as appreciable portions of external load may be carried by twist action.

(8) The development of appreciable horizontal compressive stresses at and parallel to the downstream face, decreasing in magnitude from the abutment slopes toward the center of the dam.

(9) The development of appreciable horizontal tensile stresses at and parallel to the upstream face of the dam, with possible resultant cracking, decreasing in magnitude and effect from the abutment slopes toward the center of the dam.

(10) Wherever the deflection curves of the horizontal elements may indicate the possible existence of relatively high tensile stresses, diagonal cracking may occur. This condition may exist especially near the points of contraflexure of horizontal elements in the upper portions of the dam.

It is seen from the above summary that both beneficial and detrimental effects on loads, stresses, and stability factors for straight gravity dams may accrue by twist action. The lateral transfer of load to the abutments causes some reduction in load on the high cantilevers at the lower ends of the abutment slopes. However, the beneficial results of such reductions are usually of minor importance in comparison with the detrimental effects of load increases on the shorter end cantilevers. In some cases, sliding factors at the bases of these shorter cantilevers are increased to more than unity; hence the sections theoretically would move downstream if they were not held in place by the shear resistance and weight of the mass of the dam. Fortunately, shear-friction factors of safety at the bases of gravity sections increase as the heights of the sections decrease. Consequently, the shear resistance at the bases of the shorter end cantilevers is usually great enough to prevent failure even though the sliding factor in these regions may be greater than unity.

Theoretically, it may sometimes be possible to save concrete by reducing slightly the thickness of the cross section at regions where twist action is indicated to be beneficial. In practice, however, it is usually desirable to keep the slopes of the faces constant throughout the length of the dam for economy of construction. Another reason for not making reductions in cross section to allow for

TABLE *E-2.-Maximum effects of twist action in some gravity dams with principal dimensions of twisted structure.* –DS2-2(T3)

GENERAL DIMENSIONS AND DATA

ITEM		Madden	Norris	O'Shaughnessy	Grand Coulee	Friant	Marshall Ford	Davis
Location		Panama Canal Zone	Tennessee	California	Washington	California	Texas	Ariz.-Nevada
River		Chagres	Clinch	Tuolumme	Columbia	San Joaquin	Lower-Colo.	Lower-Colo.
Maximum height of twisted section		210	260	382	458	267	268	153
Length of twisted section		950	1580	850	4118	3390	2700	402
Width at top of dam		22	20	27.5	30	20	30	32
Width at base of maximum section		176	210	308	394	220	216	110
Upstream projection at base		12	15	16	26	17	11	0
Downstream projection at base		144	175	264	338	183	175	78
Loading condition analyzed		Full Reservoir	Full Reservoir	Full Reservoir +Earthquake	Full Reservoir +Earthquake	Full Reservoir +Earthquake	Full Reservoir +Earthquake	Full Reservoir +Earthquake
SLIDING FACTORS	Maximum increase at any position	0.40-0.75	0.44-0.69	0.43-1.49	0.35-0.48	0.80-0.84	0.73-0.74	
					0.42-1.19	0.84-0.93	0.39-0.68	0.07-3.84
	Maximum decrease at any position	0.90-0.62	0.64-0.37	0.87-0.46	0.79-0.41	0.88-0.82	0.66-0.48	
					0.79-0.55	0.88-0.79	0.66-0.49	0.50-0.29
SHEAR-FRICTION FACTOR	Maximum increase at any position			22.7-37.4	32.4-54.2	5.9-6.3	31.0-34.1	
					24.3-31.5	7.2-7.9	5.9-8.0	30.1-77.6
	Maximum decrease at any position			37.7-10.9	17.4-12.8	18.3-17.8	12.7-12.5	
					76.5-52.0	10.5-9.2	92.1-72.9	129.4-8.7
CANTILEVER STRESSES COMPRESSION	Maximum increase at upstream face	66-94	81-123	1-126	126-217	31-37	98-124	
				1-228	130-278	22-53	101-161	-11-140
	Maximum decrease at downstream face	198-156	224-163	508-279	511-366	382-374	220-180	
				508-93	520-282	318-274	286-197	292-75
Remarks:				Designed as Gravity Dam Radius 700 ft				Concrete Gravity Penstock Section

Notes:

Figures above line-Joints ungrouted. Figures below line- Joints grouted

Dimensions in feet, Stresses in p.s.i., Stresses act parallel to face.

the effects of beneficial twist action is that effects of nonlinear distribution of stress throughout the sections would probably overshadow the beneficial effects of twist action.

Hydraulic Data and Tables

F-1. ***Lists of Symbols and Conversion Factors.*** -The following list includes symbols used in hydraulic formulas given in chapters IX and X and in this appendix. Standard mathematical notations and symbols having only very limited applications have been omitted.

Symbol	Description
A, a	An area; area of a surface; cross-sectional area of flow in an open channel; cross-sectional area of a closed conduit
a_g	Gross area of a trashrack
a_n	Net area of a trashrack
b	Bottom width of a channel
c	A coefficient; coefficient of discharge
C_d	Coefficient of discharge through an orifice
C_i	Coefficient of discharge for an ogee crest with inclined upstream face
C_o	Coefficient of discharge for a nappe-shaped ogee crest designed for an H_o head
C_s	Coefficient of discharge for a partly submerged crest
D	Diameter; conduit diameter; height of a rectangular conduit or passageway; height of a square or rectangular orifice
d	Depth of flow in an open channel; height of an orifice or gate opening
d_c	Critical depth
d_H	Depth for high (subcritical) flow stage (alternate to d_L)
d_j	Height of a hydraulic jump (difference in the conjugate depths)
d_L	Depth for low (supercritical) flow stage (alternate to d_H)
d_m	Mean depth of flow
d_{m_c}	Critical mean depth
d_n	Depth of flow measured normal to channel bottom

Symbol	Description
d_s	Depth of scour below tailwater in a plunge pool
d_t	Depth of flow in a chute at tailwater level
E	Energy
E_m	Energy of a particle of mass
F	Froude number parameter for defining flow conditions in a channel, $F = \frac{v}{\sqrt{gd}}$
F_t	Froude number parameter for flow in a chute at the tailwater level
f	Friction loss coefficient in the Darcy-Weisbach formula $h_f = \frac{fL}{D}\frac{v^2}{2g}$
g	Acceleration due to the force of gravity
H	Head over a crest; head on center of an orifice opening; head difference at a gate (between the upstream and downstream water surface levels)
H_A	Absolute head above a datum plane, in channel flow
H_a	Head above a section in the transition of a drop inlet spillway
H_1	Head measured to bottom of an orifice opening
H_2	Head measured to top of an orifice opening
h	Head; height of baffle block; height of end sill
h_a	Approach velocity head
h_b	Head loss due to bend
h_c	Head loss due to contraction
H_D	Head from reservoir water surface to water surface at a given point in the downstream channel
h_d	Difference in water surface level, measured from reservoir water surface to the downstream channel water surface
H_E	Specific energy head
H_{E_C}	Specific energy head at critical flow
H_e	Total head on a crest, including velocity of approach

Symbol	Description
he	Head loss due to entrance
h_{ex}	Head loss due to expansion
h_f	Head loss due to friction
Δh_f	Incremental head loss due to friction
h_g	Head loss due to gates or valves
h_L	Head losses from all causes
Σh_{L_u}	Sum of head losses upstream from a section
Δh_L	Incremental head loss from all causes
$\Sigma(\Delta h_L)$	Sum of incremental head losses from all causes
H_o	Design head over ogee crest
h_o	Head measured from the crest of an ogee to the reservoir surface immediately upstream, not including the velocity of approach (crest shaped for design head ***Ho)***
H_s	Total head over a sharp-crested weir
h_s	Head over a sharp-crested weir, not including velocity of approach
HT	Total head from reservoir water surface to tailwater, or to center of outlet of a free-discharging pipe
h_t	Head loss due to trashrack
h_v	Velocity head; head loss due to exit
h_{v_c}	Critical velocity head
K	A constant factor for various equations; a . coefficient
k	A constant
K_a	Abutment contraction coefficient
K_b	Bend loss coefficient
K_c	Contraction loss coefficient
K_e	Entrance loss coefficient
K_{ex}	Expansion loss coefficient
K_g	Gate or valve loss coefficient
K_L	A summary loss coefficient for losses due to all causes
K_p	Pier contraction coefficient
K_t	Trashrack loss coefficient
K_v	Velocity head loss coefficient
L	Length; length of a channel or a pipe; effective length of a crest; length of a hydraulic jump; length of a stilling basin; length of a transition
AL	Incremental length; incremental channel length
L_I, L_{II}, L_{III}	Stilling basin lengths for different hydraulic jump stilling basins
L'	Net length of a crest
M	Momentum
M_d	Momentum in a downstream section
M_u	Momentum in an upstream section
ΔM	Difference in momentum between successive sections
m	Mass
N	Number of piers on an overflow crest; number of slots in a slotted grating dissipator
n	Exponential constant used in equation for defining crest shapes; coefficient of roughness in the Manning equation
P	Approach height of an ogee weir, hydrostatic pressure of a water prism cross section
p	Unit pressure intensity; unit dynamic pressure on a spillway floor; wetted perimeter of a channel or conduit cross section
Q	Discharge; volume rate of flow
ΔQ	Incremental change in rate of discharge
4	Unit discharge
QC	Critical discharge
q_c	Critical discharge per unit of width
Q_i	Average rate of inflow
Q_o	Average rate of outflow
R	Radius; radius of a cross section; crest profile radius; vertical radius of curvature of the channel floor profile; radius of a terminal bucket profile
r	Hydraulic radius; radius of abutment rounding
R_b	Radius of a bend in a channel or pipe
R_s	Radius of a circular sharp-crested weir
S	Storage
A s	Incremental storage
s	Friction slope in the Manning equation; spacing
s_b	Slope of the channel floor, in profile
s_{ws}	Slope of the water surface
T	Tailwater depth; width at the water surface in a cross section of an open channel
T_{max}	Limiting maximum tailwater depth
T_{min}	Limiting minimum tailwater depth
t	Time
At	Increment of time
T_s	Tailwater sweep-out depth
T. W.	Tailwater; tailwater depth
U	A parameter for defining flow conditions in a closed waterway, U $= \frac{v}{\sqrt{gD}}$
v	Velocity
Δv	Incremental change in velocity
v_a	Velocity of approach
v_c	Critical velocity
v_t	Velocity of flow in a channel or chute, at tailwater depth
W	Weight of a mass; width of a stilling basin
w	Unit weight of water; width of chute and baffle blocks in a stilling basin

Symbol	Description
X	A coordinate for defining a crest profile; a coordinate for defining a channel profile; a coordinate for defining a conduit entrance
A X	Increment of length
x_c	Horizontal distance from the break point, on the upstream face of an ogee crest, to the apex of the crest
x_s	Horizontal distance from the vertical upstream face of a circular sharp-crested weir to the apex of the undernappe of the overflow sheet
Y	Drop distance measured from the crest of the overflow to the basin floor, for a free overfall spillway
Y	A coordinate for defining a crest profile; a coordinate for defining a channel profile; a coordinate for defining a conduit entrance
$\bar{y}$	Depth from water surface to the center of gravity of a water prism cross section
Δy	Difference in elevation of the water surface profile between successive sections in a side channel trough
y_c	Vertical distance from the break point, on the upstream face of an ogee crest, to the apex of the crest
y_s	Vertical distance from the crest of a circular sharp-crested weir to the apex of the undernappe of the overflow sheet
Z	Elevation above a datum plane
A Z	Elevation difference of the bottom profile between successive sections in an open channel
z	Ratio, horizontal to vertical, of the slope of the sides of a channel cross section
α	A coefficient; angular variation of the side wall with respect to the structure centerline
β	Deflection angle of bend in a conduit
θ	Angle from the horizontal; angle from vertical of the position of an orifice; angle from the horizontal of the edge of the lip of a deflector bucket

Table F-l presents conversion factors most frequently used by the designer of concrete dams to convert from one set of units to another-for example, to convert from cubic feet per second to acre-feet. Also included are some basic conversion formulas such as the ones for converting flow for a given time to volume.

F-2. *Flow in Open Channels.* -(a) *Energy and Head.-If* it is assumed that streamlines of flow in an open channel are parallel and that velocities at all points in a cross section are equal to the mean velocity v, the energy possessed by the water is made up of two parts: kinetic (or motive) energy and potential (or latent) energy. Referring to figure F-l, if W is the weight of a mass m, the mass possesses Wh_2 foot-pounds of energy with reference to the datum. Also, it possesses Wh_1 foot-pounds of energy because of the pressure exerted by the water above it. Thus, the potential energy of the mass m is $W(h_1 + h_2)$. This value is the same for each particle of mass in the cross section. Assuming uniform velocity, the kinetic energy of m is $W\left(\frac{v^2}{2g}\right)$.

Thus, the total energy of each mass particle is

$$E_m = W\left(h_1 + h_2 + \frac{v^2}{2g}\right) \tag{1}$$

Applying the above relationship to the whole discharge Q of the cross section in terms of the unit weight of water w,

$$E = Qw\left(d + Z + \frac{v^2}{2g}\right) \tag{2}$$

where E is total energy per second at the cross section.

The portion of equation (2) in the parentheses is termed the absolute head, and is written:

$$H_A = d + Z + \frac{v^2}{2g} \tag{3}$$

Equation (3) is called the Bernoulli equation.

The energy in the cross section, referred to the bottom of the channel, is termed the specific energy. The corresponding head is referred to as the specific energy head and is expressed as:

$$H_E = d + \frac{v^2}{2g} \tag{4}$$

Where Q = av, equation (4) can be stated:

TABLE F-1 .-*Conversion factors and formulas.* -288-D-3199(1/2)

[To reduce units in column 1 to units in column 4, multiply column 1 by column 2
To reduce units in column 4 to units in column 1, multiply column 4 by column 3]

CONVERSION FACTORS			
Column 1	Column 2	Column 3	Column 4
LENGTH			
In.	2.54 0.0254	0.3937 39.37	Cm. M.
Ft.	0.3048	3.2808	M.
Miles	1.609	0.621	Km.
AREA			
Sq. in.	6.4516	0.1556	Sq. cm.
Sq. m.	10.764	.0929	sq. ft.
Sq. miles	 30.976×10⁵ /640.27.8784×10⁶2.59	0.3587×10^{-7} .15625X10- .3228x16-4 .386	sq. ft. Acres (1 section). Sq yd. Sq. km.
Acre	43,560.0 4,046.9 4,840.0	0.22957x16-4 $.2471\times10^{-3}$ $.2066\times10^{-3}$	sq. ft. sq. m. Sq. yd.
VOLUME			
cu. ft.	1,728.0 7.4865 6.2321	0.5787X10-S .13368 16046	cu. in. Gal. Imperial gal.
Cu. m.	35.3145 1.3679	0.028317 .76456	cu. ft. Cu. yd.
Gal.	231.0 3.7854	0.4329×10^{-2} .26417	cu. in. Liters.
Million gal.	133,681.0 3.0689	0.74805×10^{-5} .32585	cu. ft. Acre-ft
Imperial gal.	1.2003	0.83311	Oal.
Acre-in	3,630.0	$.27548\times10^{-3}$	cu. ft.
Acre-ft.	1,233.5 43.560.0	0.81071×10^{-3} $.22957\times10^{-4}$	Cu. m. cu. ft.
In. on 1 sq. mile	232.32×10^{4} 53.33	0.43044x16-4 .01875	cu. ft. Acre-ft.
Ft. on 1 sq. mile	278.784×10^{5} 640.0	0.3587×10^{-7} $.15625\times10^{-2}$	cu. ft. Acre-ft.
VELOCITY AND GRADE			
Miles/hr.	1.4667	0.68182	Ft./sec.
M./sec.	3.2808 2.2369	.3048 .44704	Ft./see. Miles/hr.
Fall in ft./mile	189.39×10^{-6} 	5.28×10^{3}	Fall/ft.

CONVERSION FACTORS			
Column 1	Column 2	Column 3	Column 4
now			
cu. ft./sec. (c.f.s.) (second-feet) (sec.-ft.).	60.0 86,400.0 31.536X1 448.83 646, 317.0 1.98347 723.88 725.70 55.54 57.52 59.50 61.49 50.0 40.0 38.4 35.7 0.028317 1.699 0.99173	0.016667 $.11574\times10^{-4}$ $.31709\times10^{-1}$ $.2228\times10^{-2}$ $.15472\times10^{-5}$ .50417 .13813X16- .13778X16--I .018005 .017385 .016806 .016262 .020 .025 .026042 .028011 35.31 .5886 1.0083	Cu. ft./min. Cu. ft./day. cu. ft./yr. Gal./min. Gal./day. Acre-ft./day. Acre-ft./365 days. Acre-ft./366 days. Acre-ft./28 days. Acre-ft./29 days. Acre-ft./30 days. Acre-ft./31 days. Miner's Inch in Idaho, Kans., Nebr., N. Men., N. Dak., 8. Dak., and Utah. Miner's Inch in Ariz., Calif., Mont., Nev., and Oreg. Miner's Inch In Colo. Miner's Inch in British Columbia. Cu. m./sec. Cu. m./min. Acre-in./hr.
Cu. ft./min	7.4805 10,772.0	0.13368 $.92834\times10^{-4}$	Gal./min. Gal./day.
10⁶ gal./day	1.5472 694.44 3.0689	0.64632 $.1440\times10^{-2}$ .32585	C.f.s. Gal./min. Acre-ft./day.
In. depth/hr.	645.33	0.15496×10^{-2}	C.f.s./sq. mile.
In. depth/day	26.889 53.33	0.63719 .01878	C.f.s./sq. mile. Acre-ft./sq. mile.
C.f.s./sq. mile	1.0413 1.0785 1.1157 1.1529 13.574 13.612	0.96032 .92720 .89630 .86738 .073668 .073467	In. depth/28 days. In. depth/29 days. In. depth/36 days. In. depth/31 days. In. depth/365 days. In. depth/366 days.
Acre-ft./day	226.24 20.17 19.36	0.442x10-r .0496 .0517	Gal./min. Miner's Inch in Calif. Miner's inch In Colo.
Gal./sec	5.347 5.128	0.187 .195	Miner's inch In Callf. Miner's Inch in Colo.
PERMEABILITY			
Meinzer (gal./day through 1 sq. It. under unit gradient).	48.8	0.02049	Bureau of Reclamation (cu. ft./yr. through 1 sq. ft. under unit gradient).

TABLE F-1 .-*Conversion factors and* formulas.-Continued.-288-D-3199(2/2)

CONVERSION FACTORS

Column 1	Column 2	Column 3	Column 4
POWER AND ENERGY			
Hp.	555.0	0.18182×10^{-2}	Ft.-lb./sec.
	0.746	1.3405	Kw.
	6,535.	0.15303×10^{-3}	Kw.-hr./yr.
	42.4	.0236	B.t.u./min.
	1.0	1.0	C.f.s. falling 6.8 ft.
Hp.-hr.	0.746	1.3405	Kw.-hr.
	198.0×10^{4}	0.505×10^{-6}	Ft.-lb.
	2,545.0	$.393\times10^{-3}$	B.t.u.
Kw.	8,760.0	0.11416×10^{-3}	Kw.-hr./yr.
	737.56	$.1354\times10^{-2}$	Ft.-lb./sec.
	11.8	.0846	C.f.s. falling 1 ft.
	3,412.0	$.29308\times10^{-3}$	B.t.u./hr.
Kw.-hr.	0.975	1.025	Acre-ft. falling 1 ft.
B.t.u.	778.0	0.1285×10^{-2}	Ft.-lb.
	0.1x10-3 to $.834\times10^{-4}$	10,000 to 12,000	Lb. of coal.
PRESSURE			
Ft. water at max. density	62.425	0.01602	Lb./sq./ft.
	0.4335	2.3087	Lb./sq. in.
	.0295	33.93	Atm.
	.8826	1.133	In. Hg at 30° F.
	773.3	0.1293×10^{-2}	Ft. air at 32° F. and atm. pressure.
Ft. avg. sea water	1.026	0.9746	Ft. pure water.
Atm., sea level, 32° F	14.697	.06804	Lb./sq. in.
Millibars	295.299×10^{-4}	33.663	In. Hg.
	75.008×10^{-2}	1.3331	Mm. Hg
Atm	29.92	33.48X10-J	I”. Hg
WEIGHT			
P.p.m.	0.00136	735.29	Tons/acre-ft.
	.0584	17.123	Gr./gal.
	8.345	0.1198	Lb./10^{6} gal.
Lb.	7.0×10^{3}	0.14286X16-J	Gr.
Gm.	15.432	.064799	Gr.
Kg.	2.2046	.45359	Lb.
Lb. water at 39.1° F	27.6612	0.03612	Cu. in.
	0.11983	3.345	Gal.
	.09983	10.016	Imperial gal.
	.453617	2.204	Liters.
	.01602	62.425	Cu. ft. pure water.
	.01560	64.048	Cu. ft. sea water.
Lb. water at 62° F	0.01604	62.355	Cu. ft. pure water
	.01563	63.976	cu. ft. sea water.

FORMULAS

VOLUME

Average depth in inches, or acre-inch per acre

$$= \frac{(\text{c.f.s.})\ (\text{hr.})}{\text{acres}}$$

$$= \frac{(\text{gal./min.})\ (\text{hr.})}{450\ (\text{acres})}$$

$$= \frac{(\text{miner's in.})\ (\text{hr.})}{(40^{*})\ (\text{acres})}$$

*Where 1 miner's in. = 1/40 c.f.s.

Use 50 where 1 miner's in. = 1/50 c.f.s.

Conversion of inches depth on area to c.f.s.

$$\text{c.f.s.} = \frac{(645)\ (\text{sq. miles})\ (\text{in. on area})}{(\text{time in hr.})}$$

POWER AND ENERGY

$$\text{hp.} = \frac{(\text{c.f.s.})\ (\text{head in ft.})}{8.8}$$

$$= \frac{(\text{c.f.s.})\ (\text{pressure in lb./sq. in.})}{3.3}$$

$$= \frac{(\text{gal./min.})\ (\text{head in ft.})}{3{,}960}$$

$$= \frac{(\text{gal./min.})\ (\text{pressure in lb./sq. in.})}{1{,}714}$$

$$\text{b. hp.} = \frac{\text{water hp.}}{\text{pump efficiency}}$$

kw.-hr./1,000 gal. pumped/hr.

$$= \frac{(\text{head in ft.})\ (0.00315)}{(\text{pump efficiency})\ (\text{motor efficiency})}$$

Kw.-hr. = (plant efficiency) (1.025) (head in ft.) (water in acre-ft.)

$$\text{Load factor} = \frac{(\text{kw.-hr. in time } t)}{(\text{kw. peak load})\ (\text{time } t \text{ in hr.})}$$

SEDIMENTATION

Tons/acre-ft. = (unit weight/c”. ft.) (21.78)

Tons/day = (c.f.s.) (p.p.m.) (0.0027)

TEMPERATURE

$$°\text{C.} = \frac{5}{9}\ (°\text{F.}-32°) \qquad °\text{F.} = \frac{9}{5}\ °\text{C.} + 32°$$

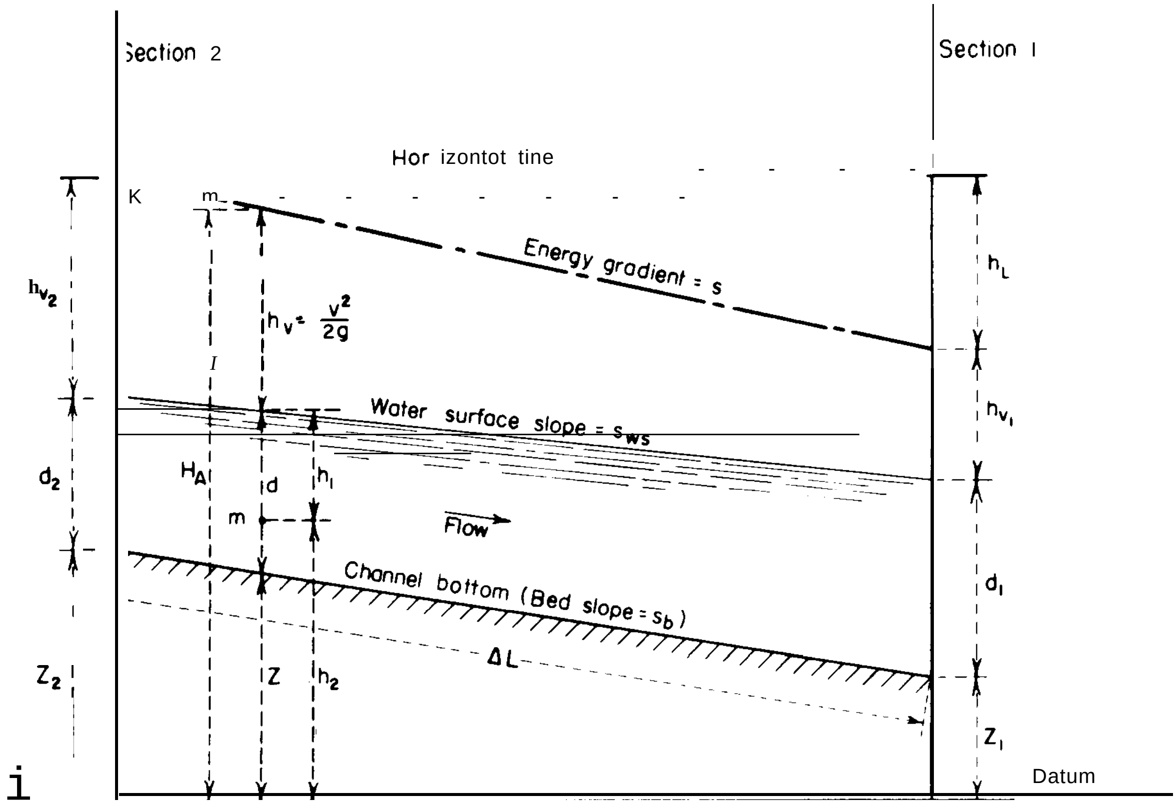


Figure F-I. **Characteristics of open-channel flow.-288-D-2550**

$$H_E = d + \frac{Q^2}{2ga^2} \qquad (5)$$

For a trapezoidal channel where b is the bottom width and z defines the side slope, if q is expressed as $\frac{Q}{b}$ and a is expressed $d(b + zd)$, equation (5) becomes:

$$H_E = d + \frac{q^2}{2gd^2\left(1+\frac{zd}{b}\right)^2} \qquad (6)$$

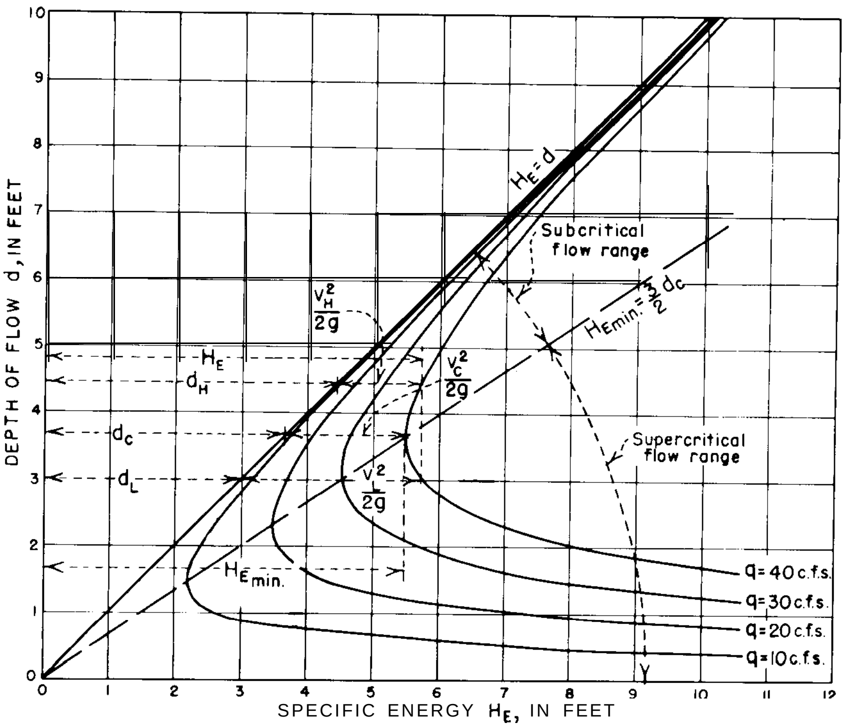
Equation (5) is represented in diagrammatic form on figure F-2 to show the relationships between discharge, energy, and depth of flow in an open channel. The diagram is drawn for several values of unit discharge in a rectangular channel.

It can be seen that there are two values of d, d_H, and d_L for each value of H_E, except at the point where H_E is minimum, where only a single value exists. The depth at energy $H_{E\,min}$ is called the critical depth, and the depths for other values of H_E are called alternate depths. Those depths lying above the trace through the locus of minimum depths are in the subcritical flow range and are termed subcritical depths, while those lying below the trace are in the supercritical flow range and are termed supercritical depths.

Figure F-3 plots the relationships of d to H_E as stated in equation (6), for various values of unit discharge q and side slope z. The curves can be used to quickly determine alternate depths of flow in open channel spillways.

(b) *Critical* Flow. -Critical flow is the term used to describe open channel flow when certain relationships exist between specific energy and discharge and between specific energy and depth. As indicated in section F-2(a) and as demonstrated on figure F-2, critical flow terms can be defined as follows:

(1) *Critical discharge.* -The maximum



$H_E = d + \frac{V^2}{2g} = d + \frac{q^2}{2gd^2}$ where q = discharge per unit width

$d_c = \left(\frac{q_c}{\sqrt{g}}\right)^{\frac{2}{3}} = \frac{2}{3} H_{E_{min.}}$ where d_c = critical depth
q_c = critical discharge per unit width
$H_{E_{min.}}$ = minimum energy content

Figure F-2. Depth **of flow and specific energy for rectangular section in open channel.-288-D-255 1**

discharge for a given specific energy, or the discharge which will occur with minimum specific energy.

(2) Critical depth. -The depth of flow at which the discharge is maximum for a given specific energy, or the depth at which a given discharge occurs with minimum specific energy.

(3) Critical velocity. -The mean velocity when the discharge is critical.

(4) Critical slope. -That slope which will sustain a given discharge at uniform critical depth in a given channel.

(5) Subcritical flow. -Those conditions of flow for which the depths are greater than critical and the velocities are less than critical.

(6) Supercritical flow. -Those conditions of flow for which the depths are less than critical and the velocities are greater than critical.

More complete discussions of the critical flow theory in relationship to specific energy are given in most hydraulic textbooks [1, 2, 3, 4, 5] .[1] The relationship between cross section and discharge which must exist in order that flow may occur at the critical stage is:

$$\frac{Q^2}{g} = \frac{a^3}{T} \qquad (7)$$

where:

a = cross-sectional area in square feet, and
T = water surface width in feet.

[1] **Numbers in brackets refer to items in the bibliography, sec. F-5.**

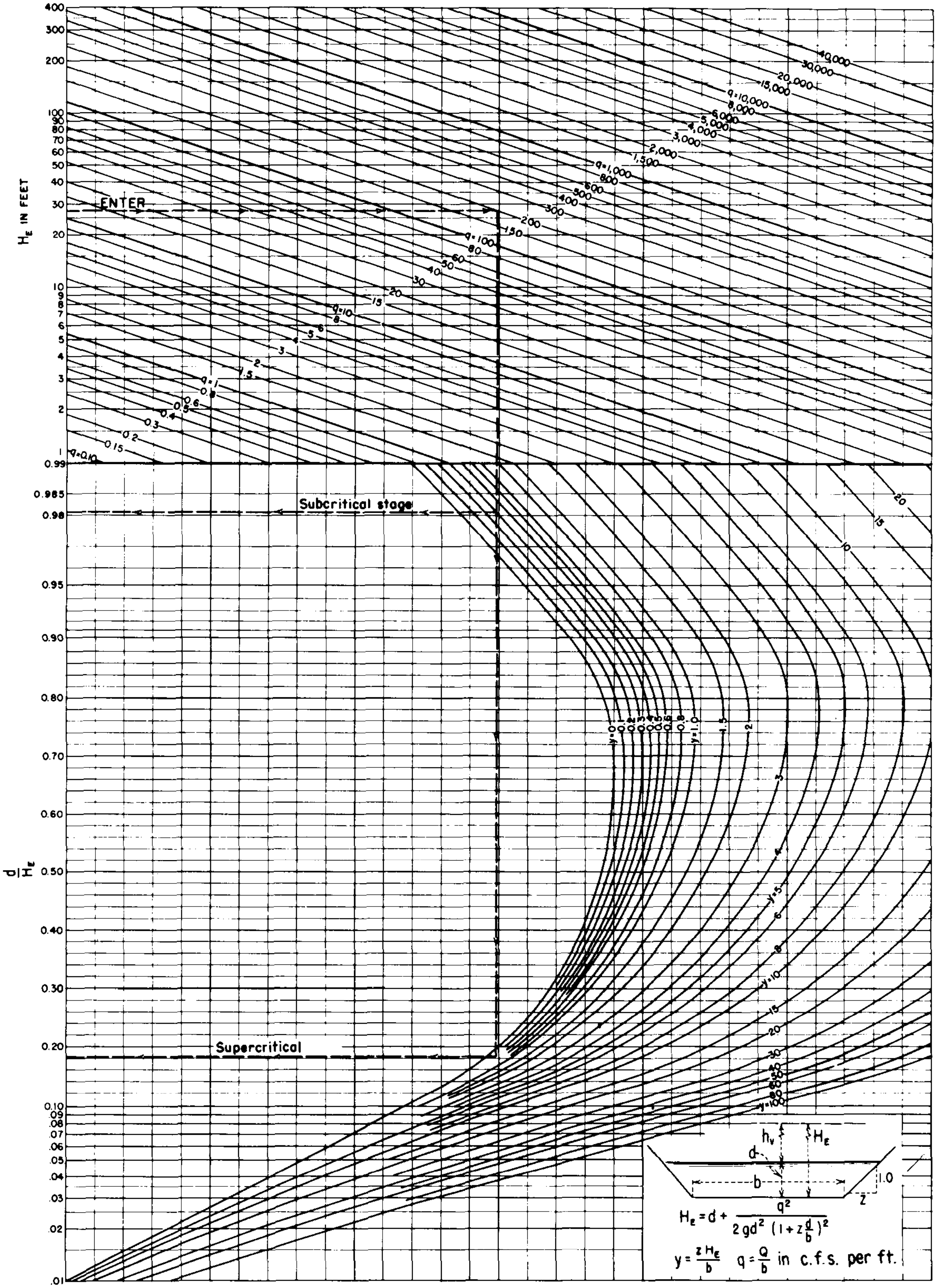


Figure F-3. **Energydepth curves for rectangular and trapezoidal channels.-288-D-3193**

Since $Q^2 = a^2 v^2$, equation (7) can be written:

$$\frac{v_c^2}{2g} = \frac{a}{2T} \quad (8)$$

Also, since $a = d_m T$, where d_m is the mean depth of flow at the section, and $\frac{v_c^2}{2g} = h_{v_c}$, equation (8) can be rewritten:

$$h_{v_c} = \frac{d_{m_c}}{2} \quad (9)$$

Then equation (4) can be stated

$$H_E = d_c + \frac{d_{m_c}}{2} \quad (10)$$

From the foregoing, the following additional relations can be stated:

$$d_{m_c} = \frac{v_c^2}{g} \quad (11)$$

$$d_{m_c} = \frac{Q_c^2}{a^2 g} \quad (12)$$

$$v_c = \sqrt{g d_{m_c}} \quad (13)$$

$$v_c = \sqrt{\frac{ag}{T}} = 5.67\sqrt{\frac{a}{T}} \quad (14)$$

$$Q_c = a\sqrt{g d_{m_c}} \quad (15)$$

For rectangular sections, if q is the discharge per foot width of channel, the various critical flow formulae are:

$$H_{E_c} = \frac{3}{2} d_c \quad (16)$$

$$d_c = \frac{2}{3} H_{E_c} \quad (17)$$

$$d_c = \frac{v_c^2}{g} \quad (18)$$

$$d_c = \sqrt[3]{\frac{q_c^2}{g}} \quad (19)$$

$$d_c = \sqrt[3]{\frac{Q_c^2}{b^2 g}} \quad (20)$$

$$v_c = \sqrt{g d_c} \quad (21)$$

$$v_c = \sqrt[3]{g q_c} \quad (22)$$

$$v_c = \sqrt[3]{\frac{g Q_c}{b}} \quad (23)$$

$$q_c = d_c^{3/2}\sqrt{g} \quad (24)$$

$$Q_c = 5.67 b d_c^{3/2} \quad (25)$$

$$Q_c = 3.087 b H_{E_c}^{3/2} \quad (26)$$

The critical depth for trapezoidal sections is given by the equation:

$$d_c = \frac{v_c^2}{g} - \frac{b}{2z} + \sqrt{\frac{v_c^4}{g^2} + \frac{b^2}{4z^2}} \quad (27)$$

where z = the ratio, horizontal to vertical, of the slope of the sides of the channel.

Similarly, for the trapezoidal section,

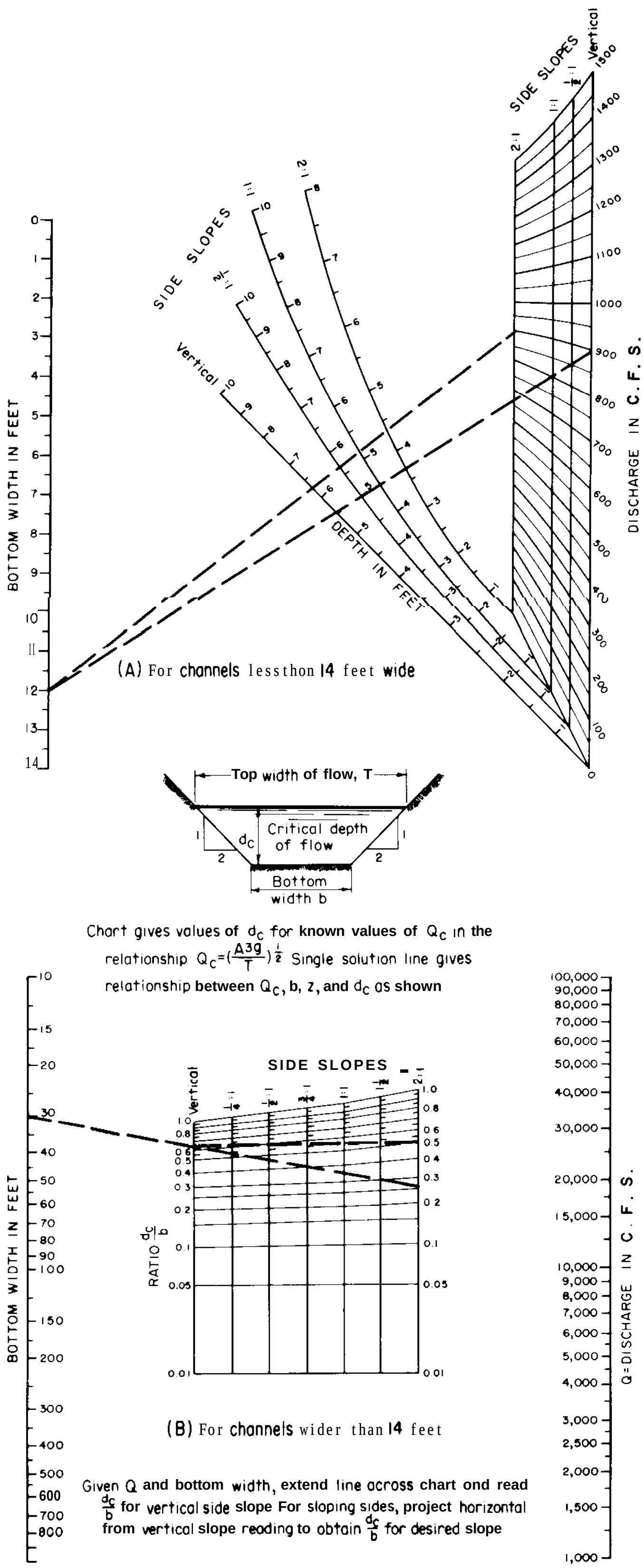
$$v_c = \sqrt{\left(\frac{b + z d_c}{b + 2z d_c}\right) d_c g} \quad (28)$$

and

$$Q_c = d_c^{3/2}\sqrt{\frac{g(b + z d_c)^3}{b + 2z d_c}} \quad (29)$$

The solutions of equations (25) and (29) are simplified by use of figure F-4.

(c) Manning Formula. -The formula developed by Manning for flow in open channels is used in most of the hydraulic analyses discussed in this text. It is a special form of Chezy's formula; the complete development is contained in most textbooks on elementary fluid mechanics. The formula is written as follows:



Example No I

Q_c = 900 c.f.s.
Bottom width "b" = 12'

Side slope	Critical depth "d_c" (feet)
2 I	4 4
Vertical	5 6

Example No 2

Q = 15.000 c.f.s.
Bottom width "b" = 30'
Side slope = 2 I
Vertical "d_c" = .68b = 20'+
"d_c" for 2 , = (0.5)(30) = 15'

Figure F-4. **Critical depth in trapezoidal section.-288-D-3194**

$$v = \frac{1.486}{n} r^{2/3} s^{1/2} \tag{30}$$

or

$$Q = \frac{1.486}{n} a r^{2/3} s^{1/2} \tag{31}$$

where :

Q = discharge in cubic feet per second (c.f.s.),
a = the cross section of flow area in square feet,
v = the velocity in feet per second,
n = a roughness coefficient,
r = the hydraulic radius $= \frac{\text{area } (a)}{\text{wetted perimeter}(p)}$, and
s = the slope of the energy gradient,

The value of the roughness coefficient, n, varies according to the physical roughness of the sides and bottom of the channel and is influenced by such factors as channel curvature, size and shape of cross section, alinement, and type and condition of the material forming the wetted perimeter.

Values of *n* commonly used in the design of artificial channels are as follows:

Description of channel	Values of *n*		
	Minimum	Maximum	Average
Earth channels, straight and uniform	0.017	0.025	0.0225
Dredged earth channels	.025	.033	.0275
Rock channels, straight and uniform . . .	.025	.035	.033
Rock channels, jagged and irregular	.035	.045	.045
Concrete lined	.012	.018	.014
Neat cement lined . .	.010	.013	. . . . . .
Grouted rubble paving	.017	.030	. . . .
Corrugated metal . .	.023	.025	. . .024 .

(d) ***Bernoulli Theorem.*** -The Bernoulli theorem, which is the principle of conservation of energy applied to open channel flow, may be stated: The absolute head at any section is equal to the absolute head at a section downstream plus intervening losses of head. Referring to figure F- 1, the energy equation (3) can be written:

$$Z_2 + d_2 + h_{v_2} = Z_1 + d_1 + h_{v_1} + h_L \tag{32}$$

where h_L represents all losses in head between section 2 (subscript 2) and section 1 (subscript 1). Such head losses will consist largely of friction loss, but may include minor other losses such as those due to eddy, transition, obstruction, impact, etc.

When the discharge at a given cross section of a channel is constant with respect to time, the flow is steady. If steady flow occurs at all sections in a reach, the flow is continuous and

$$Q = a_1 v_1 = a_2 v_2 \tag{33}$$

Equation (33) is termed the equation of continuity. Equations (32) and (33), solved simultaneously, are the basic formulas used in solving problems of flow in open channels.

(e) Hydraulic and Energy Gradients. -The hydraulic gradient in open channel flow is the water surface. The energy gradient is above the hydraulic gradient a distance equal to the velocity head. The fall of the energy gradient for a given length of channel represents the loss of energy, either from friction or from friction and other influences. The relationship of the energy gradient to the hydraulic gradient reflects not only the loss of energy, but also the conversion between potential and kinetic energy. For uniform flow the gradients are parallel and the slope of the water surface represents the friction loss gradient. In accelerated flow the hydraulic gradient is steeper than the energy gradient, indicating a progressive conversion from potential to kinetic energy. In retarded flow the energy gradient is steeper than the hydraulic gradient, indicating a conversion from kinetic to potential energy. The Bernoulli theorem defines the progressive relationships of these energy gradients.

For a given reach of channel **AL,** the average slope of the energy gradient is $\frac{\Delta h_L}{\Delta L}$, where Ah, is the cumulative losses through the reach. If these losses are solely from friction, Δh_L will become Δh_f and

$$\Delta h_f = \left(\frac{s_2 + s_1}{2} \right) \Delta L \qquad (34)$$

Expressed in terms of the hydraulic properties at each end of the reach and of the roughness coefficient,

$$\Delta h_f = \frac{n^2}{4.41} \left[\left(\frac{v_2}{r_2^{2/3}} \right)^2 + \left(\frac{v_1}{r_1^{2/3}} \right)^2 \right] \Delta L \quad (35)$$

If the average friction slope, s_f, is equal to $\frac{s_2 + s_1}{2} = \frac{\Delta h_f}{\Delta L}$, and s_b is the slope of the channel floor, by substituting $s_b \Delta L$ for $Z_2 - Z_1$, and H_E for $(d + h_v)$, equation (32) may be written:

$$\Delta L = \frac{H_{E_1} - H_{E_2}}{s_b - s_f} \qquad (36)$$

(f) ***Chart for Approximating Friction Losses in*** Chutes.-Figure 9-26 is a nomograph from which approximate friction losses in a channel can be evaluated. To generalize the chart so that it can be applied for differing channel conditions, several approximations are made. First, the depth of flow in the channel is assumed equal to the hydraulic radius; the results will therefore be most applicable to wide, shallow channels. Furthermore, the increase in velocity head is assumed to vary proportionally along the length of the channel. Thus, the data given in the chart are not exact and are intended to serve only as a guide in estimating channel losses.

The chart plots the solution of the equation $s = \frac{dh_f}{dx}$, integrated between the limits from zero to L, or

$$h_f = \int_o^L s\, dx,$$

where, from the Manning equation,

$$s = \frac{v^2}{\left(\frac{1.486}{n} \right)^2 r^{4/3}}$$

F-3. ***Flow in Closed Conduits.-(a) Partly Full Flow in Conduits.-The*** hydraulics of partly full flow in closed conduits is similar to that in open channels, and open channel flow formulas are applicable. Hydraulic properties for different flow depths in circular and horseshoe conduits are tabulated in tables F-2 through F-5 to facilitate hydraulic computations for these sections.

Tables F-2 and F-4 give data for determining critical depths, critical velocities, and hydrostatic pressures of the water prism cross section for various discharges and conduit diameters. If the area at critical flow, a_c, is represented as $k_1 D^2$ and the top width of the water prism, T, for critical flow is equal to $k_2 D$, equation (7) can be written:

$$\frac{Q_c^2}{g} = \frac{(k_1 D^2)^3}{k_2 D}, \text{ or } Q_c = k_3 D^{5/2} \qquad (37)$$

Values of k_3, for various flow depths, are tabulated in column 3. The hydrostatic pressure, P, of the water prism cross section is $wa\bar{y}$, where $\bar{y}$ is the depth from the water surface to the center of gravity of the cross section. If $a_c = k_1 D^2$ and $\bar{y} = k_4 D$, then

$$P = k_5 D^3 \qquad (38)$$

Values of k_5, for various flow depths, are tabulated in column 4. Column 2 gives the values of h_{v_c} in relation to the conduit diameter, for various flow depths.

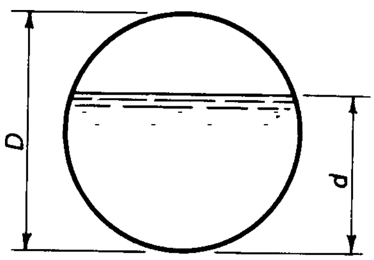
Tables F-3 and F-5 give areas and hydraulic radii for partly full conduits and coefficients which can be applied in the solution of the Manning equation. If $A = k_6 \frac{\pi D^2}{4}$ and $r = k_7 D$, Manning's equation can be written:

$$Q = \frac{1.486}{n} \left(k_6 \frac{\pi D^2}{4} \right) (k_7 D)^{2/3} s^{1/2},$$

or

TABLE F-2.—*Velocity head and discharge at critical depths and static pressures in circular conduits partly full.*—288-D-3195

D= Diameter of pipe.
d=Depth of flow.
h_{v_c}= Velocity head for a critical depth of
Q_c= Discharge when the critical depth is *d.*
P= Pressure on cross section of water prism in cubic units of water. To get *P* in pounds, when **d** and *D* are in feet, multiply by 62.5.



$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$
1	2	3	4	1	2	3	4	1	2	3	4
0.01	0.0033	0.0006	0.0000	0.34	0.1243	0.6657	0.0332	0. 67	0. 2974	2.4464	0.1644
.02	.0067	.0025	.0000	.35	1284	.7040	.0356	.68	.3048	2.5182	1700
.03	.0101	.0055	0001	.36	.1326	.7433	.0381	.69	.3125	2.5912	.1758
.04	.0134	.0098	.0002	.37	.1368	.7836	.0407	.70	.3204	2.6656	1816
.05	.0168	.0153	.0003	.38	.1411	.8249	.0434	.71	.3286	2. 7414	1875
.06	.0203	.0220	.0005	.39	.1454	.8671	.0462	.72	.3371	2. 8188	.1935
.07	.0237	.0298	.0007	.40	.1497	.9103	.0491	.73	.3459	2.8977	1996
.08	.0271	.0389	0010	.41	.1541	.9545	.0520	.74	.3552	2.9783	.2058
.09	.0306	.0491	.0013	.42	1586	.9996	.0551	.75	3648	3.0607	2121
1 0	.0341	.0605	.0017	.43	.1631	1.0458	.0583	i6	.3749	3. 1450	2185
.11	.0376	.0731	.0021	.44	.1676	1.0929	.0616	.77	.3855	3. 2314	.2249
.12	.0411	.0868	.0026	.45	1723	1.1410	.0650	78	.3967	3.3200	2314
.13	.0446	1016	.0032	.46	1769	1.1899	.0684	79	.4085	3.4112	.2380
.14	.0482	1176	.0038	.47	.1817	1.2399	.0720	.80	.4210	3.5050	2447
.15	.0517	.1347	.0045	.48	.1865	1.2908	.0757	.81	.4343	3.6019	2515
1 6	.0553	1530	.0053	.49	.1914	1.3427	.0795	.82	.4485	3.7021	.2584
.17	.0589	1724	.0061	.50	.1964	1.3955	.0833	.83	.4638	3.8061	.2653
.18	.0626	.!928	.0070	.51	.2014	1.4493	.0873	.84	.4803	3.9144	.2723
.19	.0662	.2144	.0080	.52	.2065	1.5041	.0914	.85	.4982	4.0276	.2794
.20	.0699	.2371	.0091	.53	.2117	1.5598	.0956	.86	.5177	4. 1465	.2865
.21	.0736	.2609	.0103	.54	.2170	1.6164	.0998	.87	.5392	4.2721	.2938
.22	.0773	.2857	.0115	.55	.2224	1.6735	.1042	.88	.5632	4.4056	.3011
.23	.0811	.3116	.0128	.56	2279	1.7327	1087	.89	.5900	4.5486	.3084
2 4	.0848	.3386	.0143	.57	.2335	1. 7923	1133	.90	.6204	4.7033	.3158
2 5	.0887	.3667	.0157	.58	.2393	1.8530	1179	.91	6555	4.8725	.3233
.26	.0925	.3957	.0173	.59	.2451	1. 9146	122i	.92	.6966	5.0603	.3308
.27	.0963	.4259	.0190	.60	.2511	1.9773	1276	.93	.7459	5.2726	.3384
.28	.1002	.4571	.0207	.61	.2572	2.0409	.1326	.94	.8065	5.5183	.3460
.29	.1042	.4893	.0226	.62	.2635	2.1057	.1376	.95	.8841	5.8118	.3537
.30	1081	.5225	.0255	.63	.2699	2.1716	.1428	.96	.9885	6.1787	.3615
.31	1121	.5568	.0266	.64	.2765	2.2386	.1481	.97	1.1410	6.6692	.3692
.32	1161	.5921	.0287	.65	.2833	2. 3067	.1534	.98	1.3958	7.4063	.3770
.33	.1202	.6284	.0309	.66	.2902	2.3766	.1589	.99	1.9700	8.8263	.3848
								1.00	......	.....	.3927

$$\frac{Qn}{D^{8/3}s^{1/2}} = k_6\frac{1.486\pi}{4}(k_7)^{2/3} = k_8 \quad \textbf{(39)}$$

Values of k_8, for various flow depths, are tabulated in column 4. If $D = k_9 d$, equation (39) can be written:

$$\frac{Qn}{d^{8/3}s^{1/2}} = \frac{1.486\pi}{4}k_6(k_7)^{2/3}(k_9)^{8/3} = k_{10} \quad \textbf{(40)}$$

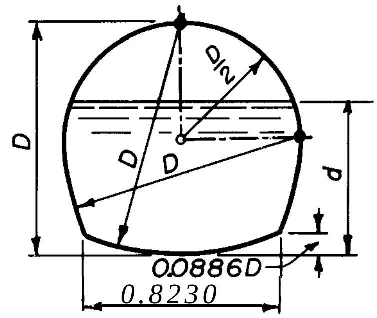
TABLE F-3.-Uniform *flow* in circular sections *flowing* partly full. -288-D-3196

d = Depth of flow.
D = Diameter of pipe.
A = Area of flow.
r = Hydraulic radius.
Q = Discharge in c.f.s. by Manning's formula.
n = Manning's coefficient.
s = Slope of the channel bottom and of the water surface.

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$
1	2	3	4	5	1	2	3	4	5
0.01	0.0013	0.0066	0.00007	15.04	0.51	0.4027	0.2531	0.239	1.442
.02	.0037	.0132	.00031	10.57	.52	.4127	.2562	.247	1.415
.03	.0069	.0197	.00074	8.56	.53	.4227	.2592	.255	1.388
.04	.0105	.0262	.00138	7.38	.54	.4327	.2621	.263	1.362
.05	.0147	.0325	.00222	6.55	.55	.4426	.2649	.271	1.336
.06	.0192	.0389	.00328	5.95	.56	.4526	.2676	.279	1.311
.07	.0242	.0451	.00455	5.47	.57	.4625	.2703	.287	1.286
.08	.0294	.0513	.00604	5.09	.58	.4724	.2728	.295	1.262
.09	.0350	.0575	.00775	4.76	.59	.4822	.2753	.303	1.238
.10	.0409	.0635	.00967	4.49	.60	.4920	.2776	.311	1.215
.11	.0470	.0695	.01181	4.25	.61	.5018	.2799	.319	1.192
.12	.0534	.0755	.01417	4.04	.62	.5115	.2821	.327	1.170
.13	.0600	.0813	.01674	3.86	.63	.5212	.2842	.335	1.148
.14	.0668	.0871	.01952	3.69	.64	.5308	.2862	.343	1.126
.15	.0739	.0929	.0225	3.54	.65	.5404	.2882	.350	1.105
.16	.0811	.0985	.0257	3.41	.66	.5499	.2900	.358	1.084
.17	.0885	.1042	.0291	3.28	.67	.5594	.2917	.366	1.064
.18	.0961	.1097	.0327	3.17	.68	.5687	.2933	.373	1.044
.19	.1039	.1152	.0365	3.06	.69	.5780	.2948	.380	1.024
.20	.1118	.1206	.0406	2.96	.70	.5872	.2962	.388	1.004
.21	.1199	.1259	.0448	2.87	.71	.5964	.2975	.395	0.985
.22	.1281	.1312	.0492	2.79	.72	.6054	.2987	.402	.965
.23	.1365	.1364	.0537	2.71	.73	.6143	.2998	.409	.947
.24	.1449	.1416	.0585	2.63	.74	.6231	.3008	.416	.928
.25	.1535	.1466	.0634	2.56	.75	.6319	.3017	.422	.910
.26	.1623	.1516	.0686	2.49	.76	.6405	.3024	.429	.891
.27	.1711	.1566	.0739	2.42	.77	.6489	.3031	.435	.873
.28	.1800	.1614	.0793	2.36	.78	.6573	.3036	.441	.856
.29	.1890	.1662	.0849	2.30	.79	.6655	.3039	.447	.838
.30	.1982	.1709	.0907	2.25	.80	.6736	.3042	.453	.821
.31	.2074	.1756	.0966	2.20	.81	.6815	.3043	.458	.804
.32	.2167	.1802	.1027	2.14	.82	.6893	.3043	.463	.787
.33	.2260	.1847	.1089	2.09	.83	.6969	.3041	.468	.770
.34	.2355	.1891	.1153	2.05	.84	.7043	.3038	.473	.753
.35	.2450	.1935	.1218	2.01	.85	.7115	.3033	.477	.736
.36	.2546	.1978	.1284	1.958	.86	.7186	.3026	.481	.720
.37	.2642	.2020	.1351	1.915	.87	.7254	.3018	.485	.703
.38	.2739	.2062	.1420	1.875	.88	.7320	.3007	.488	.687
.39	.2836	.2102	.1490	1.835	.89	.7384	.2995	.491	.670
.40	.2934	.2142	.1561	1.797	.90	.7445	.2980	.494	.654
.41	.3032	.2182	.1633	1.760	.91	.7504	.2963	.496	.637
.42	.3130	.2220	.1705	1.724	.92	.7560	.2944	.497	.621
.43	.3229	.2258	.1779	1.689	.93	.7612	.2921	.498	.604
.44	.3328	.2295	.1854	1.655	.94	.7662	.2895	.498	.588
.45	.3428	.2331	.1929	1.622	.95	.7707	.2865	.498	.571
.46	.3527	.2366	.201	1.590	.96	.7749	.2829	.496	.553
.47	.3627	.2401	.208	1.559	.97	.7785	.2787	.494	.535
.48	.3727	.2435	.216	1.530	.98	.7817	.2735	.489	.517
.49	.3827	.2468	.224	1.500	.909	.7841	.2666	.483	.496
.50	.3927	.2500	.232	1.471	1.00	.7854	.2500	.463	.463

TABLE F-4.– ***Velocity head and discharge at critical depths and static pressures in horseshoe conduits partly full.*** **-288-D-3197**

D= *Diameter* of horseshoe.
d= Depth of flow.
h_{v_c} = Velocity head for a critical depth of d.
Q_c= Discharge when the critical depth is d.
P= Pressure on cross section of water prism in cubic units of water. To get P in pounds, when d and D are in feet, multiply by 62.5.



$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$	$\frac{d}{D}$	$\frac{h_{v_c}}{D}$	$\frac{Q_c}{D^{5/2}}$	$\frac{P}{D^3}$
1	2	3	4	1	2	3	4	1	2	3	4
0.01	0.0033	0.0009	0.0000	0.35	0.1472	0.8854	0.0449	0.69	0.3362	2.8922	0.1999
.02	.0067	.0035	.0000	.36	.1518	.9296	.0478	.70	.3413	2.9702	.2062
.03	0100	.0079	.0001	.37	.1563	.9746	.0508	.71	.3528	3.0499	.2125
.04	.0134	.0139	.0002	.38	.1609	1.0205	.0540	.72	.3615	3.1311	.2190
.05	.0168	.0217	.0004	.39	.1655	1.0673	.0572	.73	.3707	3.2140	.2255
.06	.0201	.0312	.0007	.40	.1702	1.1148	.0605	.74	.3802	3.2987	.2321
.07	.0235	.0425	0010	.41	.1749	1.1633	.0639	75	.3902	3.3853	.2385
.08	.0269	.0554	.0014	.42	.1795	1.2125	.0675	:76	.4006	3.4740	.2457
.09	.0305	.0703	.0018	.43	.1843	1.2626	.0711	.77	.4116	3.5650	.2525
10	.0351	.0879	.0024	.44	.1890	1.3135	.0748	.78	.4232	3.6584	.2595
.11	.0397	.1069	.0030	.45	.1938	1.3652	.0786	.79	.4354	3.7544	.2666
.12	.0443	1272	.0037	.46	.1986	1.4178	.0825	.80	.4484	3.8534	.2737
.13	.0489	.1487	.0045	.47	.2035	1.4712	.0865	.81	.4623	3.9557	.2809
.14	.0534	.1714	.0054	.48	.2084	1.5253	.0907	.82	.4771	4.0616	.2882
.15	.0579	.1953	.0063	.49	.2133	1.5803	.0949	.83	.4930	4.1716	.2956
.16	.0624	.2203	.0074	.50	.2183	1.6361	.0992	.84	.5102	4.2863	.3030
.17	.0669	.2465	.0085	.51	.2234	1.6928	.1036	.85	.5289	4.4063	.3105
.18	.0714	.2736	.0098	.52	.2285	1.7505	.1081	.86	.5494	4.5325	.3181
.19	.0758	.3019	.0111	53	.2337	1.8992	.1127	.87	.5719	4.6660	.3258
.20	.0803	.3312	.0125	:54	.2391	1.8686	.1174	.88	.5969	4.8080	.3335
.21	.0847	.3615	.0140	.55	.2445	1.9294	1223	.89	.6251	4.9605	.3413
.--							:		.6570	5.1256	.3492
.23 2	.0936	.4251	.0173	.57	.2557	2.0537 1.9911	.1322	1272 ..91	.6939	5.3065	.3572
.24	.0980	.4583	.0191	.58	.2615	2.1174	.1373	.92	.7371	5.5077	.3653
.25	1024	.4926	.0210	.59	.2674	2.1821	.1425	.93	.7889	5.7354	.3733
.26	.1069	.5277	.0229	.60	.2735	2.2479	1478	.94	.8528	5.9996	.3813
.27	.1113	.5638	.0250	.61	.2797	2.3148	.1532	.95	.9345	6.3157	.3894
.28	.1158	.6009	0271	.62	.2861	2.3828	.1587	.96	1.0446	6.7114	.3976
.29	.1202	.6389	.0294	.63	.2926	2.4519	.1643	.97	1.2053	7.2417	.4058
.30	.1247	.6777	.0317	64	.2994	2.5221	.1700	.98	1.4742	8.0892	4140
.31	.1292	.7175	.0342	.65	.3063	2.5936	.1758	.99	2.0804	9.5780	.4223
.32	1337	.7582	.0367	.66	.3134	2.6663	1817	1.00	....	....	.4306
.33	.1382	.7997	.0393	.67	.3208	2.7402	.1877				
.34	.1427	.8421	.0421	.68	.3283	2.8155	.1937				

Values of $k_{1\,0}$, for various flow depths, are tabulated in column 5.

(b) *Pressure Flow in Conduits.* -Since factors affecting head losses in conduits are independent of pressure, the same laws apply to flow in both closed conduits and open channels, and the formulas for each take the same general form. Thus, the equation of continuity, equation (33), $Q = a_1 v_1 = a_2 v_2$, also applies to pressure flow in conduits.

TABLE F-5 .-*Uniform flow in horseshoe sections flowing partly full.* -288-D-3198

d=Depth of flow.
D=Diameter.
A=Area of flow.
r=Hydraulic radius.

Q= Discharge in c.f.s. by Manning's formula.
n=Manning's coefficient.
s=Slope of the channel bottom and of the water surface.

$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$	$\frac{d}{D}$	$\frac{A}{D^2}$	$\frac{r}{D}$	$\frac{Qn}{D^{8/3}s^{1/2}}$	$\frac{Qn}{d^{8/3}s^{1/2}}$
1	2	3	4	5	1	2	3	4	5
0.01	0.0019	0.0066	0.00010	21.40	0.51	0.4466	0.2602	0.2705	1.629
.02	.0053	.0132	.00044	14.93	.52	.4566	.2630	.2785	1.593
.03	.0097	.0198	00105	12.14	.53	.4666	.2657	.2866	1.558
.04	.0150	.0264	00198	10.56	.54	.4766	.2683	.2946	1.524
.05	.0209	.0329	00319	9.40	.55	.4865	.2707	,303	1.490
.06	.0275	.0394	00473	8.58	.56	.4965	.2733	.311	1.458
.07	.0346	.0459	.00659	7.92	.57	.5064	.2757	,319	1.427
.08	.0421	0524	00876	7.37	.58	.5163	.2781	.327	1.397
.09	.0502	.0590	.01131	6.95	.59	.5261	.2804	,335	1.368
.10	.0585	.0670	.01434	6.66	.60	.5359	.2824	.343	1.339
.11	.0670	.0748	.01768	6.36	.61	.5457	.2844	.351	1.310
.12	.0753	.0823	.02117	6.04	.62	.5555	.2864	.359	1.283
.13	.0839	0895	.02495	5.75	.63	.5651	.2884	.367	1.257
.14	.0925	.0964	.02890	5.47	.64	.5748	.2902	.374	1.231
.15	.1012	.1031	.0331	5.21	.65	.5843	.2920	.382	1.206
.16	.1100	.1097	.0375	4.96	.66	.5938	.2937	.390	1.181
.17	.1188	.1161	.0420	4.74	.67	.6033	.2953	,398	1.157
.18	.1277	.1222	.0467	4.52	.68	.6126	.2967	.405	1.133
.19	.1367	.1282	.0516	4.33	.69	.6219	.2981	.412	1.109
.20	.1457	.1341	.0567	4.15	70	.6312	.2994	,420	1.087
.21	.1549	.1398	.0620	3.98	.71	.6403	.3006	.427	1.064
.22	.1640	.1454	.0674	3.82	.72	.6493	.3018	.434	1.042
.23	.1733	.1508	.0730	3.68	.73	.6582	.3028	,441	1.021
.24	.1825	.1560	.0786	3.53	.74	.6671	.3036	.448	1.000
.25	.1919	.1611	.0844	3.40	.75	.6758	.3044	.454	0.979
.26	.2013	1662	.0904	3.28	.76	.6844	.3050	.461	.958
.27	.2107	:1710	.0965	3.17	.77	.6929	.3055	.467	.938
.28	.2202	.1758	.1027	3.06	.78	.7012	.3060	.473	,918
.29	.2297	.1804	.1090	2.96	.79	.7094	.3064	,479	.898
.30	.2393	.1850	.1155	2.86	.80	.7175	.3067	,485	,879
.31	.2489	.1895	.1220	2. 7:	.81	.7254	.3067	.490	.860
.32	.2586	.1938	.1287	2.69	.82	.7332	.3066	.495	.841
.33	.2683	.1981	.1355	2.61	.83	.7408	.3064	.500	.822
.34	.2780	.2023	1424	2.53	.84	.7482	.3061	,505	.804
.35	.2878	.2063	.1493	2.45	.85	.7554	.3056	.509	.786
.36	.2975	.2103	.1563	2.38	.86	7625	.3050	.513	.768
.37	.3074	.2142	.1635	2.32	.87	.7693	.3042	.517	,750
.38	.3172	.2181	.1708	2.25	.88	.7759	.3032	.520	.732
.39	.3271	.2217	.1781	2.19	.89	.7823	.3020	.523	.714
.40	.3370	.2252	1854	2.13	.90	.7884	.3005	,526	.696
.41	.3469	.2287	.1928	2.08	.91	.7943	.2988	,528	.678
.42	.3568	.2322	.2003	2.02	.92	.7999	.2969	.529	.661
.43	.3667	.2356	.2079	1.973	.93	.8052	.2947	.530	.643
.44	.3767	.2390	.2156	1.925	.94	.8101	.2922	.530	,625
.45	.3867	.2422	.2233	1.878	.95	.8146	.2893	.529	.607
.46	.3966	.2454	.2310	1.832	.96	.8188	.2858	.528	.589
.47	.4066	.2484	.2388	1.788	.97	.8224	.2816	.525	569
.48	.4166	.2514	.2466	1.746	.98	8256	.2766	,521	,550
.49	.4266	2544	.2545	1.705	.99	.8280	.2696	.513	,527
.50	.4366	.2574	.2625	1.667	1.00	.8293	.2538	,494	,494

A mass of water, as such, does not have pressure energy. Pressure energy is acquired by contact with other masses and is, therefore, transmitted to or through the mass under consideration. The pressure head-$ (where p is the pressure intensity in pounds per square foot and w is unit weight in pounds per cubic foot), like velocity and elevation heads, also expresses energy. Thus, to be applicable to pressure flow in a conduit, the Bernoulli equation for flow in open channels, equation (3), can be rewritten:

$$H_A = \frac{p}{w} + Z + \frac{v^2}{2g} \qquad (41)$$

The Bernoulli theorem for flow in a reach of pressure conduit (as shown on fig. F-5) is:

$$\frac{p_1}{w} + Z_1 + h_{v_1} = \frac{p_2}{w} + Z_2 + h_{v_2} + \Delta h_L \qquad (42)$$

where Δh_L represents the head losses within the reach from all causes. If H_T is the total head and v is the velocity at the outlet, Bernoulli's equation for the entire length is:

$$H_T = \Sigma(\Delta h_L) + h_v \qquad (43)$$

As in open channel flow, the Bernoulli theorem and the continuity equation are the basic formulas used in solving problems of pressure conduit flow.

(c) *Energy and Pressure Gradients.* - If piezometer standpipes were to be inserted at various points along the length of a conduit flowing under pressure, as illustrated on figure F-5, water would rise in each standpipe to a level equal to the pressure head in the conduit at those points. The pressure at any point may be equal to, greater than, or less than the local atmospheric pressure. The height to which the water would rise in a piezometer is termed the pressure gradient. The energy gradient is above the pressure gradient a distance equal to the velocity head. The fall of the energy gradient for a given length of conduit represents the loss of energy, either from friction or from friction and other influences. The relationship of the energy gradient to the pressure gradient reflects the variations between kinetic energy and pressure head.

(d) *Friction Losses.* -Many empirical formulas have been developed for evaluating the flow of fluids in conduits. Those in most common use are the Manning equation and the Darcy-Weisbach equation, previously given in this appendix and further discussed in chapter X.

The Manning equation assumes that the energy loss depends only on the velocity, the dimensions of the conduit, and the magnitude of wall roughness as defined by the friction coefficient n. The n value is related to the physical roughness of the conduit wall and is independent of the size of the conduit or of the density and viscosity of the water.

The Darcy-Weisbach equation assumes the loss to be related to the velocity, the dimensions of the conduit, and the friction factor f. The factor fis a dimensionless variable based on the viscosity and density of the fluid and on the roughness of the conduit walls as it relates to the size of the conduit.

Data and criteria for determining J'values for large pipe are given in a Bureau of Reclamation engineering monograph [61.

F-4. ***Hydraulic Jump.*** -The hydraulic jump is an abrupt rise in water surface which may occur in an open channel when water flowing at high velocity is retarded. The formula for the hydraulic jump is obtained by equating the unbalanced forces acting to retard the mass of flow to the rate of change of the momentum of flow. The general formula for this relationship is:

$$v_1^2 = g\,\frac{a_2\bar{y}_2 - a_1\bar{y}_1}{a_1\left(1 - \frac{a_1}{a_2}\right)} \qquad (44)$$

where:

v_1 = the velocity before the jump,
a_1 and a_2 = the areas before and after the jump, respectively, and

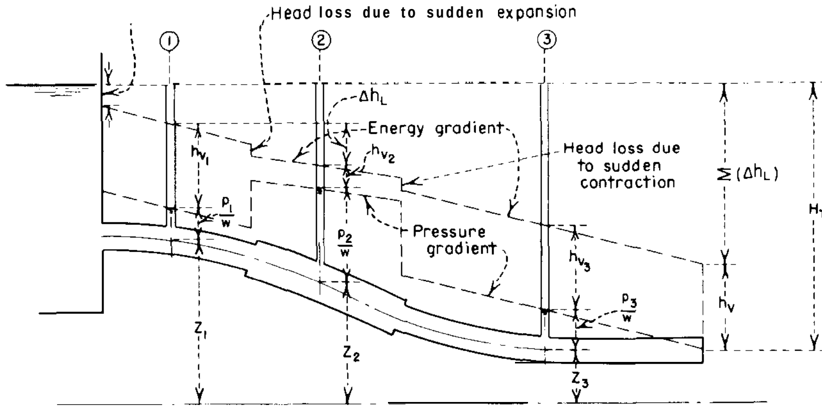


Figure F-S. **Characteristics of pressure flow in conduits,-288-D-2555**

$\bar{y}_1$ and $\bar{y}_2$ = the corresponding depths from the water surface to the center of gravity of the cross section.

The general formula expressed in terms of discharge is:

$$Q^2 = g\frac{a_2\bar{y}_2 - a_1\bar{y}_1}{\frac{1}{a_1} - \frac{1}{a_2}} \qquad (45)$$

or:

$$\frac{Q^2}{ga_1} + a_1\bar{y}_1 = \frac{Q^2}{ga_2} + a_2\bar{y}_2 \qquad (46)$$

For a rectangular channel, equation (44) can be reduced to $v_1{}^2 = \frac{gd_2}{2d}(d_2 + d_1)$, where d_1 and d_2 are the flow depths before and after the jump, respectively. Solving for d_2 :

$$d_2 = -\frac{d_1}{2}+\sqrt{\frac{2v_1{}^2 d_1}{g} + \frac{d_1{}^2}{4}} \qquad (47)$$

Similarly, expressing d_1 in terms of d_2 and v_2 :

$$d_1 = -\frac{d_2}{2}+\sqrt{\frac{2v_2{}^2 d_2}{g} + \frac{d_2{}^2}{4}} \qquad (48)$$

A graphic solution of equation (47) is shown on figure F-8.

If the Froude number $F_1 = \frac{v_1}{\sqrt{gd_1}}$ is substituted in the equation (47):

$$\frac{d_2}{d_1} = \frac{1}{2}\left(\sqrt{8F_1{}^2 + 1} - 1\right) \qquad (49)$$

Figure F-6 shows a graphical representation of the characteristics of the hydraulic jump. Figure F-7 shows the hydraulic properties of the jump in relation to the Froude number, as determined from experimental data [7]. And figure F-8 is a nomograph showing the relation between variables in the hydraulic jump.

Data are for jumps on a flat floor with no chute blocks, baffle piers, or end sills. Ordinarily, the jump length can be shortened by incorporation of such devices in the designs of a specific stilling basin.

F-5. *Bibliography.*

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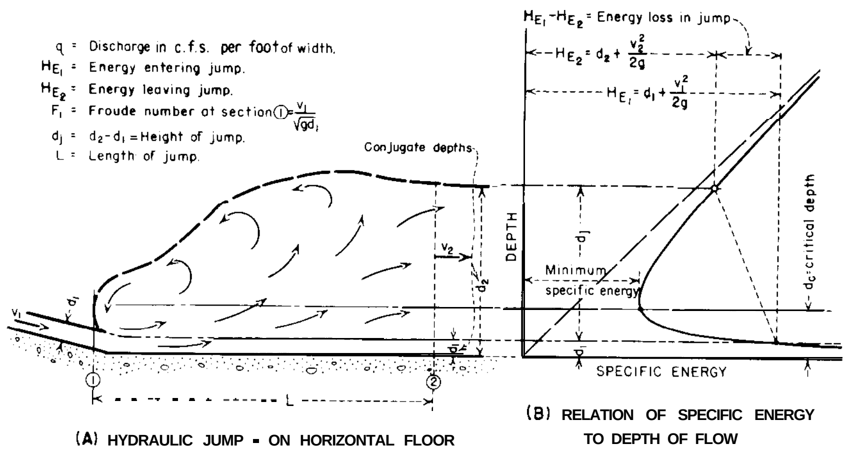


Figure F-6. **Hydraulic jump symbols and characteristics.-288-D-3190**

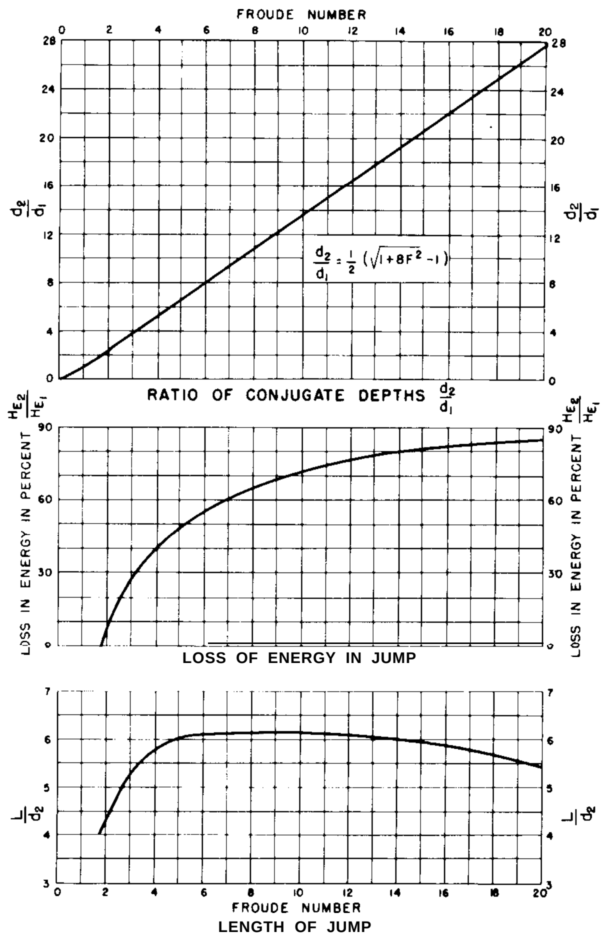


Figure F-7. **Hydraulic jump properties in relation to Froude number.-288-D-2558**

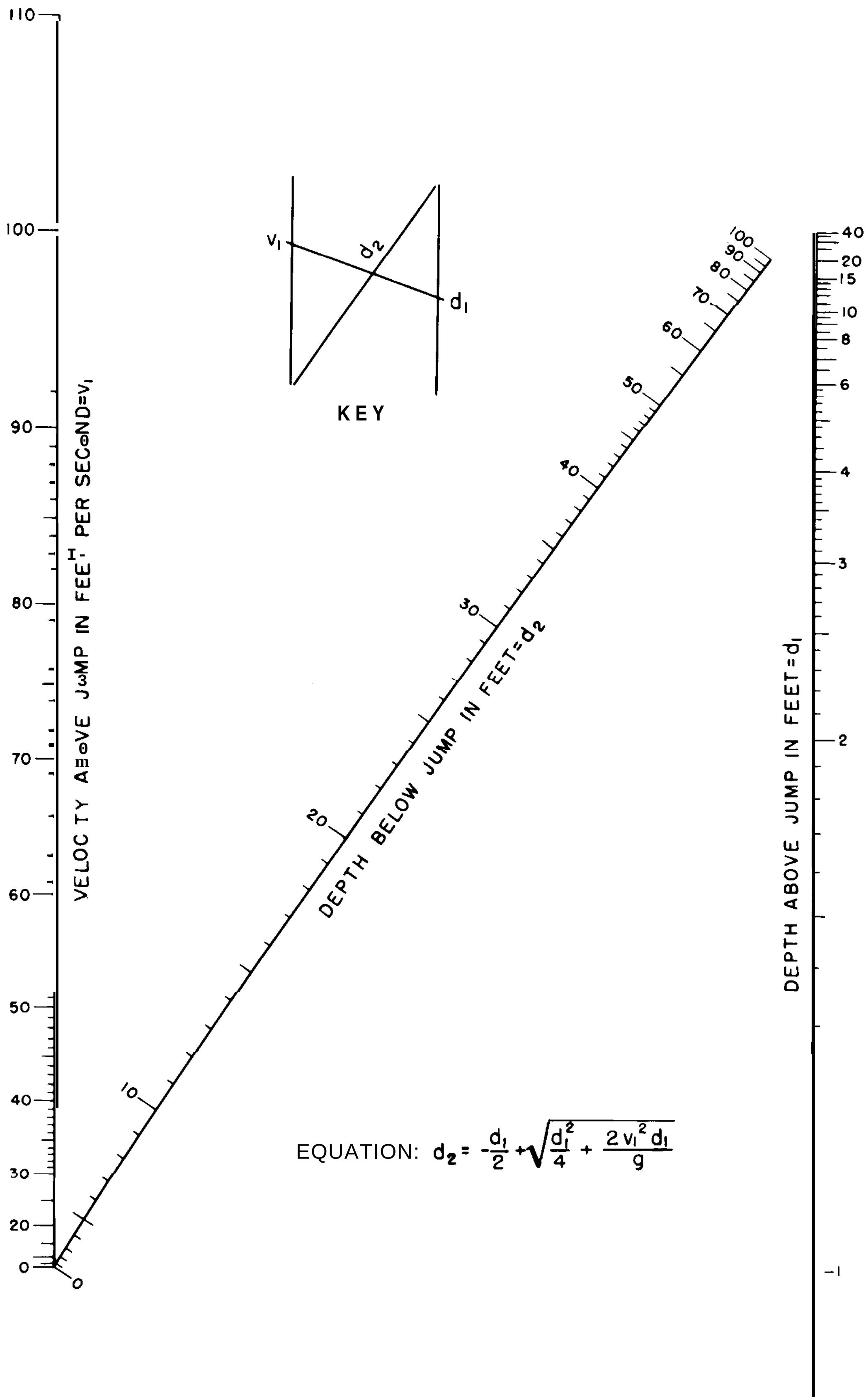


Figure F-8. **Relation between variables in the hydraulic jump.-288-D-2559**

Inflow Design Flood Studies

G-1. Introduction. -A 1970 report of the United States Committee on Large Dams (USCOLD) [1] [1] gives a definition of an inflow design flood (IDF) as:

> "The reservoir inflow-discharge hydrograph used in estimating the maximum spillway discharge capacity and maximum surcharge elevation finally adopted as a basis for project design . . . ."

An inflow design flood selected for design of a dam impounding considerable storage located where partial or total failure would cause sudden release of water and create major hazards to life or property downstream should be equal to a probable maximum flood (PMF). The USCOLD report defines a probable maximum flood as:

> "Estimates of hypothetical flood characteristics (peak discharge, volume and hydrograph shape) that are considered to be the most severe ***reasonably possible*** at a particular location, based on relatively comprehensive hydrometeorological analyses of critical runoff producing precipitation (and snowmelt, if pertinent) and hydrologic factors favorable for maximum flood runoff."

This appendix discusses flood hydrology studies relating to estimates of an inflow design flood equal to a probable maximum flood, as defined in the USCOLD report. The phrase "relatively comprehensive hydrometeorological analyses" in the preceding definition refers to studies by hydrometeorologists directed towards estimation of ***the physical upper limits of storm rainfall and maximum snow accumulation and melt rates.*** The resulting estimates of the physical upper limits to storm rainfall in a basin or region are usually called the "probable maximum storm" or "probable maximum precipitation" [2]. Both of these terms are used in this text but with more precise meanings attached to each term as discussed in sections G- 14 through G 17 on design storm studies.

Bureau of Reclamation policy in design of dams located where failure might create major hazards requires an inflow design flood estimated by evaluating the runoff from the most critical of the following situations:

(1) A probable maximum storm in conjunction with severe, but not uncommon, antecedent conditions.

(2) A probable maximum storm for the season of heavy snowmelt, in conjunction with a major snowmelt flood somewhat smaller than the probable maximum.

(3) A probable maximum snowmelt flood in conjunction with a major rainstorm less severe than the probable maximum storm for that season.

(a) ***Items to be Evaluated.*** -Depending on meteorological conditions for the basin above a damsite, on the size of the drainage area and, to a lesser extent, on the proposed size of reservoir and type of dam, it may be necessary to evaluate:

(1) Each of the above assumptions.

(2) Each of the two assumptions in which snowmelt is a factor.

(3) Where snowmelt is not a factor,

[1] Numbers in brackets refer to items in the bibliography, sec. G-32.

two probable maximum storms-a storm causing the maximum peak inflow, and a storm causing the maximum volume of inflow.

It is beyond the scope of this text to present a complete manual of all procedures used for estimating inflow design floods, because selection of procedures is dependent on available hydrological data and individual watershed characteristics.

(b) *Discussions in This Text.* -Discussions in this text will provide design engineers information about the problems encountered and some methods for their solution. Broad discussions accompany presentation of the information which concerns:

(1) Hydrologic data for estimating floodflows and data sources in the United States.

(2) Analyses of basic data.

(3) Unit hydrograph procedures for synthesizing the distribution of runoff of a basin above a damsite.

(4) Sources of generalized probable maximum precipitation values.

(5) An example of computation of a preliminary inflow design flood hydrograph and establishment of reservoir routing criteria for the flood.

Designers also need estimates of floodflows that may occur at the damsite during the construction period in order to estimate requirements for streamflow diversion. Such estimates are usually included in an inflow design flood study. Sections G-28 and G-29 discuss selected methods of estimating flood magnitudes and frequency of occurrence at the damsite.

Every damsite presents one or more unique problems to probable maximum flood estimates. An inflow design flood (IDF) used for final designs of a dam should be based on estimates by an experienced hydrometeorologist of probable maximum precipitation values *for the basin above the damsite*, not on generalized probable maximum precipitation values for a region. The methods of preparing a study which yields generalized estimates of probable maximum precipitation inherently result in values that are somewhat greater than values obtained from an individual basin study.

Sections G-14 through G-17 present a general discussion of methods and assumptions that a hydrometeorologist may use in the preparation of hydrometeorological studies for individual basins. The physical characteristics of a basin may vary as to: drainage area size, relatively small to extremely large; runoff characteristics, similar throughout the basin or including tributary areas with markedly dissimilar runoff producing conditions; contribution from snowmelt; etc. Sections G-23 through G-26 describe some methods of estimating the contribution of snowmelt runoff to inflow design floods.

The final IDF study converting probable maximum precipitation values to an IDF hydrograph should be prepared by experienced flood hydrologists. Remarks regarding considerations for development of a final IDF study are included throughout the text and a brief summary of these considerations is given in sections G-30 and G-3 1.

Computational procedures given in this text are oriented toward step by step "long-hand" solutions, recognizing that the ever-increasing advances in computer technology provide greatly expanded capability in all phases of flood hydrology studies. One should be mindful, though, as stated in World Meterological Organization (WMO) Technical Note No. 98 [2] that: "While the computer is a powerful tool, it must be recognized that it is simply that, and results are no better than the basic logic and methods of application."

The bibliography, section G-32, includes selected references to hydrometeorological studies in addition to those specifically referred to in the text.

A. COLLECTION OF HYDROLOGIC DATA FOR USE IN ESTIMATING FLOODFLOWS

G-2. General.-For all flood studies, compilation and judgment as to quality of all available streamflow, precipitation, and watershed data are most important. Mathematical procedures cannot improve the quality of input data, and analyses procedures must be compatible with the data available.

G-3. ***Streamflow Data.*** -The hydrologic data most directly useful in determining floodflows are actual streamflow records of considerable length at the location of the dam. Such records are rarely available. The engineer should obtain the streamflow records available for the general region in which the dam is to be situated. Locations of stream gaging stations and precipitation stations in the United States are shown on a series of maps entitled "River Basin Maps Showing Hydrologic Stations," edition 1961,[2] prepared under the supervision of the National Weather Service. Such data collecting stations are subject to change in location, discontinuation, or initiation of new stations. These maps cannot be kept current, and information thereon must be supplemented by additional investigations in order to be sure of the location and operation of stations in a given area. The engineer should consult the water supply papers, catalogs, maps, and indexes of the U.S. Geological Survey' and, if possible, confer with the Survey's district engineer. He should also make a search of the records of other Federal agencies which may have collected information in the region, and the records of State water conservation agencies or State geological surveys; and he should determine whether any information may be available from other State departments, from county engineer offices, from municipalities in the vicinity, or from utility companies. Where streamflow records are not available, some agencies or inhabitants of the vicinity may have information about high-water marks caused by specific historic floods.

With respect to the character of the streamflow data available, floodflows at the damsite may be determined under one of the following conditions:

(1) ***Streamflow record at or near the*** *damsite.* -If such a record is available and covers a period of 20 years or more, the floodflows shown by the record may be analyzed to provide flood frequency values. Hydrographs of outstanding flood events can be analyzed to provide runoff factors for use in determining the maximum probable flood.

If such a record is available but covers only a few years, it may not include any flood of great magnitude within its limits and, if used alone, it would give false indication of flood potential. Analysis may, however, give some or all of the runoff factors needed to compute the probable maximum flood. Frequency values obtained from a short record should not be used without analysis of data from nearby watersheds of comparable runoff characteristics.

(2) Streamflow record available on the stream itself, but at a considerable distance from the *damsite.* -Such a record may be analyzed to provide unitgraph characteristics and frequency data which may be transferred to the damsite by appropriate area and basin-characteristic coefficients. This transfer can be made directly from one drainage area to another if the areas have comparable characteristics. Often damsites are located within the transition zone from mountains to plains and the stream gaging stations are located well out on the plains; in such instances, special care must be exercised when using the plains record for determination of floodflows at the damsite.

(3) *No* ***adequate*** *streamflow* ***data***

*Published by the Government Printing Office and available in libraries designated as depositories of Government publications; most important libraries in the United States are so designated.

available on the specific stream, but a satisfactory record for a drainage basin of similar characteristics in the same region. -Such a record may be analyzed for unitgraph characteristics and frequency data, and these data transferred to the damsite by appropriate area and basin-characteristic coefficients.

(4) Streamflow records in the region, but not satisfactorily useful for application and analysis under one of the above methods.-These records may be assembled and analyzed as reference information on general runoff characteristics.

(5) Use of high-water marks. -High-water marks pointed out by inhabitants of the valley should be used with caution in estimating flood magnitudes. However, where there are a number of high-water marks in the vicinity of the project, and particularly if such marks are obtained from the records of public offices (such as State highway departments or county engineers), they may be used as the basis of a separate supplemental study. These records may be used to determine the water cross-sectional area and the water surface slope for the flood to which they refer, and from these data an estimate of that particular flood peak may be prepared using the slope-area method described in appendix B of the Bureau of Reclamation publication "Design of Small Dams" [31].

Whenever it appears that there will be one or more flood seasons between the selection of the damsite and construction of the dam, facilities for securing a streamflow record for the project should be set up as promptly as possible. This is of particular importance in order to obtain watershed data directly applicable to the computation of the inflow design flood for the dam, although a record usable for frequency computations cannot be secured. The, facilities for obtaining such a record should be the best possible depending on the circumstances. A detailed discussion of these facilities, which may consist of either nonrecording or recording gages, is included in the following publications: "Equipment for Current-Meter Gaging Stations," U.S. Geological Survey Water Supply Paper 371; "Stream-Gaging Procedure," U.S. Geological Survey Water Supply Paper 888; and "Stream Flow," by Grover and Harrington, John Wiley & Sons, Inc., New York, 1943. The advice of Geological Survey engineers will be helpful in the site selection and installation, operation, and interpretation of records obtained.

A series of manuals "Techniques of Water-Resources Investigations of the United States Geological Survey," describes procedures for planning and executing specialized work in water-resources investigations. The material is grouped under major subject headings called books and further subdivided into sections and chapters; section A of book 3 is on surface water. The unit of publication, the chapter, is limited to a narrow field of subject matter. This format permits flexibility in revision and publication as the need arises.

Provisional drafts of chapters are distributed to field offices of the U.S. Geological Survey for their use. These drafts are subject to revision because of experience in use or because of advancement in knowledge, techniques, or equipment. After the technique described in a chapter is sufficiently developed, the chapter is published and is for sale by the Superintendent of Documents.[2]

The importance of utilizing records of runoff originating from the watershed above the damsite cannot be overemphasized. In the case of a damsite located on an ungaged stream, the establishment of measuring facilities as discussed above may produce basic data which would justify "eleventh hour" revision of the plans, thus improving the design of the dam.

G-4. ***Precipitation Data.-In*** each of the situations outlined in the preceding section, precipitation data are needed to evaluate factors for use in computing the probable maximum flood. The engineer should assemble the information with respect to precipitation

[2] In loc. cit. p. 437

during the greater storms in the region, and particularly for those storms for which runoff records are available. Such information can be obtained from publications of the National Weather Service[3] and Environmental Data Service. At present (1974), daily precipitation data for each month for each State are contained in the publication "Climatological Data." Hourly data for each month for each State obtained by recording precipitation gages are contained in the publication "Hourly Precipitation Data."[4] In areas where large storms have occurred, often precipitation data obtained by the National Weather Service precipitation stations have been supplemented by "bucket survey" data, i.e., information on rainfall amounts of unusual storms obtained from residents within the storm area by personnel of the National Weather Service and other Government agencies.

Locations of precipitation stations as of 1961 are shown on the series of maps "River Basin Maps showing Hydrologic Stations," previously referred to.

If plans are made to install streamflow measuring facilities as discussed in the preceding section, provision should also be made for obtaining precipitation records. An important item to consider is the selection of the location (or locations) of the precipitation gage, so that the catch will be a representative sample of average precipitation over the watershed. A comprehensive discussion of types of precipitation gages and observational procedures is contained in the National Weather Service publication "Instructions for Climatological Observers," Circular B, eleventh edition, January 1962.

G-5. Watershed Data. -All available information concerning watershed characteristics should be assembled. A map of the area above the damsite should be prepared showing the drainage system, contours if available, drainage boundaries, and locations of any precipitation stations and streamflow gaging stations. Available data on soil types, cover, and land usage provide valuable guides to judgment of runoff potential. Soil maps prepared by the U.S. Department of Agriculture will prove helpful when the watershed lies within areas so mapped. These surveys (if in print) are available for purchase from the Superintendent of Documents, Washington, D.C. Out-of-print maps and other unpublished surveys may be available for examination from the U.S. Department of Agriculture, county extension agents, colleges, universities, and libraries.

The hydrologist preparing the flood study should make an inspection trip over the watershed to verify drainage area boundaries and soil and cover information, and to determine if any noncontributing areas are included within the drainage boundaries. The trip should also include visits to nearby watersheds if it is anticipated that records from nearby watersheds will be used in the study.

B. ANALYSES OF BASIC HYDROLOGIC DATA

G-6. ***General.*** -A flood hydrologist first directs attention to individual large flood events, seeking procedures whereby a good estimate may be made of the hydrograph that will result from a given amount of precipitation. As floods which consist of combined snowmelt and rainfall runoff are difficult to separate into their two components, usually snowmelt floods and rain floods are analyzed separately. Analyses of rain floods only are discussed in these sections G-6 through G-8 with inclusion of examples of some mathematical computations. Considerations for runoff contribution from snowmelt are discussed separately in sections G-22 through G-26. Flood analyses of rainfall

[3] Official designation: U.S. Department of Commerce, National Oceanic and Atmospheric Administration, National Weather Service.

[4] Subscription to these publications may be made through the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402.

data are interrelated to analyses of respective runoff data, so that discussions of procedures for one must include some references to the other. In the discussion that follows, analysis of storm rainfall is described first and is followed by a description of the analysis of the resulting flood runoff. Procedures used to analyze streamflow data for estimating the frequency of occurrence of flood magnitudes are discussed in sections G-28 and G-29.

G-7. Estimating Runoff From Rainfall. — (a) *General.*—The hydrometeorological approach to analyzing flood events and using the information obtained to estimate the magnitude of hypothetical floods requires a firm estimate of the difference between precipitation and the resulting runoff. From a flood determination point of view, this difference is considered loss, that is, loss from precipitation in the form of water over a given watershed. A simple solution to derive this loss value appears to be in finding the rate at which water will infiltrate the soil. If this infiltration rate is known, along with the amount of precipitation, a simple subtraction should give the amount of runoff. However, there are other precipitation losses in addition to infiltration, such as interception by vegetative cover, surface storage, and evaporation, that may have material effect on runoff amounts.

Various types of apparatus have been devised to test the infiltration rates of soils, and studies have been made of interception and evaporation losses. Although maps to an extremely large scale could define most of the surface storage area, it is apparent that an accurate volumetric evaluation of all the loss factors can be made only for a highly instrumented, small plot of ground and that such an evaluation is not practical for a natural watershed composed of many square miles of varying type soils, vegetative cover, and terrain features. For this reason, hydrologic literature contains arguments against the "infiltration rate approach" to determination of runoff amounts. However, the infiltration rate approach is applied on an empirical basis to obtain a practical solution to the problem of determining amounts of runoff, recognizing that the values used are of the nature of index values rather than ***true*** values.

Natural events are studied and the difference between rainfall and runoff determined. Since this difference includes all the losses described above, it is usually called a ***retention loss*** or a ***retention rate.*** Such retention rates derived from available records may be adjusted to ungaged watersheds by analogy of soil type and cover.

The characteristics of a hydrograph must be understood so that respective amounts of runoff and precipitation are compared for estimating retention rates (and for other comparisons described later). A hydrograph of storm runoff obtained at a streamflow gaging station represents one or more of the following types of runoff from the watershed: channel runoff, surface runoff, interflow, and base flow. Brief definitions of these types are:

Channel runoff. —Caused by rain falling on the water surface of the stream. It begins with the start of precipitation and may be discernible from a slight rise of the hydrograph just after rainfall begins, but the quantity of channel runoff is so small that it is ignored in hydrograph analyses.

Surface runoff.—Occurs only when the rainfall rate is greater than the retention loss rate. This type of runoff causes most floods and the computational procedures in this text consider this type of runoff dominant.

Interflow. —Occurs when rainfall infiltrating the soil surface encounters an underground zone of lower permeability, travels above the zone to the surface downhill, and reappears to become surface runoff. This type of flow may also be called subsurface ***flow*** or ***quick return flow.***

Base flow.—The fairly steady flow of a stream from natural storage as shown by hydrographs during nonstorm (or nonactive snowmelt) periods.

In flood hydrology it is customary to deal separately with base flow and to combine all other types of flow into ***direct runoff*** in unknown proportions as assumed in this text.

Making studies to compare rainfall with

runoff requires a knowledge of the units of measurement used and the factors for conversion to common units. These conversion factors are given in appendix F. In the United States, precipitation is measured in inches and runoff is measured in cubic feet per second (abbreviated c.f.s.).

It is necessary to know the watershed area contributing the runoff at a given measuring point, in order to express the runoff volume of inches of depth over the watershed for comparison with precipitation amounts. When making such comparisons, the amount of runoff, expressed as inches, is termed ***rainfall*** excess, and the difference between the rainfall excess and the total precipitation is considered retention loss as just discussed.

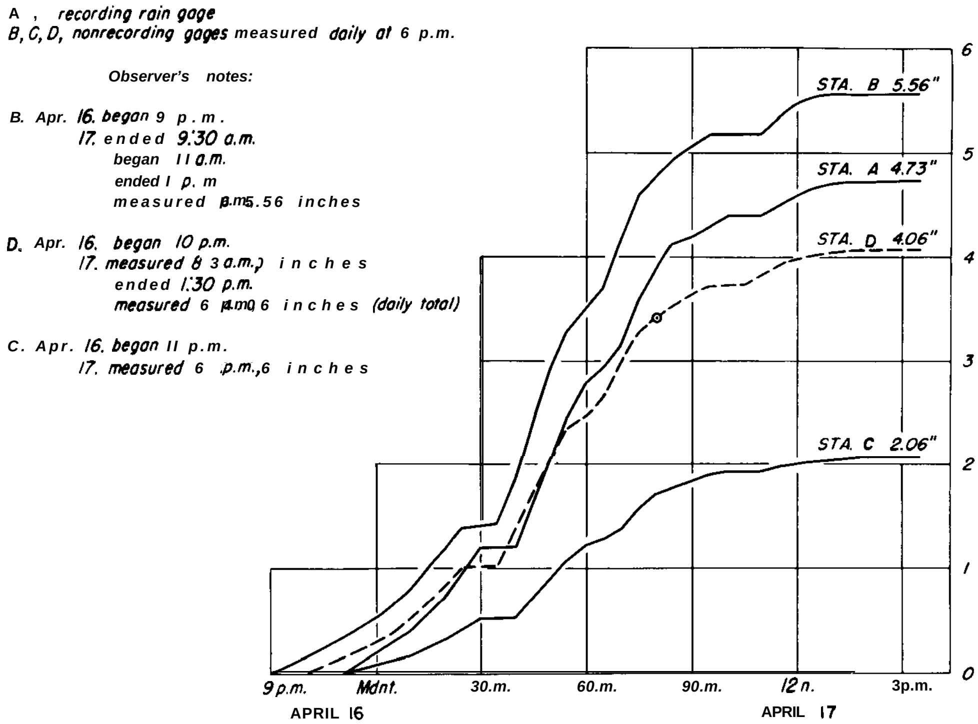
The following method of making a rainfall-runoff analysis has been selected for description in this text. The objectives of such analyses are: (1) the determination of a retention rate, and (2) the determination of the duration time interval of rainfall excess. A comparison of retention rates derived from several analyses leads to adoption of a rate for design flood computations. The determination of the duration of excess rainfall is necessary for the hydrograph analyses computations involving determinations of unitgraphs and lag-times, which are discussed later in this section and in sections beginning with G-9. In all such analyses, the runoff volume which is compared with precipitation amounts is that which relates directly to the rainfall under study. Therefore, the base flow of the streamflow hydrograph must be subtracted out before comparisons are made (see sec. G-8(c)).

(b) ***Analysis of Observed Rainfall Data***.–

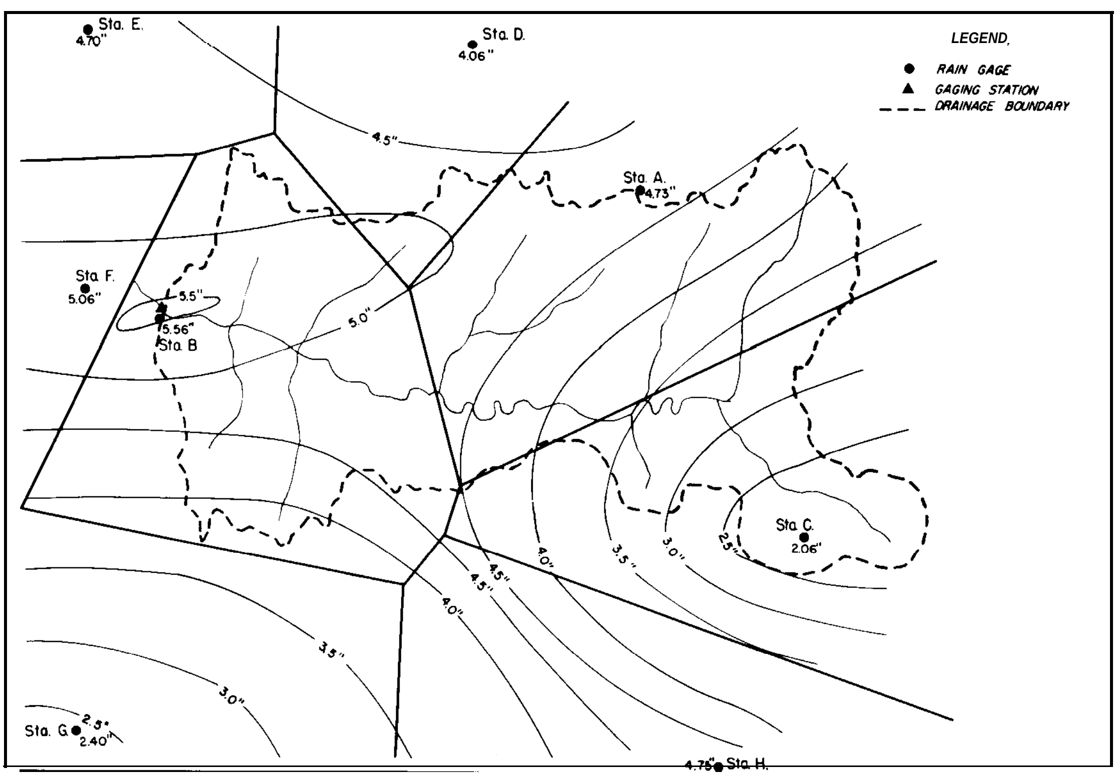
(1) ***Mass curves of rainfall.*** -Mass curves of cumulative rainfall during the storm period should be plotted for all precipitation stations in and near the basin as shown on figure G-l(A). To show clearly the relation of rainfall to runoff, it is sometimes desirable to plot the mass curves to the same time scale as the discharge hydrograph of storm runoff. Usually, however, the curves should be given a more expanded time scale than it is desirable to use for the hydrograph analysis. When only one recording station is located nearby, and in the absence of better information, the mass curve of precipitation at a nonrecording station is usually considered to be proportional in shape to that of the recording station, except as otherwise defined by the observer's readings and notes (fig. G-l(A)). The speed and direction of travel of the rainburst should be taken into account. Many rainfall observers enter the times of beginning and ending on the same line as the current daily reading. The notes may therefore refer to the previous day, especially when the gage is regularly read in the morning.

(2) Isohyetal maps.-The total amounts of rainfall occurring during the portion of the storm that produced the flood hydrograph under study should be determined from the mass curves for each station in and near the drainage area. For a flood hydrograph consisting of a single event, this will be the total depth of precipitation occurring during the storm period. For a compound hydrograph, in which individual portions of the hydrograph are studied separately, temporary cessations of rainfall will usually be indicated in the mass curves, and from inspection it usually will be apparent which of the increments of rainfall caused the runoff event under study. The appropriate depths of rainfall are then used to draw an isohyetal map, using standard procedures. A typical isohyetal map for plains-type terrain is shown on figure G-l(B). Isohyets are generally drawn smoothly, interpolating between precipitation stations. The interpolation should not be excessively mechanical.

Extreme caution should be used in drawing the isohyetal pattern in mountainous areas where the orographic effect is an important factor in the area1 distribution of rainfall. For example, if there is a precipitation station in a valley on one side of a mountain range and another station in a valley on the opposite side of the range with no intervening station, it cannot be assumed that the rainfall during a storm would vary linearly between the two stations. It is likely that the rainfall would increase with increases in elevation on the windward side of the divide, whereas on the leeward side, precipitation would decrease



(A) MASS CURVES OF RAINFALL



(B) ISOHYETS AND THIESSEN POLYGONS

Figure G-l. Analysis of observed rainfall data.-288-D-3158 rapidly with distance from the divide. This type of distribution can usually be verified in mountainous areas where there are sufficient precipitation stations to define the isohyetal pattern accurately.

A storm isohyetal pattern for mountainous terrain may be constructed by the isopercental technique, discussed in WMO Technical Note No. 98 [2] as follows:

"In mountainous regions the simple interpolation technique would yield unsatisfactory isohyets. Yet to prepare a valid isohyetal pattern in a mountainous region is not easy. One commonly used procedure is the isopercental technique, excellent under certain limited conditions stated in the next paragraph. This method requires a base chart of either mean annual precipitation, or preferably mean precipitation for the season of the storm, such as winter, summer, or monsoon months. In this method the ratio of the storm precipitation to the mean annual or mean seasonal precipitation (base precipitation) is plotted at each station. Isolines are drawn smoothly to these numbers. The ratios on the lines are then multiplied by the original base chart values at a large number of points to yield the storm isohyetal chart. Thus the storm isohyetal gradients and locations of centers tend to resemble the features of the base chart, which in turn is influenced by terrain.

"The first requirement for success of the isopercental technique is that a reasonably accurate mean annual or mean seasonal precipitation chart be available as a base. The base chart is of more value if it contains precipitation stations in addition to those reporting in the storm than if both charts are drawn exclusively from data observed at the same stations. The value of the base chart is also enhanced, in regions where the runoff of streams is a large percentage of the precipitation, if the precipitation shown on the chart has been adjusted not only for topographic factors, but also adjusted to agree with seasonal streamflow. In regions where a large percentage of the precipitation evaporates, adjustment to runoff volumes would be of dubious value.

"An additional requirement for success of the isopercental technique is that most of the annual or seasonal precipitation in the region result from storms with relatively the same wind direction, and from storms with minimal convective activity. Under these circumstances an individual storm will have a strong resemblance to the mean chart, as the latter is an average of kindred storms.

"In the Tropics with the dominance of convective activity and with lighter winds, the isopercental technique is of less value in analysis of an individual storm than in middle latitude locations that meet the other requirements."

After the preliminary hydrographs and the isohyetal maps have been drawn, the atypical flood events for unit hydrographs determination may readily be eliminated. *Those floods having a combination of large volume, uniform intensities, isolated periods of rainfall, and uniform area1 distribution oj rainfall, should be chosen for further study.*

(3) Average rainfall by Thiessen polygons.-The average rainfall on a drainage area can be determined from precipitation station records by the Thiessen polygon method. A sample computation of average hourly rainfall from the mass curves on figure G-l(A), using Thiessen polygons indicated on figure G-l(B), is given in table G-l.

The first step is to construct the Thiessen polygons, which are the areas bounded by the perpendicular bisectors of lines joining adjacent precipitation stations. The percentage of the drainage area controlled by each station's polygon is planimetered and entered in table G-l. Next, the average depth of rainfall over each station's polygon is determined by planimetering areas between isohyets on figure G-l(B). A factor to be used in weighing station rainfall values is obtained by multiplying the percentage of the drainage area controlled by each station's polygon by the ratio of the average depth of rainfall over each station's polygon to the observed rainfall at the station, and dividing by 100.

Hourly incremental rainfall values are determined for each precipitation station from

Table *G-l .-Computation of rainfall increments*

COMPUTATION OF STATION WEIGHTS

Station (1)	Average rainfall over Thiessen polygon (2)	Percent of basin area (3)	Rainfall at station (4)	Weight, col. (2) x col. (3) / 100 x col. (4) (5)
A	4.3	38.9	4.73	0.35
B	4.6	37.0	5.56	.31
C	2.8	21.1	2.06	.29
D	5.0	3.0	4.06	.04

COMPUTATION OF WEIGHTED AVERAGE HOURLY RAINFALL OVER BASIN

Time, hours	Station A			Station B			Station C			Station D			Weighted average, sum of cols. (3)
	Mass rf. (1)	Δ rf. (2)	0.35xΔrf. (3)	Mass rf. (1)	Δ rf. (2)	0.31xΔrf. (3)	Mass rf. (1)	Δ rf. (2)	0.29xΔrf. (3)	Mass rf. (1)	Δrf. (2)	0.04xΔrf (3)	
0				0									
1				.17	0.17	0.053				0			0.053
2	0			.33	.16	.050	0			.15	0.15	0.006	.056
3	.20	0.20	0.070	.52	.19	.059	.09	0.09	0.026	.29	.14	.006	.161
4	.40	.20	.070	.80	.28	.087	.17	.08	.023	.52	.23	.009	189
5	.73	.33	116	1.20	.40	.124	.32	.15	.044	.84	.32	.013	.297
6	1.20	.47	.164	1.41	.21	.065	.52	.20	.058	1.01	.17	.007	.294
7	1.20	0	0	1.85	.44	136	.52	0	0	1.34	.33	.013	.149
8	2.05	.85	.298	2.91	1.06	.329	.89	.37	.107	2.05	.71	.028	.762
9	2.80	.75	.262	3.49	.58	.180	1.22	.33	.096	2.47	.42	.017	.555
10	3.15	.35	.122	4.19	.70	.217	1.37	.15	.044	3.00	.53	.021	.404
11	3.90	.75	.262	4.79	.60	.186	1.70	.33	.096	3.40	.40	.016	.560
12	4.20	.30	.105	5.08	.29	.090	1.83	.13	.038	3.63	.23	.009	.242
13	4.40	.20	.070	5.18	10	.031	1.92	.09	.026	3.73	10	.004	.131
14	4.40	0	0	5.18	0	0	1.92	0	0	3.83	10	.004	.004
15	4.59	.19	.066	5.49	.31	.096	2.00	.08	.023	3.97	.14	.006	.191
16	4.70	.11	.038	5.56	.07	.022	2.04	.04	.012	4.04	.07	.003	.075
17	4.73	.03	.010	5.56	0	0	2.06	.02	.006	4.06	.02	.001	.017
Total		4.73	1.653		5.56	1.725		2.06	.599		4.06	163	4.140

the mass curves of figure G-l(A) and are multiplied by the appropriate weight factors as shown in table G-l, to obtain the total for the drainage area.

Additional information on determining average rainfall is given in "Cooperative Studies Technical Paper No. 1," published by the National Weather Service, and in references [2] and [17].

(4) Determination of rainfall excess. -Two methods may be used to determine rainfall excess: by assuming a constant average retention rate throughout the storm period, and by assuming a retention rate varying with time. The capacity rate of retention decreases progressively throughout the storm period until a constant minimum rate is reached if the rain is sufficiently prolonged. With dry antecedent conditions, the initial capacity rate will be greater and will decline faster. Because the use of a varying retention rate requires a complicated method of computation, it is often preferable to assume an average retention rate (sometimes referred to as infiltration index) with an estimate of initial loss being made if antecedent conditions are relatively dry.

The method of determining the period of rainfall excess, when an average retention rate is used, is a trial-and-error process in which a retention rate is assumed and subtracted from hourly rainfall increments determined as the average over the basin. Various retention rates are assumed until the total of the computed rainfall excess equals the measured storm runoff. An example of this procedure is given in table G-2. If the correct retention rate has not been assumed after two trials, a rainfall

Table G-2.–*Computation of rainfall excess*

Time, hours	Rainfall increment (basin average), inches	First trial: Assumed retention rate, inches per hour	First trial: Rainfall excess, inches	Second trial: Assumed retention rate, inches per hour	Second trial: Rainfall excess, inches	Third trial: Assumed retention rate, inches per hour	Third trial: Rainfall excess, inches
0							
1	0.05	0.25		0.15		0.17	
2	.06						
3	.16				0.01		
4	.19				.04		0.02
5	.30		0.05		.15		.13
6	.29		.04		.14		.12
7	.15		0		0		0
8	.76		.51		.61		.59
9	.56		.31		.41		.39
10	.40		.15		.25		.23
11	.56		.31		.41		.39
12	.24		0		.09		.07
13	.13				0		0
14	0				0		0
15	.19				.04		.02
16	.08						
17	.02	.25		.15		.17	
Total	4.14		1.37		2.15		1.96

Total rainfall, 4.14 inches; observed runoff, 2.0 inches; total retention in 17 hours, 2.1 inches. The average retention rate of 0.17 inch per hour assumed in the third trial gives the best agreement of computed rainfall excess with measured runoff.

excess-retention curve will facilitate the solution. In the example of table G-2, the curve could be drawn through the two points represented by the coordinates 0.25, 1.37, and 0.15, 2.15. The correct retention rate corresponding to a rainfall excess of 2.0 inches would then be taken from this curve.

The duration time of excess rainfall is that time during which rainfall increments exceed the average retention rate. In the third trial, table G-2, the duration time may be taken as either 8 or 9 hours, or as two periods, one of 2 or 3 hours, and the other of 5 hours (the final 0.02 inch of precipitation being disregarded), according to the characteristics of the hydrograph. A small amount of excess rain in a marginal period is frequently assumed to have occurred within only a small part of that period and may be neglected.

(5) Discussion of observed rainfall analyses procedures. -The above classic procedure of rainfall-runoff analysis is simple and satisfactory, given rainfall data such as used in the illustration and a relatively homogeneous watershed not exceeding a few hundred square miles in area. As stated earlier in section G-7(a): "A comparison of retention rates derived from several analyses leads to adoption of a rate for design flood computations." Experienced judgment is needed for such comparison with due reconsideration given to the characteristics of the data for each analysis and of the watershed. The selected rate is not necessarily the minimum rate computed. Mass curves of rainfall and isohyetal patterns should always be constructed as described in sections G-7(b)(2) and (3) to obtain good results from any rainfall-runoff analysis.

The importance in flood computations of good estimates of retention losses is evident. As the ratio of retention loss to flood causative precipitation increases, the relative effect of retention loss estimates on resulting flood magnitudes increases. Research studies directed towards improved understanding and evaluation of all processes contributing to retention losses are increasing yearly. Many complex functions are being tested by electronic computer programs to model such processes. However, the most practical approach for estimating natural watershed retention losses continues to be use of empirically derived relationships, preferably from records within the watershed.

Often, relationships as percentages of runoff to rainfall, runoff coefficients, are obtained by analyses and judicially used in flood studies. This approach may be practical in cases where basic data are meager.

The following extract from WMO Technical Note No. 98 [2] gives information of a method that may be used.

> "... For a particular river basin with records of streamflow and precipitation, a common procedure is to develop multiple variable rainfall-runoff correlations. Such correlations may be derived either graphically or analytically. They usually involve at least four variables, (i) depth of storm rainfall over the basin, (ii) surface runoff volume from the storm event, (iii) an index of moisture conditions in the basin

prior to the storm, and (iv) a seasonal factor. In some cases storm duration is included as a fifth variable. The methods of determining these factors from the observational records in a basin or a region and graphical and analytic procedures for multiple-variable correlation analyses are outlined in the WMO Guide to Hydrometeorological Practices, Annex A, WMO 168.TP.82."

A hydrologist making an inflow design flood study seldom finds rainfall-runoff records for the watershed above a particular damsite adequate to establish a good estimate of retention loss for the watershed. Recourse is then made to information of analyses for other watersheds having similar runoff characteristics. For example, hydrologists of the Soil Conservation Service, U.S. Department of Agriculture, have made extensive analyses of runoff from small experimental watersheds having individually homogeneous soil and cover characteristics but such characteristics differing between watersheds. A procedure was developed from these studies for estimating runoff from precipitation for any watershed for which certain soil and cover data are known; such soil and cover data are usually obtainable or subject to reasonable approximations [3].

The SCS procedure with modifications to fit specific purposes is described in appendix A of the Bureau of Reclamation publication "Design of Small Dams," second edition [31]. An abridgement of that description is given in the following subsection. (The descriptive items have been renumbered for convenience.)

(6) Method of estimating retention losses.-This method consists of the following steps:

(I). Classification of watershed soils into hydrologic groups A, B, C, or D, and estimation of percent of area1 extent of each in the watershed.

(II). Identification of land use characteristics dominant for each hydrologic group.

(III). The combination of a hydrologic group and its land use characteristics to ***give*** a ***hydrologic soil-cover complex*** identification for entering tables from which respective runoff curve numbers, CN, may be obtained.

(IV). Runoff values are obtained from a family of curves on a plot of rainfall versus runoff or by solution of the equation used to define the curves.

(V). Three antecedent moisture conditions, AMC, of a watershed are considered in relation to curve numbers; namely, AMC-I, AMC-II, AMC-III.

The mathematical procedure is given in this text with minimum definitions of the terms used in the procedure and without inclusion of a list of about 4,000 soil-type names and respective hydrologic group classifications compiled by the Soil Conservation Service. A full discussion of the procedure including the list of soil-type names is given in "Design of Small Dams" [31]. Information on the development of the runoff curves may be found in the SCS National Engineering Handbook [3].

Further explanation of each of the above steps follows.

(I) *Hydrologic* ***soil*** groups. -Four major soil groups are used. The soils are classified on the basis of intake of water at the end of long-duration storms occurring after prior wetting and opportunity for swelling, and without the protective effects of vegetation.

In the definitions that follow, the ***infiltration rate*** is the rate at which water enters the soil at the surface and which is controlled by surface condition, and the ***transmission rate*** is the rate at which the water moves in the soil and which is controlled by the soil horizons. The hydrologic soil groups, as defined by SCS soil scientists, are as follows:

Group A (low runoff potential).-Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

Group B. -Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

Group C.-Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

Group D (high runoff potential). -Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

(II). ***Land use and treatment classes.*** -These classes are used in the preparation of hydrologic soil-cover complexes (identified herein as item III), which in turn are used in estimating direct runoff. Types of land use and treatment are classified on a flood runoff-producing basis. The greater the ability of a given land use or treatment to increase total retention, the lower it is on a flood runoff-production scale. Land use or treatment types not described here may be classified by interpolation.

Crop rotations.-The sequence of crops on a watershed must be evaluated on the basis of its hydrologic effects. Rotations range from ***poor*** (or weak) to ***good*** (or strong) largely in proportion to the amount of dense vegetation in the rotation. ***Poor rotations*** are those in which a row crop or small grain is planted in the same field year after year. A poor rotation may combine row crops, small grains, or fallow, in various ways. ***Good rotations*** will contain alfalfa or other close-seeded legumes or grasses, to improve tilth and increase infiltration. For example, a 2-year rotation of wheat and fallow may be a good rotation for crop production where low annual rainfall is a limiting factor, but hydrologically it is a poor rotation.

Native pasture and range.-Three conditions are used, based on hydrologic considerations, not on forage production. ***Poor pasture or range*** is heavily grazed, has no mulch, or has plant cover on less than about 50 percent of the area. ***Fair pasture or range*** has between about 50 and 75 percent of the area with plant cover and is not heavily grazed. ***Good pasture or range*** has more than about 75 percent of the area with plant cover, and is lightly grazed.

Farm woodlots.-The classes are based on hydrologic factors, not on timber production. ***Poor*** *woodlots* are heavily grazed and regularly burned in a manner that destroys litter, small trees, and brush. ***Fair*** *woodlots* are grazed but not burned. These woodlots may have some litter, but usually these woods are not protected. ***Good*** *woodlots* ***are*** protected from grazing so that litter and shrubs cover the soil.

Forests. -See hydrologic soil-cover complex, item III following.

Straight-row farming.-This class includes up-and-down and cross-slope farming in straight rows. In areas of 1 or 2 percent slope, cross-slope farming in straight rows is almost the same as contour farming. Where the proportion of cross-slope farming is believed to be significant, it may be classed halfway between straight-row and contour farming in the table G-3(A).

Co ntouring. -Contour furrows used with small grains and legumes are made while planting, are generally small, and tend to disappear due to climatic action. Contour furrows, and beds on the contour, as used with row crops are generally large. They may be made in planting and later reduced in size by cultivation, or they may be insignificant after planting and become large from cultivation. Average conditions are used in table G-3(A).

Surface runoff reductions due to contour farming are greater as land slopes decrease. The curve numbers for contouring shown in table G-3(A) were obtained using data from experimental watersheds having slopes of 3 to 8 percent.

Contour furrows in pasture or range land are usually of the permanent type, Their dimensions and spacing generally vary with climate and topography. Table G-3(A) considers average conditions in the Great Plains.

Terracing. -Terraces may be graded, open-end level, or closed-end level. The effects of graded and open-end level terraces are considered in table G-3(A), and the effects of both contouring and the grass waterway outlets are included.

When considering land use and treatment classes for hydrologic soil groups within a large watershed, the above definitions should be applied broadly, estimating percentage of land use in each group, assigning proper CN and computing a weighed CN for each particular soil group.

(III) ***Hydrologic soil-cover complexes.*** -Combinations of hydrologic soil groups and land use and treatment classes into hydrologic soil-cover complexes with respective curve numbers are given in table G-3(A), (B), (C). The numbers show the relative value of the complexes as direct runoff-producers. The higher the number, the greater the amount of direct runoff to be expected from a storm. Table G-3(A) is applicable to farm lands and related areas, and table G-3(B) is applicable to forested watersheds. A more detailed method of estimating curve numbers for heavy forested land in humid regions is given in appendix A of "Design of Small Dams," second edition [31].

Table G-3(C) is applicable for forest-range areas in the Western United States. Descriptions of the types of cover listed are as follows:

Herbaceous. -Grass-weed-brush mixtures with brush the minor element.

Oak-Aspen. -Mountain brush mixtures of oak, aspen, mountain mahogany, bitter brush, maple, and other brush.

Juniper-Grass. -Juniper or pinon with an understory of grass.

Sage-Grass. -Sage with an understory of grass.

(IV) ***Rainfall-runoff curves for estimating direct runoff amounts.***-The curves of figure G-2 are obtained using the equation:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (1)$$

where :

Q = direct runoff, in inches
P = storm rainfall, in inches, and
S = maximum potential difference between P and Q, in inches, at time of storm's beginning.

There is some loss of rainfall before runoff begins due principally to interception, infiltration, and surface storage, so provision for an initial abstraction I_a is included in the runoff equation (see diagram on figure G-2). With the condition that I_a cannot be greater than P, an empirical relationship of $I_a = 0.2S$ was adopted in developing the equation, obtaining the empirical relationship of I_a and S from data from watersheds in various parts of the country.

For convenience in interpolation, the curves of figure G-2 are numbered from 100 to zero. The numbers are related to S as follows:

$$\text{Curve number, CN} = \frac{1{,}000}{10 + S} \qquad (2)$$

The procedure recommended in this text for estimating incremental rainfall excesses from design storm rainfall using appropriate CN and figure G-2 or the runoff equation is given in section G-19. In the process of hydrograph analyses, preliminary estimates of curve numbers for a watershed can be quickly obtained from figure G-2 by using total storm rainfall and runoff amounts. However, such preliminary estimates have to be revised by trial computations of rainfall excesses using the procedure given later in section G-19.

(V) ***Antecedent moisture conditions.*** -The following generalized criteria define three antecedent moisture conditions of watersheds used in the development of the runoff curve numbers.

AMC-I. -A condition of watershed soils where the soils are dry but not to the

Table *G-3.-Hydrologic soil-cover complexes and respective curve numbers (CN)*

(A) RUNOFF CURVE NUMBERS (CN) FOR FARMLANDS AND RELATED AREAS

[FOR WATERSHED CONDITION AMC-II]

Land use or cover	Treatment or practice	Hydrologic condition for infiltrating	Hydrologic soil group A	B	C	D
Fallow	SR	....	77	86	91	94
Row crops	SR	Poor	72	81	88	91
	SR	Good	67	78	85	89
	C	Poor	70	79	84	88
	C	Good	65	75	82	86
	C & T	Poor	66	74	80	82
	C & T	Good	62	71	78	81
Small grain	SR	Poor	65	76	84	88
	SR	Good	63	75	83	87
	C	Poor	63	74	82	85
	C	Good	61	73	81	84
	C & T	Poor	61	72	79	82
	C & T	Good	59	70	78	81
Close-seeded legumes [1] or rotation meadow.	SR	Poor	66	77	85	89
	SR	Good	58	72	81	85
	C	Poor	64	75	83	85
	C	Good	55	69	78	83
	C & T	Poor	63	73	80	83
	C & T	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	C	Poor	47	67	81	88
	C	Fair	25	59	75	83
	C	Good	6	35	70	79
Meadow (permanent).		...do...	30	58	71	78
Woods (farm woodlots).	1	Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads	...		59	74	82	86
Roads (dirt)[2] (hard surface).[2]		....	72	82	87	89
			74	84	90	92

[1] Close-drilled or broadcast.
[2] Including right-of-way.
SR = Straight row.
C-Contoured.
T = Terraced.
C&T = Contoured and terraced.

(U.S. Soil Conservation Service.)

(B) RUNOFF CURVE NUMBERS (CN) FOR FORESTED WATERSHEDS

COMMERCIAL OR NATIONAL FOREST, FOR WATERSHED CONDITION AMC-II

Hydrologic condition class	Hydrologic soil group A	B	C	D
I. Poorest	56	75	86	91
II. Poor	46	68	78	84
III. Medium	36	60	70	76
IV. Good	26	52	62	69
V. Best	15	44	54	61

(C) RUNOFF CURVE NUMBERS (CN) FOR FOREST RANGE AREAS IN WESTERN UNITED STATES (AMC-II)

Cover	Condition	Soil groups A	B	C	D
Herbaceous	Poor	---	78	85	92
	Fair	---	68	81	88
	Good	---	59	71	84
Sage-Grass	Poor	---	64	78	---
	Fair	---	46	67	---
	Good	---	35	46	---
Oak-Aspen	Poor	---	63	71	---
	Fair	---	40	54	---
	Good	---	30	40	---
Juniper-Grass	Poor	---	73	84	---
	Fair	---	54	70	---
	Good	---	40	59	---

wilting point, and when satisfactory plowing or cultivation takes place. (This condition is not considered applicable to the design flood computation methods presented in this text.)

AMC-II. -The average case for ***annual jloods,*** that is, an average of the conditions which have preceded the occurrence of the maximum annual flood on numerous watersheds.

AMC-III. -Heavy rainfall has occurred during the 5 days previous to the given storm and the soil is nearly saturated.

Curve numbers in table G-3(A), (B), (C) for hydrologic soil-cover complexes all relate to AMC-II. Table G-4(A) lists curve numbers for AMC-II with respective S values (column (4)) and $0.2S$ values (column (5)) which may be used to solve the runoff equation on figure

G-2. Curve numbers for AMC-I and AMC-III respective to the CN for AMC-II in column (1) are listed in columns (2) and (3). This information is useful for estimating retention losses. If data are available for analyzing observed storms and resulting runoff, an estimate of antecedent moisture condition of a watershed may be made from table G-4(B).

G-8. Analyses of Streamflow Data. —Streamflow data at a given location may consist of: (1) a continuous hydrograph of discharges obtained from waterstage recording mechanisms; (2) mean (average) daily discharges computed from waterstage recorders or from once or twice daily observed water stages; or, in some instances (3) peak discharges computed from flood marks or crest stage gages. U.S. Geological Survey publications should be consulted for information about collection and processing these data for publication. However, one should be aware that U.S.G.S. publications give for each published station record an estimate of the degree of accuracy of field data and computed results for that record as follows:

> ***"Excellent*** means that about 95 percent of the daily discharges are within 5 percent; good, within 10 percent; and *fair,* within 15 percent. ***Poor*** means that daily discharges have less than *fair* accuracy."

Objectives of streamflow data analyses for inflow design flood computations are:

(1) Determinations of watershed retention losses (previously discussed).

(2) Determination of characteristic watershed response to precipitation; this is usually accomplished by deriving a unit hydrograph for the watershed. (Complex

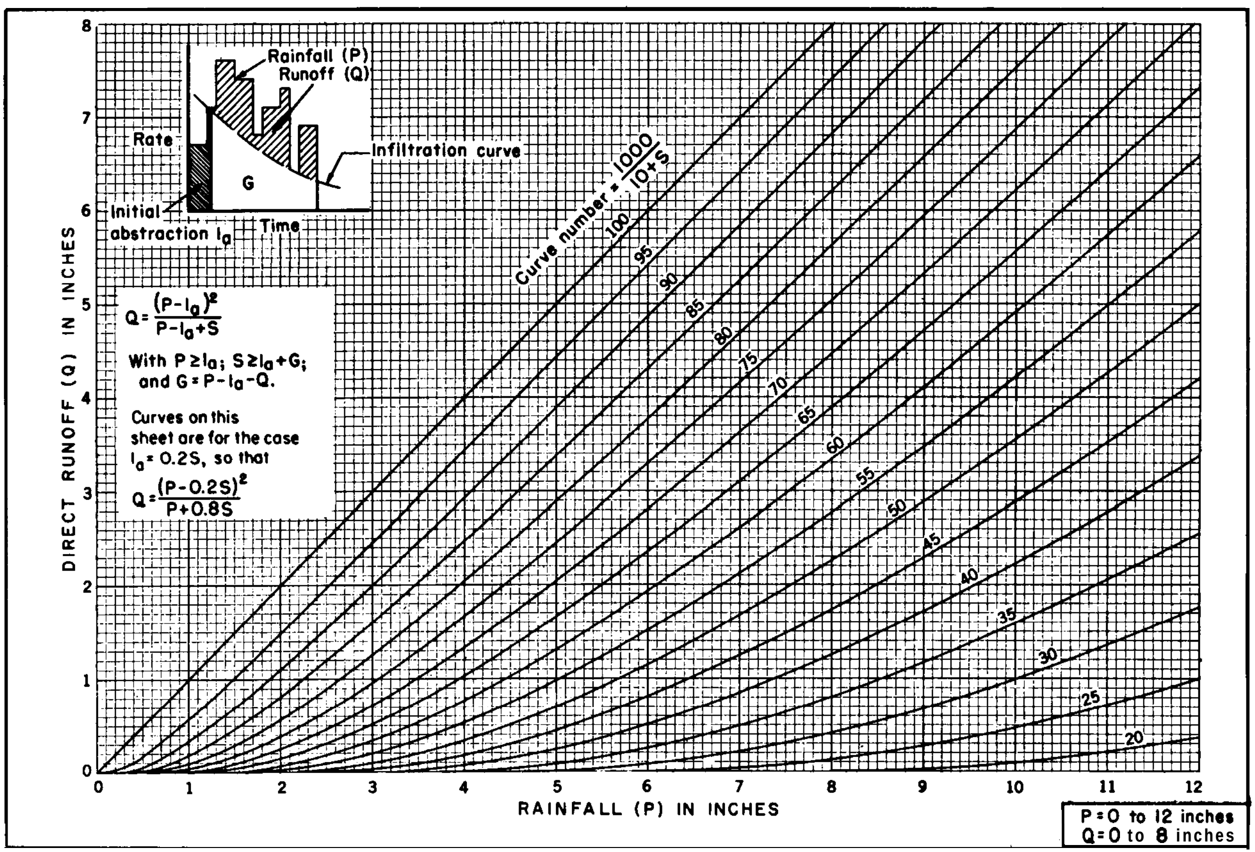


Figure G-2. Rainfall-runoff curves-solution of runoff equation, $Q = \frac{(P-0.2S)^2}{P+0.8S}$ (sheet 1 of 2) (U.S. Soil Conservation Service).-288-D-3178(1/2)

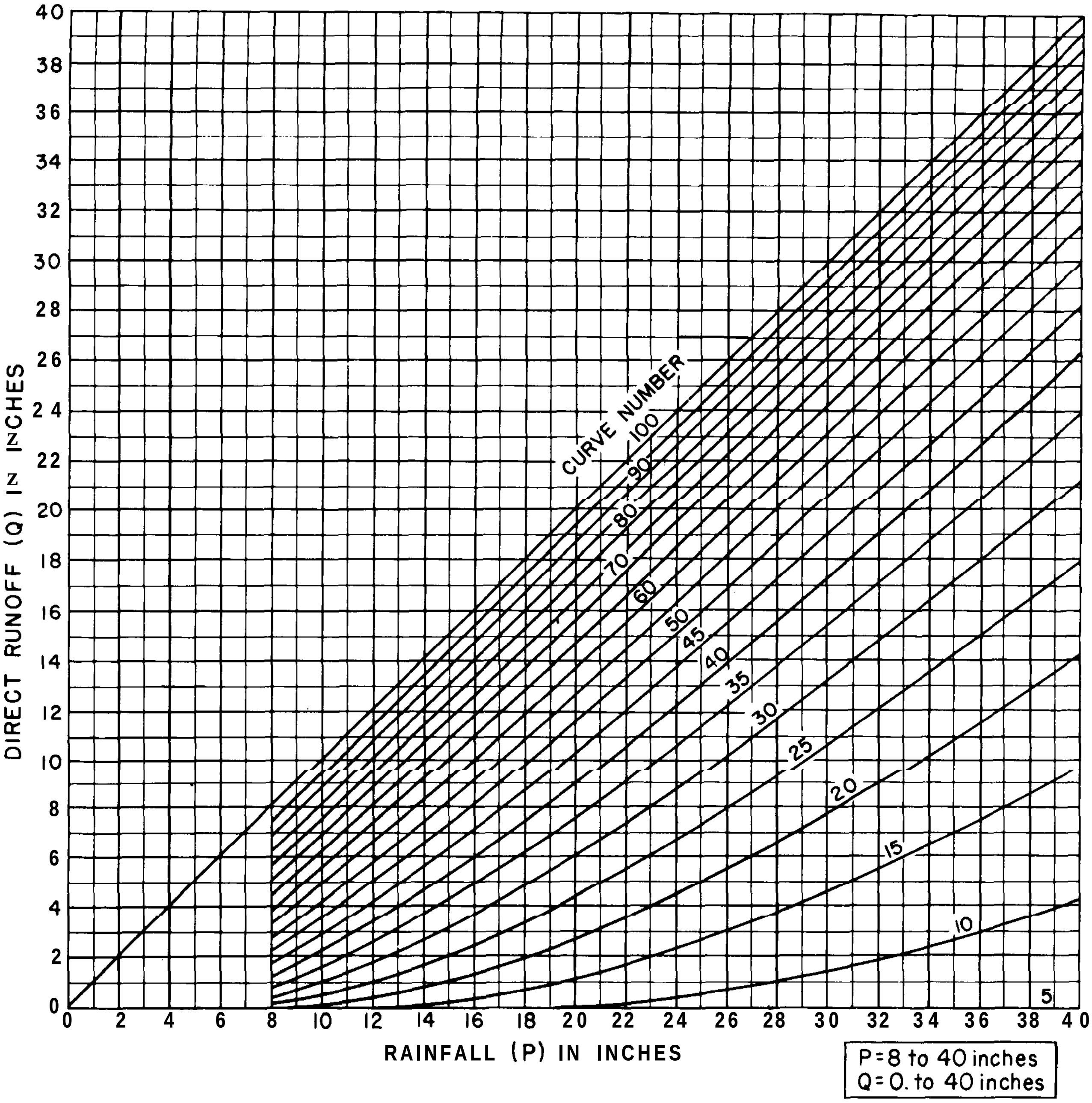


Figure G-2. Rainfall-runoff curves-solution of runoff equation, $Q = \frac{(P - 0.2S)^2}{P + 0.8S}$ (sheet 2 of 2) (U.S. Soil Conservation Service).-288-D-3178(2/2) *(Note:* Curve designated by number is *below* number.)

computer-programed watershed runoff models may use other means of estimating time distribution of runoff.)

Continuous hydrographs can provide for estimates of retention loss variations with time, with accumulative loss, or with accumulative precipitation. Mean daily discharges can provide ratio estimates of total retention loss to total storm precipitation.

Continuous hydrographs are essential to unit hydrograph derivations from recorded streamflow data. When mean daily discharges only are available, a continuous hydrograph is sketched for making unit hydrograph

Table G-4.-Curve ***numbers, constants, and seasonal rainfall limits***

(A) CURVE NUMBERS (CN) AND CONSTANTS FOR THE CASE $I_a = 0.2S$

1	2	3	4	5	1	2	3	4	5
CN for condition II	CN for conditions I	III	*S* values* *inches*	Curve * starts where *P* = *inches*	CN for condition II	CN for conditions I	III	*S* values* *inches*	Curve* starts where *P* = *inches*
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	.101	.02	59	39	77	6.95	1.39
98	94	99	.204	.04	58	38	76	7.24	1.45
97	91	99	.309	.06	57	37	75	7.54	1.51
96	89	99	.417	.08	56	36	75	7.86	1.57
95	87	98	.526	.11	55	35	74	8.18	1.64
94	85	98	.638	.13	54	34	73	8.52	1.70
93	83	98	.753	.15	53	33	72	8.87	1.77
92	81	97	.870	.17	52	32	71	9.23	1.85
91	80	97	.989	.20	51	31	70	9.61	1.92
90	78	96	1.11	.22	50	31	70	10.0	2.00
89	76	96	1.24	.25	49	30	69	10.4	2.08
88	75	95	1.36	.27	48	29	68	10.8	2.16
87	73	95	1.49	.30	47	28	67	11.3	2.26
86	72	94	1.63	.33	46	27	66	11.7	2.34
85	70	94	1.76	.35	45	26	65	12.2	2.44
84	68	93	1.90	.38	44	25	64	12.7	2.54
83	67	93	2.05	.41	43	25	63	13.2	2.64
82	66	92	2.20	.44	42	24	62	13.8	2.16
81	64	92	2.34	.47	41	23	61	14.4	2.88
80	63	91	2.50	.50	40	22	60	15.0	3.00
79	62	91	2.66	.53	39	21	59	15.6	3.12
78	60	90	2.82	.56	38	21	58	16.3	3.26
77	59	89	2.99	.60	37	20	57	17.0	3.40
76	58	89	3.16	.63	36	19	56	17.8	3.56
75	57	88	3.33	.67	35	18	55	18.6	3.12
74	55	88	3.51	.70	34	18	54	19.4	3.88
73	54	87	3.70	.74	33	17	53	20.3	4.06
72	53	86	3.89	.78	32	16	52	21.2	4.24
71	52	86	4.08	.82	31	16	51	22.2	4.44
70	51	85	4.28	.86	30	15	50	23.3	4.66
69	50	84	4.49	.90					
68	48	84	4.70	.94	25	12	43	30.0	6.00
67	47	83	4.92	.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.00
63	43	80	5.87	1.17	0	0	0	infinity	infinity
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

*For CN in column 1 (value = $0.2S$)

(B) SEASONAL RAINFALL LIMITS FOR AMC

AMC group	Total 5day antecedent rainfall, inches: Dormant season	Growing season
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	Over 1.1	Over 2.1

estimates; the chance of introducing considerable error is obvious. Discussions which follow assume continuous hydrographs obtained from continuous recording waterstage records converted to discharges expressed as cubic feet per second (c.f.s.), the degree of accuracy of the records being *excellent* or *good*.

(a) *Unit Hydrograph (Unitgraph) Principles.* -The 1970 USCOLD report [1] states: "In general the unit hydrograph method, in conjunction with the estimated probable maximum precipitation, is used in estimating probable maximum floods . . . ." The unit hydrograph principle was originally developed by Sherman [4] in 1932. Although numerous refinements have been added by other investigators, the basic principles as presented by Sherman remain the same. These principles as now applied are given and illustrated on figure G-3.

Sherman's definition of unit hydrograph did not imply a specific volume of runoff, and the term was applied to the observed hydrograph as well as to a hydrograph of l-inch volume computed from the observed graph. In present practice, observed hydrographs are usually identified as such, and the term *unitgraph* refers either to the l-inch volume unitgraph derived from a specific observed hydrograph or to a l-inch volume unitgraph representative of the watershed and used to compute synthetic floods from rainfall excess over the watershed. Random variations in rainfall rate in respect to time and area have a great effect on the shape of the runoff hydrograph. To minimize the effect of the time variations in rainfall rate, it has been found that the rainfall excess duration time of a basin unitgraph should not exceed one-fourth the basin lag-time as defined in section G-8(e), and the shorter the rainfall excess period with respect to lag-time, the better the unitgraph results are likely to be.

The term *unit hydrograph,* or *unitgraph,* as used in this text always means l-inch volume of runoff; the volume notation is seldom included. The rainfall excess unit duration time is always given for a watershed representative unitgraph.

Natural flood hydrographs at a given stream gage are assumed to give integrated results of all interdependent effects on runoff such as watershed precipitation, retention losses, and routing effects of watershed vegetative cover and channel systems. A unit hydrograph which has been derived from recorded floods at a given stream location, and which will give close reconstruction of recorded flood hydrographs from recorded respective precipitation events as affected by retention losses, is considered representative of that particular watershed and also considered representative of other watersheds having similar runoff characteristics.

On this basis, synthetic unit hydrographs for ungaged basins are derived by judging comparative watershed characteristics and adjusting "representative" unit hydrographs to tit the size and lag-time of the ungaged watershed. Mathematical watershed runoff models are currently being developed by computer integration of meteorological, hydrological, and physiographical factors. Some hydrologists prefer to use these models rather than a unitgraph. However, each model includes constants related to watershed characteristics that must be empirically determined by trial analyses of recorded flows. As in the application of synthetic unitgraphs, transference of a mathematical model from a gaged to an ungaged watershed also requires experienced judgment of the effect from variations in watershed characteristics.

The use of the unit hydrograph is limited in the following ways:

(1) The principle of the unit hydrograph is applicable to basins of any size. However, it is desirable in the derivation of unitgraphs to use storms that are well distributed over the entire basin and produce runoff nearly concurrently from all parts of it. Such storms rarely occur over large areas. The extent of the basin for which a unitgraph may be derived from observed data is therefore limited in each case to the area1 extent of rainfalls that have been observed.

(2) Hydrographs containing more than small amounts of snowmelt runoff are

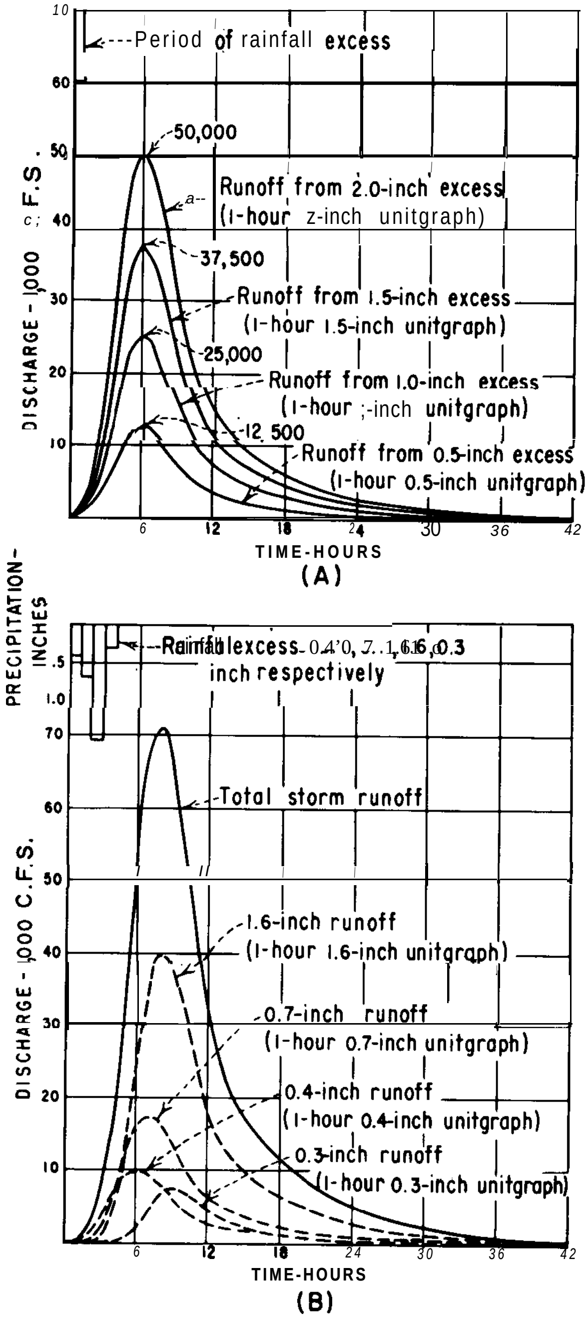


Figure **G-3. Unit hydrograph principles (sheet 1 of 2).-288-D-3179(1/2)**

Definitions :

Unitgraph - A hydrograph of direct runoff* at a given point that will result from an isolated event of rainfall excess occurring within a unit of time and spread in an average pattern over the contributing drainage area. Identified by by the unit time and volume of the excess rainfall, that is 1-hour 1-inch unitgraph

Rainfall excess-That portion of rainfall that enters a stream channel as direct runoff and produces the runoff hydrograph at the measuring point, base flow included

Basic Assumptions:

(1) The effects of all physical characteristics of a given drainage basin are reflected in the shape of the direct runoff hydrograph for that basin.

(2) At a given point on a stream, discharge ordinates of different unitgraphs of the same unit time of rainfall excess are mutually proportional to respective volumes. See (A) at left.

(3) A hydrograph of storm discharge that would result from a series of bursts of excess rain or from continuous excess rain of variable intensity may be constructed from a series of over-lapping unitgraphs each resulting from a single increment of excess rain of unit duration. See (B) at left.

Practical Application:

For a given runoff contributing area, a unitgraph representing exactly one inch of runoff (rainfall excess) for a selected unit time interval is computed. Increments of rainfall excess for the same unit time interval are determined for a storm. A total hydrograph of direct runoff from the storm is then computed using assumptions (2) and (3) above. See graph (B) at left.

**Note*: Direct runoff is defined in section G-8.

Figure **G-3. Unit hydrograph principles (sheet 2 of 2).-288-D-3179(2/2)**

usually unsuitable sources of unitgraphs.

(3) The observed hydrograph of storm discharge is a smooth curve, because it is actually made up of unitgraphs produced by infinitely short increments of excess rain. It cannot be reproduced perfectly by the use of rainfall increments of measurable duration. When unitgraphs are combined they produce a regular undulation similar to a harmonic with a period equal to that of the rainfall increments, superimposed upon the fundamental hydrograph. Another obstacle to exact reproduction is the fact that the successive rainfall increments do not have the same isohyetal pattern and a single form of unitgraph is not strictly applicable to all of them. These phenomena contradict, to a certain extent, the third basic assumption of the unit hydrograph (fig. G-3). They can be disregarded in the synthesis of hydrographs, but frequently cause difficulty in the use of arithmetical procedures for analyzing them.

An engineer attempting unitgraph analyses or researching literature regarding unitgraphs soon becomes aware that the three basic assumptions listed on figure G-3 are not theoretically supportable. However, experience has shown that this does not negate use of the method as a practical tool.

(b) Selection ***of Hydrographs to Analyze.*** -The statement made in section G-7(b)(2) bears enough importance to unit hydrograph studies to be repeated: "Those floods having a combination of large volume, uniform intensity, isolated periods of rainfall, and uniform area1 distribution of rainfall, should be chosen for further study."

Streamflow discharge records and basin precipitation records must be examined jointly for selection of hydrographs to analyze for unit hydrograph derivation. Isolated floods likely to merit investigation are easily identified by a rapid rise to a single peak and a smooth curve recession to low flow. Preferably, volumes of selected hydrographs should be equivalent to about one-half inch or more of runoff from the watershed. Preliminary estimates of hydrograph volumes can be made by summing the daily mean daily discharges in c.f.s.-days for the flood period. A sum of c.f.s.-days equal in number to 15 times the drainage area size in square miles is equivalent to 0.56 inch of runoff from the area. A useful equation for converting discharge volume to equivalent inches of rainfall is:

$$P_e = \frac{V}{26.89\ \boldsymbol{A}} \qquad (3)$$

where :

P_e = rainfall excesses, inches, average depth over basin,
$\boldsymbol{V}$ = volume of runoff, c.f.s.-days, and
$\boldsymbol{A}$ = drainage area in square miles.

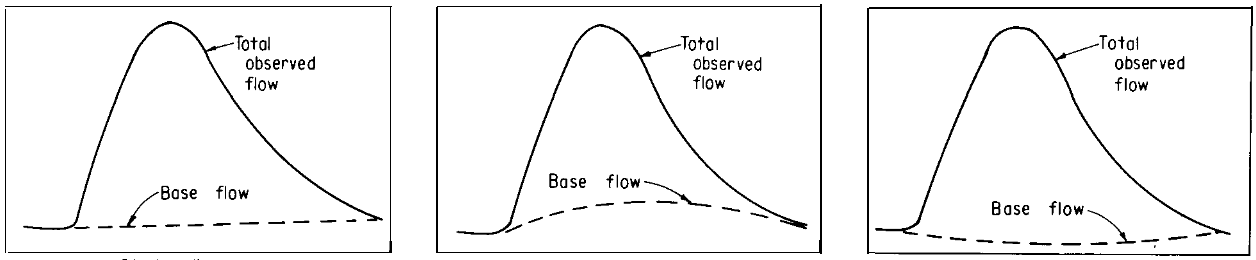
Hydrographs with volume sum of c.f.s.-days less than five times the drainage area size, 0.19 inch runoff, are almost always unsuitable for unit hydrograph analyses.

After noting dates of all flood hydrographs that satisfy preliminary volume criteria, rainfall records for respective flood events are examined for conformance with the ideal corn bination of short duration, uniform intensity, and uniform area1 distribution of rainfall over the entire watershed. Those storms approaching nearest to the ***ideal*** criteria are analyzed as previously described in section G-7. If enough rainfall data are not available to do a good storm analysis for some of the isolated flood events having satisfactory volumes, the flood hydrographs may be analyzed for unitgraph comparisons as discussed in section G-8(e) by assuming that the beginning of rainfall excess coincides with the beginning of a sharp rise of the hydrograph, provided there is enough information available to reasonably assume the rainfall covered the total watershed.

Unit hydrograph derivations are difficult in regions where isolated flood events are rare and, instead, flood hydrographs commonly have two or more peaks caused by storms which usually persist for several days. Procedures for analyzing multipeaked flood hydrographs cannot be included in this text but can be found in publications listed in the bibliography, section G-32.

(c) ***Hydrograph Analyses-Base Flow Separation.*** -The purpose of flood hydrograph analyses is to determine for a watershed the time-distribution of the runoff which ***quickly*** reaches a particular point on a stream when rain falls on the watershed. The portion of the rainfall that infiltrates through the soil mantle into the ground-water supply will not reach the stream until days or months after the storm. Ground-water supply to a stream, base flow, may be a large proportion of that stream's total yearly discharge, but the base flow volume during an isolated flood is small in ratio to the total flood volume. However, base flow must be estimated and subtracted from the total discharge hydrograph in order to determine the direct runoff hydrograph. The schematic graphs on figure G-4 show three common approaches for estimating base flow discharges [6]. Base flow estimates are usually made graphically after plotting total flood discharges on linear or semilogarithmic graph paper.

(d) ***Hydrograph Analysis of Direct Runoff-Need for Synthetic Unit*** *Hydrographs.*—It is often necessary to use synthetic unit hydrographs for inflow design flood estimates and for obtaining indices for



Type A - Straight line base flow between beginning and end of direct runoff.

Type B. - Base flow peaked at some time shortly after peok direct runoff.

Type C - Base flow depletes for some time after beginning of direct runoff. then straight line to end of direct runoff.

Figure G-4. Three common approaches for estimating base flow discharges.-288-D-3180

synthetic unitgraph estimates. Suitable records of observed discharge are seldom available at the exact stream point for which a unitgraph is needed; in this discussion, at a proposed damsite. Even if such records are available, often the proposed reservoir will be large enough to inundate several miles of stream channels above the damsite, thus causing watershed runoff to enter a full reservoir more quickly than the respective runoff would arrive at the damsite through natural channels. Therefore, a unitgraph usable for estimating floods at the damsite under natural conditions must be properly adjusted to be usable to estimate inflow to a full reservoir.

The shape of a representative watershed unitgraph can be obtained by a proper average of several unitgraphs computed from observed discharge records at a gage, or occasionally by a single unitgraph from an intense rainburst, well centered and distributed. If there are available several isolated direct runoff hydrographs suitable for simple conversion to l-inch volume unitgraphs by multiplying the hydrograph discharge ordinates by the ratio of 1 inch to the direct runoff volume in inches, only those unitgraphs having *equal duration times of rainfall* excess can be directly averaged. Most likely, rainfall excess duration time will be different for each l-inch unitgraph. A general similarity in shape of the unitgraphs will be recognized, but they may show pronounced differences in their relative steepness and time of peak discharge.

It is possible to eliminate these differences to a large degree by adjusting the ordinates and abscissae of each unitgraph in proportion to some index related to both the duration of rainfall excess and to the average time interval between the rainfall excess and some representative point near the center of the respective runoff unitgraph. The index used for this purpose is known as *lag-time* which, for procedures to be described in this text is defined as: *The time interval between the mid-time **of** rainfall excess duration and the time **of** occurrence **of** one-half the volume **of** the hydrograph.*

Lag-time may be used as later described to convert each unitgraph into a dimensionless-graph form and the dimensionless-graphs can then be averaged. *(Note:* In this text, the hyphenated term dimensionless-graph refers to the particular form used within the Bureau of Reclamation. The two words, dimensionless graph(s) refer in general to graphs expressing time versus discharge as ratios.) Lag-time is also an index of time-of-concentration (time interval between end of rainfall excess and point of inflection on recession limb of direct runoff hydrograph) of runoff for a watershed, and can be correlated with certain measurable physical features common to all watersheds such as area, stream channel length, and slope. Correlations between lag-times derived from recorded floods and respective watershed features, in the form of *lag-time curves,* provide means for estimating lag-time at any desired ungaged stream point on the basis of watershed features above that point.

A synthetic unitgraph may be estimated for a watershed area, given a representative lag-time curve and dimensionless graph *based*

on the same lag-time definition. Hydrology textbooks and published professional papers give many different definitions of lag-time, several different dimensionless graph forms, and many variations in correlations of basin features with lag-times.

Investigators are continually striving to improve estimates of time-distribution of runoff from rainfall. Only the lag-time versus basin factor relationships and related dimensionless-graph form used most often in Bureau of Reclamation inflow design flood studies will be described in detail in this text.

(e) Hydrograp h Analysis of Direct Runoff-Dimensionless-Graph Computations and Lag-Time Estimates.-A direct runoff hydrograph may be converted to dimensionless-graph form using a function of lag-time. A lag-time for the flood event may also be computed if sufficient rainfall data are available to define the duration time of rainfall excess.

All hydrographs may be converted to dimensionless-graph form by the mathematical procedure to be described, but experienced judgment must be employed to select those that are suitable for further considerations. Lag-time is the basic index; however, a related value known as ***lag-plus-semiduration*** is the actual index used for dimensionless-graph computations. Lag-plus-semiduration is obtained by adding one-half of the duration time of rainfall excess to the lag-time. This addition provides a means of obtaining comparable dimensionless-graphs for unitgraphs of different rainfall excess durations, as, by definition, a unitgraph starts at the beginning of rainfall excess and the measurement of lag-time starts at the mid-time of rainfall excess duration. Lag-plus-semiduration is the elapsed time between the beginning of the major rise of the hydrograph and the point of 50 percent of runoff volume. Thus, in the analysis of an observed direct runoff hydrograph for which rainfall excess can be established and begins concurrently with the start of the major rise of the hydrograph, lag-time is computed as lag-plus-semiduration minus one-half of the rainfall excess duration.

When analyzing direct runoff hydrographs by the dimensionless-graph method, it is not necessary to first convert each hydrograph to a volume equivalent to 1 inch of runoff. In practice, selected observed direct runoff hydrographs are converted to dimensionless-graph form as follows. The elapsed time from the beginning of a hydrograph to the point of 50 percent volume is computed; this is the lag-plus-semiduration value for the hydrograph. The abscissae of the hydrograph is converted from actual hours into percent of the lag-plus-semiduration value. Each ordinate of the hydrograph, cubic feet per second (or c.f.s.), is multiplied by the lag-plus-semiduration value, and the product is divided by the total direct runoff hydrograph volume expressed as c.f.s.-days. The converted ordinates and abscissae are dimensionless and may be plotted for comparisons and averaging with other dimensionless-graphs similarly obtained.

The above method of eliminating the effect of rainfall excess duration time by lag-time relations is considered satisfactory in the comparison and averaging of a group of dimensionless-graphs when the maximum value of the rainfall excess duration, expressed in percent of lag-time, does not exceed about four times the minimum value found in the same group, expressed in the same way. When the duration of rainfall excess cannot be determined with reasonable accuracy, lag-plus-semiduration can frequently be measured directly from the start of rise of the direct runoff hydrograph. Thus, dimensionless-graphs may be obtained from recorded floods from watersheds where streamflows are gaged but precipitation data are meager or not collected. Use of this procedure increases the data available for synthetic unitgraph derivations.

To determine the average shape of a group of dimensionless-graphs, first determine the average of the peak ordinates and the average of the corresponding abscissae. These two values become the coordinates of the peak of the average graph. Points on the lower portions of the accession and recession are averaged ***on the horizontal,*** that is, an ordinate is assumed and the average of the abscissae corresponding to that ordinate is determined. If the plotting is on semilog paper and the recessions end in tangents, only two averages are needed to define the mean tangent. The ***shoulder*** portions of the mean graph are best sketched in by visual inspection. Arithmetical averages should not be used near the peak unless the ordinates of the points averaged are taken at a fixed percentage of the respective peak ordinates, or unless the individual peaks as plotted are at virtually the same height.

(1) ***Procedures.-A*** method of complete hydrograph analyses for obtaining a dimensionless-graph and lag-time estimate from a selected isolated flood event is given as a step-by-step outline with pertinent comments, graphically illustrated on figure G-5, and supplemented by a table of computation, table G-5. For illustrative purposes, computations included in table G-5 are more detailed than necessary in practice. An outline of procedures follows:

(a) Plot recorded hydrograph on Cartesian coordinate paper and on semilog paper:

① on figure G-5(A), and

① on figure G-5(B)

Hypothetical total flood discharges are listed in table G-5. A hyetograph of average hourly basin rainfall, if obtainable, plotted as shown on the same coordinate paper with the total flood hydrograph, is helpful for determining the coincidence of beginning time of rainfall excess and direct runoff. The plot on semilog paper helps in making base flow estimates.

(b) Estimate base flow, ② on figure G-5(A) and (B), by trial and error. Subtract base flow from recorded

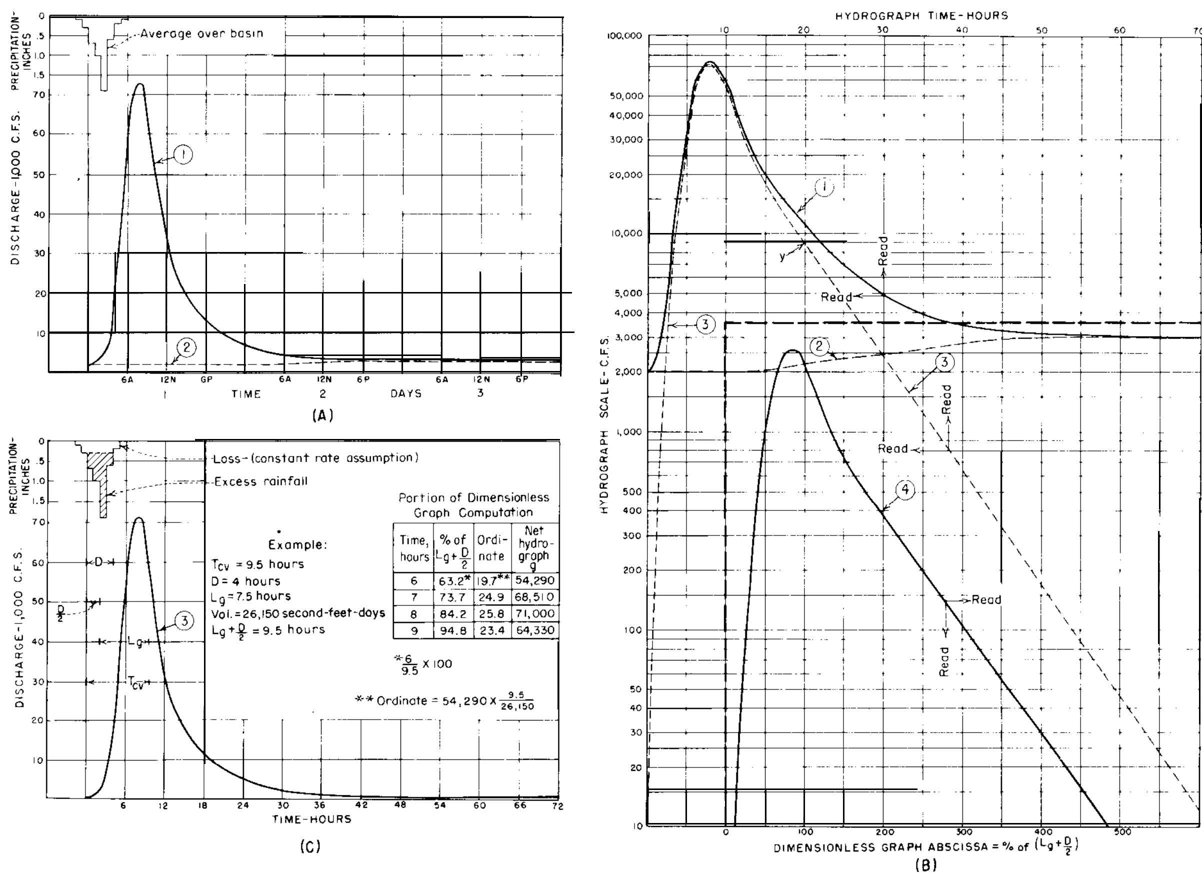


Figure G-S. **Hydrograph** analysis.-288-D-2457.

hydrograph and plot net hydrograph, 3 on figure G-5(B). If the base flow has been estimated correctly, the descending limb of hydrograph 3 on figure G-5(B) will be a straight line (exponential recession) [7]. (3 = 1 minus 2 on figure G-5(B).)

Large base flow discharges were used in this example to improve graphical illustration.

(c) Compute volume of net hydrograph 3 as follows (method 1, table G-5):

1. Add average hourly discharges (in c.f.s.-hours) to a point such as y on the exponential recession, 3 on figure G-5(B).
2. Compute hourly recession constant, k_{hr}, from two points on exponential recession line by use of following equation:

$$k_{hr} = \sqrt[t]{\frac{q_t}{q_o}} \qquad (4)$$

where :

q_o = discharge at first point,
q_t = discharge at second point, and
t = time interval, in hours, between points 1 and 2.

3. Storage, or volume after point y (in c.f.s.-hours) equals:

$$\frac{-q_y}{\log_e k_{hr}} \qquad (5)$$

where:

q_y = discharge in c.f.s. at point y, and
$\log_e k_{hr} = 2.3026 (\log_{10} k_{hr})$.

4. Total volume is sum of volume to y plus volume after y.

(d) For comparison with rainfall data, convert volume of 3 to inches of runoff:

Inches of runoff =

$$\frac{\text{volume in c.f.s.-hours}}{(\text{area in sq. mi.}) \times 645.3^*} \qquad (6)$$

*(1 inch P_{e1}/sq. mi. = 26.888 c.f.s.-days: (26.888)(24) = 645.3 c.f.s.-hrs.)

(e) Analyze rainfall data, if available; determine period D of rainfall excess.

(f) Compute time of occurrence of one-half volume of hydrograph 3, figure G-5(C). The time to center of volume, T_{cv}, equals time from beginning of rise of net hydrograph to time one-half volume has passed measuring point.

(g) Find lag, Lg, time in hours from midpoint of excess rainfall period to time of occurrence of one-half volume.

(h) Compute dimensionless graph as follows and plot on semilog paper, 4 on figure G-5(B).

1. Abscissa-hours from beginning of excess rain expressed as percent of $(Lg + D/2)$.
2. Ordinates-discharge in c.f.s. of 3 (at respective abscissa) multiplied by $(Lg + D/2)$, all divided by net hydrograph volume expressed as c.f.s.-days = $\left(\frac{\text{c.f.s.-hours}}{24}\right)$.

(2) Lag-time curves.—Lag-time is a key function for estimating synthetic unitgraphs. An average lag-time value for a watershed is obtained by averaging the results of several good analyses of stream gage records. Such average values for different gages on a stream and/or different streams of similar runoff characteristics can be correlated empirically with certain measurable watershed features. The correlation equation most often used in the Bureau of Reclamation is of the form:

$$\text{Lag-time, hours} = C\left[\frac{LL_{ca}}{\sqrt{S}}\right]^x \qquad (7)$$

where: C and x are constants,

Table G-5 .-*Hydrograph analysis computations*

BASIC DATA:

Name of streamgage = (Hypothetical for this table) — A, drainage area, sq. mi. = 319
Date of flood = (Assume May 1-3,1970) — Volume, c.f.s.-days, net = 26,150
Time, beginning of direct runoff-net hydrograph = 12:00 p.m., 30 April
Time, point of 50 percent volume of net hydrograph, T_{cv} = 9:30 a.m., 1 May

Lag-plus-semiduration, hrs.; $\left(Lg + \frac{D}{2}\right) = 9.5$

Duration of rainfall excess, D, hrs. = 4 (obtained by storm analysis)

Lag-time, hrs. = $\left(Lg + \frac{D}{2}\right) - \left(\frac{D}{2}\right) = 7.5$

Q = instantaneous discharge, c.f.s.

T		Hydrograph			Net v ume		Dimensionless-graph	
Hour and day	Net Σ hr.	Total flood, Q	Base flow Q	Net Q	Increm.[2] c.f.s.-hrs.	Accum. 1,000 c.f.s.-hrs.	Abscissae, percent of $Lg + \frac{D}{2}$	Ordinates, net Q x $\left[\frac{Lg + \frac{D}{2}}{\text{vol}}\right]$
12P30	0	2,000	2,000	0	0	0	0	0
1A1	1	2,250	2,000	250	125	.12	10.5	0.09
2A1	2	3,560	2,000	1,560	905	1.03	21.1	0.57
3A1	3	8,120	2,000	6,120	3,840	4.87	31.6	2.22
4A1	4	18,640	2,000	16,640	11,380	16.25	42.1	6.0
5A1	5	36,040	2,000	34,040	25,340	41.59	52.6	12.4
6A1	6	56,290	2,000	54,290	44,165	85.76	63.2	19.7
7A1	7	70,510	2,000	68,510	61,400	147.16	73.7	24.9
8A1	8	73,000	2,000	71,000	69,755	216.91	84.2	25.8
9A1	9	66,330	2,000	64,330	67,665	284.58	94.8	23.4
10A1	10	55,360	2,000	53,360	58,845	343.42	105.8	19.4
11A1	11	43,250	2,000	41,250	47,305	390.72	115.8	15.0
12N1	12	33,520	2,000	31,520	36,385	427.11	126.4	11.4
1P1	13	26,900	2,020	24,880	28,200	455.31	136.9	9.0
2P1	14	22,830	2,050	20,780	22,830	478.14	147.4	7.5
3P1	15	19,810	2,080	17,730	19,255	497.40	158.0	6.4
4P1	16	17,230	2,100	15,310	16,520	513.92	168.5	5.6
5P1	17	15,390	2,120	13,270	14,290	528.20	179.0	4.8
6P1	18	13,780	2,150	11,630	12,450	540.66	189.5	4.2
8P1	[1]20	11,090	2,200	8,890	(20,520)	(561.18)	210.6	3.23
12P1	24	7,460	2,300	5,160	(28,100)	(589.28)		
6A2	30	4,840	2,500	2,340	(22,500)	(611.78)	[3]315.9	[3].85
12N2	36	3,700	2,650	1,050	(10,170)	(621.94)		
6P2	42	3,305	2,830	475	(4,575)	(626.52)	[3]442.3	[3].17
12P2	48	3,215	3,000	215	(2,070)	(628.59)		
6A3	54	3,100	3,000	100	(960)	(629.55)		
12N3	60	3,045	3,000	45	(420)	(629.97)		
6P3	66	3,020	3,000	20	(180)	(630.15)		
12P3	72	3,010	3,000	10	(90)	(630.24)		
6A4	78	3,000	3,000	0	(30)	(630.27)		

[1]Note variations in time intervals for listing discharges (optional).

[2]c.f.s-hrs. = $\left(\frac{Q_1 + Q_2}{2}\right)$ x (time interval, hrs.)

[3]For plot on semilog paper, only enough points to define a straight line need be computed.

Table G-S.-Continued

Equations for dimensionless-graph:

$$\text{Abscissae} = \frac{\text{net } \Sigma \text{ hr.}}{Lg + \frac{D}{2}} \times 100$$

$$\text{Ordinates} = \text{net Q} \times \frac{Lg + \frac{D}{2}}{\text{vol., c.f.s.-days}}$$

$$\left[\text{c.f.s.-days} = \left(\frac{\text{c.f.s.-hours}}{24} \right) \right]$$

Lag-plus-semiduration:

1/2 volume is between net Σ hrs. 9 and 10
By linear interpolation:

Volume, method 1,

$$Lg + \frac{D}{2} = 9.50 \text{ hrs.}$$

Volume, method 2,

$$Lg + \frac{D}{2} = 9.52 \text{ hrs.}$$

Except for very small watersheds, lag-plus-semiduration values are rounded to nearest 1/10 hr.

For dimensionless-graph equations:

Use: $Lg + \frac{D}{2} = 9.5$

Volume = 26,150 c.f.s.-days

Lag estimate:

$D = 4$ hrs.

$$Lag = 9.5 - \frac{D}{2} = 7.5 \text{ hrs.}$$

Net volume computations:

Method 1, by equations.

q_o: Q at net Σ hr. 20 = 8,890 c.f.s.
q_t: Q at net Σ hr. 30 = 2,340 c.f.s.
t: time interval, q_o to q_t = 10 hrs.

$$k_{hr} = \sqrt[t]{\frac{q_t}{q_o}} = \sqrt[10]{\frac{2,340}{8,890}}$$

$$k_{hr} = \sqrt[10]{0.263} = 0.875$$

$$\text{Volume after net } \Sigma \text{ hr. } 20 = \frac{-q_o}{\log_e k_{hr}}$$

$$= \frac{-8,890}{-0.1336}$$

= 66,540 c.f.s.-hrs.

Σ net volume, hrs. O-20 = 561,180 c.f.s.-hrs.
Total net volume = 627,720 c.f.s.-hrs.
= 26,150 c.f.s.-days
½ total net volume = 313,860 c.f.s.-hrs.

Method 2.
Ordinates of total net hydrograph used as shown in table at left,
Discharges of recession limb read at time intervals for which recession curve can be approximated as a straight line.

Total volume = 630,270 c.f.s.-hrs.
= 26,260 c.f.s.days
½ volume = 315,140 c.f.s.-hrs.

L = length of longest watercourse from point of interest to watershed divide, measured in miles,
ca = centroid of basin-usually found by vertically suspending a cardboard cutout of basin shape successively from two or more points and finding intersection of plumb lines from each point,
L_{ca} = length of watercourse from point of interest to intersection of perpendicular from ca to stream alinement, and
S = overall slope in feet per mile of longest watercourse from point of interest to divide.

Values for the constants C and x are obtained empirically from plots on log-log paper of $\frac{LL_{ca}}{\sqrt{S}}$ values versus lag-time, hours, and fitting a straight line, either "by eye" or by leas t-squares computations. The lag-time indicated by the curve for an $\frac{LL_{ca}}{\sqrt{S}}$ value of 1 .O is the constant C, and the "slope" of the line on log-log paper is the constant x.

A lag-time curve for a watershed should be based on as many hydrograph analyses as can be obtained from the data available within the watershed and for other watersheds with similar runoff characteristics. When developing a lag-time curve, a consistent method of hydrograph analyses should be used and measurements of watercourse lengths should be made on maps of the same scale. If suitable data are limited to only one stream gage location, a lag-time curve can be constructed by drawing a line with slope of 0.33 through the point plotted on log-log paper of average lag-time versus $\frac{LL_{ca}}{\sqrt{S}}$ value.

In the absence of any runoff data suitable for hydrograph analyses, preliminary estimates of lag-times for ***direct runoff*** for watersheds having rapid runoff characteristics can be made by the following generalized equation:

$$\text{Lag-time, hours} = 1.6 \left[\frac{LL_{ca}}{\sqrt{S}}\right]^{0.33}$$

The above equation gives values acceptable as preliminary estimates of direct runoff lag-times for many streams in the plains and southwestern regions of the United States and for foothill streams of the Rocky Mountains. Certain types of watersheds have large variations in lag-times that are not adequately reflected by the generalized C value given. These include watersheds which have physical features tending to retard surface runoff such as near level terrain, dense vegetative cover, etc.; and those in which the streams extend into high, well-forested mountains or whose streamflow records show pronounced interflow contribution. Lag-time estimates for such watersheds should be made by an experienced hydrologist.

C. SYNTHETIC UNIT HYDROGRAPH

G-9. Synthetic Unitgraphs by Lag-Time Dimensionless-Graph Method.-Computation of a unitgraph for a watershed above a specific location by this method is done by reversing the mathematical process used to derive a dimensionless-graph. The important factors for obtaining a representative unitgraph for a given watershed are the selections of a proper lag-time curve and proper dimensionless-graph. An example of a unitgraph derivation for an ungaged watershed follows, given as a step-by-step outline with pertinent comments and graphically illustrated on figure G-6.

(1) Outline drainage boundary, determine area (fig. G-6(A)).

(2) Find basin center of area, *ca* and project to the nearest point on the longest watercourse. Measure ***L*** (to divide at head of longest watercourse) and L_{ca} miles. (Refer to sec. G-8(e)(2).) Determine S (for upper elevation, estimate average elevations along divide in vicinity of head of longest watercourse, not the specific elevation at the point of extention of longest watercourse to divide).

(3) Compute $\frac{LL_{ca}}{\sqrt{S}}$.

(4) Enter graph, lag-time curve (fig. G-6(B)), with $\frac{LL_{ca}}{\sqrt{S}}$ value and read the corresponding lag-time. (Lag-time curve (B) represents mean curve drawn "by eye" through plotted lag-times obtained from hydrograph analyses versus respective $\frac{LL_{ca}}{\sqrt{S}}$ for basins of similar runoff characteristics.)

(5) Select a dimensionless-graph (fig. G-6(C)) (usually an average dimensionless-graph of a number of dimensionless-graphs derived for the same stream or for streams of similar characteristics).

(6) Select a unit rainfall duration time; this should be one-fourth or less of lag-time for basin. (Unit times are selected for

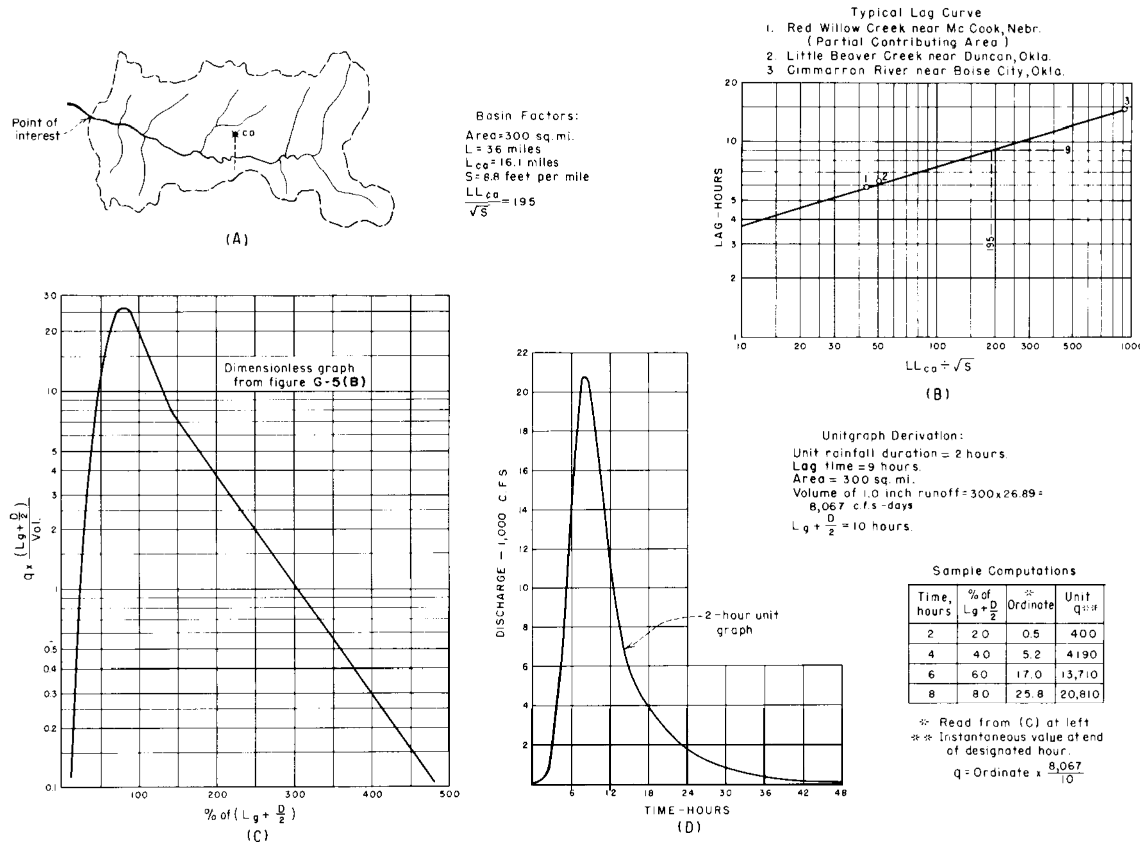


Figure G-6. Unitgraph derivation for **ungaged** area.-288-D-3182

computational convenience, usually 1-, 2-, 3-, 4-, or 6-hour units for lag-times of 4 hours or greater. Unit times larger than 6 hours are seldom used. Units of one-half or one-quarter hour are used for lag-times less than 4 hours.)

(7) Compute unitgraph (fig. G-6(D)) using:

(a) Basin area, square miles.

(b) Lag-time plus one-half selected unit rainfall duration time.

(c) Dimensionless-graph.

(d) Notes regarding computational procedure.

1. Equations for deriving a dimensionless-graph are given in table G-5. Unitgraph computation requires solving for instantaneous discharges at end of successive unit time intervals.

2. Time, hours, accumulative by unit time intervals are listed, and each accumulative value expressed as percent of lag-plus-semiduration.

3. Dimensionless-graph (fig. G-6(C)) is entered with successive lag-plus-semiduration values, and respective ordinates read from the graph. Ordinates are substituted in the ordinate equation for solution of discharge values. When done by desk calculator, discharges are rounded.

(Note: Dimensionless-graph ordinates listed in the table of sample computations (fig. G-6) do not agree numerically at respective accumulative time values with dimensionless-graph ordinates in table G-5, because the dimensionless-graph ordinates in the table were derived at intervals of 10.5 percent of lag-plus-semiduration but the ordinates for 2-hour unitgraph derivation in figure G-6 were read at intervals of 20 percent of a different lag-plus-semiduration value.)

4. Caution. -The volume of a synthetic unitgraph should always be checked before being used, to be sure it has a volume within 1 percent of l-inch runoff volume for the watershed area. All of the ordinates of a unitgraph ((D) of fig. G-6) may be computed by reading the entire dimensionless-graph (C) and summing the ordinates to check the volume.

Another procedure may be used if the selected dimensionless-graph has an exponential recession limb such as on figure G-6(C). Unitgraph ordinates are obtained by reading the dimensionless-graph forward to an ordinate that is on the beginning portion of the exponential limb of the dimensionless-graph (see sec. G-8(e)(l)(b)). The volume of the unitgraph thus far obtained is computed and subtracted from the volume of 1 inch of runoff for the watershed area, giving the remaining volume, V_x . A recession constant, k, for the selected unit time interval can be computed by the equation,

$$\log_e k = -\frac{q}{V_x} \tag{8}$$

where :

q = the discharge ordinate, c.f.s., on the exponential limb, and
V_x = the remaining volume expressed in unit time (c.f.s.-hours).

The factor k is used to compute the ordinates of the unitgraph following the last ordinate obtained by reading the dimensionless-graph. This procedure assures correct unitgraph volume.

G-10. ***Trial Reconstruction of Past Floods.*** -Final decisions regarding appropriate lag-time, dimensionless-graph, and retention losses for a gaged watershed are made empirically by computing hydrographs of past recorded floods. Retention losses believed appropriate are applied to the observed storm precipitation data for each flood to be reconstructed to determine unit time increments of rainfall excess equivalent to the respective hydrograph volume. These increments are applied to a representative unitgraph according to basic assumption (3), figure G-3. The hydrograph thus computed is compared with the recorded hydrograph for *goodness of fit*; preliminary conclusions regarding appropriate factors are revised, if necessary, until an acceptable *fit* is obtained. These *test* trial reconstructions should be made for the large floods. Preferably, the largest flood of record should be excluded from the set of hydrographs selected for analyses and the parameters resulting from analyses tested by the *fit* achieved using them to reconstruct the largest flood.

G-l 1. ***Synthetic Unitgraphs by Other Methods.*** -Descriptions of several different methods of estimating synthetic unitgraphs may be found in technical publications. Among those often used are the S-curve hydrograph [8], Snyder's method [9], and basin routing methods based on the Clark approach [5],[10],[11],[12].

D. STREAMFLOW ROUTING

G-12. ***General.*** -Computation of an inflow design flood (IDF) hydrograph often requires that floodflows from several subareas within the drainage area be computed separately. Beginning with the farthest upstream subarea, hydrographs are transferred downstream by some method of streamflow routing, the flows being consecutively combined with other flood hydrographs, and the total inflow design flood hydrograph obtained for the proposed reservoir. Watershed features above a damsite which indicate the need to subdivide the basin into subareas include:

(1) Large tributary areas which have different sizes, shapes, and cover characteristics.

(2) Existing reservoirs or natural lakes which control runoff from significant portions

of the drainage area above a proposed damsite. The flood runoff from the portion of the design storm for the total drainage area that occurs above such an existing feature should be reservoir-routed through the feature to obtain an outflow hydrograph before routing on downstream. If an existing dam impounds a large-capacity reservoir, the capability of the existing dam to safely withstand the computed inflow flood must be determined. Should the upstream dam be found to have an inadequate spillway capacity (or structural weakness), steps should be taken to get the owners of the upstream dam to make modifications as necessary to safely pass the inflow design flood. Or as an alternative, failure of the structure should be assumed and provision made at the proposed downstream dam and reservoir to safely handle the flood wave surge that might be expected with failure and an additional inflow volume equivalent to the capacity of the upstream reservoir.

(3) Drainage areas in which storm potential varies to an extent that an assumption of average precipitation over the total area during a design storm is unreasonable.

(4) Drainage areas in which during design storm conditions some streams will have snowmelt runoff in addition to rainfall runoff and other streams have only rainfall runoff.

G-13. *Practical Methods of Streamflow Routing Computations.* -Streamflow routing, the determination of a flood discharge hydrograph at any point on a stream from a discharge hydrograph at some point upstream, requires solution of the movement of flood waves in natural open channels which are extremely complex. A discussion of the theoretical and mathematical bases of flood routing methods is beyond the scope of this text. Many different methods and procedures have been described in engineering literature. If streamflow routing is necessary in the derivation of an inflow design flood hydrograph and the damsite is located on a stream that has discharge records at two or more locations, an applicable routing method may be selected from descriptions in publications, for example, “Hydrology for Engineers” [13].

Usually, inflow design flood derivations that include streamflow routing computations involve ungaged streams. Description of two practical methods of mathematical streamflow routing which can be used on the basis of an estimate of peak discharge travel time between two points on a reach of natural stream channel follows. These methods have been found to give acceptable results when tested by using recorded discharge hydrographs.

(a) *Tatum’s Method* [141 .-This method is also known as the *Method* ***of Successive Averages.*** Factors used when applying this method are travel time of peak discharge through the channel reach, *Tin* hours; selected routing interval between discharges of the upstream hydrograph to be routed, t in hours; and routing constants listed in table G-6 for respective number of routing steps. Definite rules for selecting lengths of stream channel reaches for each routing computation cannot be set, but use of extremely long reaches may give very poor results. When computing an inflow design flood hydrograph, channel reaches are those on the main stream between points of inflow from subareas. Thus, inflow from a subarea can be added to the routed flow at the subarea inflow point to obtain a combined floodflow for routing through the next reach. After estimating travel time T believed applicable for a reach, a routing interval t is selected choosing an interval small enough to define well the hydrograph, and the number of routing steps for that reach computed by the equation:

$$\text{Number of routing steps} = 2T/t \qquad (9)$$

Computed fractional steps are rounded to the nearest whole number. The computational procedure is illustrated in table G-7. In actual practice when using a desk calculator, the routing constants are copied in a column on a separate sheet of paper and used as a slide beside the column of discharges to be routed. Products of the multiplications of constants and respective discharges are accumulated in the machine and only the total of each set of multiplications recorded. Constants for larger numbers of routing steps than given in table

Table ***G-&-Coefficients for jloodrouting by** Tatum's **method***

Routing constants	Number of uting st									
	1	2	3	4	5	6	7	8	9	10
C_1	0.5000	0.2500	0.1250	0.0625	0.0313	0.0156	0.0078	0.0039	0.0020	0.0010
C_2	.5000	.5000	.3750	.2500	.1562	.0937	.0547	.0313	.0176	.0098
C_3		.2500	.3750	.3750	.3125	.2344	.1641	.1094	.0703	.0440
C_4			.1250	.2500	.3125	.3126	.2734	.2187	.1641	.1172
C_5				.0625	.1562	.2344	.2734	.2734	.2460	.2050
C_6					.0313	.0937	.1641	.2187	.2460	.2460
C_7						.0156	.0547	.1094	.1641	.2050
C_8							.0078	.0313	.0703	.1172
C_9								.0039	.0176	.0440
C_{10}									.0020	.0098
C_{11}										.0010

G-6 may be computed from the expression $(\frac{1}{2} + \frac{1}{2})^n$ by the general equation for each term of a binomial expansion, n as the number of steps. Streamflow routing by Tatum's method using a desk calculator becomes tedious and time consuming when more than eight routing steps are used. The procedure may be easily programed for computer use.

(b) ***Translation and Storage Method.***-In a paper describing a graphical reservoir-routing method, Wilson [15] also discusses streamflow routing, pointing out that it is partly analogous to reservoir routing but that natural channel storage produces less "flattening" effect on an inflow hydrograph than does reservoir storage. He suggested that in streamflow routing, the out flow (routed) hydrograph would lie between a hydrograph obtained by applying the graphical reservoir-routing method and the inflow hydrograph translated downstream a time interval equivalent to the reach travel time, and presented an example in which the routed hydrograph showed half translatory effect and half storage effect.

A report of the California Division of Water Resources [161 presented a streamflow routing method based on an adaptation of Wilson's graphical routing method showing that translation effect (travel time) and channel storage effect (attenuation) on the shape of a flood hydrograph moving downstream can be treated separately. In their studies, each effect was found to have approximately equal weight.

The translation and storage method of streamflow routing was devised[5] on the basis of evaluating separately the effects of travel time and channel storage and assuming equal weight for each effect in natural stream channels having "usual" storage characteristics. An equation for mathematical application of Wilson's graphical routing method was given in the U.S. Department of Agriculture, Soil Conservation Engineering Handbook, Supplement A, 1956. The given equation is used in the translation and storage method of streamflow routing as follows:

$$O_2 = O_1 + K(I_1 + I_2 - 2\,O_1) \qquad (10)$$

where:

I_1, I_2 = inflow, consecutive incremental instantaneous discharges at the head of a stream reach, and

O_1, O_2 = outflow, successive incremental instantaneous discharges at the end of a stream reach; O_2 is the outflow resulting from I_1 and I_2 and the preceding outflow O_1.

The routing constant, ***K***, in the above equation, is obtained as follows:

T = travel time, hours, of peak flow through the reach consisting of:

[5] Described in unpublished memoranda, Flood Hydrology Section, Engineering and Research Center, Bureau of Reclamation, Denver, Colo.

Table *G-I.-Illustrative example of streamflow routing by Tatum's method*

HYPOTHETICAL PROBLEM: Streamflow-route total flood hydrograph, table G-5, through channel reach having travel time of 4 hours.

If selected t = 1 hr., routing steps = $\frac{(2)(4)}{1} = 8$

If selected t = 2 hrs., routing steps = $\frac{(2)(4)}{2} = 4$

Hour and date	Upstream Q 1,000 c.f.s	t = 1 hr., 8 routing steps Illustrative positioning of routing constants[2]				Routed [3]Q 1,000 c.f.s.	t = 2 hrs., 4 routing steps Illustrative positioning of routing constants[2]				Routed [3]Q 1,000 c.f.s.
4P30	'2.0										
5P	2.0	*0.0039*									
6P	2.0	.0313					0.0625				
7P	2.0	.1094									
8P	2.0	.2184					.2500				
9P	2.0	.2734									
10P	2.0	.2187					.3750				
11P	2.0	.1094									
12P30	2.0	.0313					.2500				
1A1	2.3	.0039				[4]2.0					
2A	3.6						.0625	0.0625			[4]2.1
3A	8.1		*0.0039*								
4A	18.6		.0313	*3.0039*				.2500	I.0625		
5A	36.1)		.1094	.0313	).0039						
6A	56.3		.2187	.1094	.0313			.3750	.2500	3.0625	
7A	70.5		.2734	.2187	.1094						
8A	73.0		.2187	.2734	.2187			.2500	.3750	.2500	
9A	66.3		.1094	.2187	.2734						
10A	55.4		.0313	.1094	.2187			.0625	.2500	.3750	47.7
11A	43.2		.0039	.0313	.1094	61.3					
12N1	33.5			.0039	.0313	'64.8			.0625	.2500	[6]58.6
1P1	26.9				.0039	61.7					
2P	22.8									.0625	37.8

[1]Constant base flow of 2,000 c.f.s. assumed to precede flood event.
[2]All routing constants are placed opposite respective Q's at t intervals.
[3]Discharge at bottom of reach; each Q is instantaneous discharge at time given in column 1.
[4]Sum of products of each constant times respective Q.
'Peak discharge of routed hydrograph, occurs 4 hours later than upstream peak.
[6]Peak discharge of routed hydrograph, agrees in time with routing t = 1 hr., but differs in magnitude because of longer routing interval.

T_r = translation time component, hours (when assuming equal weight to storage effect, $T_r = 0.5T$)

T_s = storage time component, hours (when assuming equal weight to translation effect, $T_s = 0.5T$)

and

$$T = T_r + T_s$$

Then for stream routing evaluation of storage time effect,

$$K = \frac{t}{2T_s + t}$$

where:

t = routing time interval, hours, with $t \leq 0.5T_s$.

Solving the equation for O_2 gives an instantaneous discharge value at the end of the incremental time interval designated by I_2. If I_1, I_2, etc., are designated by time at the head of a reach, the time of occurrence of O_2 at the bottom of the reach is obtained by adding the translation time component, T_r to the time of respective I_2.

Use of the above equation with an assumption that the travel time for the reach is divided equally into translation time, T_r, and storage time, T_s, gives as acceptable results as those obtained by using Tatum's Method but requires less computational time when doing manual routing. A detailed example of application of the translation and storage method is shown in table G-8. Of course, in practice, such a detailed table is not necessary.

The translation and storage method, in addition to being easy to apply to stream reaches for which Tatum's method might be used, is also versatile enough to be applied to stream reaches having more or less storage effect than "usual." The relationship of storage time and translation time is not rigid, but may be varied depending on channel reach characteristics. If hydrographs are available at the head and bottom of a stream reach, a few trial routings will give an acceptable value for each component. Characteristics of ungaged stream channels are judged by comparison with characteristics of gaged streams when necessary to use streamflow routing methods.

*(c) **Comparison of Methods.*** -An illustration of results of applying the above two methods of streamflow routing is shown on figure G-7 on which the hypothetical flood hydrograph, with discharges listed in table G-5, is plotted. This hydrograph was routed downstream assuming a reach travel time of 4 hours: first, by Tatum's method assuming routing intervals of 1 hour and 2 hours; and secondly, by the translation and storage method using a routing interval of 1 hour. Routed (downstream) hydrographs are also plotted on figure G-7 (computations are not included). The two routed hydrographs obtained by Tatum's method differ because of different routing intervals; the routing by l-hour intervals is the more representative because the upstream hydrograph is best defined in l-hour intervals. The routed hydrograph obtained by the translation and storage method is acceptably similar to the hydrographs obtained by Tatum's method.

E. DESIGN STORM STUDIES

G-14. General.-Major floods, except those associated with dam failure, earthquakes, or landslides, result from a combination of severe meteorological and hydrological conditions. It follows that estimates of meteorological conditions which may approach the physical upper limits of rainfall or snow accumulation and melt rates must be considered where an inflow design flood (IDF) is required. This section is concerned only with rainfall studies. For the purpose of this text, the following terminology is used in regard to estimates of the physical upper limits of storm rainfall in a basin or region.

(a) ***Probable Maximum Precipitation (PMP).*** -Probable maximum precipitation values represent an envelopment of maximized intensity-duration values obtained from all types of storms. It is recognized that probable maximum precipitation values for all durations and all areas may not occur from only one type of storm. For example, a maximized thunderstorm is very likely to provide probable maximum precipitation over an area of 50 square miles for a duration of 6 hours or less, but the controlling values for longer durations or for larger areas generally will be obtained from general-type storms.

(b) ***Probable Maximum Storm (PMS).***—The probable maximum storm values represent an envelopment of maximized intensity-duration values obtained from storms of a single type. Consideration is given to storm type and variations of precipitation with respect to

Table G-&-Translation and storage method of streamflow routing

Equation: $O_2 = O_1 + K(I_1 + I_2 - 2\,O_1)$

T = 12 hours $K = \frac{t}{2T_s + t}$

$T_r = 6$ hours $K = \frac{3}{12+3}$

$T_s = 6$ hours $K = 0.20$

t = 3 hours

(For definitions of symbols, see sec. G-13 (b).)

(1) Time, hours[1]	(2) Inflow, c.f.s.	(3) $I_1 + I_2$, c.f.s.	(4) $2\,O_1$	(5) (3) − (4)	(6) (K)(5)	(7) Outflow,[2] c.f.s.	(8) Time, hours[4]
0	300					[3]300	6
3	300	600	600	0	0	300	9
6	415	715	600	115	23	323	12
9	1,604	2,019	646	1,373	275	598	15
12	5,458	7,062	1,196	5,866	1,173	1,771	18
15	10,093	15551	3,542	12,009	2,402	4,173	21
18	16,567	26,660	8,346	18,314	3,663	7,836	24
21	17,924	34,491	15,672	18,819	3,764	11,600	27
24	18,608	26,532	23,200	13,332	2,666	14,266	30
27	19,244	37,852	28,532	9,320	1,864	16,130	33
30	19,772	39,016	32,260	6,756	1,351	17,481	36
33	25,913	45,685	34,962	10,723	2,145	19,626	39
36	23,499	49,412	39,252	10,160	2,032	21,658	42
39	20,552	44,051	43,316	735	147	21,805	45
42	17,377	31,929	43,610	-5,681	-1,136	20,669	48
45	14,703	32,080	41,338	-9,258	-1,852	18,817	51
48	12,054	26,757	37,634	-10,877	-2,175	16,642	54

[1]Time of instantaneous discharge at head of reach.
[2]Discharge at end of reach; (6) + preceding value in (7).
[3]Constant flow in reach assumed.
[4]Time of instantaneous discharge at end of reach. Translation time, T_r, added to time at head of reach.

location, area1 coverage of a watershed, and storm duration.

(c) Design Storm. -The precipitation values selected for computing an inflow design flood are usually referred to as a design storm. These design storm values may or may not be equal to the PMP. The hydrometeorological report which describes the considerations and computations leading to the recommendation of a design storm for a particular watershed is usually called a "Design Storm Study."

(d) *Additional* References. -It is beyond the scope of this text to discuss in detail the meteorological considerations and computations involved in obtaining the "maximized intensity-duration values" cited in the above definitions. A comprehensive discussion of this subject is given in chapter 2, "Maximum Rainfall," of WMO Technical Note No. 98 [2]. A brief discussion on estimation of probable maximum storms is given in subsequent paragraphs. Also included in this section are generalized precipitation charts for estimating probable maximum precipitation values east of the 105° meridian and general-type design storm values west of the 105° meridian for watersheds in the 48 conterminous United States. These charts also are presented in chapter III of "Design of Small Dams," second edition [31], associated with procedures for

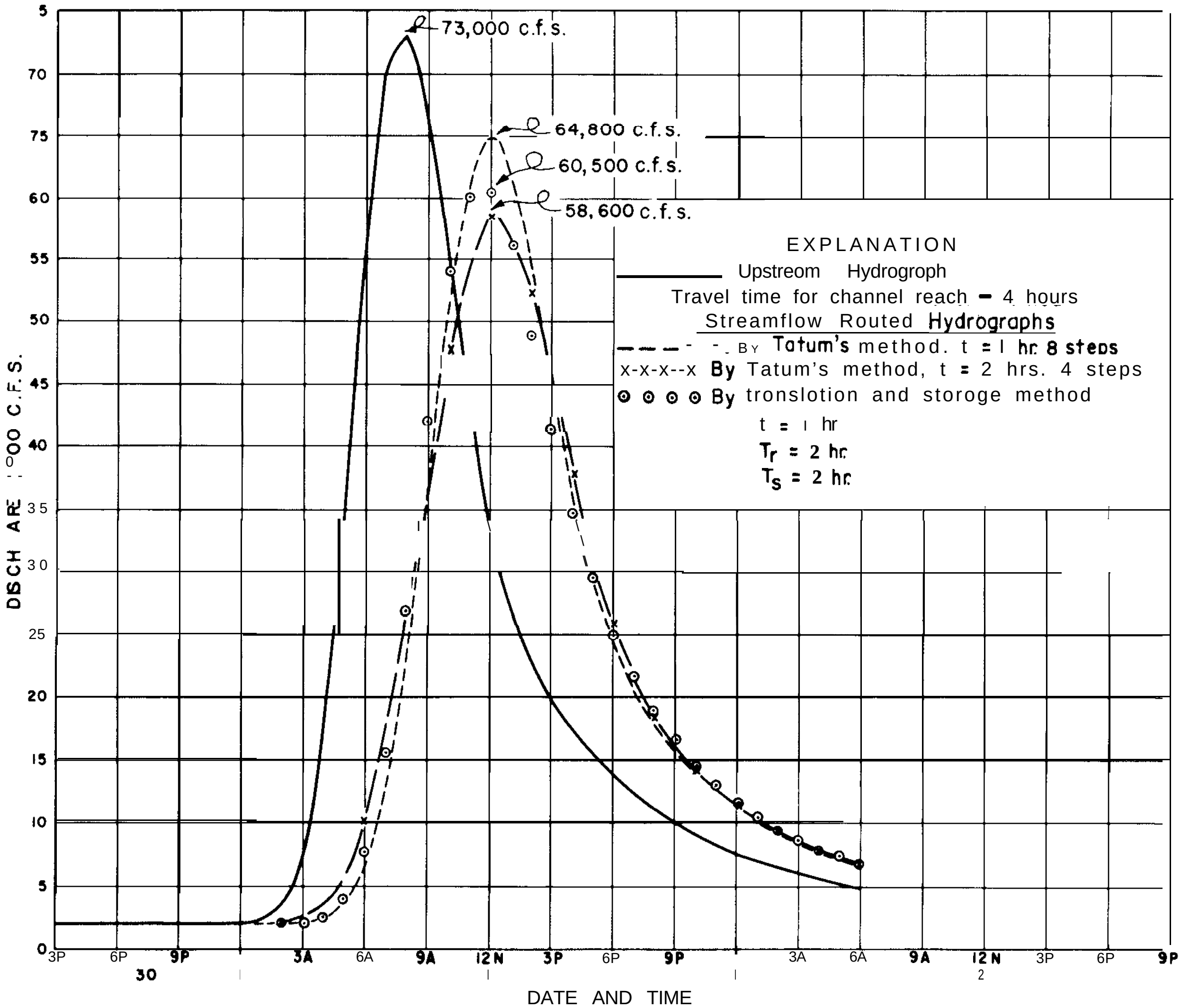


Figure **G-7. Comparison of results of streamflow routings.–288-D-3183**

estimating inflow design floods for small dams.

Discussion of design thunderstorm rainfall has been omitted in this text, anticipating that readers will be concerned generally with damsites controlling drainage areas large enough to preclude the use of thunderstorm rainfall. However, thunderstorm rainfall should never be ignored completely, as it may prove critical under some circumstances.

G-15. ***Probable Maximum Storm Considerations.*** -Estimates of probable maximum storms (PMS) are based on analyses which consist of three steps: (1) determination of the area1 and temporal distribution of the larger storms of record in the general area; (2) augmentation of these observed storms through moisture adjustment; and (3) consideration of storm transposition.

One objective of the first step cited above is the determination of maximum values of storm rainfall for selected durations and area. Depth-area-duration (DAD) values of each total storm are analyzed without regard to watershed boundaries [17]. Comparison of DAD values will indicate which storms are best suited for further analysis. If hydrographs of floods for specific watersheds associated with the storms are available for analyses, determination of rainfall data for these specific watersheds can be included as a part of the analyses.

Technical literature [2] should be consulted for a detailed discussion of the theoretical assumptions included in the computational procedures for storm maximization, step (2), and storm transposition, step (3). An abridged discussion of a procedure often used for maximization and transposition of storms in plains-type terrain follows. Discussion of procedures for storm maximization and limited transposition in mountainous terrain is beyond the scope of this text.

G-16. ***Procedure for Storm Maximization, Plains- Type Terrain.*** -This procedure is based on assuming a saturated air-mass with a pseudoadiabatic lapse rate. Moisture content under these circumstances is a unique function of surface dewpoint temperature, so that dewpoint temperatures may be used to quantitatively estimate total atmospheric water vapor or precipitable water values. Tables [18] have been published which list ambient temperatures for various elevations or pressures above a 1,000-mb. (1,000-millibar) surface, approximately equivalent to mean sea level, for selected temperatures in a saturated atmosphere with a pseudoadiabatic lapse rate.

Tables [18] also list, for each 1,000-mb. dewpoint temperature, values of precipitable water in inches for layers between the 1,000-mb. surface and various elevations to extreme heights in a saturated, pseudoadiabatic atmosphere. These precipitable water values may be used as an index to the moisture content of a unit column of air between sea level and the top of a moisture-bearing air-mass. Maps with isotherms of maximum 12-hour persisting 1,000-mb. dewpoint temperatures (° F.) of record for each month for the 48 conterminous states are available in the "Climatic Atlas of the United States" [19] .

Computational procedures for storm maximization and transposition, plains-type terrain, follow:

(a) Maximization of a Storm in Place of Occurrence.

(1) *Observed storm dewpoint.* -A representative 12-hour persisting surface dewpoint temperature is obtained for the storm period under study from temperature stations in the path of the inflowing moist air. If the rainfall is of a frontal type, the surface dewpoints within the rainfall area will be lower than those of the inflowing moist air, thus giving a low estimate of storm moisture content. Distance and direction from the storm center to the representative dewpoint station or stations should be recorded.

(2) Adjustment to 1,000-mb. surface. -Since during major storms the airmass will be saturated, the dewpoint temperature at the representative station can be adjusted to a 1,000-mb. surface temperature assuming a saturated, pseudoadiabatic lapse rate of temperature.

(3) Precipitable water values. -From the 1,000-mb. dewpoint temperature determined in (2) above, obtain two precipitable water values, W_p , for the observed storm:

(a) W_{p-1} is the precipitable water between 1,000 mb. and the top of the moist layer for the storm system; an elevation of 40,000 feet, or pressure of 200 mb., is usually assumed.

(b) W_{p-2} is the precipitable water between 1,000 mb. and the mean surface elevation of the central portion of the observed storm. If the inflowing moist air has passed over a topographical barrier with a higher elevation than at the central portion of the storm, W_{p-2} is obtained using the inflow barrier elevation.

(4) Observed storm's precipitable water, W_s.—Compute the observed storm's moisture content or available precipitable water, W_s, as W_{p-1} minus W_{p-2}.

(5) Probable maximum precipitable water for the storm, W_x. -An estimate of the probable maximum moisture content indicated for the storm is obtained as follows:

(a) From the "Climatic Atlas of the United States" [19], the maximum 12-hour persisting dewpoint temperature of record can be determined for the date of storm occurrence and the location of the representative dewpoint for the observed storm. Frequently, the maximum recorded dewpoint temperature within a period of plus or minus 15 days is used.

(b) From the maximum dewpoint of record, precipitable water is obtained for the same layers as used in W_{p-1} and W_{p-2} above. These precipitable water values are designated W_{r-1} and W_{r-2} .

(c) The estimated probable maximum precipitable water, W_x , will be W_{r-1} minus W_{r-2}.

(6) *Moisture maximization factor, M_f.* -The moisture maximization factor, *Mf,* is computed as the ratio of the probable maximum precipitable water to the precipitable water observed during the storm, or $Mf = W_x / W_s$.

(7) Maximized storm values. -Maximized storm values are computed by multiplying depth-area-duration (DAD) values of the observed storm by the maximization factor, M_f.

Note: This procedure assumes that the magnitude of rainfall in a storm is a function only of the inflow moisture charge. It also assumes that the most effective combination of storm efficiency and inflow wind has occurred or has been closely approached in the major storms of record. The procedure may not always prove adequate, particularly for regions where rainfall is strongly influenced by orographic effects [2].

(8) Example of computations–maximization in place.

(a) Dewpoint observation station: elevation 1000 feet.

Location: 100 miles southeast of storm center.

Representative 12-hour storm dewpoint: 69° F.

Sea level, 1,000 mb., dewpoint: 71° F.

(b) Surface elevation, storm center: 1500 feet.

W_{p-1} = 2.38 inches (at 40,000 feet)
W_{p-2} = 0.32 inch (at 1500 feet)
W_s = 2.06 inches

(c) Maximum dewpoint of record, observed 100 miles southeast of storm center: 78° F. 1191.

W_{r-1} = 3.35 inches (at 40,000 feet)
W_{r-2} = 0.41 inch (at 1500 feet)
W_x = 2.94 inches

(d) Moisture maximization factor:

$$M_f = 2.94/2.06$$
$$M_f = 1.43$$

(b) *Maximization of Transposed* Storm.-When a storm is transposed and maximized for moisture content, the maximization factor is usually computed for the storm only at its transposed location. Computation of available precipitable water for the observed storm, W_s, remains the same as described above.

The moisture maximization factor is computed by determining the surface elevation at the center of the storm at its transposed position or the height of the mean inflow barrier to that location. The maximum dewpoint of record is obtained from the charts of dewpoints [191 at *the same distance from the transposed center and in the same direction as the observed storm dewpoint was obtained.*

(1) *Example of computations-moisture maximization of transposed storm,*

(a) Assume that the storm used in the previous example is transposed to a location where the elevation of the storm center is 2500 feet and that there is not a higher inflow barrier between the transposed center and the moisture source.

(b) Mark the location of the transposed center on the charts of maximum recorded dewpoint temperatures and measure 100 miles southeast to determine the maximum dewpoint of record; for example 77° F.

(c) Observed storm precipitable water remains the same; W_s = 2.06 inches.

(d) Maximum precipitable water for a dewpoint of 77° F:

W_{r-1} = 3.19 inches (at 40,000 feet)
W_{r-2} = 0.64 inch (at 2500 feet)
W_x = 2.55 inches

(e) Moisture maximization factor for the transposed storm:

$$M_f = 2.55/2.06$$
$$M_f = 1.24$$

Note: If an M_f factor greater than 2.0 is computed, reexamine the computations and all meteorological aspects of the transposed storm. An M_f factor greater than 2.0 has not been used in Bureau of Reclamation design storm studies.

(2) Maximized transposed storm values. -The maximized values for the transposed storm are computed by multiplying the DAD values of the observed storm by the maximization factor for the transposed location.

G-17. ***Design Storm-Probable Maximum Precipitation (PMP) or Probable Maximum Storm (PMS) Estimates for a Watershed.*** -Estimates of PMP or PMS, whether made by storm transposition and procedure of dewpoint adjustment described above or by more detailed theoretical computations [20] [6], are based generally on the results of analyses of observed storms. In the United States, passage of the Flood Control Act of 1936 led to the development of a National Storm Study Program under the primary sponsorship of the U.S. Army Corps of Engineers. Under this program more than 600 storms throughout the United States have been analyzed in a uniform manner and summary sheets distributed to Government agencies and the engineering profession [21]. An example of a storm analysis summary sheet from the publication "Storm Rainfall in the United States" [21] is shown on figure G-8. Each storm analyzed has been assigned a designation such as MR 4-24 on the figure. Unfortunately, not all of the summary sheets have a reference to the observed storm dewpoint, such as shown on figure G-8(A). Depth-area-duration (DAD) data for each storm analyzed are given in a table, such as the one at the bottom of figure G-8(A).

A storm location map and a few selected mass rainfall curves are given on figure G-8(B). Summaries of observed storm data such as presented in "Storm Rainfall in the United States," provide broad outlines of storm magnitudes and their seasonal and geographical variations.

A simplified example of the derivation of design storm values for a particular watershed follows. Sources of numerical values used are referenced when possible. The isohyetal patterns and watershed map are not presented. This example may provide the reader with information that will be useful in a better understanding of how preliminary design storm estimates are obtained from the generalized PMP charts given later.

(a) ***Example of a Design Storm Study.*** -(Final-type design storm studies should be prepared by experienced hydrometeorologists.) Let us assume that design storm values representing PMS estimates are required for a watershed with a 200-square-mile area at longitude 99°30′ west, latitude 41°00′ north, a region where storm transposition and maximization by dewpoint adjustment is an acceptable approach. Procedural steps are described first, then numerical computations are given.

(1) Transposition limits of major storms. -The broad limits within which major observed storms can be transposed should be established first. This will require consultation with an experienced hydrometeorologist. However, for the United States east of the 105° meridian, guidelines have been established in Hydrometeorological Report 33 [20].

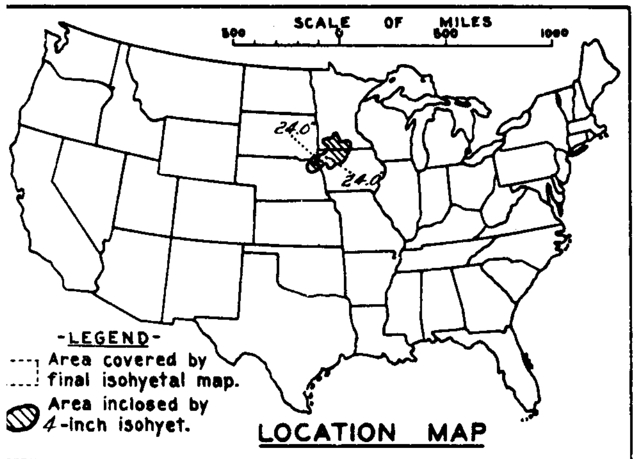
(2) Inventory of data of major storms. -Referring to "Storm Rainfall in the United States" [21], rainfall depth-duration values can be obtained for an area of 200 square miles for all major storms that have been analyzed in the region for which transposition is applicable. Analysis may be required for recent major storms in the region in order to complete the inventory.

(3) Selection of storms for further study.-***Several*** of the larger storms are

[6] Includes 23 separate reports.

DEPARTMENT OF THE ARMY CORPS OF ENGINEERS

STORM STUDIES - PERTINENT DATA SHEET



Storm of 17-19 September 1926
Assignment MR 4-24
Location Ia, Minn, Nebr. S.D. & Wisc
Study Prepared by:
Missouri River Division
Omaha District Office

Part I Reviewed by H. M. Sec. of Weather Bureau, 8/5/47
Part II Approved by Office, Chief of Engineers for Distribution of Factual Data, 12/23/47
Remarks: Centers near Boyden & Maurice, Ia.
Dewpt. 70° - Ref. Pt. 175 SSE Grid C-15

DATA AND COMPUTATIONS COMPILED

PART I

Preliminary isohyetal map, in 2 sheets, scale 1:500,000

Precipitation data and mass curves: (Number of Sheets)

Form 5001-C (Hourly precip. data)	8
Form 5001-B (24-hour " ")	-
Form 5001-D ("" " ")	11
Miscl. precip. records, meteorological data, etc.	29
Form 5002 (Mass rainfall curves)	27

PART II

Final isohyetal maps, in 1 sheet, scale 1:1,000,000

Data and computation sheets:

Form S-10 (Data from mass rainfall curves)	3
Form S-11 (Depth-area data from isohyetal map)	2
Form S-12 (Maximum depth-duration data)	17
Maximum duration - depth- area curves	1
Data relating to periods of maximum rainfall	7

MAXIMUM AVERAGE DEPTH OF RAINFALL IN INCHES

Area in Sq. Mi.	Duration of Rainfall in Hours									
	6	12	18	24	30	36	[illegible]	54		
Max. Station	18.4	23.8	24.0	24.0	24.0	24.0	24.0	24.0		
10	15.1	20.7	21.7	21.7	21.7	21.7	21.7	21.7		
100	12.8	17.1	17.8	17.8	17.8	17.8	17.8	17.8		
200	11.7	15.8	16.6	14.4	16.6	16.4	16.6	16.6		
500	9.4	12.4	13.3	13.3	13.3	13.3	13.3	13.3		
1,000	7.5	10.1	10.4	10.6	10.5	10.6	10.6	10.6		
2,000	5.9	6.0	8.2	8.4	8.5	8.6	8.6	8.6		
5,000	4.1	6.3	6.4	6.6	6.6	6.6	6.4	6.6		
30,000	3.0	5.2	5.4	5.5	5.6	5.6	5.6	5.6		
20,000	2.1	4.1	4.3	4.4	4.4	4.8	4.9	4.9		
50,000	1.4	2.7	2.9	3.0	3.2	3.6	3.8	3.8		
63,000	1.2	2.4	2.4	2.7	2.9	3.3	3.5	3.5		

Form S-2

(A)

Figure G-8. Example of summary sheet, "Storm Rainfall in the U.S." (sheet 1 of 2).-288-D-3184(1/2)

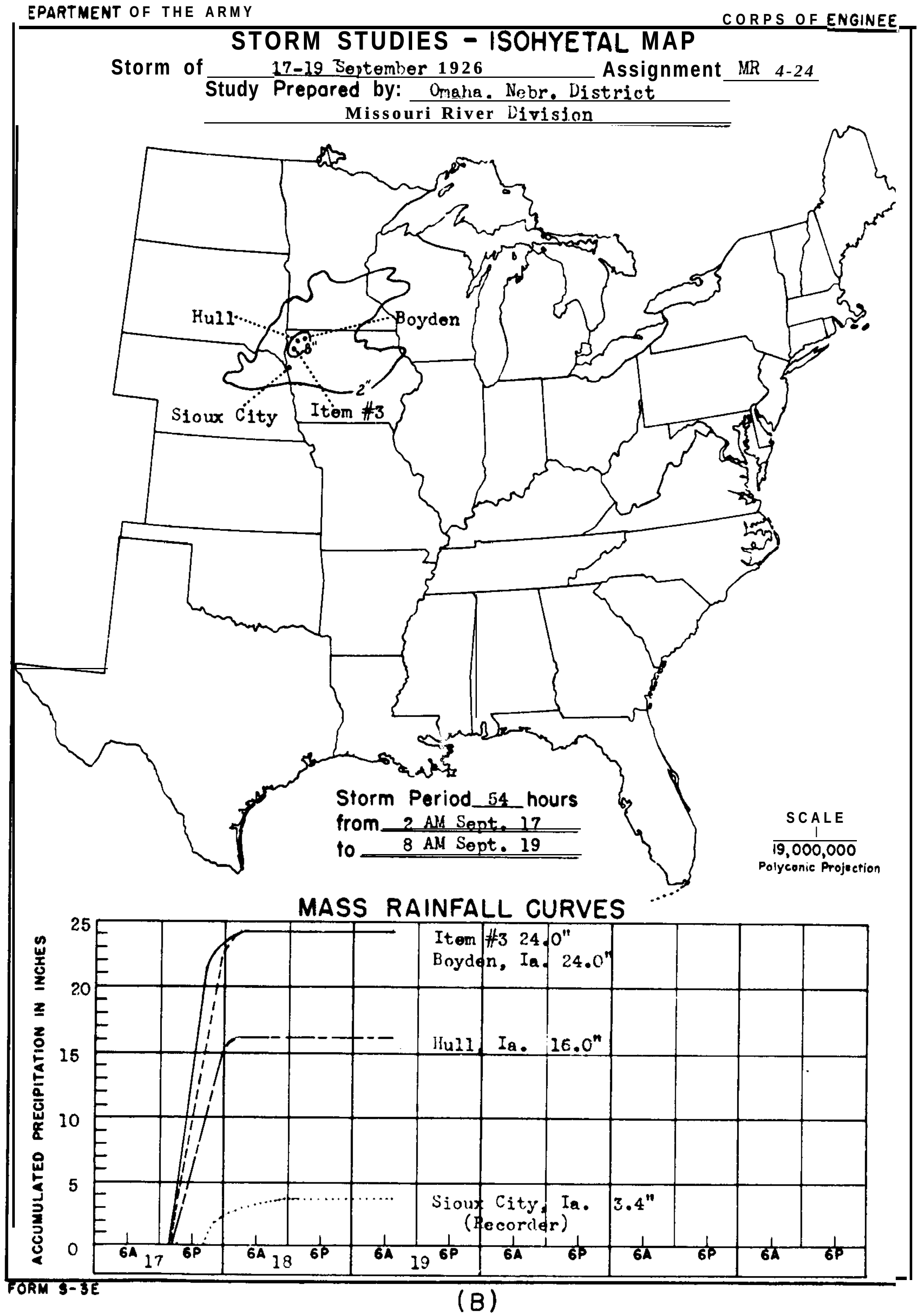


Figure G-8. Example of summary sheet, "Storm Rainfall in the U.S." (sheet 2 of 2).-288-D-3184(2/2) assumed transposed and the depth-duration values for 200 square miles maximized for maximum moisture charge to identify those storms that give the greatest values. Any individual storm may not yield maximum values for all durations. It may be necessary, therefore, to consider a number of storms in the final analysis.

(4) Transposition of isohyetal patterns. -The isohyetal patterns of the storms which yield large values should be obtained, and these patterns then overlaid individually on a map of the subject watershed. The position, within limits, that gives the greatest total basin average rainfall depth should be used. In positioning a transposed storm isohyetal pattern, the orientation of the observed storm pattern is maintained generally within limits of plus or minus 20°.

(5) Average watershed precipitation of transposed storm. -The average storm rainfall within the watershed boundaries of each transposed storm isohyetal pattern is obtained by planimetry. The depth of precipitation for a given area for the total storm was obtained from a DAD tabulation similar to that shown on figure G-8(A). These values were, of course, measured from the isohyetal pattern in the original storm without regard to any watershed boundaries. Obviously, only an assumption of a *perfect fit* of the transposed isohyetal pattern to the basin configuration would give the same total basin rainfall for the transposed storm as that listed in the DAD tables.

(6) Fit-factor. -A fit-factor, F_f, that is, the ratio of the watershed average rainfall depth to the storm pattern rainfall depth, for equal areas, is computed for each transposed storm. The importance of the fit-factor to PMS estimates varies depending on the size, shape, and orientation with respect to major storm patterns of each individual watershed. In the example region, watersheds are typically long and narrow with their major axis oriented generally east-west, so that a fit-factor in this region is quite important, except for extremely large drainage basins.

If $\overline{P}_o$ represents the average rainfall depth for the total observed storm for a given area and $\overline{P}_{tr}$ represents the average rainfall depth measured from the isohyetal pattern of the observed storm, as transposed, then

$$F_f = \frac{\overline{P}_{tr}}{\overline{P}_o} \qquad (11)$$

It should be obvious that $F_f \leq 1$.

(7) Total maximization adjustment factor, Ad_f. -The total maximization adjustment factor, Ad_f, for a storm, as transposed to a watershed, is the product of the storm moisture maximization factor, M_f, and the fit-factor, F_f, or,

$$Ad_f = (M_f)(F_f) \qquad (12)$$

(8) Design storm values, depth-duration curve. -The maximized depth-duration values for each storm, as transposed to a watershed, are computed by multiplying the observed storm depth-duration values by the respective maximization adjustment factor, Ad_f. The computed values for each storm should be plotted with accumulative time in hours as the abscissa versus the accumulative rainfall depths in inches as the ordinate.

A design storm depth-duration curve is obtained by drawing a smooth curve. An enveloping curve will give design storm values approaching PMP for a watershed. A curve drawn through the data for one storm only will give selected PMS values.

Since the depth-duration curve is ordered in such a manner as to show only the maximum values of rainfall for various durations, the curve does not indicate a realistic sequence of rainfall increments which might occur during the actual design storm. Incremental design storm values obtained from the smooth depth-duration curve should be arranged in realistic sequence for flood computation.

For storms of long duration (several days), the design storm depth-duration curve may not be smooth throughout but have two or more periods of intense rainfall separated by periods of little or no rainfall. Such storms are frequently critical for very large basins or basins in tropical regions. In these instances, incremental design storm values may be arranged in any realistic sequence, within the limitation that the separate periods will not be so combined as to produce a rainfall sequence that would have exceeded the recommended design storm depth-duration curve at any point.

*(9) **Numerical computations.*** -Table G-9 presents numerical values for procedures described in the subsections above. Maps showing the transposed storm isohyetal patterns as fitted to the watershed and the planimetry notes for determination of average basin rainfall for each transposed storm are not included. A plot of depth-duration values of the transposed storms, as maximized, and the recommended depth-duration curve of the design storm are shown on figure G-9. In this instance, the design storm duration is 17 hours and rainfall values approach PMP. The enveloping curve on figure G-9 was drawn "by eye" as adequate for a preliminary PMS estimate. Design storm values read from the curve at l-hour intervals are listed in table G-10 because a flood hydrologist may wish to use a l-hour unitgraph to compute an inflow design flood hydrograph for this size watershed.

(b) ***Generalized Precipitation Charts.*** -Maps showing smoothed isohyets of PMP for the United States east of the 105° meridian and PMS values for the United States west of the 105° meridian are presented here to provide a means of quickly obtaining ***preliminary design storm values*** for selected watersheds above proposed damsites. It is impossible to show on the generalized charts all of the refinements and variations that can influence the magnitude of design storm values for individual watersheds. Design storm values obtained from the generalized charts represent a reasonable upper limit and, in most instances, will exceed the values obtained for a specific watershed by a detailed hydrometeorological study, as previously discussed.

(1) ***Generalized chart for the United States east of the 105° meridian.*** -Figure G-10 shows probable maximum 6-hour precipitation values for any area of 10 square miles for the United States east of the 105° meridian. This chart is based on one presented in Hydrometeorological Report No. 33, prepared by the Hydrometeorological Section of the National Weather Service in collaboration with the U.S. Army Corps of Engineers [20]. These 6-hour values for IO-square-mile areas can be modified for durations in excess of 6 hours and for larger areas up to 1,000 square miles by use of figure G-l 1. No variation is assumed between point and 1 O-square-mile precipitation. For durations shorter than 6 hours, the time distribution of precipitation can be obtained from curve C, figure G-12. Subsequent to the publication of Hydrometeorological Report No. 33, the Corps of Engineers have recommended[7] that the following adjustment percentages be applied to the depth-duration values obtained from figure G-l 0 in order to provide for the imperfect ***fit*** of the isohyetal patterns of observed storms to the shape of a particular basin.

Drainage area, square miles	*Adjustment factor applicable to H.R. 33 rainfall values, percent*
1,000	90
500	90
200	89
100	87
50	85
10	80

(2) Generalized chart for the United States west of the 105° meridian. -Figure G- 13 shows probable maximum 6-hour point general-type storm values for areas of the United States west of the 105° meridian. This chart is based on the results of approximately 330 design storm analyses prepared by the Bureau of Reclamation for specific drainage basins west of the 105° meridian, as well as consideration of numerous design storm analyses made by the Special Studies and Hydrometeorological Branches of the National Weather Service.

The variable topography of this part of the United, States greatly influences the storm potential and permits only limited transposition of storms. These point storm values can be applied to areas up to 1,000 square miles by use of the curves presented on figure G-14. The 6-hour general-type storm values can be extended for longer duration periods by multiplying the 6-hour value by the

'Engineer Circular No. 1110-z-27, dated August 1, 1966, "Policies and Procedures Pertaining to Determination of Spillway Capacities and Freeboard Allowances for Dams."

Table *G-9.-Example of design storm derivation for area east of 105° meridian*

BASIC DATA:
Watershed location: 99°30′ W, 41°00′ N
Drainage area: 200 sq. mi.
Inflow barrier: 2,500 feet

(A) MAJOR STORMS SELECTED FOR TRANSPOSITION

Designation No.	Approximate geographic location-name	Date of storm	Inflow barrier, feet	Observed storm dewpoint °F.[1]	Observed storm dewpoint Ref. pt.	Total storm Duration, hrs.	Total storm [2]$\overline{P}_o$	Reference
MR4-24	Boyden, Iowa	9/17-19/26	1,200	70	175 mi. SSE	54	16.6	Fig. G-8A
MR4-5	Grant Township, Nebr.	6/3-4/40	1,200	66[3]	120 mi. S	20	11.2	[21]
MR6-15	Stanton, Nebr.	6/10-13/44	1,500	70	125 mi. SSE	78	14.4	[21]
R10-1-1[4]	Greeley, Nebr.	8/12-13/66	2,000	71	80 mi. SSE	17	13.3	[4]

[1] 1,000 millibars, or mean sea level.
[2] Average rainfall depth, 200 sq. mi.
[3] Revised value in lieu of 63° F. [21]
[4] Recent storm analysis, preliminary, Bureau of Reclamation, Engineering and Research Center, Denver, Colo.

(B) STORM TRANSPOSITION AND MAXIMIZATION

(Column heading symbols as previously defined in text.)

Storm No.	Observed storm: Dwpt., °F.	Barrier, feet	$W_{p\text{-}1}$	$W_{p\text{-}2}$	W_s	$\overline{P}_o$	Transposed storms: Dwpt.,[1] °F.	Barrier, feet	$W_{r\text{-}1}$	$W_{r\text{-}3}$	W_x	$\overline{P}_{tr}$	Maximizing factors: M_f	F_f	Ad_f
MR4-24	70	1,200	2.21	0.25	2.02	16.6	76	2,500	3.04	0.62	2.42	12.3	1.20	0.74	0.89
MR4-5	66	1,200	1.86	.22	1.64	11.2	76	2,500	3.04	.62	2.42	9.6	1.48	.86	1.27
MR6-15	70	1,500	2.27	.31	1.96	14.4	76	2,500	3.04	.62	2.42	13.0	1.23	.90	1.11
R10-1-1	71	2,000	2.38	.42	1.96	13.4	77	2,500	3.19	.64	2.55	12.4	1.30	.93	1.21

[1] From Climatic Atlas of United States [19]

(C) MAXIMUM OBSERVED DEPTHS, INCHES

Storm	Duration in hours: 3	6	9	12	15	18	24	30	36	48	60	72
MR4-24		11.7		15.8		16.6	16.6	16.6	16.6	16.6	[1]16.6	
MR4-5	5.5	9.6	11.1	11.2	11.2	11.2	[2]11.2					
MR6-15		11.1		12.9		12.9	12.9	12.9	13.1	14.1	14.3	[3]14.4
R10-1-1	6.7	9.4	12.5	13.1	13.2	[4]13.4						

[1] Storm ended at 54 hrs.
[2] Storm ended at 20 hrs.
[3] Storm ended at 78 hrs., depth = 14.4 in.
[4] Storm ended at 17 hrs.

(D) MAXIMUM TRANSPOSED DEPTHS, INCHES

Storm No.	Ad_f	Duration in hours: 3	6	9	12	15	18	24	30	36	48	60	72
MR4-24	0.89		10.4		14.1		14.8	14.8	14.8	14.8	14.8	[1]14.8	
MR4-5	1.27	7.0	12.2	14.1	14.2	14.2	14.2	[2]14.2					
MR6-15	1.11		12.3		14.3		14.3	14.3	14.3	14.5	15.7	15.9	[3]16.0
R10-1-1	1.21	8.1	11.4	15.1	15.9	16.0	[4]16.2						

[1] At 54 hrs.
[2] At 20 hrs.
[3] Also at 78 hrs.
[4] At 17 hrs.

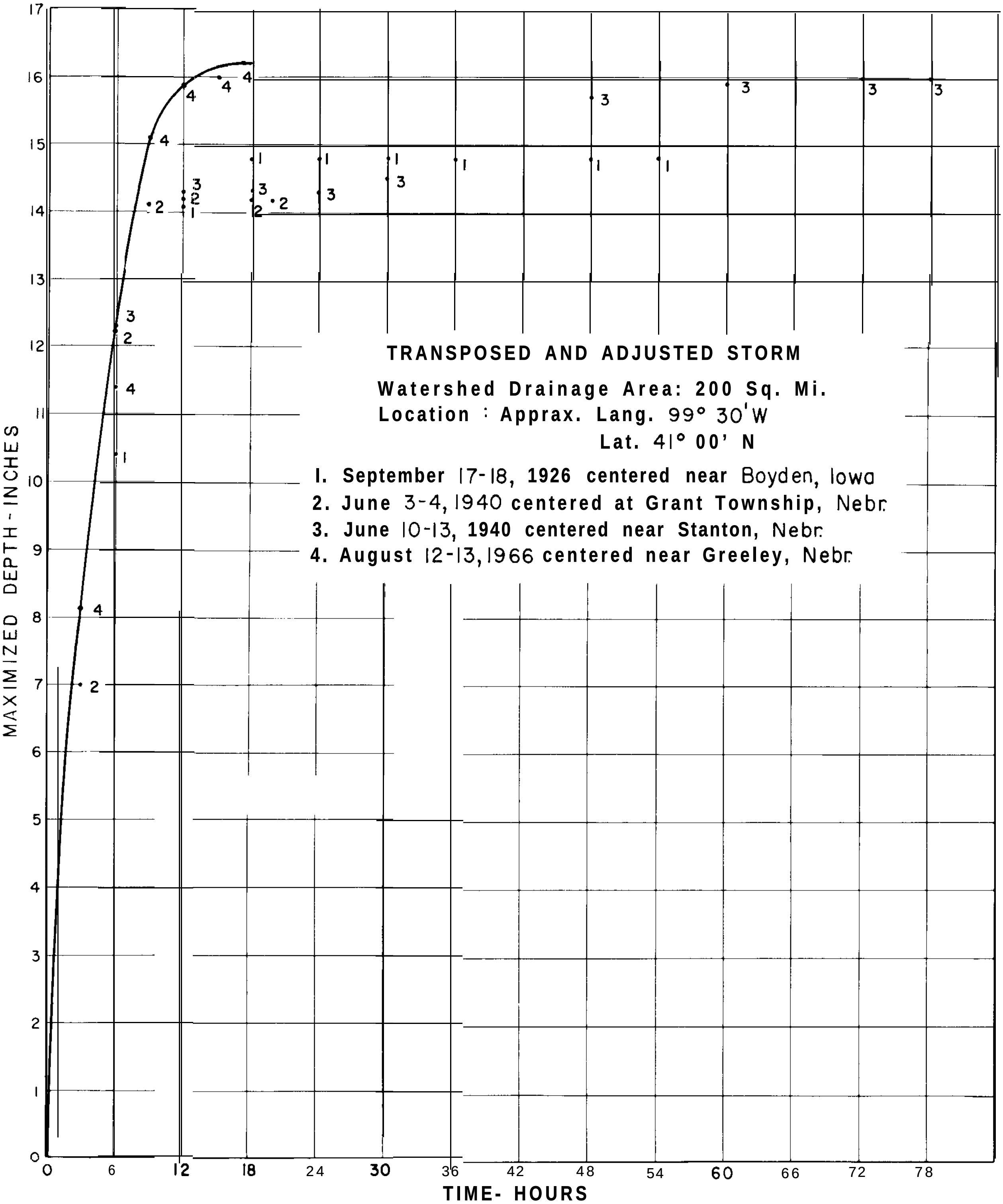


Figure G-9. Design storm-depth-duration values.-288-D-3185

Table ***G-lO.-Design storm depth-duration values, inches***

BASIC DATA: Hypothetical example.
Watershed area = 200 sq. mi.
Location = approximately 99°30′ W, 41°00′ N

Time, ending at hour	Accumulated depth, inches	Incremental depth, inches
0	*0*	0
	4.20	4.20
2	6.40	2.20
3	8.10	1.70
4	9.70	1.60
5	11.10	1.40
6	12.30	1.20
	13.30	1.00
8	14.30	1.00
9	15.10	.80
10	15.45	.35
11	15.70	.25
12	15.90	.20
13	16.00	.10
14	16.10	.10
15	16.15	.05
16	16.20	.05
17	16.20	
18	16.20	8

appropriate factor shown in table G-l 1. Values for duration of less than 6 hours can be obtained from the appropriate curve of figure G-12.

(3) Use of generalized charts. -Design storm values for any watershed of a 1,000-square-mile area or less in the conterminous 48 United States may be obtained from the generalized charts, but it must be noted that such design storm values should be considered as only preliminary estimates for watersheds controlled by large dams. Design storm values obtained from figures G-l 0 and G-13 show considerable difference at their common boundary along the 105° meridian. This is due to the techniques used in determining the values shown on the charts.

Preliminary design storm values for a particular watershed obtained from either generalized chart should be plotted on coordinate paper and an enveloping depth-duration curve drawn. Plotting offers a method of checking the computations, as a smooth curve should be indicated, and also provides the means of obtaining hourly design storm values for the total storm period if needed. Incremental values from the depth-duration curve may be arranged in any sequence desired by a flood hydrologist for computation of a preliminary inflow design flood.

The generalization charts for estimating preliminary design storm values have been limited to an area of 1,000 square miles because generalizations of criteria become more difficult as the size of the area increases. Preliminary design storm estimates can be made for areas greater than 1,000 square miles in regions of nonorographic rainfall by the procedure described in section G-l 7. The step of determining a fit-factor is omitted. A depth-duration curve is drawn on the basis of information compiled in a tabulation such as table G-9(D), using the moisture maximization factor, M_f, instead of the total adjustment factor, Ad_f, to compute values for the table. Preliminary design storm estimates for large mountainous basins (with predominately orographic rainfall) should be obtained from a hydrometeorologist.

F. PRELIMINARY INFLOW DESIGN FLOOD, RAINFALL ONLY

G- 18. General.-This subchapter outlines procedures for estimating preliminary inflow design flood (IDF) hydrographs using: (1) design storm values from the generalized precipitation charts, figures G-10 and G-13; (2) an estimation of incremental rainfall excesses from runoff curves, section G-7(b)(6); and (3) the lag-time dimensionless-graph method of obtaining unitgraphs, section G-9. An example is given of computation of preliminary inflow design flood hydrographs for a watershed east of the 105° meridian, with accompanying

Figure **G-IO. Probable maximum precipitation (inches) east of the 105° meridian for an area of 10 square miles and 6 hours' duration.-288-D-3191**

discussions directed toward considerations applicable to all inflow design flood studies. Procedures applicable to watersheds west of the 105° are outlined. A discussion of preparing recommendations for routing preliminary inflow design flood hydrographs through proposed reservoirs concludes this presentation.

G 19. Example-Preliminary Inflow Design Flood Hydrographs, Watersheds East of 105° Meridian. -A hypothetical watershed in a general location east of the 105° meridian has been assumed in order to illustrate several of the problems encountered in IDF computations, all of which would not likely be presented by a specifically located watershed.

(a) ***Basin Description.-A*** map of the assumed watershed above a proposed damsite is shown on figure G-15. The center of the basin is assumed to be located in zone 4 somewhere along the 30-inch, 6-hour PMP for 1 O-square-mile isohyet, figure G- 10. An outline

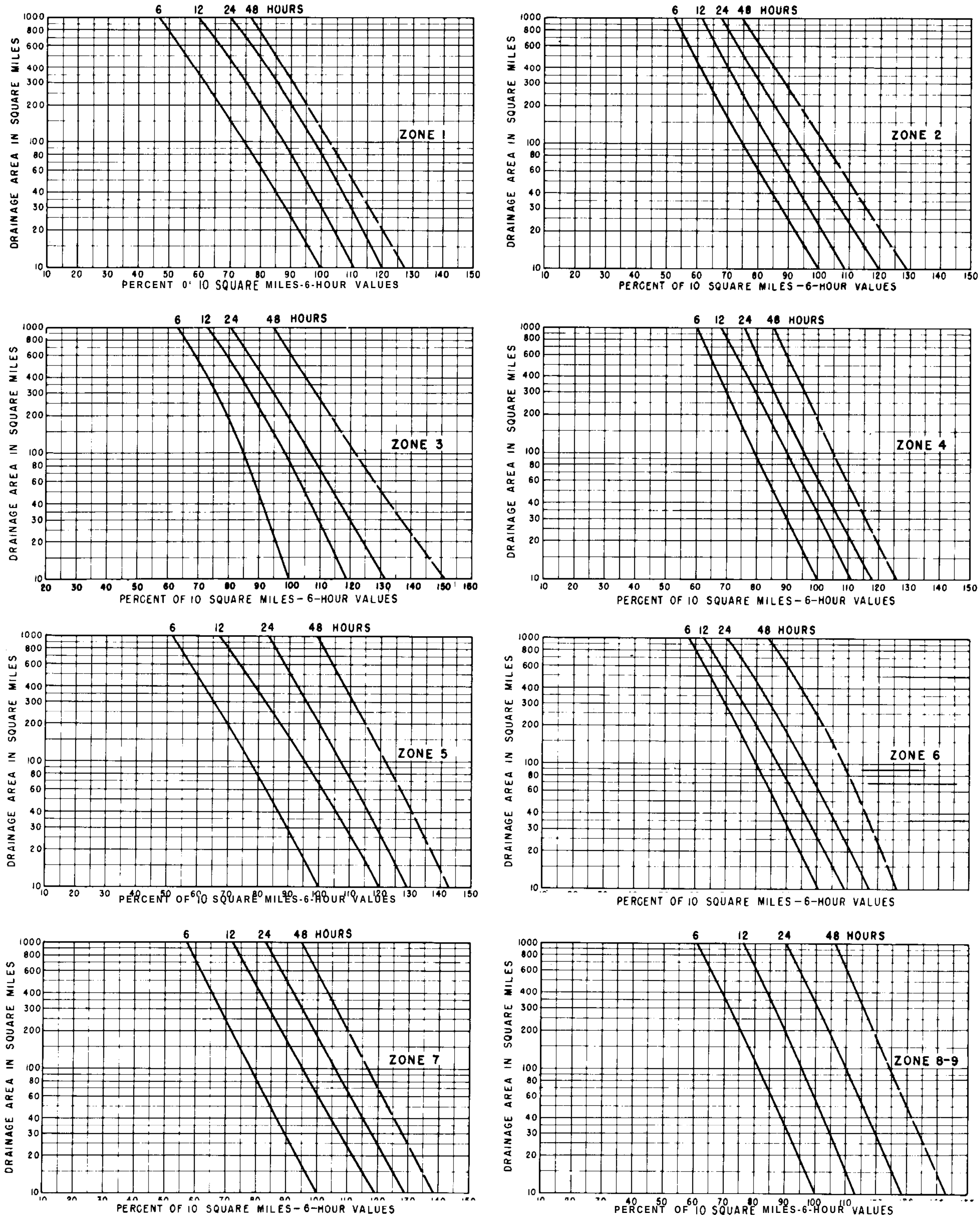


Figure G-11. Depth-area-duration relationships-percentage to be applied to 10 square miles, 6-hour probable maximum precipitation values.-288-D-2450

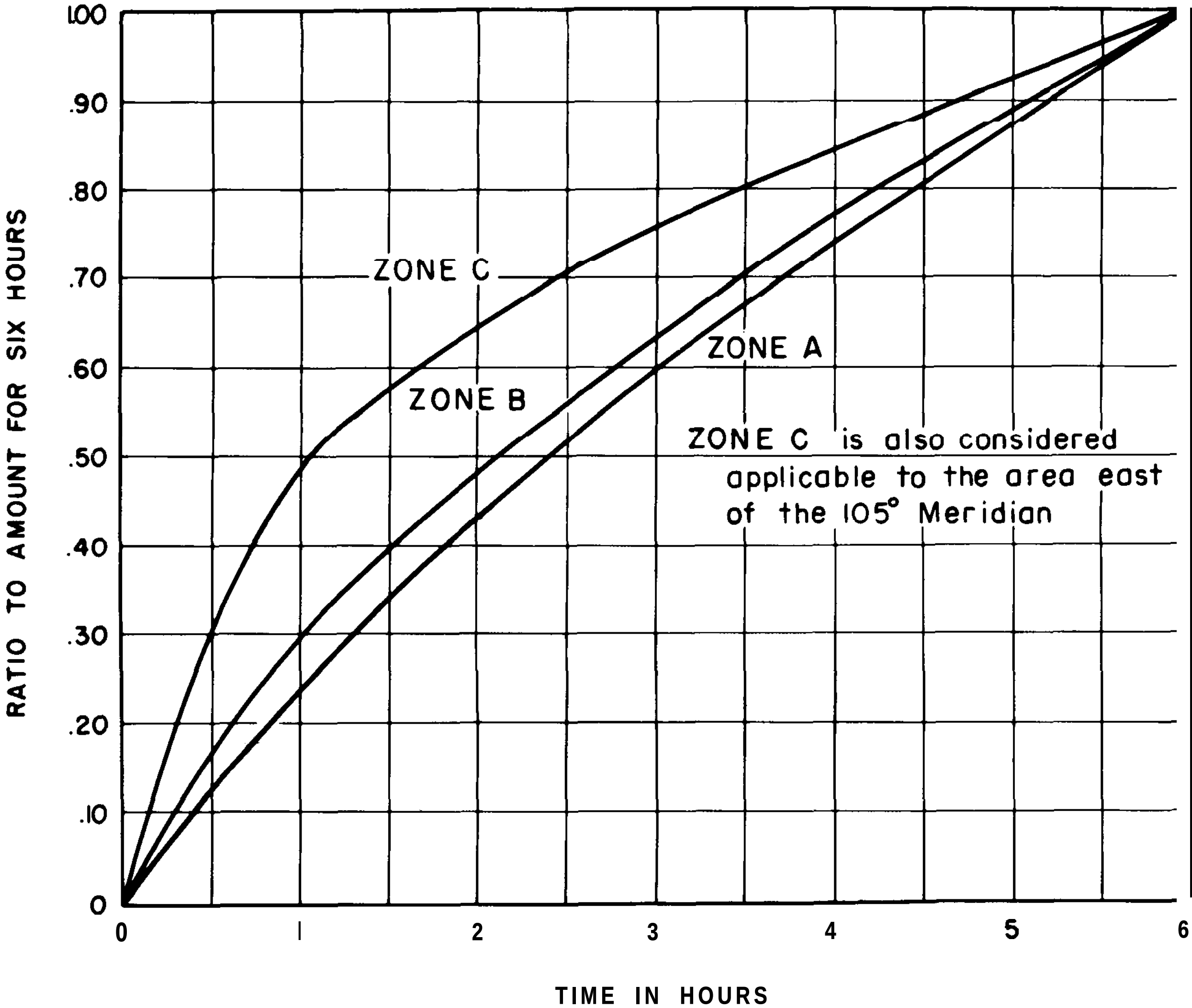


Figure G-12. Distribution of 6-hour rainfall for area west of 105° meridian (see fig. G-13 for area included in each zone).-288-D-2758

of the proposed reservoir surface at normal water storage capacity is shown, because the length of natural stream channels to be submerged influences lag-time calculations. It is assumed that runoff characteristics of the areas drained by the two main tributaries differ enough to warrant consideration of dividing the watershed into two subareas, A and B, as there is information available indicating that subarea A definitely has rapid runoff characteristics. All of the area enclosed by the natural divides contributes runoff.

(1) ***Drainage areas*** are:

Total basin	800 square miles
Subarea A	240 square miles
Subarea B	560 square miles
Reservoir surface	26 square miles

As the reservoir surface area is about 3 percent of the total basin area in this example, reservoir surface may be considered as land area, except for lag-time computations. Whenever there is found a reservoir surface area of about 10 percent or more of total contributing drainage area, computations should be made separately of the runoff originating from the land area, to which

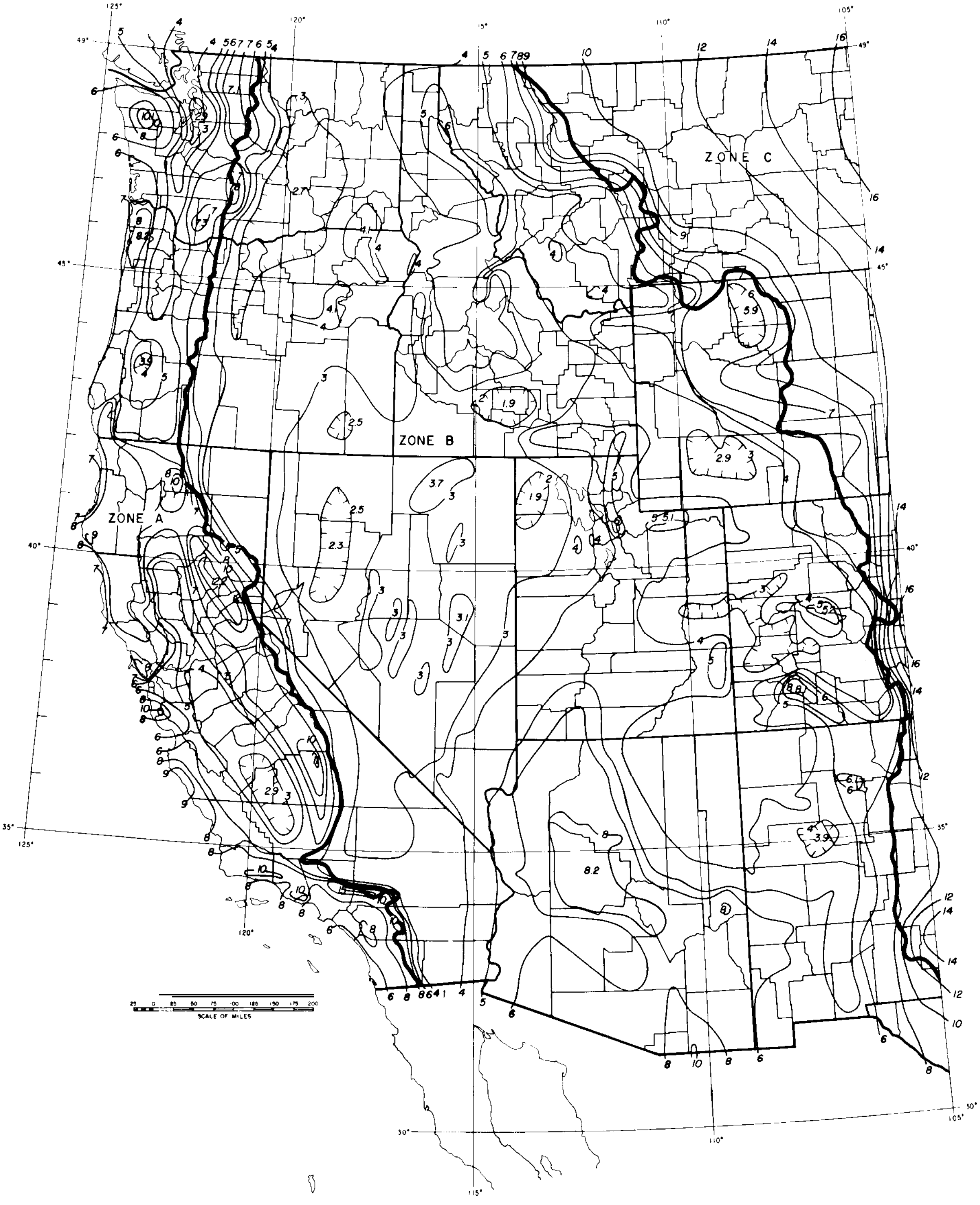


Figure G-13. **Probable maximum 6-hour point precipitation values in inches for general-type storms west of the 105^{o} meridian.-288-D-3192**

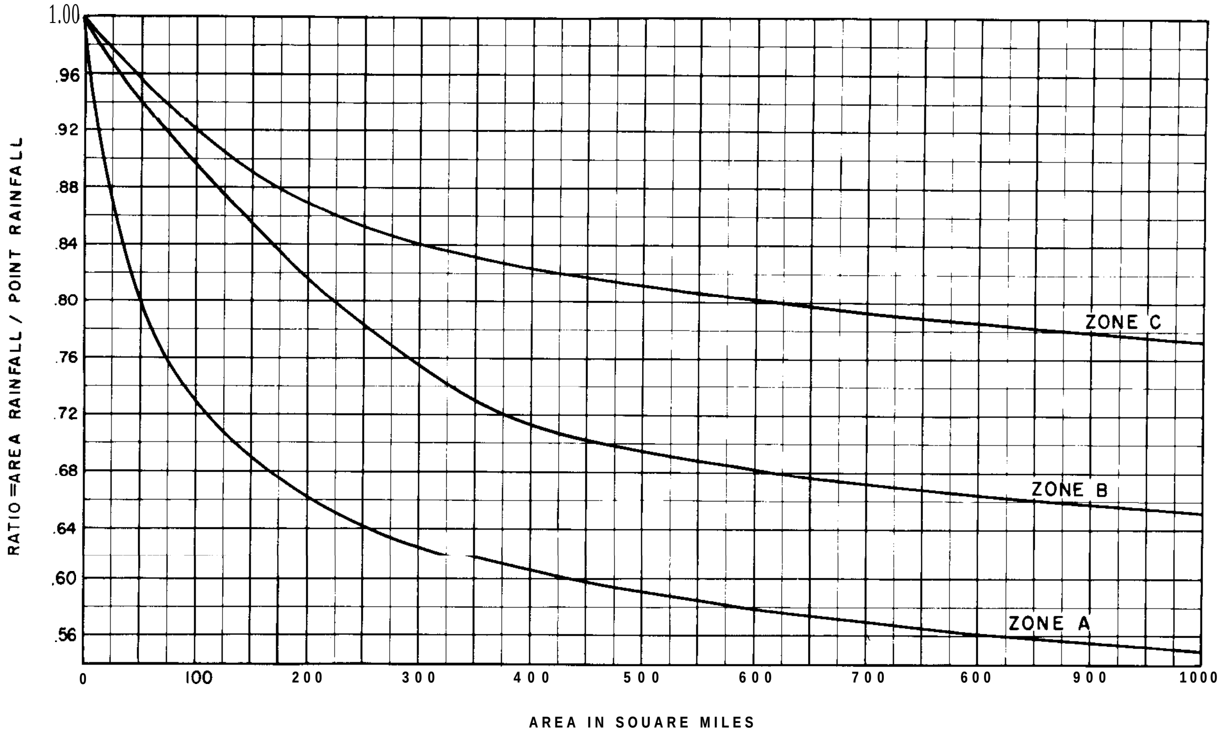


Figure G-14. General-type storm-conversion ratio from 6-hour point rainfall to area rainfall for area west of 105° meridian.—288-D-2759

Table G-11 .-*Constants for extending 6-hour general-type design-storm values west of 105° meridian to longer duration periods*[1]

Duration, hours[2]	Constants		
	Zone A	Zone B	Zone C
8	1.20	1.18	1.14
10	1.39	1.36	1.26
12	1.58	1.53	1.36
14	1.76	1.66	1.43
16	1.93	1.77	1.50
18	2.10	1.87	1.57
20	2.26	1.95	1.64
22	2.42	2.03	1.71
24	2.57	2.10	1.78
30	2.95	2.28	1.97
36	3.26	2.38	2.15
42	3.55	2.40	2.25
48	3.79	2.41	2.28
60	4.14		
72	4.34		

[1]Multiply 6-hour point rainfall from figure G-13 by indicated constant.
[2]For durations shorter than 6 hours, the time distribution of storm values can be obtained from the appropriate curve presented on figure G-12.

retention losses are applicable to design storm rainfall, and the increased inflow to the reservoir due to design rainfall on the reservoir surface area where retention losses are zero. There are instances where rain falling on reservoir surfaces supplies the major portion of inflow. When rain falling on a reservoir surface must be considered, rainfall increments in inches are converted to equivalent incremental flow in cubic feet per second and combined with respectively timed increments of inflow from the land area. Watersheds in which a reservoir will submerge miles of mainstream channel, and numerous side tributaries flow directly into the reservoir, the watershed should be divided into at least two subareas, the subarea above the head of the reservoir and the area directly tributary to the reservoir. Subarea B, figure G 15, approaches this situation. If a final-type IDF study were made for the example watershed, a better evaluation of a final-type IDF would be obtained by dividing subarea B into two subbasins and

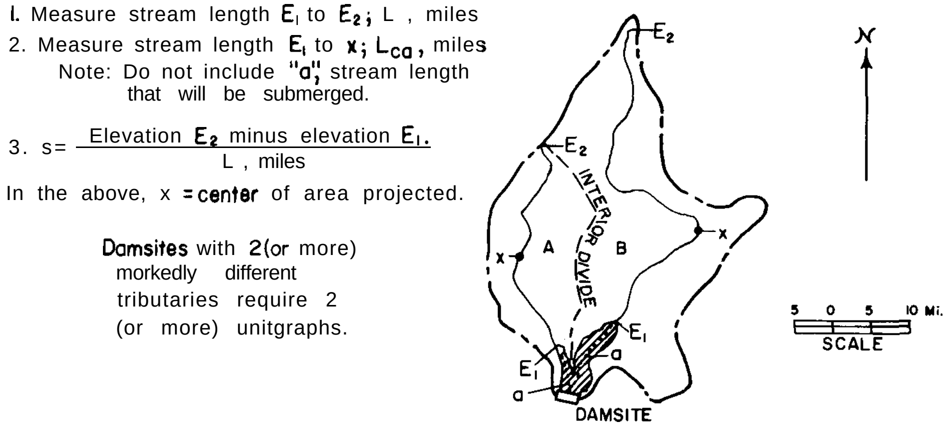


Figure G-15. Basin map-example of preliminary inflow design flood computation.-288-D-3186

deriving a unitgraph for each; the subbasins would be above and below the head of the reservoir, point E_1, figure G- 15.

(2) *Streamflow* records.-Two assumptions are made for lag-time illustrating purposes: first, that there are no streamflow records available for analysis; second, that tributary B has been gaged at the mouth near the damsite, and hydrograph analyses have indicated a lag-time of 22 hours for subarea B.

(3) ***Soils*** and cover.-Use of runoff curves requires hydrologic classification of watershed soils and cover, discussed in section G-7(b)(6), for selection of applicable runoff curve number, These classifications are made by field inspections, examination of soils maps, etc. For this example, it is assumed available information indicates:

Subarea A:

Soils, hydrologic group C
Land use, mostly poor pasture
Runoff curve, AMC-II CN86 (table G-3(A))

Subarea B:

Soils, hydrologic group B
Land use, mostly small grain, contour terraced
Runoff curve, AMC-II CN70 (table G-3(A))

(b) ***Dimensionless-Graph*** Selection.-As hydrograph analyses cannot be made in the first instance because of lack of streamflow records, a dimensionless-graph must be selected from other sources. The dimensionless-graph shown as (C), figure G-6, which was derived from a flood hydrograph in the general region of the assumed location of the watershed, has been selected as applicable to both subareas of the watershed. It is also used in the second example, where streamflow records are available.

(c) Lug-Times.-A cutout of each subarea, including the respective reservoir portion in each, was made, the center of area of each determined and projected to the main streams at the points marked x on the stream channels as shown on figure G-1 5 (see sec. G-9(2)). Longest watercourse lengths listed below were measured from the map. Slope values for this example, S in feet per mile, were selected from general data. In the usual study, elevations for computing slope values for a given watershed are obtained from topographic maps.

Subarea A:

L = 29.0 miles from head of reservoir to divide, E_1 to E_2, figure G- 15.
L_{ca} = 12.7 miles from head of reservoir to center of area projected, E_1 to x, figure G 15.
S = 23.2 feet per mile (assumed in this example).

Subarea B: (Assumption of no streamflow records.)

L = 48.9 miles from head of reservoir to divide, E_1 to E_2, figure G-15.
L_{ca} = 15.4 miles from head of reservoir to center of area (projected), E_1 to x, figure G-15.
S = 12.6 feet per mile (assumed for this example).

For use in assumption that streamflow records have indicated a lag-time of 22 hours for tributary B:

L = 59.8 miles from mouth (gage) to divide.
L_{ca} = 26.3 miles from mouth (gage) to center of area, x.
S = 16.5 feet per mile (assumed for this example).

Two sets of lag-times are estimated for this example on the basis of the two assumptions regarding available streamflow records. Under the assumption that no streamflow records are available, the generalized lag-time equation is considered applicable.

$$\text{Lag-time hours} = 1.6\left[\frac{LL_{ca}}{\sqrt{S}}\right]^{0.33} \text{(Sec. G-8(e)(2).)}$$

Estimated lag-times are:

Subarea A:

$$\frac{LL_{ca}}{\sqrt{S}} = \frac{(29.4)(12.7)}{\sqrt{23.2}} = 77.5$$

Lag-time = 6.7 hours.

Subarea B:

$$\frac{LL_{ca}}{\sqrt{S}} = \frac{(48.9)(15.4)}{\sqrt{12.6}} = 212.2$$

Lag-time = 9.4 hours,

Under the assumption that hydrograph analyses for streamflow gaged near the mouth of tributary B indicates a lag-time of 22 hours for subarea B, the following lag-times are estimated:

Subarea A:

No change, lag-time = 6.7 hours.

Subarea B:

Referring to section G-8(e)(2), if a reliable lag-time for a basin is found by hydrograph analyses at a gaging station, a lag-time for an ungaged portion of the basin may be obtained by passing a curve with slope 0.33 through the point plotted on log-log paper, $\frac{LL_{ca}}{\sqrt{S}}$ versus lag hours. An $\frac{LL_{ca}}{\sqrt{S}}$ value for subarea B above the assumed gaging station is:

$$\frac{(59.8)(26.3)}{\sqrt{16.5}} = 386.7$$

If the generalized lag-time curve has been plotted on log-log paper, plot 387 versus the lag-time of 22 hours and draw a line through the plotted point parallel to the generalized lag-time curve. Read a lag-time of 18 hours for the $\frac{LL_{ca}}{\sqrt{S}}$-value of 212 from the constructed curve. In this example, the proposed reservoir has the effect of reducing the lag-time for subarea B from 22 hours for natural conditions to 18 hours after the dam is built. The effect of a proposed reservoir on natural lag-times should not be overlooked in the preparation of inflow design flood hydrographs.

Of course, the lag-time of 18.0 hours can also be obtained without plotting the curves, by solving the equation,

$$\text{Lag-time} = C\left[\frac{LL_{ca}}{\sqrt{S}}\right]^{0.33}$$

for C, substituting 22 hours for lag-time and 386.7 for $\frac{LL_{ca}}{\sqrt{S}}$; this gives C = 3.08. Then, using this computed value for C, and 212.2 for $\frac{LL_{ca}}{\sqrt{S}}$, lag-time in hours equals 18.0.

(d) *Preliminary Design Storm Values.*-A specific watershed location is identified on the generalized charts, figures G-10 and G-13, by county boundaries within the States and reading the zone and 6-hour PMP values applicable to the watershed. A specific location for the watershed for this example has not been designated other than it is assumed to be in zone 4 where 6-hour probable maximum precipitation (PMP) for 10 square miles is 30 inches (figure G 10). Computation of preliminary design storm values are shown in table G-12. The design storm is assumed to cover the entire watershed area of 800 square miles. Percentages of the 6-hour PMP for 10 square miles applicable to 800 square miles were read from the depth-area-duration relationships on the chart for zone 4, figure G-11, and PMP values for 6, 12, 24, and 48 hours for 800 square miles computed. These values were adjusted to 90 percent of the computed values in accordance with the fit adjustment factors given in section G-17(b)(1). Hourly depth-duration values for the maximum 6-hour period of the storm were computed by percentages read from curve C on figure G-12. Depth-duration values, line 5 of table G-12, were plotted and a preliminary design storm depth-duration curve drawn as shown on figure G-16.

(e) *Arrangement of Design Storm Rainfall Increments and Computation* ***of*** *Increments* ***of*** *Rainfall Excess.* -Arrangement of increments of rainfall of a preliminary design storm estimated from figure G-10 is illustrated in table G-13, along with the computation of respective increments of excess rainfall. Computation of table G-13 is explained in the following paragraphs. General comments on design storm arrangements are included.

(1) *Selection* ***of*** *design storm unit time interval.* -Design storm increments and respective rainfall excesses obtained therefrom must be for the same unit time interval as the unitgraph to which the excesses will be applied to compute an inflow design flood (IDF) hydrograph. Unit time of a unitgraph is related to the lag-time of a basin, being one-fourth or less of the lag-time (sec. G-9(6)). In this example, a l-hour unitgraph is required for subarea A because a lag-time of 6.7 hours has been estimated for that subarea. A 2-hour unitgraph could be used for subarea B, lag-time 9.4 hours. However, the computed hydrographs for the two subareas must be combined to give the preliminary inflow design flood hydrograph. A better definition of the IDF hydrograph will be obtained if the unitgraphs for the two subareas have the same unit time interval. A l-hour unitgraph for each subarea was used in this example. Hourly values of preliminary design storm rainfall were read to the nearest tenth inch from the depth-duration curve, figure G-16, from 1 to 24 hours and tabulated in column 2 of table G 13. Hourly increments of rainfall are listed in column 3 of table G- 13.

Table G-12.–*Preliminary design storm estimate for hypothetical watershed, east of 105° meridian*

BASIC DATA:
Location: Hypothetical
Reference: Figure G-10, zone 4, 6-hr. PMP[1], 10 sq. mi.: 30 inches
Areas: Total basin, 800 sq. mi.; subarea A, 240 sq. mi.; subarea B, 560 sq. mi.

	Item	Time in hours									Text reference
		1	2	3	4	5	6	12	24	48	
1.	Percent of 6-hr. PMP[1] for 800 sq. mi.						62	70	77	87	Fig. G-11
2.	Computed PMP, 800 sq.mi., inches						18.6	21.0	23.1	26.1	
3.	PMP, adjusted to 90 percent						16.7	18.9	20.8	23.5	Sec.G-17(b)(l)
4.	Ratios to 6-hr. rainfall	0.49	0.64	0.75	0.85	0.93	1.00				Fig. G-12, zone C
5.	Design PMP, 800 sq. mi., inches	8.2	10.7	12.5	14.2	15.5	16.7	18.9	20.8	23.5	Fig.G-16

[1]PMP = probable maximum precipitation.

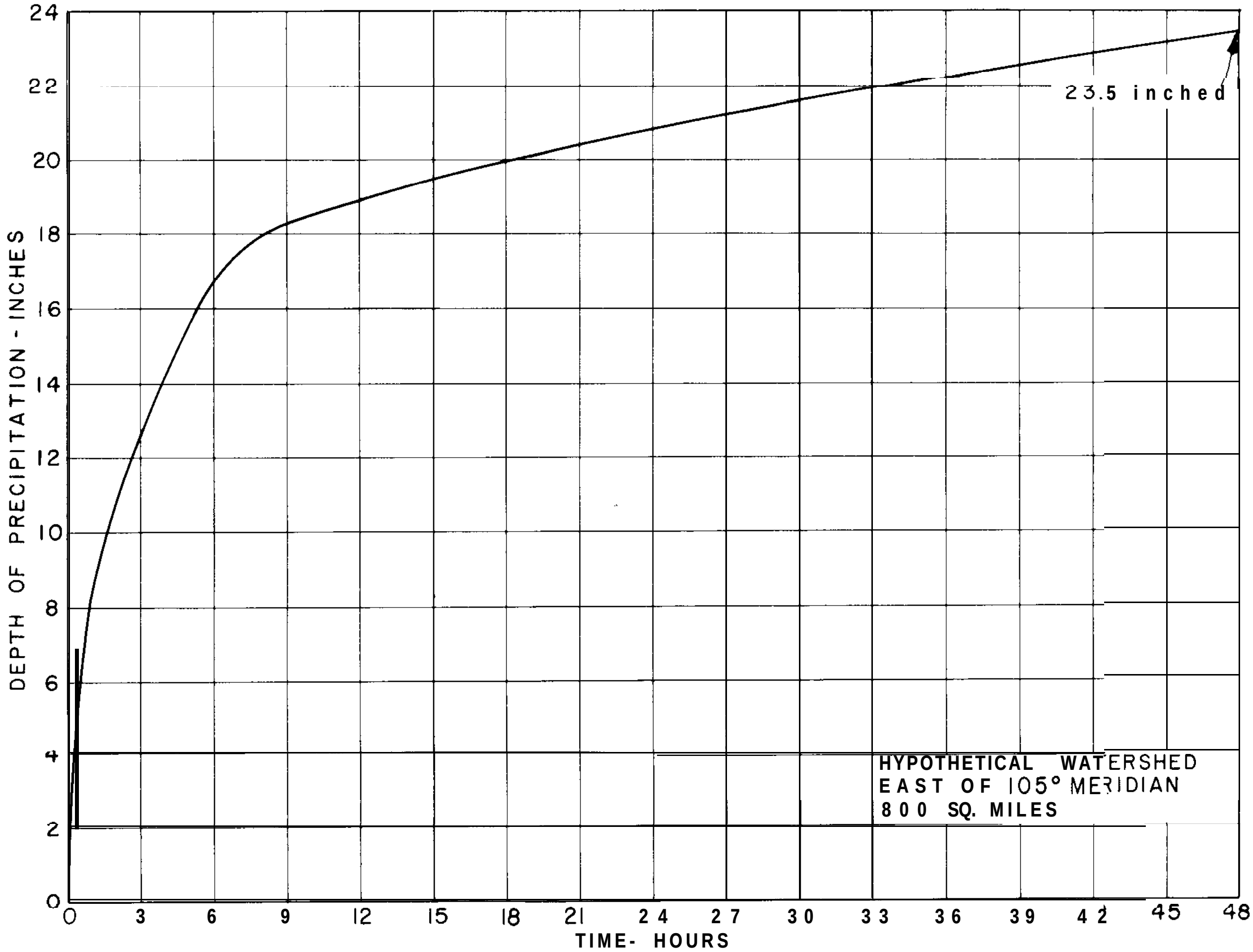


Figure G-16. **Preliminary design storm-depth-duration curve.-288-D-3187**

(2) Arrangement of design storm incremental rainfall. -Normally, the arrangement with respect to time of increments of design storm rainfall is not established in a design storm study (sec. G-17(a)(8)). Flood hydrologists arrange design storm increments to give rainfall excesses that produce the most critical inflow design flood hydrograph. Except for basins having several thousands of square miles of drainage area, design storm rainfall is assumed to occur with the same time sequence over the total watershed area. If a constant retention loss rate is used to compute rainfall excesses, a critical arrangement may be easily found by arranging design storm increments opposite the ordinates of the unitgraph for the basin, so that the largest rainfall increment (which would give the largest excess increment) is opposite the largest ordinate; and the second largest rainfall increment is opposite the second largest ordinate, etc.

This arrangement is then reversed to give the design storm arrangement in correct time sequence, because rainfall excesses are reversed in sequence of natural occurrence when being applied to unitgraph ordinates by calculators. Otherwise, much additional work must be done: (1) computing discharges for each ordinate of the unitgraph for each excess increment; (2) tabulating the individual discharges in correct time sequence; and (3) adding respectively timed incremental discharges to get the total flood hydrograph. If a retention loss rate which varies with time is used, a critical design storm arrangement is found by trial.

Table G-13.–*Preliminary design storm east of 105° meridian-arrangement of incremental rainfall; computation of incremental excesses, ΔP_e, for subareas A and B*

BASIC DATA:
Total area (for design storm estimate)-800 sq. mi.
Subarea size and retention data:
Subarea A: 240 sq. mi.; CN 86, selected minimum loss rate, 0.12 in./hr.
Subarea B: 560 sq. mi.; CN 70, selected minimum loss rate, 0.24 in./hr.

1	2	3	4	5	6	7	8	9	10	11
Time, ending hour	Design rainfall depth duration		Arrangement of design rainfall		Rainfall excesses, P_e					
					Subarea A			Subarea B		
	ΣP, inches	ΔP, inches	ΔP, inches	ΣP, Inches	[2] ΣP_e, inches	ΔP_e, inches	A loss, inches	[3] ΣP_e, inches	ΔP_e, inches	A loss, inches
1	8.2	8.2	1.2	1.2	0.30	0.30	0.90	0.02	0.02	1.18
2	10.7	2.5	1.7	2.9	1.57	1.27	.43	.66	.64	1.06
3	12.5	1.8	1.8	4.7	3.18	1.61	.19	1.82	1.16	.64
4	14.2	1.7	8.2	12.9	11.13	7.95	.25	8.88	1.06	1.14
5	15.5	1.3	2.5	15.4	13.51	2.38	[4] .12	11.14	2.26	[5] .24
6	16.7	1.2	1.3	16.7	14.69	1.18	.12	12.20	1.06	.24
7	17.4	.7	.7	17.4	15.27	.58	.12	12.66	.46	.24
8	17.9	.5	.5	17.9	15.65	.38	.12	12.92	.26	.24
9	18.2	.3	.3	18.2	15.83	.18	.12	12.98	.06	.24
10	18.5	.3	.3	18.5	16.01	.18	.12	13.04	.06	.24
11	18.7	.2	.2	18.7	16.09	.08	.12	13.04	0	.24
12	18.9	.2	.2	18.9	16.17	.08	.12			
13	19.1	.2	.2	19.1	16.25	.08	.12			
14	19.3	.2	.2	19.3	16.33	.08	.12			
15	19.5	.2	.2	19.5	16.41	.08	.10			
16	19.6	.1	.1	19.6	16.41	0	.12			
17	19.8	.2	.2	19.8	16.49	.08	.12			
18	20.0	.2	.2	20.0	16.57	.08	.12			
19	20.1	.1	.1	20.1	[6]	[6]				
20	20.2	.1	.1	20.2						
21	20.4	.2	.2	20.4						
22	20.6	.2	.2	20.6						
23	20.7	.1	.1	20.7						
24	[1] 20.8	.1	.1	20.8						

[1] Balance of design rainfall considered lost to retention.

[2] By equation $\Sigma P_e = \frac{(P - 0.2S)^2}{(P + 0.8S)}$ for CN 86, S = 1.63; $0.2S$ = 0.33, $0.8S$ = 1.30 (table G-4).

[3] By above equation, for CN 70, S = 4.28; $0.2S$ = 0.86, $0.8S$ = 3.42 (table G-4).

[4] ΔP_e by CN 86 indicates A loss = 0.03 in., which is less than 0.12 in. Use 0.12 in. loss/hr.

[5] ΔP_e by CN 70 indicates A loss = 0.15 in., which is less than 0.24 in. Use 0.24 in. loss/hr.

[6] Total of remaining excess not significant for preliminary IDF.

A definite arrangement of design storm increments has been specified for preliminary design storm values obtained from each generalized precipitation chart, figures G-10 and G-13, because the selected general method of computing rainfall excesses using rainfall runoff curves has "built-in" varying retention loss rates. The arrangement specified for preliminary design storm values east of the 105° meridian is illustrated by the arrangement of rainfall increments in column 4, table G-13. The maximum 6-hour period of design rainfall is assumed to occur during the first 6-hour period of the design storm. Hourly precipitation amounts within the maximum 6-hour period are arranged in the following order of magnitude: 6, 4, 3, 1, 2, 5. Increments of design storm rainfall after the sixth hour decrease and are taken directly from the design storm depth-duration curve.

(3) Computation of increments of rainfall excess. -The method of estimating excess rainfall increments given in section G-7(b)(6) has been taken from the SCS National Engineering Handbook [3] with the following modifications introduced to give a procedure applicable to preliminary design storm rainfall obtained from generalized precipitation charts.

The rainfall-runoff relationships shown by the curves of figure G-2 were developed by Soil Conservation Service hydrologists from analyses of rainfall and respective runoff records at numerous small area experimental watersheds. The relationships were developed for use with daily nonrecording rainfall data, which are more plentiful in the United States than are recording rainfall data. Data used in the development are totals for one or more storms occurring in a calendar day and nothing is known about their time distributions. The relationships developed, therefore, exclude time as an explicit variable which means that rainfall intensity is ignored.

Strict adherence to use of the runoff curves on figure G-2 results in hourly runoff increments almost equal to hourly precipitation increments after a few hours for many of the design storm values obtained from generalized precipitation charts. Infiltrometer studies indicate that all but impervious clay soils have minimum constant infiltration rates after saturation that may range from 0.05 inch per hour to greater than 1.00 inch per hour, depending on the type of soil. Therefore, to utilize the rainfall-runoff relationships in the computational procedures given in this text, time-sequences of incremental rainfall for a design storm are specified and precipitation excesses are then computed using the runoff curve relationships, with the provision that hourly retention rates indicated by use of the runoff curves be tabulated for each hourly rainfall increment. Progressively through the arranged precipitation sequence, these hourly retention rates are compared with the tabulated minimum retention rates assigned to the four hydrologic soil groups (see table G-1 4). When the retention rate given by use of a runoff curve becomes less than an assigned minimum retention rate, the minimum rate is used to' compute excesses thereafter for the remainder of the storm.

For this example, determination of applicable runoff curve numbers, AMC-II, for subareas A and B has been assumed as described earlier in section G-1 9(a)(3) on soils and cover. East of the 105° meridian, soil moisture within a watershed which has similar to average conditions present before occurrence of the maximum annual flood (AMC-II) is considered a reasonable assumption for occurrence of a design storm. Therefore, the curve numbers referred to above were obtained from table G-3(A), which lists curve numbers for AMC-II; CN 86 was selected for subarea A and CN 70 for subarea B, to compute rainfall excesses. Minimum retention rates selected are those for general cases, table G-14: 0.12 inch per hour for subarea A, hydrologic soil group C; and 0.24 inch per hour for subarea B, hydrologic soil group B.

Computations of rainfall excesses are made to hundredths of an inch, as shown in table G- 13. Runoff curves, figure G-2, cannot be accurately read to hundredths unless plotted to a large scale, so it is recommended that rainfall excesses be computed by the equation shown on figure G-2. The symbol P_e is used in this text to designate direct runoff values, rainfall excesses, in lieu of Q shown on figure G-2. Values of S and $0.2S$ in inches for each curve number are listed in table G-4. Referring to table G-13, computations of hourly rainfall excesses for subarea A are described. This procedure applies to all such computations.

(1) Obtain S and $0.2S$ values from table G-4 for CN 86. Compute $0.8S$ value.

(2) Fill in column 5, ΣP, by summing the arranged design storm increments.

Table G-14.–*Minimum retention rates for hydrologic soil groups*

Hydrologic soil group	Range of minimum retention rates, inches per hour	Recommended rate for use in general case, inches per hour
A	0.30-0.45	0.40
B	0.15-0.30	0.24
C	0.08-0.15	0.12
D	0.02-0.08	0.04

(3) To obtain column 6, begin with the first ΣP value that exceeds the applicable $0.2S$ value and, successively by hours, compute ΣP_e by the equation:

$$\Sigma P_e = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad (13)$$

Each successive ΣP value in column 5 of table G-1 3 becomes the P for the equation, and the values of $0.2S$ and $0.8S$ are those obtained as in (1) above.

(4) Determine increment of excess rain, ΔP_e for each hour, and tabulate in column 7, then substract ΔP_e from respective ΔP, column 4, and enter Δ loss value thus obtained in column 8.

(5) As successively computed, compare Δ loss value with assigned minimum retention rate: 0.12 inch per hour for subarea A. If loss is greater than 0.12 inch per hour, proceed to next hour and repeat procedure; if loss is less than 0.12 inch, do not use the computed ΔP_e value. Drop use of runoff equation and use the constant hourly loss rate of 0.12 inch per hour to compute that hour's excess and the rest of the hourly increments of excess rainfall. This change occurred at hour 5 in the example in table G 13.

The hourly increments of excess rainfall listed in column 7 will be applied to a l-hour unitgraph for subarea A.

In all cases when the generalized precipitation charts are used to estimate preliminary design storm values for a watershed, hourly increments of excess rainfall should be obtained by the above procedure. If a 2-, 3-, or 4-hour unitgraph is to be used for the watershed, the computed hourly rainfall excesses are grouped into respective 2-, 3-, or 4-hour sums and applied to the chosen unitgraph.

(f) ***Computation of Preliminary Inflow Design Flood Hydrographs.*** -Computation of an inflow design flood (IDF) hydrograph is a routine mathematical process after decisions are made regarding selection of dimensionless-graph, lag-time, retention rate, and design storm values and arrangement. Procedural steps for obtaining a synthetic unitgraph for a watershed have been given in section G-9(7). The principle of obtaining a total flood hydrograph resulting from successive increments of excess rainfall is illustrated on figure G-3. Therefore, detailed tables showing computation of unitgraphs for subareas A and B and the application of respective sets of rainfall excesses to respective unitgraphs are omitted. In lieu thereof, copies of the printouts from the Bureau's Automatic Data Processing (ADP) program for application of the dimensionless-graph lag-time method of computing flood hydrographs are included as tables G 1.5 and G- 16. Table G-l 5 is a simulated printout of the computed preliminary design flood contribution from subarea A resulting from the incremental rainfall excesses listed in column 7 of table G-13. The program is designed to compute discharges to the nearest cubic foot per second (c.f.s.) so the ordinates of the l-hour unitgraph for a lag-time of 6.7 hours, listed in the third column of table G-1 5, are more exact than warranted by the basic data. (The same comment applies also to the computed flood hydrograph discharges.) Table G- 16 is a similar printout for subarea B.

(1) ***Preliminary*** *inflow* ***design flood hydrograph using generalized lag-time curve for both subareas.*** -Design flood contributions for each subarea are tabulated, combined, and total preliminary IDF discharges listed in table G-1 7. Subarea hydrographs and the total hydrograph are shown on figure G- 17. (In usual practice, only the total flood hydrograph is plotted.) A base flow has not been added to computed flood discharges, because base flow discharges are insignificant in relation to the computed flood discharges in this example. A method of obtaining the volume of the IDF hydrograph is detailed in table G- 17.

(2) Preliminary inflow design flood h ydrograph, watershed not divided into subareas. -Under the assumption that no streamflow records are available within the watershed and that the same dimensionless-graph, lag-time curve, and preliminary design storm values are to be used for both subareas, a preliminary inflow design flood hydrograph may be computed using one unitgraph for the

Table G-l *5 .-Simulated automatic data processing printout-preliminary inflow design flood (IDF) contribution, subarea A*

EXAMPLE PRELIMINARY IDF SUBAREA A

UNIT GRAPH DEVELOPED FROM DIMENSIONLESS GRAPH

DIMENSIONLESS FIGURE G-5 DESIGN OF GRAVITY DAMS TUN = 1.00

LAG DATA GENERAL CURVE SUBAREA A LAG = 6.70

AREA = 240.000 SQ MI UNITGRAPH RECES COEF = 0.828781 AT 18.00 HRS

EXCESS OR STORM VOLUME = 16.570 INCHES

HYDROGRAPH VOLUME IN INCHES = 16.573 AND IN AC FT = 212132.7

HOURS	EXCESSES INCHES	UNITGRAPH CFS	HYDROGRAPH CFS
.00	,000	[illegible]	0
1.00	.300	174	52
2.00	1.270	,247	595
3.00	1.610	4988	3361
4.00	7.950	12887	13591
5.00	2.380	20571	40900
6.00	1.180	2313,	96644
7.00	.580	2043,	184518
8.00	.380	15042	268585
9.00	.180	10787	306688
10.00	.180	7795	29163,
11.00	.080	6238	242607
12.00	.080	5107	192229
13.00	.080	4217	150359
14.00	.080	3575	121417
15.00	.080	2980	99867
16.00	.000	2502	83365
17.00	.080	2140	70906
18.00	.080	1900	60650
19.00	,000	1575	52146
20.00	,000	,305	45261
21.00	.000	,082	39936
22.00	.000	897	34392
23.00	,000	743	29058
24.00	.000	616	24031
25.00	.000	510	19680
26.00	.000	423	16123
27.00	.000	35,	13290
28.00	.000	291	11018
29.00	.000	24,	9144
30.00	,000	200	7602
31.00	.000	165	6324
32.00	,000	137	5255
33.00	.000	114	4363
34.00	.000	94	3631
35.00	.000	78	3020
36.00	.000	65	2503
37.00	,000	54	2074
38.00	.000	44	1719
39.00	.000	37	,425
40.00	.000	31	1181
41.00	.000	25	979
42.00	.000	21	811
43.00	.000	17	672
44.00	.000	14	557
45.00	.000	12	462
46.00	.000	10	383
47.00	.000	8	317
48.00	,000	7	263
49.00	.000	6	218
50.00	.000	5	181
51.00	.000	4	150

Table *G- 16. -Simulated automatic data processing printout-preliminary inflow design flood (IDF) contribution, subarea B*

EXAMPLE PRELIMINARY IDF SUBAREA B

UNIT GRAPH DEVELOPED FROM DIMENSIONLESS GRAPH

DIMENSIONLESS FIGURE G-5 DESIGN OF GRAVITY DAMS TUN = 1.00

LAG DATA GENERAL CURVE SUBAREA B LAG = 9.40

AREA = 560.000 SQ MI UNITGRAPH RECES COEF = 0.872335 AT 24.00 HRS

EXCESS OR STORM VOLUME = 13.040 INCHES

HYDROGRAPH VOLUME IN INCHES = 13.041 AND IN AC FT = 389484.2

HOURS	EXCESSES INCHES	UNITGRAPH CFS	HYDROGRAPH CFS
.00	.000	0	0
1.00	.020	140	3
2.00	.640	842	106
3.00	1.160	2824	757
4.00	7.060	7558	3921
5.00	2.260	16848	14710
6.00	1.060	26830	42072
7.00	.460	35618	98125
0.00	.260	38938	194141
9.00	.060	38289	304039
10.00	.060	32412	404552
11.00	.000	25421	459442
12.00	.000	19932	467687
13.00	.000	15560	424020
14.00	.000	12603	356437
15.00	.000	10754	290340
16.00	.000	9518	232873
17.00	.000	8093	188750
18.00	.000	6930	157275
19.00	.000	6232	134614
20.00	.000	5464	114743
21.00	.000	4806	98479
22.00	,000	4229	86598
23.00	.000	3721	75960
24.00	.000	3558	66750
25.00	.000	3103	58891
26.00	.000	2707	52122
27.00	.000	2362	47810
28.00	.000	2060	42485
29.00	,000	1797	37420
30.00	.000	1568	32799
31.00	.000	1368	28697
32.00	.000	1193	25054
33.00	.000	1041	21874
34.00	.000	908	19081
35.00	.000	792	16645
36.00	.000	691	14520
37.00	.000	603	12667
38.00	.000	526	11050
39.00	,000	459	9639
40.00	.000	400	8408
41.00	,000	349	7335
42.00	.000	304	6399
43.00	.000	266	5582
44.00	.000	232	4869
45.00	.000	202	4247
46.00	.000	176	3705
47.00	.000	154	3232
48.00	.000	134	2820
49.00	.000	117	2460
50.00	.000	102	2146
51.00	.000	89	,872
52.00	.000	78	,633
53.0Q	.000	68	,424
54.00	.000	59	,242
55.00	.000	52	1084
56.00	.000	45	945
57.00	.000	39	825
58.00	.000	34	719
59.00	.000	30	628
60.00	.000	26	547
61.00	.000	23	478
62.00	.000	20	417
63.00	.000	17	363
64.00	.000	15	317
65.00	.000	13	277
66.00	.000	11	24,
67.00	,000	10	210

Table G-11.-*Preliminary inflow design flood hydrograph, east of 105° meridian-same lag-time curve for both subareas*

Time, ending at hour	Discharges,[1] 1,000 c.f.s. Subarea A	Subarea B	Prelim. IDF
0	0.00	0.0	0.0
1	.05	.0	.1
2	.6	.1	.7
3	3.4	.8	4.2
4	13.6	3.9	17.5
5	40.9	14.7	55.6
6	96.6	42.1	138.7
7	184.5	98.1	282.6
8	268.6	194.1	462.7
9	306.7	304.0	610.7
10	291.6	404.6	696.2
11	242.6	459.4	702.0
12	192.2	467.7	659.9
13	150.4	424.0	574.4
14	121.4	356.4	477.8
15	99.9	290.3	390.2
16	83.4	232.9	316.3
17	10.9	188.8	259.7
18	60.7	157.3	218.0
19	52.1	134.6	186.7
20	45.3	114.7	160.0
21	39.9	98.5	138.4
22	34.4	86.6	121.0
23	29.1	76.0	105.1
24	24.0	66.8	90.8
25	19.7	58.9	78.6
26	16.1	52.1	68.2
27	13.3	47.8	61.1
28	11.0	42.5	53.5
29	9.1	31.4	46.5
30	7.6	32.8	40.4
[2]33	4.4	21.9	26.3
36	2.5	14.5	17.0
39	1.4	9.6	11.0
42	.8	6.4	1.2
45	.5	4.2	4.7
48	.3	2.8	3.1
51	.2	1.9	2.1
54	[3].1	1.2	1.3
57	<.1	.8	.8
60		.5	.5
63		.4	.4
66		.2	.2

Computation of IDF volume:

Sum, discharges, 0-29 hrs.	6,977,200
½ discharge, hr. 30	20,200
Volume. 0-30 hrs.	6,997,400 c.f.s.-hrs.
½ discharge, hr. 30	20,200
Sum, discharges, 33-63 hrs.	74.400
½ discharge, hr. 66	100
Sum	94,700
Volume, 30-66 hrs., (3 times 94,700)	284,100 c.f.s.-hrs.
Total IDF volume	7,281,500 c.f.s.-hrs.
Equivalent to	303,400 c.f.s.-24 hrs.
Equivalent to	600,800 ac.-ft.

For a check, compare with the sum of volumes in tables G-15 and G-16, or 601,600 ac.-ft.

[1]Instantaneous at designated hour.
[2]Larger time intervals may be used for lower portions of hydrograph recessions.
[3]If needed, discharges "cut off' to shorten computations (see table G-15) may be extended using the hydrograph's recession coefficient.

total watershed area. Estimating a total basin lag-time by weighting subarea lag-time proportional to the areas of 240 and 560 square miles gives a lag-time of 8.6 hours. A weighted runoff curve number, CN 7.5, and weighted minimum retention rate, 0.20 inch per hour, are obtained as shown in table G-18. The calculations are shown because this method of weighting curve numbers is used to obtain a weighted CN for a basin (or subbasin) which contains various areas of different soil and cover complexes. Table G-18 shows the computation of incremental rainfall excesses which were applied to a l-hour unitgraph for the watershed, lag-time 8.6 hours, area 800 square miles. Ordinates of the computed preliminary IDF hydrograph, peak discharge 768,600 c.f.s., volume 597,700 acre-feet, are plotted on figure G-17.

Either of the preliminary IDF hydrographs shown on figure G-17 could be recommended for use for preliminary designs. Under the assumptions made for computing these hydrographs, an acceptable result is obtained by considering the basin as a whole or by dividing the basin into two subareas.

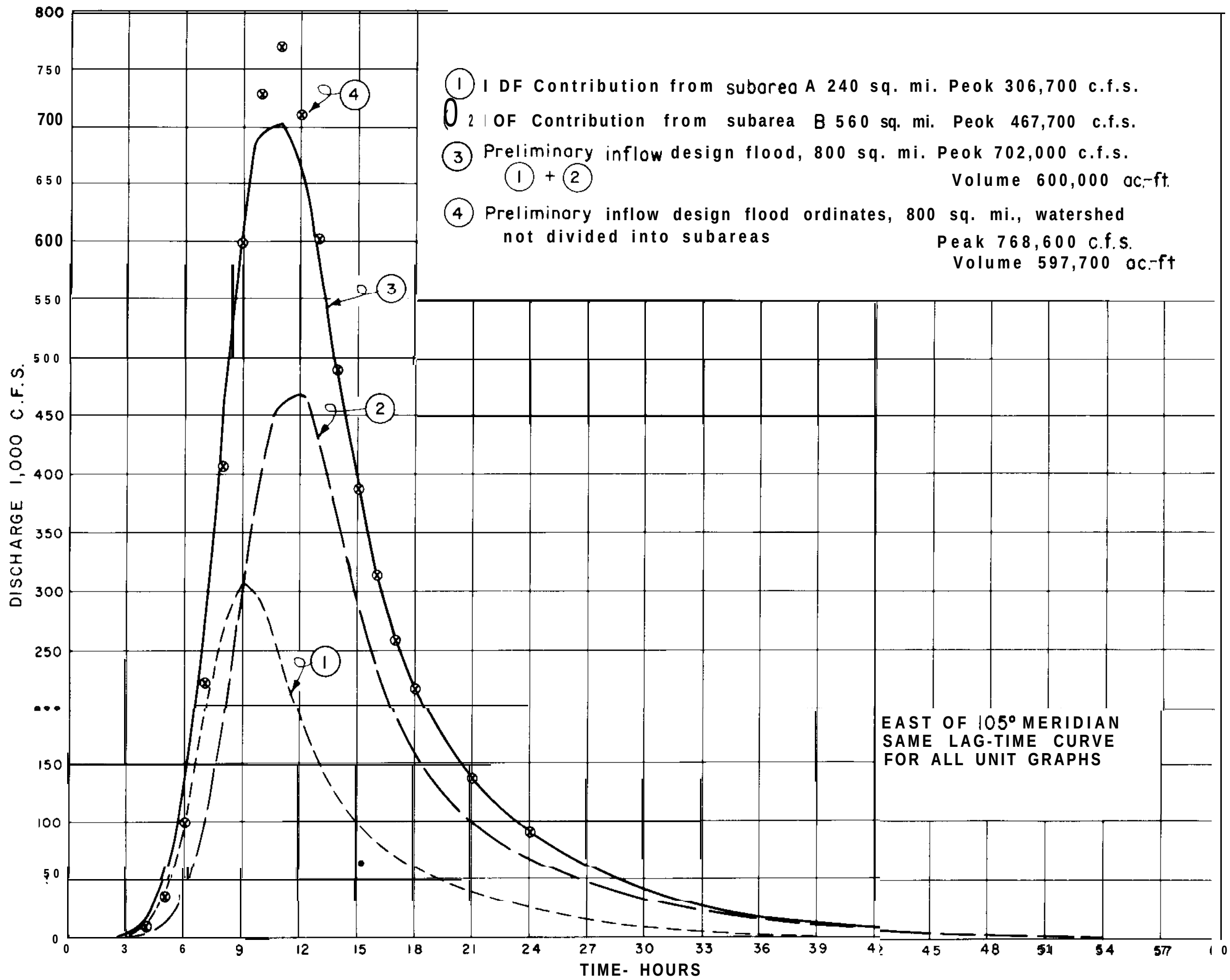


Figure G-17. Example of preliminary inflow design flood hydrographs-same lag-time curve for all unitgraphs.-288-D-3188

(3) Preliminary inflow design flood hydrograph using a different lag-time curve for each subarea. -As lag-time differences between subarea drainage systems within a basin increase, added consideration needs to be given to dividing the basin into subareas and obtaining the design flood contribution from each subarea for combination to form the inflow design flood. This is demonstrated by the hydrographs shown on figure G-18. Using the assumption given in section G-19(a)(2) that tributary B had streamflow records giving a lag-time of 22.0 hours from which a lag-time of 18.0 hours is obtained for subarea B for inflow to the proposed reservoir (sec. G-19(c)), a 1-hour unitgraph for subarea B was computed. The design flood contribution from subarea A shown on figure G-17 (1) is not changed and is replotted on figure G-18 (1).

The increment of rainfall excesses for subarea B, table G-13, column 10, applied to the new unitgraph for subarea B gives the flood contribution shown on figure G-18 (2). Combining the hydrographs from the two subareas, table G-19, gives a preliminary inflow design flood hydrograph, figure G-18 (3), having two peaks, the maximum of which is a peak discharge of 332,500 c.f.s. (as estimated when plotting the graphs) and a 72-hour volume of 597,000

Table ***G-M-Preliminary inflow design flood, east of 105° meridian-computation of incremental excesses, ΔP_e, considering basin as a whole, and using an areal weighted CN and minimum loss rate.***

BASIC DATA:
Subarea A: AMC-II CN 86; min. loss, 0.12 in./hr.; area, 240 sq. mi.
Subarea B: AMC-II CN 70; min. loss., 0.24 in./hr.; area. 560 sa. mi.
WEIGHTED VALUES FOR USE:

$$\frac{(86)(240) + (70)(560)}{800} = 74.8\text{: use AMC-II CN } 75$$

$$\frac{(0.12)(240) + (0.24)(560)}{800} = 0.204\text{; use } 0.20 \text{ in./hr}$$

Time, ending at hour	ΔP,[1] inches	ΣP, inches	Rainfall excesses, P_e		
			ΣP_e,[3] inches	ΔP_e, inches	Δ loss, inches
1	1.2	1.2	0.07	0.07	1.13
2	1.7	2.9	.89	.82	.88
3	1.8	4.7	2.21	1.32	.48
4	8.2	12.9	9.61	7.40	.80
5	2.5	15.4	11.91	2.30	[4].20
6	1.3	16.7	13.01	1.10	.20
7	.7	17.4	13.51	.50	.20
8	.5	17.9	13.81	.30	.20
9	.3	18.2	13.91	.10	.20
10	.3	18.5	14.01	.10	.20
11	.2	18.7	14.01	0	.20
12	.2	18.9			

[1]Arranged design rainfall, see column 4, table G-l 3.
[2]Balance of rainfall less than retention loss in this approach.
[3]By equation, $P_e = \frac{(P - 0.2S)^2}{(P + 0.8S)}$; for CN 75, S = 3.33, $0.2S$ = 0.67, $0.8S$ = 2.66 (table G-4).
[4]ΔP_e by equation indicates Δ loss of 0.10 in., less than 0.20 in.; use 0.20 in./hr.

acre-feet. Ordinates of a flood hydrograph computed using a l-hour unitgraph having a basin weighted lag-time of 14.6 hours and incremental rainfall excesses listed in table G-1 8 are shown as 4 on figure G-l 8. This flood hydrograph has a peak of 492,000 c.f.s., excessively high in comparison with the flood hydrograph obtained by combining the two subarea flood hydrographs. The procedure of considering the watershed as a whole does not give an acceptable preliminary IDF hydrograph in this instance.

G-20. Preliminary Inflow Design Flood Estimates, Watersheds West of 105° Meridian. -It is very likely that runoff from snowmelt will contribute a portion of the discharges of an inflow design flood (IDF) hydrograph for large dams at sites west of the 105° meridian. In many instances though, design rainstorm potential is so great that runoff from a design rainstorm gives the major portion of an inflow design flood. Preliminary inflow design flood estimates for many areas west of the 105° meridian can be made using preliminary design storm values obtained from figure G-1 3 and associated procedures, the methods of arranging design storm incremental rainfall and computing rainfall excesses given in this section, and adding appropriate base flows to the computed rain flood hydrograph. In general, for western mountainous watersheds having seasonal snowmelt runoff which reaches a maximum after mid-May, base flows for addition to the hydrograph computed from a preliminary design rainstorm may be estimated as those discharges likely to occur during the last 5 days of the maximum 15-day period of a 1 percent chance maximum annual 15-day

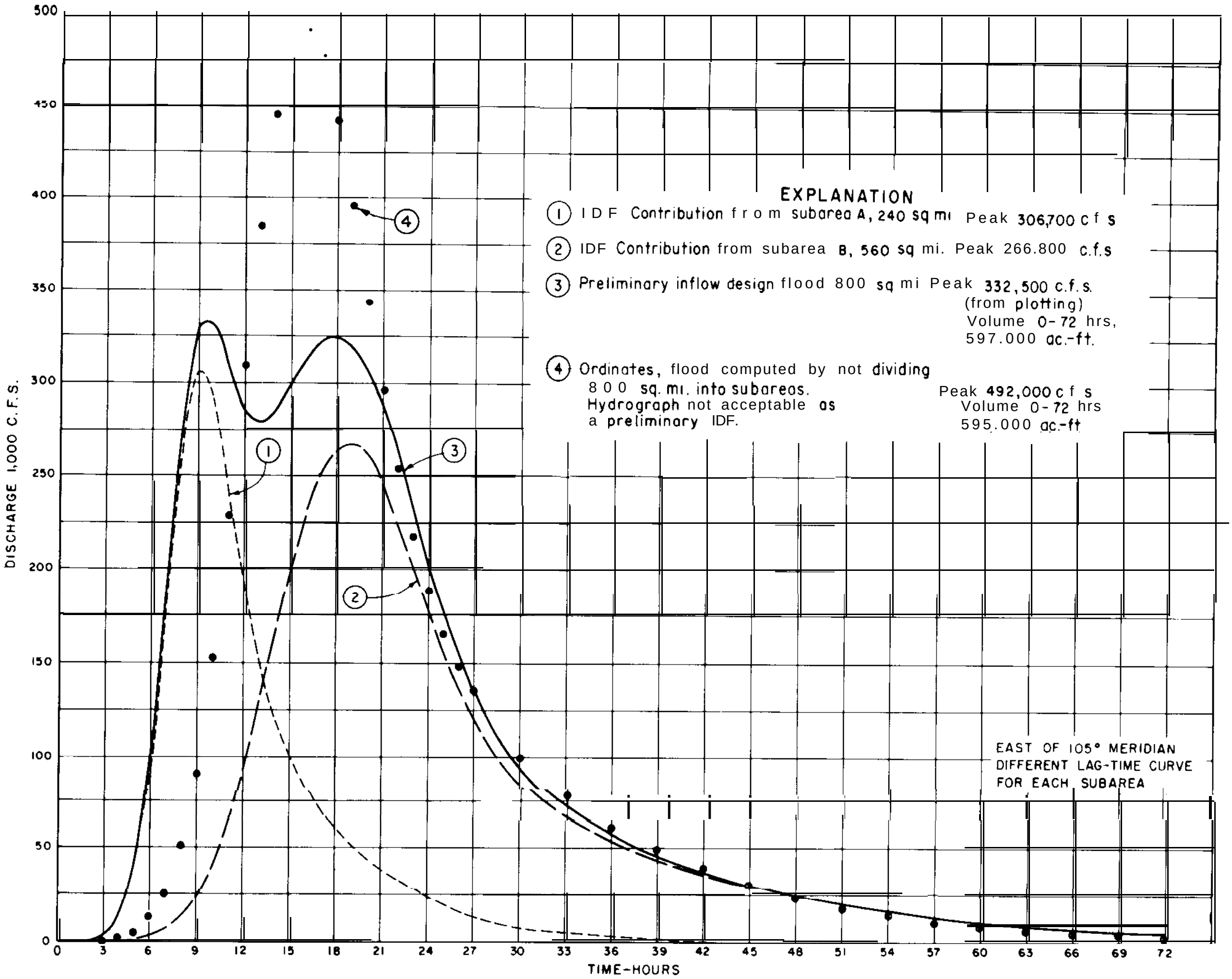


Figure G-18. **Example of preliminary inflow design flood hydrograph-different lag-time curve for each subarea.-288-D-3189**

seasonal snowmelt runoff flood. (See secs. G-28 and G-29 for a discussion of statistical analyses-frequency studies.) However, this general approach cannot be used for mountainous watersheds where maximum storm potential occurs during the winter months October through April. Examples are: Sierra Nevada Mountains in California and Nevada, Cascade Range in Oregon and Washington, and Mogollon Rim in Arizona. Extreme floods on streams in these regions result from rain falling on snow-covered watersheds. Estimation of rain-on-snow floods requires special procedures as discussed in sections G-22 through G-26. Exception also must include those watersheds having a large percentage of total basin drainage area at relatively low elevations where the ground may be frozen and winter rain falling on a light snow cover can cause large floods.

Procedures for estimating the rain-flood portion of a preliminary inflow design flood hydrograph from preliminary general-type design storm values for a watershed west of the 105° meridian differ in two respects from the procedures which have been given for watersheds east of the 105° meridian; namely, arrangement of design storm rainfall increments, and assignment of appropriate runoff curve number, CN.

Table G-19.—*Preliminary inflow design flood hydrograph, east of 10.? meridian-different lag-time curve for each subarea*

Time, ending at hour	Discharges, 1,000 c.f.s.[1] Subarea A[2]	Subarea B[3]	Prelim. IDF
0	0.00	0.0	0.0
1	.05	<.1	.1
2	.6	<.1	.6
3	3.4	.1	3.5
4	13.6	.5	14.1
5	40.9	1.3	42.2
6	96.6	3.2	99.8
7	184.5	6.9	191.4
8	268.6	13.1	281.1
9	306.7	23.5	330.2
10	291.6	39.5	331.1
11	242.6	62.8	305.4
12	192.2	94.1	286.3
13	150.4	129.1	279.5
14	121.4	165.3	286.7
15	99.9	200.7	300.6
16	83.4	231.0	314.4
17	70.9	252.5	323.4
18	60.7	263.5	324.2
19	52.1	266.8	318.9
20	45.3	260.5	305.8
21	39.9	244.4	284.3
22	34.4	224.4	258.8
23	29.1	200.8	229.9
24	24.0	177.5	201.5
25	19.7	156.4	176.1
26	16.1	137.6	153.7
27	13.3	120.8	134.1
28	11.0	105.9	116.9
29	9.1	94.9	104.0
30	7.6	86.0	93.6

Time, ending at hour	Discharges 1,000 c.f.s. Subarea A	Subarea B	Prelim. IDF
33	4.4	67.6	72.0
36	2.5	53.2	55.1
39	1.4	43.2	44.6
42	.8	35.2	36.0
45	.5	28.7	29.2
48	.3	24.8	25.1
51	.2	20.3	20.5
54	.1	16.4	16.5
57	<.1	13.2	13.2
60		10.6	10.6
63		8.5	8.5
66		6.8	6.8
69		5.5	5.5
72		4.4*	4.4

*Continuing discharges may be computed at 3-hour intervals using recession coefficient of 0.8031. Volume after hour 72:

$$\text{Vol.} = \frac{-4}{\log_e k_3}$$

$$\text{Vol.} = \frac{-4.,400}{-0.21928}$$

Vol. = 20,060 c.f.s.-3 hrs.
2,508 c.f.s.-24 hrs.
4,970 ac.-ft.

Vol. (O-72 hrs.), 301,050 c.f.s.-hrs.
597,100 ac.-ft.

[1] Instantaneous at designated hour.
[2] Same discharges as for subarea A, table G-17.
[3] 1-hr. unitgraph, lag-time 18.0 hrs., used to compute discharges. Excesses column 10, table G-13.

(a) ***Preliminary Design Storm Values, Watersheds West of 105° Meridian.*** -By geographical location (county) obtain probable maximum 6-hour point rainfall value from figure G- 13. Note zone designation, A, B, or C, in which watershed is located.

(1) Compute 6-hour basin rainfall by multiplying 6-hour point rainfall by ratio obtained from applicable zone curve, figure G 14, for watershed drainage area, square miles.

(2) Make a tabulation of design storm depth-duration values at l-hour intervals for a design storm duration extending to the hour beyond which hourly rainfall increments are equal to or less than the minimum hourly retention loss rate for the watershed. Hourly distribution of maximum 6-hour rainfall is obtained from the applicable curve of figure G-1 2. Design storm values beyond 6 hours are computed at 2-hour intervals by appropriate constants listed in table G-l 1. From 6 to 24 hours, use average of even-numbered 2-hour accumulative rainfall for the intervening odd-numbered hour. If hourly rainfall increments are needed after 24 hours, draw depth-duration curve for rainfall amounts computed by constants in table G-l 1 and read

hourly values. Compute depth-duration rainfall values to nearest hundredth of inch.

(b) Arrangement ***of Design Storm Increments of Rainfall.*** -Beginning with the second largest 6-hour design storm rainfall amount, hours 6-12 of depth-duration values, arrange hourly increments of design rainfall in ascending order of magnitude for the first 6 hours of arranged design storm values. For hours 7 through 12, arrange hourly increments of maximum 6-hour rainfall in the following order of magnitude: 6, 4, 3, 1, 2, 5. Hourly rainfall amounts after the 12th hour are arranged in descending order of magnitude.

(c) Assignment of Runoff Curve Number, CN, and Computation of Increments of Excess Rainfall. -Watershed soils, cover and land use data are used to estimate an applicable runoff curve number from the information given in section G-7(b)(6). The estimated curve number, CN, is for antecedent moisture condition II, AMC-II. This number is then coverted to the respective AMC-III CN listed in table G-4 and the AMC-III CN used to compute hourly rainfall excesses by the method illustrated in table G-13. Antecedent moisture condition III is assumed for watersheds west of the 105° meridian, because late May and June design storm potential is likely to be concurrent with, or immediately after, snowmelt runoff while watershed soil moisture is high.

If a unit time period longer than 1 hour is used for obtaining a unitgraph, the two largest increments of rainfall excesses should be grouped together. If such grouping of hourly excesses results in only 1 hourly excess increment in a unit time period at the beginning and/or end of excess rainfall period, the l-hour increment of excess is assumed as total excess for the unit time period.

(d) ***Floods From Design Thunderstorm Rainfall.*** -Data for estimating design thunderstorm rainfall have not been included in this text. If an estimate of a preliminary inflow design flood (IDF) caused by design thunderstorm rainfall is required, preliminary design thunderstorm rainfall estimates for watersheds west of the 105° meridian may be obtained from generalized data in the publication "Design of Small Dams," second edition, [31] along with data for estimating increments of excess rainfall to be applied to a unitgraph. The procedures which have been described in this text for developing a unitgraph can be used to obtain a unitgraph ***for that portion of a watershed*** over which a design thunderstorm might occur. In the event that this type of preliminary IDF estimate proves critical for design, a hydrometeorologist should be consulted for an estimate of design thunderstorm rainfall for the specific watershed.

G-21. Recommendations for Routing Preliminary Inflow Design Floods Through a Proposed Reservoir. -It is necessary for designers to assume an elevation of the reservoir pool at the start of an inflow design flood for reservoir routing studies to determine required spillway capacity. Normally, the reservoir pool is assumed to be full to the top of planned conservation storage capacity or, when either inviolate or joint use flood control capacity is proposed, full to the top of either type of flood control capacity at the beginning of a preliminary inflow design flood. If large capacities of flood control space are being considered in preliminary planning, criteria for routing a final-type IDF as discussed in sections G-30 and G-31 should be established to the extent possible with information available.

G. SNOWMELT RUNOFF CONTRIBUTIONS TO INFLOW DESIGN FLOODS

G-22. ***General.*** -"Hydraulic engineers re sp on si ble for planning and designing multiple-purpose storage reservoirs recognize snow as a form of precipitation possessing certain characteristics which can be evaluated and applied to advantage, both from a hydrologic and an economic viewpoint, in the planning and design of multipurpose storage reservoirs. In northern latitudes and at high elevations, snow falls and accumulates on the earth's surface in frozen crystalline form and usually remains until a proper sequence of meteorologic events provides the thermodynamic conditions essential for either evaporation or melting. Periodic snow surveys provide a reliable index of the relative snow accumulation. With knowledge of the processes of storage, evaporation, and melting, the engineer can predict, with reasonable accuracy (for normal climatic conditions and for known snowpack) the characteristics and amount of streamflow to be expected * * * In the Western United States, the economy of the arid and semiarid lands lying between the mountain ranges is increasingly dependent on development of multiple-purpose storage reservoirs to utilize the streamflow originating in the high mountain snow packs. Engineers of the Western States accept as a blessing the fact that ***the predictable characteristics*** (italics added) of this streamflow enable economies in planning and designing multiple-purpose reservoirs by the joint use of space allocated to the various functions and by reduction of spillway capacities."

The above extract from Mr. H. S. Riesbol's paper "Snow Hydrology for Multiple-Purpose Reservoirs" [22] is quoted to point out the importance of snow in hydrologic studies and the predictable characteristics of streamflow originating from snowpacks. These predictable characteristics often make possible employment of simple empirical correlations which give acceptable estimates of snowmelt runoff, although this runoff results from a complex thermodynamic process. Discussion of empirical methods of estimating snowmelt runoff as related to inflow design flood estimates is the main objective in these sections. Readers interested in more information about the physical and thermodynamic characteristics of snow and snowmelt processes may consult "Snow Hydrology" [23] and "Handbook of Applied Hydrology" [24].

As previously stated in section G-1, Bureau of Reclamation policy does not provide for combining probable maximum snowmelt runoff with probable maximum rainfall runoff for estimation of an inflow design flood. It is believed that such combinations are unreasonably severe. It is considered more reasonable to combine runoff from a probable maximum rainstorm that could occur during the snowmelt season with a major snowmelt flood, or to combine runoff from a major rainstorm that could occur during the snowmelt season with probable maximum snowmelt runoff. In regions where maximum probable rainstorms can occur during winter months when watersheds may have a large amount of snow on the ground, the amount of snow melted during the design rainstorm must be estimated and runoff calculated from the total combined rain and melted snow water available on the ground surface. Procedures have been developed for computing this type of rain-on-snow floods, utilizing data and analyses described in detail in the report "Snow Hydrology" [23]. One should be mindful that each individual IDF study requires some variations within the framework of a general approach, depending upon watershed characteristics, location, basic data available, and proposed operational capacity of the future reservoir.

G-23. Major Snowmelt Runoff During Seasonal Melt Period for Combination With Probable Maximum Storm Runoff. -A method of estimating snowmelt runoff contribution for this type of combination has been described briefly in connection with preliminary IDF estimates for watersheds west of the 105° meridian. Additional items need be considered when making "best possible" preliminary IDF or final-type IDF estimates. Inclusion of flood control capacity and its amount in a proposed reservoir may have a direct bearing on the time duration of flow required in estimation of an inflow design flood hydrograph.

(a) ***Damsites for Reservoirs With no Flood Control Capacity Proposed.*** -These projects are intended to store seasonal snowmelt runoff as rapidly as possible, allowing only minimum required releases until reservoir capacity becomes full to top of conservation storage. A duration time of 15 days is usually adequate for an inflow design flood hydrograph for this type of structure, as a reservoir may be assumed full to top of conservation capacity at the beginning of the 15-day period. A 1 percent chance (100 year) 15-day volume of snowmelt runoff is usually considered as a major snowmelt flood. It is obtained from a frequency study of maximum annual 15-day snowmelt runoff volumes using runoff records for the contributing watershed, if available, or records for similar nearby watersheds. The 15-day volume indicated by the frequency computations (secs. G-28 and G-29) is adjusted to the specific watershed above a damsite by area relationships.

Caution : Occasionally there will be found references or data of an extremely large snowmelt flood exceeding all recently recorded floods and, perhaps, exceeding the 1 percent chance value indicated by frequency analyses of more recent records. These data should not be ignored without making full effort to incorporate the data into the snowmelt flood estimate.

(1) ***Assembly of basic stream-flow data for frequency analyses.*** -Concurrently with tabulation of maximum annual 15-day seasonal snowmelt runoff values from streamflow records, climatological data should be examined to determine if each year's 15-day runoff volume was snowmelt runoff or was increased by rainfall amounts large enough to cause runoff during that period (small rainfall events may be ignored). If a large snowmelt volume is indicated, an estimate of the rain-flood portion can be made and subtracted by plotting the daily discharge values on semilogarithmic paper and sketching an estimated snowmelt recession (due to lower temperatures accompanying rainfall) under the obvious rain-flood portion. This procedure may have to be used in a few regions where almost every year some rainfall runoff is concurrent with snowmelt runoff.

(2) Daily distribution of 1 percent chance 15-day snowmelt runoff volume. -Springtime snowmelt runoff coordinates closely with temperature fluctuations. Large areas usually have about the same daily temperature sequence. Usually snow-fed streams in a given vicinity have similar daily distribution patterns of runoff, magnitudes of discharges reflecting individual watershed snowmelt contributing areas. These distribution patterns will also be similar year to year. Therefore, a distribution pattern for one of the larger 15-day volumes recorded for the stream where a damsite is located, or for a nearby similar watershed, can be selected and the 1 percent chance 15-day snowmelt runoff volume for the damsite distributed into daily discharges proportional to the selected recorded flood. An approximately symmetrical 15-day pattern with the maximum daily discharge occurring within the 7th to 10th day of the 15-day period is usually selected. An additional refinement may be included in selecting the distribution pattern, if by chance climatological records show that a small rain event occurred a day or two after the maximum daily discharge of a large recorded 15-day volume and discharges decreased due to lowered temperatures associated with the rain event. This sequence of events agrees with the pattern of natural conditions assumed by the occurrence of a probable maximum rainstorm a day or two after the maximum day of snowmelt runoff.

(3) Combination of probable maximum rain flood with 1 percent chance 15-day snowmelt flood. -Selection of an appropriate day within a 15-day period of snowmelt runoff as a beginning time of design rain-flood runoff is a matter of engineering judgment. One reasonable assumption is a 2-day interval between the day of maximum temperature and the beginning of runoff caused by a design storm. Under this assumption, the apparent lag-time in days between maximum temperature and maximum daily snowmelt discharge from a watershed should be considered. The lag-time may be quickly determined by plotting a few of the larger annual maximum 15-day mean daily discharges and respective daily maximum temperatures from an *index temperature record. Depending on size and runoff characteristics of a watershed, the time interval between maximum temperature and resulting daily maximum snowmelt discharges at a damsite may vary from zero to 3 or more days. If the time interval is zero days, design rain-flood runoff is added to the snowmelt runoff, beginning on the third day after the peak of the snowmelt flood. As the lag-time interval between maximum temperature and peak of snowmelt runoff increases, the beginning time for a design rain-flood hydrograph is advanced closer to the peak of the snowmelt flood by l-day intervals. Thus, for large watersheds, it may be reasonable to combine a design rain flood with the maximum daily discharges of a snowmelt flood.

(b) *Damsites for Reservoirs With Proposed Joint Use Flood Control Capacity,* -A reservoir which has a joint use flood control capacity allocation is intended to control seasonal snowmelt discharges downstream from the dam to a limit of safe channel capacity throughout the entire snowmelt season, and also to store enough water to assure that the reservoir is full to the top of the joint use capacity at the end of each snowmelt season. Forecasts of seasonal snowmelt runoff volumes are a necessary part of this kind of operation.

A seasonal major snowmelt flood as a part of an inflow design flood (IDF) hydrograph usually is required when joint use flood control capacity is proposed. However, if planned joint use capacity is small and there is a likelihood that snowmelt discharges preceding the maximum 15-day period of a 1 percent chance snowmelt flood may fill the joint use pool, a 15-day IDF hydrograph will be adequate. When a seasonal major snowmelt flood hydrograph for combination with a probable maximum rain-flood hydrograph is needed, first consideration is given to the use of streamflow data.

The duration period of a seasonal IDF corresponds with the seasonal duration of the largest snowmelt floods which have occurred in the vicinity. Frequency analyses include annual maximum 30-day, 60-day, and if needed 90-day periods of snowmelt volumes in addition to analysis of the annual maximum 15-day discharge period. A recorded seasonal snowmelt flood is selected as a pattern for runoff distribution. The design rain flood is combined with the estimated snowmelt runoff hydrograph according to the criteria previously discussed.

If available streamflow data are not suitable for satisfactory results using the above approach, one of the methods of temperature-runoff correlations described in the referenced publications may be found adaptable to the situation.

G-24. Probable Maximum Snowmelt Floods to be Combined With Major Rain Floods. -(a) *General.* -An estimate of probable maximum snowmelt runoff may be necessary when making an inflow design flood (IDF) study for a watershed where snowmelt runoff causes the major portion of yearly flow. The degree of refinement needed in making this type of estimate may vary from preliminary comparisons to computation by detailed procedures depending on factors such as the following: storage capacity, space allocations, and operational plans of the proposed reservoir; snowmelt runoff characteristics of the watershed; and difference in magnitudes of probable maximum rainstorm and major rainstorm potentials for the watershed. For some watersheds, a few preliminary computations may show an IDF combination of major snowmelt runoff and probable maximum rain runoff to be definitely critical for design. In other instances detailed computations of each type IDF consisting of combined snowmelt and rain runoff have to be made and both types of IDF hydrographs prepared for use in design of a dam.

Studies prepared by the Bureau of Reclamation show that usually a critical inflow design flood results from a combination of runoff of a major snowmelt flood and a probable maximum rainstorm. In most instances, an approximation of probable maximum snowmelt flood magnitude by simple correlations shows that it will not be critical for design. Development of a *best estimate* of probable maximum snowmelt runoff is a complex procedure and requires special treatment for each site. Therefore, this discussion is limited to general aspects of the problem, with references to publications containing more detailed information.

(b) *Considerations for Estimates **of** Probable Maximum Snowmelt Floods.* -Estimating probable maximum snowmelt contribution to an inflow design flood can be thought of as requiring three steps: (1) estimating probable maximum seasonal accumulation of snow on a watershed, (2) estimating critical melt rates of the snow pack, and (3) estimating the amount of snowmelt runoff and its timing at the reservoir. The probable maximum seasonal accumulation of snow on a mountainous watershed drained by one main stream can be adequately estimated by a study of winter season precipitation records in and near the watershed, supplemented by snow survey data, Special studies are required for probable maximum seasonal snow accumulation estimates for large multitributary river systems such as the Colorado River above Glen Canyon Dam. One of two basic approaches can be taken to estimate critical snowmelt rates; namely, calculation of snowmelt runoff by means of an air temperature index, or calculation of melt using generalized snowmelt equations based on energy balance considerations. Methods using some form of an air temperature index have given good results for many watersheds. There is some physical basis for using a snowmelt air temperature index. Air temperature is reasonably well correlated, at a particular time and place, with the atmospheric factors which affect melt rates, such as solar radiation and vapor pressure, although it is by no means a perfect index of these factors.

Snowmelt equations which consider energy balance are used to evaluate short-wave radiation melt, long-wave radiation melt, melt due to convective heat transfer from the atmosphere and to latent heat of water vapor condensing into the snow surface, melt due to heat of rain drops, and melt by heat conduction from the ground. The Corps of Engineers report "Snow Hydrology" [23 1 presents detailed information regarding both approaches. A Corps manual, "Runoff from Snowmelt," EM 1110-2-1406 [25], presents synopses of investigations of melting relationships, generalized basin snowmelt equations and their application in methods of computing maximum snowmelt floods. Selection of an approach to be used depends on the basic data available and the importance of snowmelt runoff contribution to an inflow design flood. Whichever approach is taken, it is necessary to test the snowmelt computation procedures for the basin in question in order to determine basin values of the coefficients involved.

Approximation of a maximum probable snowmelt flood for a period of 10 to 20 days usually is directed toward determination of volume. This volume is then distributed in time by using a large recorded snowmelt runoff hydrograph as a pattern, as previously described in section G-23(a)(2). If a temperature index has been used directly in the computations, the volume may be distributed by a synthetic temperature sequence.

(c) Springtime Seasonal Probable Maximum Snowmelt Flood Estimates. -General procedures for estimating total seasonal probable maximum snowmelt runoff are not outlined in detail in this text. Brief statements about some approaches which may be considered for use, and reference to respective specific descriptions, are given below.

(1) *Hydrothermogram approach.*-The paper, "Snow Hydrology for Multiple-Purpose Reservoirs" [22], includes a description of an approach in which during the melting season daily temperatures above a base temperature are directly related to resulting direct runoff by a device referred to as a *hydrothermogram.* A hydrothermogram is a hypothetical discharge hydrograph computed on the assumption that each effective degree of temperature above a base temperature will generate the same amount of runoff volume. This procedure, adjusted to fit individual basin problems, has been found useful in several Bureau of Reclamation IDF studies (unpublished) where probable maximum snowmelt flood estimates were important.

(2) Generalized melt equations for springtime snowmelt floods. -The Corps of Engineers Manual, "Runoff from Snowmelt" [25], includes a chapter describing probable maximum snowmelt flood derivation using generalized melt equations. The Salmon River Basin which drains 14,100 square miles of rugged, mountainous regions of central Idaho is cited as an example in the discussion.

(3) Correlations. -Correlations between temperature and runoff, snowcover and runoff,

etc., are usually evidenced because of the predictable nature of snowmelt runoff. Hydrologists knowledgeable in the use of correlation studies may find this type of approach useful.

(d) *Major Rain-Flood Estimates **for** Combination With Probable Maximum Snowmelt Runoff.* —

(1) *Major rainstorm and **runoff.*** -Design storm studies for watersheds where snowmelt runoff contributes to inflow design floods should also include a hydrometeorological estimate of a major rainstorm that could occur during the snowmelt season. For areas where major rainstorms have often occurred in the vicinity of the watershed during the snowmelt season, the largest rainstorm of record within the area of transposability is fitted to the basin. In areas where major rainstorm occurrences during the spring snowmelt season are infrequent, watershed design storm values without maximization for moisture adjustment may be considered. A hydrograph of runoff from the major rainstorm is computed by the dimensionless-graph lag-time procedures previously discussed, but special attention is given to effects of snowmelt on retention losses applicable to the major rainstorm. The portion of the watershed covered by a melting snowpack will have little or no retention capacity for rainfall, and the portion recently denuded of snow will have high moisture content, hence low retention capacity during rainfall. Guide criteria for combining rain-flood hydrographs and snowmelt flood hydrographs have been discussed in section G-23(a)(3).

(2) *Observed rain floods.* -Occasionally, streamflow data used for snowmelt runoff analyses will include a major rain flood during a snowmelt season. In these instances, special studies are made to separate the rain-flood hydrograph from the snowmelt runoff, and the separated rain-flood hydrograph is used for combination with the estimated probable maximum snowmelt flood hydrograph.

G-25. Probable Maximum Rain-On-Snow IDF Estimates. -There are many watersheds along or near the coasts of the United States where major rainstorms or probable maximum rainstorms can occur during the winter months while the watersheds are partially or completely covered with snow. In many areas, storm systems may consist of precipitation beginning as snow then changing to rain or closely spaced successive storm systems, the first system occurring as snow, the second as rain accompanied by warm temperatures. Devastating floods have resulted from certain rain-on-snow combinations; in other instances, apparently similar conditions have produced only high flows causing little damage. Detailed investigations of differences between rain-on-snow flood magnitudes point toward the following two items as the main contributors to these differences: density conditions of the snowpack at the time of rain occurrence, and convective condensation melt related to wind velocities during the rainstorm. Generalized equations for estimating snowmelt during rainfall, developed as described in "Snow Hydrology" [231, have proved very useful in procedures for estimating runoff due to rainfall on snow.

In addition to estimates of snowpack melting rates, procedures for estimating runoff caused by rain-on-snow conditions include evaluations of snowpack release of free water to the ground surface, retention losses, and distribution in time of the runoff at the point of interest. A procedure used by the Corps of Engineers is given in the manual, "Runoff from Snowmelt" [25]. The procedure used in Bureau of Reclamation studies is described in Engineering Monograph No. 35, "Effect of Snow Compaction on Runoff from Rain on Snow" [26]. In both procedures snow melting rates during rainfall are computed by the same melting equations and water released at ground surface is determined. Excesses are computed by subtracting retention losses, and are distributed in time by a basin unitgraph. Differences between the procedures lie in estimations of snowpack free-water holding capacities.

The Corps procedure establishes a limit of liquid water holding capacity of a snowpack as a percentage of snowpack water content. Nearly all data considered when developing the limit of water holding capacity were obtained from spring snowpack of densities above 35 percent. The procedure in Engineering Monograph No. 35 relates snowpack liquid water holding capacity to snowpack densities just preceding the start of rainfall, and to increases in snowpack density due to melting and added rainfall until the pack attains a density of 40 or 45 percent when release of liquid water to the ground surface is assumed to begin. Development of the procedure was directed primarily for use for evaluating wintertime conditions where a rainstorm system closely follows a snowstorm and the newly deposited snowpack has had little time to change in structure. Topics of discussion in Engineering Monograph No. 35 are a development of the procedure and reconstitution of the December 1955 flood on South Yuba River near Cisco, Calif. Estimation of a probable maximum rain-on-snow flood is not discussed in the monograph. Data required for use of the procedure for IDF computations are: (1) estimates of watershed snowcover depth and water content antecedent to a design storm occurrence; and (2) hydrometeorological data of temperatures and wind velocities concurrent with design storm rainfall increments.

G-26. Special Situations.--(a) Frozen Ground.-Frozen ground conditions seldom occur in well-forested areas or under deep snowpacks. On the other hand, open areas where periods of subfreezing temperatures and light snowfall are normal can develop frozen soil conditions such that retention losses are practically nil. These areas may experience severe winter floods due to combinations of shallow snowcover, rising temperature, and relatively minor rainfall. Frozen ground conditions may also reduce lag-time. Analyses for this type of condition require individual watershed study.

(b) *Snowmelt* ***in the Great Plains Region of the United*** States.-Probable maximum precipitation potential is so great in the Great Plains region that snowmelt runoff is not usually considered in inflow design flood studies except for large drainage areas with headwaters in the Rocky Mountains. In the northern Great Plains, major floods have resulted from rapid spring snowmelt and frozen ground conditions. Consideration of this type of flood may be necessary for large drainage areas near the northern border of the United States.

H. ENVELOPE CURVES

G-27. General. -Peak discharge envelope curves and flood volume envelope curves can be prepared by drawing curves enveloping plotted points representing maximum recorded values for various drainage areas. The values plotted should represent similar type floods (rain floods or snowmelt floods) that have occurred within the broad geographical subdivision within which the subject watershed lies, and should not be limited to events of a single small river system. Preparation of envelope curves for a general area provides an engineer with valuable information on past flood history and an indication of the flood of record comparable to the subject area. However, they should not be relied upon as a means of estimating probable maximum flood values. Design flood values purporting to be the probable maximum should be higher than those obtained from envelope curves. Only in specific instances where a watershed has definitely lower flood potential than neighboring watersheds due to soil type, surface storage, etc., would it be good judgment to adopt an inflow design flood of smaller magnitude than that of a flood which has occurred nearby.

A simple method of preparation of envelope curves is to tabulate maximum peak discharges (or volumes of a selected duration) and respective drainage areas prior to plotting points. In most instances, the drainage area above a stream gaging station or the point of a large flood discharge measurement is given in the U.S. Geological Survey water supply paper listing the flood. When it is known that only a portion of the drainage area above a point of measurement contributed to a flood, the size of that contributing portion should be used in the envelope curve analysis. Discharges or volumes are plotted versus respective drainage areas using log-log paper. Data thus plotted usually indicate a curved line envelopment on log-log paper which may be approximated by a straight line for small ranges in areas. High discharges from local thunderstorms may suggest consideration of two curves-one for smaller areas subject to such occurrences and another for larger areas where maximum discharges originate from general storms.

I. STATISTICAL ANALYSES-ESTIMATES OF FREQUENCY OF OCCURRENCE OF FLOODS

G-28. ***General.*** -Estimates of the magnitude of floods which have frequencies of 1 in 5, 1 in 10, or 1 in 25 years are helpful in estimating requirements for stream diversion during construction. These floods are often termed the "5-, 1 0-, or 25-year flood." The magnitude of more rare events such as the SO- or 100-year flood may be required for reasons such as to establish sill location of emergency spillways, etc. The usual term of expression, "x-year flood," should not lead to the wrong conclusion that the event indicated can happen only once in x years, and having occurred, will not happen again for another period of x years. It does mean that over a long span of years we can expect as many x-year floods (or larger) as there are x-year-long periods within that span. Floods occur randomly and may be bunched or spread out unevenly with respect to time. No predictions are possible for determining their distribution; the probable maximum flood *can* occur the first year after the project is built, though of course, the odds are heavily against it.

The frequency of a flood should be considered as the chances of occurrence of a flood of that size (or one larger) in any one year. Stated another way, the chances of the flood in any one year being equaled or exceeded by floods of the magnitudes indicated as the 5-, 1 0-, 25-, or 100-year floods have ratios of 20: 100, 10: 100, 4: 100, and 1: 100, respectively.

Many methods of flood frequency determinations based on streamflow data have been published. Excellent summaries of these methods, along with comments on factors affecting their accuracy and limitations, are contained in the papers entitled "Review of Flood Frequency Methods" [27] and "Methods of Flow Frequency Analysis" [28]. While the many methods of flood frequency determinations made from streamflow data are all based on acceptable statistical procedures, the difference in methodology can give appreciably different results when extensions are made beyond the range of adequate data. To provide for a uniformity in Federal water resources planning, the Water Resources Council has recommended that all Government agencies use the Log-Pearson type III distribution as a base method. The method is described in the publication "A Uniform Technique for Determining Flood Flow Frequencies" [29]. Hazen's method [301 gives results that are comparable to those obtained with the Log-Pearson type III method and is easier to use when computations are made by hand with or without the aid of mechanical calculating machines. A procedural outline for Hazen computations is presented in section 59 of "Design of Small Dams," second edition [31].

If streamflow data for a period of 20 years or more are available for the subject watershed or comparable watersheds, frequency curve computations yield acceptable results for estimates up to the 25-year flood and may be extrapolated to indicate the 100-year flood with a fair assurance of obtaining acceptable values.

G-29. ***Hydrographs for Estimating Diversion Requirements During Construction.*** -Usually, inflow design flood (IDF) studies include hydrographs of floods for different frequencies of occurrence to be used for estimation diversion requirements during construction of a dam.

The hydrograph of a particular frequency flood is usually sketched to conventional shape using the peak discharge value and corresponding volume value obtained from computed frequency curves. In some instances, a peak discharge and associated volume of a recorded flood will correspond closely with a particular frequency value, in which case the recorded flood hydrograph is used.

J. FINAL-TYPE INFLOW DESIGN FLOOD STUDIES

G-30. General. -Preparations of final-type inflow design flood (IDF) studies differ from preliminary studies only in the degree of refinement used to estimate each variable causing flood runoff. For example, a basin unitgraph may be derived from a single large flood hydrograph in a preliminary study, whereas in a final-type study several flood hydrographs are analyzed and a selected basin unitgraph tested by reproduction of recorded flood hydrographs. Perhaps the most important consideration in the preparation of final-type studies is making certain that all available hydrological and meteorological data available, including historical and recent events, have been considered properly. A hydrometeorologist prepares the design storm study for the basin, including therein design temperatures and wind velocities if rain-on-snow floods are to be considered. Preliminary estimates of each flood-producing variable are reviewed and revised if additional data so indicate, Preliminary dam and reservoir operation plans are examined for certainty that the critical IDF situation for the chosen type of design and operation has been used.

Hydrologists and hydrometeorologists must estimate effects of ever-varying natural phenomena. Studies of these phenomena as related to a particular watershed begin with the inception of a project and continue thereafter, unless the project is determined infeasible and not built.

G-3 1. ***Flood Routing Criteria.*** -Normally, the reservoir pool is assumed to be full to the top of conservation storage at the start of the routing of the inflow design flood (IDF). However, when either inviolate or joint use flood control space is provided, the determination of space available at the beginning of the inflow design flood will depend upon the spacing of preceding storms, the relative magnitude of snowmelt contribution to the design flood, and the operational criteria proposed for the reservoir.

(a) ***Preceding Storms.-In*** some areas of the west, for example areas for which the Gulf of Mexico is the moisture source, the meteorological situation is such that a major storm could occur a few days prior to the maximum possible storm. In these areas, the flood control pool is assumed to be partially or completely occupied at the start of the inflow design flood. The determination of the portion of flood control pool that is occupied depends upon the distance of the area from the moisture source and a study of historical flood events in the area.

(b) ***Seasonal Flood Hydrograph.*** -For those areas in which floods occur on a fixed seasonal basis, largely as the result of snowmelt, it is frequently desirable to prepare a flood-season hydrograph including the inflow design flood and maximum antecedent and supervening flows that could reasonably be expected to occur with the inflow design flood. This hydrograph is then routed through the reservoir with the conservation pool full at the beginning of the season inflow, if that assumption can be justified on the basis of carryover storage. Otherwise, the minimum drawdown for the beginning date of seasonal inflow is selected from project operation studies.

(c) Operational Criteria. -The assumed reservoir elevation at the start of the inflow design flood will also be dependent upon the type of flood control space, which may be a fixed inviolate amount or a varying amount, normally referred to as joint use storage space.

The varying amount of flood control storage required will be based on operational parameters which show the needed amount of flood control storage based on antecedent precipitation, or the needed amount of storage based on forecasts of the seasonal runoff expected from the snowcover measurements.

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<<Appendix H

Sample Specifications for Concrete

H-l. ***Introduction.*** -Designs of any structure are based on assumptions regarding the quality of work which will be obtained during construction. It is through the means of specifications that the assumed quality is described, and it is important that conformance to the specifications be obtained for all work.

This appendix includes sample specifications for concrete in the dam and its appurtenances. For the construction of a particular dam, these specifications will be supplemented by local conditions, selected provisions, and special measures required for the construction of the structure.

The sample specifications are written on the basis that the concrete mixes to be used in the work will be designed and controlled by the purchaser (referred to in the specifications as the Contracting Authority or simply as the Authority) within the maximum water to cement or water to cement plus pozzolan ratio and slump limitations specified, the limitations for quality and grading of aggregates, and the limitations for the other materials as specified. Also, the specifications are written on the basis that the quantity of sand and each size of coarse aggregate to be used in the concrete mixes will be determined by the purchaser. The quality limitations shown in the specifications for sand and coarse aggregate are considered as standard limits. These limits may be reduced when only substandard materials are available within economical hauling distance, and provided it has been determined by tests of concrete made with such aggregates that durable concrete meeting the design strength criteria can be produced.

Under these specifications the purchaser's own engineering force or an engineering organization retained by the purchaser would accomplish testing of proposed aggregates and other materials, perform the design of mixes, and handle the inspection and quality testing throughout the contract. If the purchaser will require the contractor to provide such mix design, inspection and control, the specifications should so provide and should include specific design compressive strength(s) at designated age(s) for the concrete. The concrete mixes should be designed to provide compressive strengths of test cylinders such that 80 percent of the cylinders will have compressive strength(s) at the specified age(s) greater than the design compressive strength [1].[1]

References to "designations" in the sample specifications refer to designations in the appendix of the Bureau of Reclamation Concrete Manual, eighth edition [1]. Where materials or other requirements are to conform to Federal specifications, or other standard specifications such as ASTM, the construction specifications for specific work should provide that the specifications for the materials or requirements concerned should be in compliance with the latest editions or revisions thereof in effect on the date bids are received or award of contract is made, whichever is appropriate.

H-2. ***Contractor's Plants, Equipment, and Construction Procedures.*** -Prior to the installation of the contractor's plants and

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[1] Numbers in brackets refer to items in the bibliography, sec. H-25.

equipment for processing, handling, transporting, storing, and proportioning concrete ingredients, and for mixing, transporting, and placing concrete, the contractor shall submit drawings covering his plans for approval by the Contracting Authority, showing proposed plant arrangement, including plans of locations and description of facilities for sampling of concrete and concrete materials as hereinafter provided. Included with the plans shall be a description of the equipment the contractor proposes to use in sufficient detail that an adequate review can be accomplished. The drawings and description of plant, equipment, and sampling and testing facilities shall be submitted at least 60 days prior to plant erection.

After completion of installation, the operation of the plant and equipment shall be subject to the approval of the Contracting Authority.

Sampling and testing facilities for use by the Authority shall be provided by the contractor and shall include power-driven mechanical sampling devices, satisfactory to the Authority, as may be necessary for procuring and handling representative test samples of aggregates and other concrete materials during batching; and for obtaining samples of concrete as discharged from the mixers, for mixer efficiency, slump, and other tests, except that power-driven mechanical sampling devices will not be required for sampling concrete from truck mixers if and when the use of truck mixers is permitted by these specifications. The concrete sampling device shall be capable of procuring samples of concrete from any point in the discharge stream as the concrete is being discharged from the mixer.

After completion of the plant installation, the operation of the sample taking facilities shall be demonstrated to the satisfaction of the Authority that they are suitable for the purpose intended. If truck mixers are used where permitted by these specifications, the contractor shall provide a stable, level platform with adequate shelter, satisfactory to the Authority, for concrete tests at the point of discharge from the truck mixers. The contractor shall also provide ample and protected working space adjacent to the batching and mixing plants, free from plant vibration; and shall furnish necessary utilities such as compressed air, water, heat, and electrical power for operation of the Authority's testing equipment and for execution of tests by Authority personnel of concrete and concrete materials at the batching and mixing plants.

Where these specifications require specific types of equipment to be used or specific procedures to be followed, such requirements are not to be construed as prohibiting use by the contractor of alternative types of equipment or procedures if it can be demonstrated to the satisfaction of the Authority that equal results will be obtained by the use of such alternatives. Approval of plants and equipment or their operation, or of any construction procedure, shall not operate to waive or modify any provisions or requirement contained in these specifications governing the quality of the materials or of the finished work.

The cost of providing facilities and working space for procuring and handling representative test samples of concrete and concrete materials at the batching and mixing plants shall be included in the prices bid in the schedule for concrete.

The contractor shall keep the Authority advised as to when batching and mixing of concrete, installation of reinforcement and forming, preparations for placing and placing of concrete, finishing, and repair of concrete will be performed. Unless inspection is waived in each specific case, these construction activities shall be performed only in the presence of a duly authorized Authority inspector.

H-3. ***Composition.*** -(a) *General.* -Concrete shall be composed of cement, pozzolan, sand, coarse aggregate, water, and admixtures as specified, all well mixed and brought to the proper consistency. It is contemplated that pozzolan will be used in all concrete except for miscellaneous items of concrete where elimination of pozzolan is directed by the Contracting Authority.

(b) ***Maximum Size of Aggregate.*** -The maximum size of coarse aggregate in concrete for any part of the work shall be the largest of the specified sizes, the use of which is practicable from the standpoint of satisfactory consolidation of the concrete by vibration.

Except where it is determined by the Authority that, owing to closely spaced reinforcement or other reasons, the use of a smaller maximum size of aggregate is necessary to obtain satisfactory placement of the concrete, the maximum size of aggregate shall be as follows:

(1) Six-inch maximum-size aggregate shall, in general, be used in concrete for the dam, stilling basins, gravity walls, and elsewhere in other equally massive portions of structures where concrete containing the 6-inch maximum-size aggregate can be properly placed.

(2) Three-inch maximum-size aggregate shall be used in concrete for walls that are 15 inches or more in thickness and in slabs that are 8 inches or more in thickness, such as in massive floors and walls, and elsewhere where concrete containing 6-inch maximum-size aggregate cannot be placed, except that the requirements of subsection (3) below shall apply for tunnels, and for structures under conditions indicated.

(3) Three-inch maximum-size aggregate shall be used in concrete in tunnels where the concrete is 12 inches or more in thickness and the reinforcement, if any, consists of only one row or will not otherwise prevent satisfactory placement of the concrete, as determined by the Authority: ***Provided,*** that the contractor may use 2½-inch maximum-size aggregate to facilitate pumping: ***Provided further,*** that the contractor may use 2½-inch maximum-size aggregate in concrete that would otherwise contain 3-inch maximum-size aggregate whenever concrete containing 2½-inch maximum-size aggregate is being used at that time in work requiring pumping. One and one-half-inch maximum-size aggregate shall be used in concrete in tunnels where the concrete is less than 12 inches in thickness and for greater thicknesses when it is determined by the Authority that concrete containing a larger maximum size of aggregate cannot be properly placed.

(4) One and one-half-inch maximum-size aggregate shall be used in concrete for walls (except tunnel walls) that are less than 15 inches in thickness and in slabs that are less than 8 inches in thickness. However, where the walls or slabs are so heavily reinforced that 1%inch size aggregate cannot be properly placed, as determined by the Authority, ¾-inch maximum-size aggregate may be permitted.

(5) In locations where concrete is to be placed against excavated surfaces and the thickness of concrete to be placed is greater than that shown on the drawings, correspondingly larger maximum size aggregate from that specified for the thickness of concrete shown on the drawings shall be used: ***Provided,*** that aggregate with a maximum size greater than that indicated above will not be required.

(c) Mix Proportions. -The proportions in which the various ingredients are to be used for different parts of the work and the appropriate water to portland cement plus pozzolan ratio will be determined by the Authority. Adjustments in the mix proportions and water to portland cement plus pozzolan ratio will be made by the Authority from time to time during the progress of the work, as tests are made of samples of the aggregates and the resulting concrete. These adjustments will have the objective of procuring concrete having suitable workability, density, impermeability, durability, and required strength, without the use of an excessive amount of cement.

It is contemplated that the composition of the concrete will be within the ranges given in the accompanying tabulation.

The proportions shown in the referenced tabulation may be modified by the Authority to suit the work or the nature of the materials, or to comply with limitations on the water to portland cement plus pozzolan ratio, and the contractor shall be entitled to no extra compensation by reason of such modification.

The net water to portland cement plus pozzolan ratio of the concrete (exclusive of water absorbed by the aggregates) shall not

Maximum size of aggregate (inches)	Cementing materials, portland cement plus pozzolan (approximate)		Sand, percent of total aggregate, by weight	Coarse aggregate, percent of total coarse aggregate only, by weight			
	Total pounds per cubic yard of concrete	Percent pozzolan (by weight of portland cement plus pozzolan)		3/16 to ¾ inch	¾ to 1% inches	1% to 3 inches	3 to 6 inches
6 3 1% ¾	(Values to be determined by laboratory tests and inserted here for specifications.)						

exceed 0.47, by weight, for concrete in thin sections of structures which will be exposed to frequent alternations of freezing and thawing, such as curbs, gutters, sills, the top 2 feet of walls, piers, and parapets; and walls of structures in the range of fluctuating water levels or subject to spray. The net water to portland cement plus pozzolan ratio shall not exceed 0.53, by weight, for other concrete in structures which will be exposed to freezing and thawing. The net water to portland cement plus pozzolan ratio shall not exceed 0.60, by weight, for mass concrete in the dam, stilling basin, gravity walls, and elsewhere in other equally massive portions of structures; and for concrete in structures that will be covered with fill material or be continually submerged or otherwise protected from freezing and thawing.

(d) *Consistency.* -The amount of water used in the concrete shall be regulated as required to secure concrete of the proper consistency and to adjust for any variation in the moisture content or grading of the aggregates as they enter the mixer. Addition of water to compensate for stiffening of the concrete before placing will not be permitted. Uniformity in concrete consistency from batch to batch will be required.

The slump of the concrete, after the concrete has been deposited but before it has been consolidated, shall not exceed 2 inches for mass concrete; for concrete in the tops of walls, piers, parapets, and curbs; and for concrete in slabs that are horizontal or nearly horizontal. Similarly, the slump shall not exceed 4 inches for concrete in sidewalls and arch of tunnel lining; and 3 inches for all other concrete. The Authority reserves the right to require a lesser slump whenever concrete of such lesser slump can be consolidated readily into place by means of the vibration specified in section H-18(c) (Consolidation). The use of buckets, chutes, hoppers, or other equipment which will not readily handle and place concrete of such lesser slump will not be permitted.

(e) *Tests.* -The compressive strength of the concrete will be determined by the Authority through the medium of tests of 6- by 12-inch cylinders made and tested in accordance with designations 29 to 33, inclusive, of the eighth edition of the Bureau of Reclamation Concrete Manual [1], except that, for all concrete samples from which cylinders are to be cast, the pieces of coarse aggregate larger than 1% inches will be removed by screening or hand picking. Slump tests will be made by the Authority in accordance with designation 22.

H-4. ***Cement.*** -(a) General. -Cement for concrete, mortar, and grout shall be furnished by the contractor. The cement shall be free from lumps, unground clinker, tramp metal, and other foreign material, and shall be otherwise undamaged when used in concrete. If the cement is delivered in paper bags, empty paper bags shall be disposed of as directed. The contractor shall inform the Contracting Authority in writing, at least 60 days before first shipments are required, concerning the mill or mills from which the cement is to be shipped; whether cement will be ordered in bulk or in bags; and the purchase order number, contract number, or other designation that will identify the cement to be used by the contractor.

When bulk cement is not unloaded from the primary carriers directly into weathertight hoppers at the batching plant, transportation from the mill, railhead, or intermediate storage to the batching plant shall be accomplished in adequately weathertight trucks, conveyors, or other means which will protect the cement completely from exposure to moisture. Separate facilities, other than those provided for pozzolan, shall be provided for unloading, transporting, storing, and handling bulk cement. Locked unloading facilities shall be provided, and unloading of cement shall be performed only in the presence of the Authority or his representative. Immediately upon receipt at the jobsite, bulk cement shall be stored in dry, weathertight, and properly ventilated bins which shall be constructed so that there will be no dead storage. All storage facilities shall be subject to approval and shall be such as to permit easy access for inspection and identification.

The bins shall be emptied and cleaned by the contractor when so directed; however, the intervals between required cleanings will normally be not less than 4 months. If cement is obtained from more than one cement plant, shipments from each plant shall be blended with those from the other plant or plants by placing the cement from the different plants in alternate layers when unloading into silos at the railhead or at the jobsite, or by any other method satisfactory to the Authority. To prevent undue aging of cement furnished in bags, after delivery, the contractor shall use the bagged cement in the chronological order in which it was delivered to the jobsite. Each shipment of cement in bags shall be stored so that it may readily be distinguished from other shipments.

The cement shall meet the requirements of Federal Specification SS-C- 192G [9], including Amendment 3 for type II, low-alkali cement, and shall meet the false-set limitation specified therein. In addition, cement for contraction joint grouting shall be air separated, and 100 percent of the finished product, after processing at the cement plant, shall pass a No. 30 United States standard sieve and 97.7 percent shall pass a No. 100 United States standard sieve. Cement for contraction joint grouting shall also be screened at the jobsite through a No. 16 crimped screen which shall be installed by the contractor between the mixer and agitator in the grout plant. The cement for contraction joint grouting shall be furnished in waterproof bags which will prevent hydration of the cement from exposure and also prevent lumping of the cement due to warehouse set for a minimum of 90 days. Cement for foundation grouting shall be furnished in bags: *Provided*, that bulk cement may be used for such grouting if a suitable method, satisfactory to the Authority, is used for weighing and accounting for the cement used.

(b) *Inspection.* -Except for sieve fineness of cement for contraction joint grouting, the cement will be sampled and tested by the Authority in accordance with Federal Test Method Standard No. 158A [11], including Change Notice 1 thereto, except that for initial penetration under method 2501.1 the rod shall be released 20 seconds after completion of mixing, and except that the note at the end of method 2501.1 concerning variations in initial penetration will be disregarded.

Fineness tests of the cement for contraction joint grouting will be made by the Authority in accordance with ASTM Designation C 184 [5], except that the tests will be performed on No. 30 and No. 100 sieves.

Acceptance tests, except for false set but including fineness tests, will be made on samples taken as bins of cement are filled and reserved for exclusive Authority use. Acceptance tests for false set will be made on samples taken from the cement at the latest time, prior to shipment in cars or trucks, that the cement is still in possession of the cement company. Cement not meeting test requirements will be rejected, and the contractor shall be entitled to no adjustments in price or completion time by reason of any delays occasioned thereby.

The contractor will be charged the cost of testing of all Authority-tested cement which has been ordered in excess of the amount of cement used for the work under these specifications. The charges to be made for the cost of testing excess cement will be at the rate of 3.5 cents per hundredweight (cwt.), which charge includes the Authority overhead, and will be deducted from payments due the contractor.

(c) Measurement and Payment.—

Measurement, for payment, of cement furnished in bags will be on the basis of the number of bags of cement used at the mixer. Measurement, for payment, of bulk cement will be on the basis of batch weights at the batching plant. Any cement, either bulk or in bags, used for grouting, finishing, or other miscellaneous work will be measured for payment in the most practicable manner. One bag of cement shall be considered as 0.94 hundredweight.

Payment will be made for cement used in concrete placed within the. pay lines for concrete; and for cement used in concrete placed outside the concrete pay lines, unless the requirement for such concrete is determined by the Authority to be the result of careless excavation, or excavation intentionally performed by the contractor to facilitate his operations. No payment will be made for cement used as follows: cement used in wasted concrete, mortar, or grout; cement used in the replacement of damaged or defective concrete; cement used in extra concrete required as a result of careless excavation; and cement used in concrete placed by the contractor in excavation intentionally performed by the contractor to facilitate his operations. As determined by the Authority, payment will be made for a reasonable amount of cement used in grout required to keep the pipelines full during the grouting operations.

Payment for furnishing and handling cement will be made at the applicable unit prices per hundredweight or bag bid therefor in the schedule, which unit prices shall include the cost of rail and truck transportation of the cement from the mill to the jobsite and the cost of storing the cement.

H-5. ***Pozzolan.*** -(a) ***General.*** -Pozzolan for concrete shall be furnished by the contractor. The contractor shall use pozzolan concrete as provided in section H-3 (Composition). The pozzolan shall be in accordance with Federal Specification SSP-570B [101.

When bulk pozzolan is not unloaded from primary carriers directly into weathertight hoppers at the batching plant, transportation from the source railhead or intermediate storage to the batching plant shall be accomplished in adequately designed trucks, conveyors, or other means which will protect the pozzolan completely from exposure to moisture. Separate facilities, other than those for cement, shall be provided for unloading, transporting, storing, and handling bulk pozzolan. Locked unloading facilities shall be provided and unloading of pozzolan shall be performed only in the presence of the Contracting Authority or his representative.

Immediately upon receipt at the jobsite, bulk pozzolan shall be stored in dry, weathertight, and properly ventilated bins. All storage facilities shall be subject to approval and shall be such as to permit easy access for inspection and identification. Sufficient pozzolan shall be in storage at all times to complete any concrete lift or placement started. The bins shall be emptied and cleaned by the contractor when so directed; however, the intervals between required cleanings will normally be not less than 4 months. The pozzolan shall be free from lumps and shall be otherwise undamaged when used in concrete.

The contractor shall inform the Authority in writing, within 60 days after date of notice to proceed, concerning the source or sources from which he proposes to obtain the pozzolan; together with information as to location, shipping point or points, purchase order number, contract number, or other designation and information that will identify the pozzolan to be used by the contractor.

(b) Inspection. -The pozzolan will be sampled and tested by the Authority in accordance with Federal Specification SS-P-570B [10]. Acceptance tests will be made on a lot or lots of pozzolan, which lot or lots shall be reserved in bulk storage in sealed bins at the source for exclusive Authority use. Untested lots shall not be intermingled or combined with tested and approved lots until such lots have been tested and approved. Pozzolan will also be sampled at the jobsite when determined necessary. Release for shipment and approval for use will be based on compliance with 7-day lime-pozzolan strength requirements and other physical and chemical and uniformity requirements for which tests can be completed by the time the 7-day lime-pozzolan strength test is completed. Release for shipment and approval for use on the above basis will be contingent on continuing compliance with the other requirements of the specifications. No pozzolan shall be shipped until notice has been given that the test results are satisfactory and all shipments will be made under supervision of the Authority. Any lot or lots of pozzolan not meeting test requirements will be rejected. Rejected pozzolan shall be replaced with acceptable pozzolan, and the contractor shall be entitled to no adjustments in price or completion time by reason of any delays occasioned thereby.

The contractor will be charged the cost of testing of all Authority-tested pozzolan which has been ordered in excess of the amount of pozzolan used for the work under these specifications. The charges to be made for the cost of testing excess pozzolan will be at the testing rate per ton plus overhead cost to the Authority and will be deducted from payments due the contractor.

(c) Measurement and *Payment.*—Measurement, for payment, of pozzolan will be made on the basis of batch weights at the batching plant with deductions made for the percentage of moisture in the pozzolan. The moisture content will be determined by heating a 500-gram sample to constant weight in an oven at 105° C. The percentage of moisture will be 100 times the quantity obtained by dividing the loss in weight, in grams, by the weight in grams of the moist sample. Any pozzolan used for miscellaneous work will be measured in the most practicable manner.

Pozzolan will be paid for on the basis of the number of tons (2,000 pounds net dry weight) used in the work covered by these specifications. No payment will be made for pozzolan used as follows: pozzolan used in wasted concrete; pozzolan used in the replacement of damaged or defective concrete; pozzolan used in extra concrete required as a result of careless excavation; and pozzolan used in concrete placed by the contractor in excavation intentionally performed by the contractor to facilitate his operations.

Payment for furnishing and handling pozzolan will be made at the unit price per ton bid therefor in the schedule, which unit price shall include the cost of rail and truck transportation of the pozzolan from the mill to the jobsite and the cost of storing the pozzolan.

H-6. ***Admixtures.*** -(a) *Accelerator.*—Calcium chloride shall not be used in concrete in which aluminum or galvanized metalwork is to be embedded or in concrete where it may come in contact with prestressed steel. The contractor shall use 1 percent of calcium chloride, by weight of the cement, in all other concrete placed when the mean daily temperature in the vicinity of the worksite is lower than 40° F. Calcium chloride shall not be used otherwise, except upon written approval of the Contracting Authority. Request for such approval shall state the reason for using calcium chloride and the percentage of calcium chloride to be used and the location of the concrete in which the contractor desires to use the calcium chloride. Calcium chloride shall not be used in excess of 2 percent, by weight of the cement. Calcium chloride shall be measured accurately and shall be added to the batch in solution in a portion of the mixing water. Use of calcium chloride in the concrete shall in no way relieve the contractor of responsibility for compliance with the requirements of these specifications governing protection and curing of the concrete.

(b) ***Air-Entraining Agents*** .-An air-entraining agent shall be used in all concrete. The agent used shall conform to ASTM Designation C 260 [6], except that the limitation and test on bleeding by concrete containing the agent and the requirement relating to time of setting shall not apply. The agent shall be of uniform consistency and quality within each container and from shipment to shipment. Agents will be accepted on manufacturer's certification of compliance with specifications: ***Provided,*** that the Authority reserves the right to require submission of and to perform tests on samples of the agent prior to shipment and use in the work and to sample and test the agent after delivery at the jobsite.

The amount of air-entraining agent used in each concrete mix shall be such as will effect the entrainment of the percentage of air shown in the following tabulation in the concrete as discharged from the mixer:

Maximum size of coarse aggregate in inches	***Total air, percent by volume of concrete***
¾	6.0 plus or minus 1
1 %	4.5 plus or minus 1
3	3.5 plus or minus 1
6	3.0 plus or minus 1

The agent in solution shall be maintained at uniform strength and shall be added to the batch in a portion of the mixing water. This solution shall be accurately batched by means of a reliable mechanical batcher which shall be so constructed that the full measure of solution added to each batch of concrete can be observed in a sight gage by the plant operator prior to discharge of the solution into the mixer. When calcium chloride is being used in the concrete, the portion of the mixing water containing the air-entraining agent shall be introduced separately into the mixer.

(c) Water-Reducing, Set-Controlling Admixture. -The contractor shall, except as hereinafter provided, use a water-reducing, set-controlling admixture, referred to herein as WRA, in all concrete. The WRA used shall be either a suitable lignosulfonic-acid or hydroxylated-carboxylic-acid type.

The WRA shall be of uniform consistency and quality within each container and from shipment to shipment. WRA will be accepted on manufacturer's certification of conformance to Bureau of Reclamation "Specifications and Method of Test for Water-Reducing, Set-Controlling Admixtures for Concrete," dated August 1, 1971: *Provided*, that the Authority reserves the right to require submission of and to perform tests on samples of the agent prior to shipment and use in the work and to sample and test the agent after delivery at the jobsite.

If Authority testing of the WRA is required, the contractor shall submit a sample of the WRA and five bags (94 pounds each) of the cement proposed for use in the work at least 90 days before use is expected. The size of the sample of WRA to be submitted shall be 1 liquid gallon.

The quantity of WRA to be used in each concrete batch shall be determined by the Authority and for the lignosulfonic-acid type shall not exceed 0.40 percent, by weight of cement plus pozzolan, of solid crystalline lignin, and for the hydroxylated-carboxylic-acid type shall not exceed 0.50 percent, by weight of cement plus pozzolan, of liquid.

Since the quantity of WRA required will vary with changing atmospheric conditions, the quantity used shall be commensurate with the prevailing conditions. The Authority reserves the right to use lesser quantities or no WRA in concrete for any part of the work, depending on climatic or other job conditions, and the contractor shall be entitled to no additional compensation by reason of reduction in or elimination of WRA in any concrete to be placed under these specifications.

The WRA solution shall be measured for each batch by means of a reliable visual mechanical dispenser. The WRA, in a suitably dilute form, may be added to water containing air-entraining agent for the batch if the materials are compatible with each other, or shall be introduced separately to the batch in a portion of the mixing water if the two are incompatible.

When requested, the contractor shall submit test data by the manufacturer showing effects of the WRA on mixing water requirements, setting time of concrete, and compressive strength at various ages up to 1 year.

The contractor shall be responsible for any difficulties arising or damages occurring as a result of the selection and use of WRA, such as delay or difficulty in concrete placing or damage to the concrete during form removal. The contractor shall be entitled to no additional compensation above the unit prices bid in the schedule for concrete by reason of such difficulties.

(d) *Furnishing Admixtures.* -Air-entraining agent, accelerator, and WRA, as required, shall be furnished by the contractor, and the cost of the materials and all costs incidental to their use shall be included in the applicable prices bid in the schedule for concrete in which the materials are used.

H-7. Water.-The water used in concrete, mortar, and grout shall be free from objectionable quantities of silt, organic matter, alkali, salts, and other impurities.

H-8. *Sand.* -(a) *General.* -The term "sand" is used to designate aggregate in which the maximum size of particles is 3/16 of an inch. Sand for concrete, mortar, and grout shall be furnished by the contractor and shall be natural sand, except that crushed sand may be used to make up deficiencies in the natural sand grading. The contractor shall maintain at least three separate stockpiles of processed sand; one to receive wet sand, one in the process of draining, and one that is drained and ready for use. Sand to be used in concrete shall be drawn from the stockpile of drained sand which shall have been allowed to drain for a minimum of 48 hours. Sand, as delivered to the batching plant, shall have a uniform and stable moisture content, which shall be less than 6 percent free moisture.

(b) *Quality.* -The sand shall consist of clean, hard, dense, durable, uncoated rock fragments. The maximum percentages of deleterious substances in the sand, as delivered to the mixer, shall not exceed the following values:

Deleterious substance	***Percent, by weight***
Material passing No. 200 screen (designation 16)	3
Lightweight material (designation 17)	2
Clay lumps (designation 13)	1
Total of other deleterious substances (such as alkali, mica, coated grains, soft flaky particles, and loam)	2

The sum of the percentages of all deleterious substances shall not exceed 5 percent, by weight. Sand producing a color darker than the standard in the colorimetric test for organic impurities (designation 14) may be rejected. Sand having a specific gravity (designation 9), saturated surface-dry basis, of less than 2.60 may be rejected. The sand may be rejected if the portion retained on a No. 50 screen, when subjected to 5 cycles of the sodium sulfate test for soundness (designation 19), shows a weighted average loss of more than 8 percent, by weight. The designations in parentheses refer to methods of tests described in the eighth edition of the Bureau of Reclamation Concrete Manual [1].

(c) *Grading.* -The sand as batched shall be well graded, and when tested by means of standard screens (designation 4) shall conform to the following limits:

Screen No.	***Individual percent, by weight, retained on screen***
4	0 to 5
8	* 5 to 15
16	*10 to 25
30	10 to 30
50	15 to 35
100	12 to 20
Pan	3 to 7

*If the individual percent retained on the No. 16 screen is 20 percent or less, the maximum limit for the individual percent retained on the No. 8 screen may be increased to 20 percent.

The grading of the sand shall be controlled so that at any time the fineness moduli (designation 4) of at least 9 out of 10 consecutive test samples of finished sand will not vary more than 0.20 from the average fineness modulus of the 10 test samples.

H-9. Coarse *Aggregate.* -(a) *General.* -The term "coarse aggregate," for the purpose of these specifications, designates aggregate of sizes within the range of 3/16 of an inch to 6 inches or any size or range of sizes within such limits. The coarse aggregate shall be reasonably well graded within the nominal size ranges hereinafter specified. Coarse aggregate for concrete shall be furnished by the contractor and shall consist of natural gravel or crushed rock or a mixture of natural gravel and crushed rock.

Coarse aggregate, as delivered to the batching plant, shall have a uniform and stable moisture content.

(b) *Quality.* -The coarse aggregate shall consist of clean, hard, dense, durable, uncoated rock fragments. The percentages of deleterious substances in any size of coarse aggregate, as delivered to the mixer, shall not exceed the following values:

	Percent, by weight
Material passing No. 200 screen (designation 16)	½
Lightweight material (designation 18)	2
Clay lumps (designation 13)	½
Other deleterious substances	1

The sum of the percentages of all deleterious substances in any size, as delivered to the mixer, shall not exceed 3 percent, by weight. Coarse aggregate may be rejected if it fails to meet the following test requirements:

(1) Los Angeles rattler test (designation 21).-If the loss, using grading A, exceeds 10 percent, by weight, at 100 revolutions or 40 percent, by weight, at 500 revolutions.

(2) Sodium sulfate test for soundness (designation 19).—If the weighted average loss after 5 cycles is more than 10 percent by weight.

(3) Specific gravity (designation 10).—If the specific gravity (saturated surface-dry basis) is less than 2.60.

The designations in parentheses refer to methods of test described in the eighth edition of the Bureau of Reclamation Concrete Manual [1].

(c) Separation. -The coarse aggregate shall be separated into nominal sizes and shall be graded as follows:

Designation of size (inches)	Nominal size range (inches)	Minimum percent retained on screens indicated	
		Percent	Size of screen (inches)
¾	3/16 to ¾	50	3/8
1 %	¾ to 1%	25	1¼
3	1½ to 3	20	2%
6	3 to 6	20	5

Coarse aggregate shall be finished screened on vibrating screens mounted over the batching plant, or at the option of the contractor, the screens may be mounted on the ground adjacent to the batching plant. The finish screens, if installed over the batching plant, shall be so mounted that the vibration of the screens will not be transmitted to, or affect the accuracy of the batching scales. The sequence of coarse aggregate handling and plant management shall be such that, if final and/or submerged cooling are used, excessive free moisture shall be removed and diverted outside of the plant by dewatering screens prior to finish screening so that a uniform and stable moisture content is maintained in the plant storage and batching bins. The method and rate of feed shall be such that the screens will not be overloaded and will operate properly in a manner that will result in a finished product which consistently meets the grading requirements of these specifications. The finished products shall pass directly to the individual batching bins. Material passing the 3/16-inch screen that is removed from the coarse aggregate as a result of the finished screening operation shall be wasted.

Separation of the coarse aggregate into the specified sizes, after finish screening, shall be such that, when the aggregate, as batched, is tested by screening on the screens designated in the following tabulation, the material passing the undersize test screen (significant undersize) shall not exceed 2 percent, by weight, and all material shall pass the oversize test screen:

Aggregate size designation (inches)	Size of **square** opening in screen (inches)	
	For undersize test	For oversize test
¾	No. 5 mesh (U.S. standard screen)	7/8
1½	5/8	1¾
3	1¼	3½
6	2½	7

Screens used in making the tests for undersize and oversize will conform to ASTM Designation E 11 [7] , with respect to permissible variations in average openings.

H-10. ***Production of Sand and Coarse Aggregate.*** -(a) ***Source of Aggregate.-Sand*** and coarse aggregate for concrete, and sand for mortar and grout may be obtained by the contractor from any approved source as hereinafter provided.

If sand and coarse aggregate are to be obtained from a deposit not previously tested and approved by the Contracting Authority, the contractor shall submit representative samples for preconstruction test and approval at least 60 days after date of notice to proceed. The samples shall consist of approximately 200 pounds each of sand and 3/16- to 3/4-inch size of coarse aggregate, and 100 pounds of each of the other sizes of coarse aggregate.

The approval of deposits by the Authority shall not be construed as constituting the approval of all or any specific materials taken from the deposits, and the contractor will be held responsible for the specified quality of all such materials used in the work.

In addition to preconstruction test and approval of the deposit, the Authority will test the sand and coarse aggregate during the progress of the work and the contractor shall provide such facilities as may be necessary for procuring representative samples.

If any deposit used by the contractor is located within an approved area owned or controlled by the Authority, no charge will be made to the contractor for materials taken from such deposit and used in the work covered by these specifications. Any royalties or other charges required to be paid for materials taken from deposits not owned or controlled by the Authority shall be paid by the contractor.

(b) *Developing Aggregate Deposit.* -The contractor shall carefully clear the area of the deposit, from which aggregates are to be produced, of trees, roots, brush, sod, soil, unsuitable sand and gravel, and other objectionable matter. If the deposit is owned or controlled by the Authority, the portion of the deposit used shall be located and operated so as not to detract from the usefulness of the deposit or of any other property of the Authority and so as to preserve, insofar as practicable, the future usefulness or value of the deposit. Materials, including stripping, removed from deposits owned or controlled by the Authority and not used in the work covered by these specifications shall be disposed of as directed.

The contractor's operations in and around aggregate deposits shall be in accordance with the provisions of the specifications sections on environmental protection.

(c) Processing Raw Materials. -Processing of the raw materials shall include screening, and washing as necessary, to produce sand and coarse aggregate conforming to the requirements of sections H-8 (Sand) and H-9 (Coarse Aggregate). Processing of aggregates produced from any source owned or controlled by the Authority shall be done at an approved site. Water used for washing aggregates shall be free from objectionable quantities of silt, organic matter, alkali, salts, and other impurities. To utilize the greatest practicable yield of suitable materials in the portion of the deposit being worked, the contractor may crush oversize material and any excess material of the sizes of coarse aggregate to be furnished, until the required quantity of each size has been secured: *Provided,* that crusher fines produced in manufacturing coarse aggregate that will pass a screen having 3/16-inch square openings shall be wasted or rerouted through the sand manufacturing plant. Crushed sand, if used to make up deficiencies in the natural sand grading, shall be produced by a suitable ball or rod mill, disk or cone crusher, or other approved equipment so that the sand particles shall be predominately cubical in shape and free from objectionable quantities of flat or elongated particles.

The crushed sand and coarse aggregate shall be blended uniformly with the uncrushed sand and coarse aggregate, respectively. Crushing and blending operations shall at all times be subject to approval by the Authority. The handling, transporting, and stockpiling of aggregates shall be such that there will be a minimum amount of fines resulting from breakage and abrasion of material caused by free fall and improper handling. Where excesses in any of the sand and coarse aggregate sizes occur, the contractor shall dispose of the excess material as directed by the Authority.

(d) *Furnishing Aggregates.* -The cost of producing aggregates required for work under these specifications and the cost of aggregates not obtained from a source owned or controlled by the Authority shall be included in the unit prices bid in the schedule for concrete in which the aggregates are used, which unit prices shall also include all expenses of the contractor in stripping, transporting, and storing the materials. The contractor shall be entitled to no additional compensation for materials wasted from a deposit, including crusher fines, excess material of any of the sizes into which the aggregates are required to be separated by the contractor, and materials which have been discarded by reason of being above the maximum sizes specified for use.

H-l 1. ***Batching.*** -(a) ***General.*** -The contractor shall provide equipment and shall maintain and operate the equipment as required to accurately determine and control the prescribed amounts of the various materials, including water, cement, pozzolan, admixtures, sand, and each individual size of coarse aggregate entering the concrete. The amounts of bulk cement, pozzolan, sand, and each size of coarse aggregate entering each batch of concrete shall be determined by separate weighing, and the amounts of water and each admixture shall be determined by separate weighing or volumetric measurement. Where bagged cement is used, the concrete shall be porportioned on the basis of integral bags of cement unless the cement is weighed.

When bulk cement, pozzolan, and aggregates are hauled from a central batching plant to the mixers, the cement and pozzolan for each batch shall either be placed in an individual compartment which during transit will prevent the cement and pozzolan from intermingling with each other and with the aggregates and will prevent loss of cement and pozzolan; or the cement and pozzolan shall be completely enfolded in and covered by the aggregates by loading the cement, pozzolan, and aggregates for each batch simultaneously into the batch compartment. The bins of batch trucks shall be provided with suitable covers to protect the materials therein from wind or wet weather. Each batch compartment shall be of sufficient capacity to prevent loss in transit and to prevent spilling and intermingling of batches as compartments are being emptied. If the cement and pozzolan are enfolded in aggregates containing moisture, and delays occur between filling and emptying the compartments the contractor shall, at his own expense, add extra cement to each batch in accordance with the following schedule:

****Hours of contact between cement and wet aggregate***	***Additional cement required***
0 to 2	0 percent
2 to 3	5 percent
3 to 4	10 percent
4 to 5	1.5 percent
5 to 6	20 percent
Over 6	Batch will be rejected.

*The Contracting Authority reserves the right to require the addition of cement for shorter periods of contact during periods of hot weather and the contractor shall be entitled to no additional compensation by reason of the shortened period of contact.

Batch bins shall be constructed so as to be self-cleaning during drawdown and the bins shall be drawn down until they are practically empty at least three times per week. Materials shall be deposited in the batch bins directly over the discharge gates. The 1%, 3-, and 6-inch coarse aggregates shall be deposited in the batcher bins through effective rock ladders, or other approved means. To minimize breakage, the method used in transporting the aggregates from one elevation to a lower elevation shall be such that the aggregates will roll and slide with a minimum amount of free fall.

Equipment for conveying batched materials from the batch hopper or hoppers to and into the mixer shall be so constructed, maintained, and operated that there will be no spillage of the batched materials or overlap of batches. Equipment for handling portland cement and pozzolan in the batching plant shall be constructed and operated so as to prevent noticeable increase of dust in the plant during the measuring and discharging of each batch of material. If the batching and mixing plant is enclosed, the contractor shall install exhaust fans or other suitable equipment for removing dust.

(b) ***Eyuipmen t.*** -The weighing and measuring equipment shall conform to the following requirements:

(1) The construction and accuracy of the equipment shall conform to the applicable requirements of Federal

Specification AAA-S-121d [8] for such equipment, except that an accuracy of 0.4 percent over the entire range of the equipment will be required.

The contractor shall provide standard test weights and any other equipment required for checking the operating performance of each scale or other measuring device and shall make periodic tests over the ranges of measurements involved in the batching operations. The tests shall be made in the presence of an Authority inspector, and shall be adequate to prove the accuracy of the measuring devices. Unless otherwise directed, tests of weighing equipment in operation shall be made at least once every month. The contractor shall make such adjustments, repairs, or replacements as may be necessary to meet the specified requirements for accuracy of measurement.

(2) Each weighing unit shall include a visible springless dial which will register the scale load at any stage of the weighing operation from zero to full capacity. The minimum clear interval for dial scale graduations shall be not less than 0.03 inch. The scales shall be direct reading to within 5 pounds for cement and 20 pounds for aggregate. The weighing hoppers shall be constructed so as to permit the convenient removal of overweight materials in excess of the prescribed tolerances. The scales shall be interlocked so that a new batch cannot be started until the weighing hoppers have been completely emptied of the last batch and the scales are in balance. Each scale dial shall be in full view of the operator.

(3) The equipment shall be capable of ready adjustment for compensating for the varying weight of any moisture contained in the aggregates and for changing the mix proportions.

(4) The equipment shall be capable of controlling the delivery of material for weighing or volumetric measurement so that the combined inaccuracies in feeding and measuring during normal operation will not exceed 1 percent for water; 1% percent for cement and pozzolan; 3 percent for admixtures; 2 percent for sand, ¾-inch aggregate, and 1½-inch aggregate; and 3 percent for 3- and 6-inch coarse aggregate.

(5) Convenient facilities shall be provided for readily obtaining representative samples of cement, pozzolan, admixtures, sand, and each size of coarse aggregate from the discharge streams between bins and the batch hoppers or between the batch hoppers and the mixers.

(6) The operating mechanism in the water-measuring device shall be such that leakage will not occur when the valves are closed. The water-measuring device shall be constructed so that the water will be discharged quickly and freely into the mixer without objectionable dribble from the end of the discharge pipe. In addition to the water-measuring device, there shall be supplemental means for measuring and introducing small increments of water into each mixer when required for final tempering of the concrete. This equipment shall introduce the added water well into the batch. Each water-measuring device shall be in full view of the operator.

(7) Dispensers for air-entraining agents, calcium chloride solutions, and WRA shall have sufficient capacity to measure at one time the full quantity of the properly diluted solution required for each batch, and shall be maintained in a clean and freely operating condition. Equipment for measuring shall be designed for convenient confirmation by the plant operator of the accuracy of the measurement for each batch and shall be so constructed that the required quantity can be added only once to each batch.

(8) The mixing plant shall be arranged so that the mixing action in at least one of the mixers can be conveniently observed from its control station. Provisions shall be made so that the mixing action of each of the other mixers can be observed from

a safe location which can be easily reached from the control station. Provisions shall also be made so that the operator can observe the concrete in the receiving hopper or buckets after being dumped from the mixers.

(9) Equipment that fails to conform to the requirements of this section shall be effectively repaired or satisfactorily replaced.

H-12. *Mixing.* -(a) ***General.*** -The concrete ingredients shall be mixed thoroughly in batch mixers of approved type and size and designed so as to positively ensure uniform distribution of all of the component materials throughout the mass at the end of the mixing period. The adequacy of mixing will be determined by the method of "Variability of Constituents in Concrete" in accordance with the provisions of designation 26 of the eighth edition of the Bureau of Reclamation Concrete Manual [1]. Mixers when tested shall meet the following criteria:

(1) The unit weight of air-free mortar in samples taken from the first and last portions of the batch as discharged from the mixer shall not vary more than 0.8 percent from the average of the two mortar weights.

(2) For any one mix, the average variability for more than one batch shall not exceed the following limits:

Number of tests	*Average variability (percent based on average mortar weight of all tests)*
3	0.6
6	.5
20	.4
90	.3

(3) The weight of coarse aggregate per cubic foot in samples taken from the first and last portions of the batch as discharged from the mixer shall not vary more than 5.0 percent from the average of the two weights of coarse aggregate.

The Contracting Authority reserves the right to either reduce the size of batch to be mixed or to increase the mixing time when the charging and mixing operations fail to produce a concrete batch which conforms throughout to the above-numbered criteria and in which the ingredients are uniformly distributed and the consistency is uniform. Water shall be added prior to, during, and following the mixer-charging operations. Overmixing, requiring addition of water to preserve the required consistency, will not be permitted. Any concrete retained in mixers so long as to require additional water in excess of 3 percent of the design mix water (net water-cement plus pozzolan ratio water, not including water absorbed by aggregates) to permit satisfactory placing shall be wasted. Any mixer that at any time produces unsatisfactory results shall be repaired promptly and effectively or shall be replaced.

Use of truck mixers in accordance with subsection (c) below will be permitted only for miscellaneous items of concrete work where and as approved by the Authority.

(b) ***Central Mixers.*** -Mixers shall not be loaded in excess of their rated capacity unless specifically authorized. The concrete ingredients shall be mixed in a batch mixer for not less than the period of time indicated in the following tabulation for various mixer capacities after all of the ingredients except the full amount of water are in the mixer, except that the mixing time may be reduced if, as determined by the Authority, thorough mixing conforming to subsections (a) (1) and (2) above can be obtained in less time.

Capacity of mixer	*Time of mixing*
2 cubic yards or less	1½ minutes
3 cubic yards	2 minutes
4 cubic yards	2% minutes
Larger than 4 cubic yards	To be determined by tests performed by the Authority

(c) ***Truck Mixers.-Use*** of truck mixers will be permitted only when the mixers and their operation are such that the concrete throughout the mixed batch and from batch to batch is uniform with respect to consistency and grading. Any concrete retained in truck mixers sufficiently long as to require additional water to permit placing shall be wasted.

Each truck mixer shall be equipped with (1) an accurate watermeter between supply tank and mixer, the meter to have indicating dials and totalizer, and (2) a reliable revolution counter, which can be readily reset to zero for indicating the total number of revolutions of the drum for each batch. Each mixer shall have affixed thereto a metal plate on which the drum capacities for both mixing and agitating are plainly marked in terms of volume of concrete in cubic yards and the maximum and minimum speeds of rotation of the drum in revolutions per minute.

Mixing shall be continued for not less than 50 nor more than 100 revolutions of the drum at the manufacturer's rated mixing speed after all the ingredients, except approximately 5 percent of the water which may be withheld, are in the drum. The mixing speed shall be not less than 5 nor more than 20 revolutions per minute. Thereafter, additional mixing, if any, shall be at the speed designated by the manufacturer of the equipment as agitating speed; except that after the addition of withheld water, mixing shall be continued at the specified mixing speed until the water is dispersed throughout the mix. After a period of agitation a few revolutions of the drum at mixing speed will be required just prior to discharging. In no case shall the specified maximum net water-cement plus pozzolan ratio be exceeded.

When a truck mixer or agitator is used for transporting concrete, the concrete shall be delivered to the site and the discharge completed within 1% hours after the introduction of the cement into the mixer. Each batch of concrete, when delivered at the jobsite from commercial ready-mix plants, shall be accompanied by a written certificate of batch weights and time of batching.

Mixers shall be examined daily for changes in condition due to accumulation of hard concrete or mortar or to wear of blades. No mixer shall be charged in excess of its rated capacity for mixing or agitating; however, if any mixer cannot produce concrete meeting the requirements heretofore specified when mixing at rated capacity, within the specified limitation on the number of revolutions of the mixing drum at mixing speed, the size of batch mixed in that mixer may be reduced until, upon testing, a uniformly mixed batch, conforming to the mixer performance tests as provided in subsection (a) above, is obtained.

H- *13. Temperature of Concrete.* -The temperature of mass concrete for the dam shall, when concrete is being placed, be not more than 50° F. and not less than 40° F. For all other concrete, the temperature of concrete when it is being placed shall be not more than 90° F. and not less than 40° F. in moderate weather or not less than 50° F. in weather during which the mean daily temperature drops below 40° F. Concrete ingredients shall not be heated to a temperature higher than that necessary to keep the temperature of the mixed concrete, as placed, from falling below the specified minimum temperature. Methods of heating concrete ingredients shall be subject to approval by the Contracting Authority.

If concrete is placed when the weather is such that the temperature of the concrete would exceed the maximum placing temperatures specified, as determined by the Authority, the contractor shall employ effective means as necessary to maintain the temperature of the concrete, as it is placed, below the maximum temperatures specified. These means may include placing at night; precooling the aggregates by cool airblast, immersion in cold water, vacuum processing, or other suitable method; refrigerating the mixing water; adding chip or flake ice to the mixing water; or a combination of these or other approved means. The contractor shall be entitled to no additional compensation on account of the foregoing requirements.

H-14. *Forms.* -(a) ***General.*** -Forms shall be used, wherever necessary, to confine the concrete and shape it to the required lines. Forms shall have sufficient strength to withstand the pressure resulting from placement and vibration of the concrete, and shall be maintained rigidly in position. Forms shall be sufficiently tight to prevent loss of mortar from the concrete. Chamfer strips shall be placed in the corners of forms so as to produce beveled edges on permanently exposed concrete surfaces. Interior angles on such surfaces and edges at formed joints will not require beveling unless requirement for beveling is indicated on the drawings. Inside forms for nearly horizontal circular tunnels having an inside diameter of 12 feet or more shall be constructed to cover only the arch and sides. The bottom 60° of the inside circumference shall be placed without forming: *Provided*, that the contractor may increase the angle of the inside circumference to be placed without forming on written approval of the Contracting Authority. Request for approval shall be accompanied by complete plans and description of the placing methods proposed to be used.

Forms for tunnel lining shall be provided with openings along each sidewall and in each arch, each opening to be not less than 2 by 2 feet. The openings shall be located in the crown and along each sidewall, as follows:

(1) Openings in the crown shall be spaced at not more than 8 feet on centers and shall be located alternately on each side of the tunnel centerline.

(2) Openings in sidewall forms for tunnels having an inside diameter less than 12 feet shall be located at midheight of the tunnel in each sidewall and shall be spaced at not more than 8 feet on centers along each sidewall.

(3) Openings in sidewall forms for tunnels having an inside diameter of 12 feet or more shall be located along two longitudinal lines in each sidewall, the locations of which are satisfactory to the Authority. The openings along the two selected longitudinal lines in each sidewall shall be staggered and shall be spaced at not more than 8 feet on centers along each longitudinal line.

The cost of all labor and materials for forms and for any necessary treatment or coating of forms shall be included in the unit prices bid in the schedule for the concrete for which the forms are used.

(b) *Form Sheathing and Lining.*-Wood sheathing or lining shall be of such kind and quality or shall be so treated or coated that there will be no chemical deterioration or discoloration of the formed concrete surfaces. The type and condition of form sheathing and lining, and the fabrication of forms for finishes F2, F3, and F4 shall be such that the form surfaces will be even and uniform. The ability of forms to withstand distortion caused by placement and vibration of concrete shall be such that formed surfaces will conform with applicable requirements of these specifications pertaining to finish of formed surfaces. Where finish F3 is specified, the sheathing or lining shall be placed so that the joint marks on the concrete surfaces will be in general alinement both horizontally and vertically. Where pine is used for form sheathing, the lumber shall be pinus ponderosa in accordance with the Standard Grading Rules of the Western Wood Products Association or shall be other lumber of a grading equivalent to that specified for pine. Plywood used for form sheathing or lining shall be concrete form, class I, grade B-B exterior, mill oiled and edge sealed, in accordance with Product Standard PS 1-66 of the Bureau of Standards [12]. Materials used for form sheathing or lining shall conform with the following requirements, or may be other materials producing equivalent results:

Required finish of formed surface	*Wood sheathing or lining*	*Steel sheathing or lining* *
F1	Any grade–S2E	Steel sheathing permitted. Steel lining permitted.
F2	No. 2 common or better, pine shiplap, or plywood sheathing or lining.	Steel sheathing permitted. Steel lining permitted if approved.
F3	No. 2 common or better pine tongue-and-groove or plywood sheathing or lining, except where special form material is prescribed.	Steel sheathing not permitted. Steel lining not permitted.
F4	For plane surfaces, No. 1 common or better pine tongue-and-groove or shiplap or plywood. For warped surfaces, lumber which is free from knots and other imperfections and which can be cut and bent accurately to the required curvatures without splintering or splitting.	Steel she-permitted. Steel lining not permitted.

*Steel "sheathing" denotes steel sheets not supported by a backing of wood boards. Steel "lining" denotes thin steel sheets supported by a backing of wood boards.

(c) Form ***Ties.*** -Embedded ties for holding forms shall remain embedded and, except where F1 finish is permitted, shall terminate not less than two diameters or twice the minimum dimension of the tie in the clear of the formed faces of the concrete. Where F1 finish is permitted, ties may be cut off flush with the formed surfaces. The ties shall be constructed so that removal of the ends or end fasteners can be accomplished without causing appreciable spalling at the faces of the concrete. Recesses resulting from removal of the ends of form ties shall be filled in accordance with section H-19 (Repair of Concrete).

(d) ***Cleaning and Oiling of Forms.-At*** the time the concrete is placed in the forms, the surfaces of the forms shall be free from encrustations of mortar, grout, or other foreign material. Before concrete is placed, the surfaces of the forms shall be oiled with a commercial -form oil that will effectively prevent sticking and will not soften or stain the concrete surfaces, or cause the surfaces to become chalky or dust producing. For wood forms, form oil shall consist of straight, refined, pale, paraffin base mineral oil. For steel forms, form oil shall consist of refined mineral oil suitably compounded with one or more ingredients which are appropriate for the purpose. The contractor shall furnish certification of compliance with these specifications for form oil.

(e) Removal of Forms.-To facilitate satisfactory progress with the specified curing and enable earliest practicable repair of surface imperfections, forms shall be removed as soon as the concrete has hardened sufficiently to prevent damage by careful form removal. Forms on upper sloping faces of concrete, such as forms on the watersides of warped transitions, shall be removed as soon as the concrete has attained sufficient stiffness to prevent sagging. Any needed repairs or treatment required on such sloping surfaces shall be performed at once and be followed immediately by the specified curing.

To avoid excessive stresses in the concrete that might result from swelling of the forms, wood forms for wall openings shall be loosened as soon as this can be accomplished without damage to the concrete. Forms for the openings shall be constructed so as to facilitate such loosening. Forms for conduits and tunnel lining shall not be removed until the strength of the concrete is such that form removal will not result in perceptible cracking, spalling, or breaking of edges or surfaces, or other damage to the concrete. Forms shall be removed with care so as to avoid injury to the concrete and any concrete so damaged shall be repaired in accordance with section H-19 (Repair of Concrete).

H-15. ***Tolerances for Concrete Construction.*** -(a) ***General.*** -Permissible surface irregularities for the various classes of concrete surface finish as specified in section H-20 (Finishes) are defined as "finishes," and are to be distinguished from tolerances as described herein. The intent of this section is to establish tolerances that are consistent with modern construction practice, yet are governed by the effect that permissible deviations will have upon the structural action or operational function of the structure. Deviations from the established lines, grades, and dimensions will be permitted to the extent set forth herein: ***Provided,*** that the Contracting Authority reserves the right to diminish the tolerances set forth herein if such tolerances impair the structural action or operational function of a structure or portion thereof.

Where specific tolerances are not stated in these specifications or shown on the drawings for a structure, portion of a structure, or other feature of the work, permissible deviations will be interpreted conformably to the tolerances stated in this section for similar work. Specific maximum or minimum tolerances shown on the drawings in connection with any dimension shall be considered as supplemental to the tolerances specified in this section, and shall govern. The contractor shall be responsible for setting and maintaining concrete forms within the tolerance limits necessary to insure that the completed work will be within the tolerances specified. Concrete work that exceeds the tolerance limits specified in these specifications or shown on the drawings shall be remedied or removed and replaced at the expense of and by the contractor.

(b) Tolerances for Dam Structures.-

(1) Variation of constructed linear outline from established position in plan	In any length of 20 feet, except in buried construction . . . . ½ inch Maximum for entire length, except in buried construction . . . . ¾ inch In buried construction . . . . . twice the above amounts
(2) Variation of dimensions to individual structure features from established positions	Maximum for overall dimension, except in buried construction . . . . 1% inches In buried construction . . . . 2% inches
(3) Variation from plumb, specified batter, or curved surfaces for all structures, including lines and surfaces of columns, walls, piers, buttresses, arch sections, vertical joint grooves, and visible arrises	In any length of 10 feet, except in buried construction . . . . ½ inch In any length of 20 feet, except in buried construction . . . . ¾ inch Maximum for entire length, except in buried construction . . . . 1% inches In buried construction . . . . twice the above amounts
(4) Variation from level or from grades indicated on the drawings for slabs, beams, soffits, horizontal joint grooves, and visible arrises	In any length of 10 feet, except in buried construction . . . . ¼ inch Maximum for entire length, except in buried construction . . . . ½ inch In buried construction . . . twice the above amounts
(5) Variation in cross-sectional dimensions of columns, beams, buttresses, piers, and similar members	Minus . . . . ¼ inch Plus . . . . ½ inch
(6) Variation in the thickness of slabs, walls, arch sections, and similar members	Minus . . . . ¼ inch Plus . . . . ½ inch
(7) Footings for columns, piers, walls, buttresses, and similar members:	
(a) Variation of dimensions in plan	Minus . . . . ½ inch Plus . . . . 2 inches
(b) Misplacement or eccentricity	2 percent of the footing width in the direction of misplacement but not more than . . . . 2 inches
(c) Reduction in thickness	. . . . 5 percent of specified thickness
(8) Variation from plumb or level for sills and sidewalls for radial gates and similar watertight joints*	. . . . Not greater than a rate of ⅛ inch in 10 feet
(9) Variation in locations of sleeves, floor openings, and wall openings	. . . . ½ inch

*Dimensions between sidewalls for radial gates shall be not more than shown on the drawings at the sills and not less than shown on the drawings at the top of the walls.

(10) Variation in sizes of sleeves, floor openings, and wall openings	¼ inch

(c) Tolerances for Tunnel Lining.-

(1) Departure from established alinement or from established grade	Free-flow tunnels and conduits	1 inch
	High-velocity tunnels and conduits	½ inch
(2) Variation in thickness, at any point	Tunnel lining	minus 0
	Conduits	minus 2% percent or ¼ inch, whichever is greater
	Conduits	plus 5 percent or ½ inch, whichever is greater
(3) Variation from inside dimensions		½ of 1 percent

(d) *Tolerances for Placing Reinforcing Bars and Fabric.-*

(1) Reinforcing steel, except for bridges:

(a) Variation of protective covering	With cover of 2% inches or less	¼ inch
	With cover of more than 2% inches	½ inch
(b) Variation from indicated spacing		1 inch

(2) Reinforcing steel for bridges:

(a) Variation of protective covering	With cover of 2% inches or less	⅛ inch
	With cover of more than 2% inches	¼ inch
(b) Variation from indicated spacing		1 inch

H-16. ***Reinforcing Bars and Fabric.*** -(a) ***Furnishing.*** -The contractor shall furnish all the reinforcing bars and fabric required for completion of the work. Reinforcing bars shall conform to ASTM Designation A 615, grade 40 or 60, or ASTM Designation A 6 17, grade 40 or 60. (See reference [3] or [4] .) Fabric shall be electrically welded-wire fabric and shall conform to ASTM Designation A 185 [2].

(b) *Placing.* -Reinforcing bars and fabric shall be placed in the concrete where shown on the drawings or where directed. Splices shall be located where shown on the drawings: ***Provided,*** that the location of splices may be altered subject to the written approval of the Contracting Authority, and ***Provided further,*** that, subject to the written approval of the Authority, the contractor may splice bars at additional locations other than those shown on the drawings. Reinforcing bars in splices located where shown on the drawings, in relocated splices approved by the Authority, or in additional splices approved by the Authority, will be included in the measurement, for payment, of reinforcing bars.

Unless otherwise prescribed, placement dimensions shall be to the centerlines of the bars. Reinforcement will be inspected for compliance with requirements as to size, shape, length, splicing, position, and amount after it has been placed.

Before the reinforcement is embedded in concrete, the surfaces of the bars and the surfaces of any bar supports shall be cleaned of heavy flaky rust, loose mill scale, dirt, grease, or other foreign substances which, in the opinion of the Authority, are objectionable.

Heavy flaky rust that can be removed by firm rubbing with burlap or equivalent treatment is considered objectionable.

Reinforcement shall be accurately placed and secured in position so that it will not be displaced during the placing of the concrete, and special care shall be exercised to prevent any disturbance of the reinforcement in concrete that has already been placed. Welding or tack welding of grade 60 or grade 75 reinforcing bars will not be permitted except at locations shown on the drawings. Chairs, hangers, spacers, and other supports for reinforcement may be of concrete, metal, or other approved material. Where portions of such supports will be exposed on concrete surfaces designated to receive F2 or F3 finish, the exposed portion of the supports shall be of galvanized or other corrosion-resistant material, except that concrete supports will not be permitted. Such supports shall not be exposed on surfaces designated to receive an F4 finish. Unless otherwise shown on the drawings, the reinforcement in structures shall be so placed that there will be a clear distance of at least 1 inch between the reinforcement and any anchor bolts, form ties, or other embedded metalwork.

(c) Reinforcement Drawings to be Prepared by the Contractor. -The contractor shall prepare and submit for approval of the Authority reinforcement detail drawings for all structures including bar-placing drawings, bar-bending diagrams, and bar lists.

The contractor's reinforcement detail drawings shall be prepared from reinforcement design drawings included with these specifications and from supplemental reinforcement design drawings to be furnished by the Authority. The position, size, and shape of reinforcing bars are not shown in all cases on the drawings included with these specifications. Supplemental reinforcement design drawings in sufficient detail to permit the contractor to prepare his reinforcement detail drawings will be furnished to the contractor by the Authority after final designs have been completed and after equipment data are received from equipment manufacturers. As the supplemental reinforcement design drawings may not be available in time to enable the contractor to purchase prefabricated reinforcing bars, it may be necessary for the contractor to purchase bars in stock lengths, and to cut and bend the bars in the field.

At least ________ days before scheduled concrete placement, the contractor shall submit to the Authority for approval three prints of each of his reinforcement detail drawings. The contractor's reinforcement detail drawings shall be prepared following the recommendations established by the American Concrete Institute's "Manual of Standard Practice for Detailing Reinforced Concrete Structures" (ACI 3 15-65) unless otherwise shown on the reinforcement design drawings. The contractor's drawings shall show necessary details for checking the bars during placement and for use in establishing payment quantities. Reinforcement shall conform to the requirements shown on the reinforcement design drawings.

The contractor's reinforcement detail drawings shall be clear, legible, and accurate and checked by the contractor before submittal. If any reinforcement detail drawing or group of drawings is not of a quality acceptable to the Authority, the entire set or group of drawings will be returned to the contractor, without approval, to be corrected and resubmitted. Acceptable reinforcement detail drawings will be reviewed by the Contracting Authority for adequacy of general design and controlling dimensions. Errors, omissions, or corrections will be marked on the prints, or otherwise relayed to the contractor, and one print of each drawing will be returned to the contractor for correction. The contractor shall make all necessary corrections shown on the returned prints. The corrected drawings need not be resubmitted unless the corrections are extensive enough, as determined by the Authority, to warrant resubmittal. Such Authority review and approval shall not relieve the contractor of his responsibility for the correctness of details or for conformance with the requirements of these specifications.

(d) ***Measurement and*** *Payment*.–Measurement, for payment, of reinforcing bars and fabric will be made only of the weight of the bars and fabric placed in the concrete in accordance with the drawings or as directed.

Payment for furnishing and placing reinforcing bars will be made at the applicable unit price per pound bid in the schedule for the various sizes of reinforcing bars and fabric, which unit prices shall include the cost of preparing reinforcement detail drawings, including bar-placing drawings and bar-bending diagrams; of submitting the drawings to the Authority; of preparing all necessary bar lists and cutting lists; of furnishing and attaching wire ties and metal or other approved supports, if used; and of cutting, bending, cleaning, and securing and maintaining in position, all reinforcing bars and fabric as shown on the drawings.

H - 17. ***Preparations for Placing.*** — (a) ***General.***-No concrete shall be placed until all formwork, installation of parts to be embedded, and preparation of surfaces involved in the placing have been approved. No concrete shall be placed in water except with the written permission of the Contracting Authority, and the method of depositing the concrete shall be subject to his approval. Concrete shall not be placed in running water and shall not be subjected to the action of running water until after the concrete has hardened. All surfaces of forms and embedded materials that have become encrusted with dried mortar or grout from concrete previously placed shall be cleaned of all such mortar or grout before the surrounding or adjacent concrete is placed.

(b) ***Foundation Surfaces.*** -Immediately before placing concrete, all surfaces of foundations upon or against which the concrete is to be placed shall be free from standing water, mud, and debris. All surfaces of rock upon or against which concrete is to be placed shall, in addition to the foregoing requirements, be clean and free from oil, objectionable coatings, and loose, semidetached, or unsound fragments. Earth foundations shall be free from frost or ice when concrete is placed upon or against them. The surfaces of absorptive foundations against which concrete is to be placed shall be moistened thoroughly so that moisture will not be drawn from the freshly placed concrete.

(c) ***Surfaces of Construction and Contraction Joints.*** -Concrete surfaces upon or against which concrete is to be placed and to which new concrete is to adhere, that have become so rigid that the new concrete cannot be incorporated integrally with that previously placed, are defined as construction joints.

All construction joints shall be cured by water curing or by application of wax base curing compound in accordance with the provisions of section H-22 (Curing). Wax base curing compound, if used on these joints, shall be removed in the process of preparing the joints to receive fresh concrete. The surfaces of the construction joints shall be clean, rough, and surface dry when covered with fresh concrete. Cleaning shall consist of the removal of all laitance, loose or defective concrete, coatings, sand, curing compound if used, and other foreign material. The cleaning and roughening shall be accomplished by wet sandblasting, washing thoroughly with air-water jets, and surface drying prior to placement of adjoining concrete: ***Provided,*** that high-pressure water blasting utilizing pressures not less than 6,000 pounds per square inch may be used in lieu of wet sandblasting for preparing the joint surfaces if it is demonstrated to the satisfaction of the Authority that the equipment proposed for use will produce equivalent results to those obtainable by wet sandblasting. High-pressure water blasting equipment, if used, shall be equipped with suitable safety devices for controlling pressures, including shutoff switches at the nozzle that will shut off the pressure if the nozzle is dropped. The sandblasting (or high-pressure water blasting if approved), washing, and surface drying shall be performed at the last opportunity prior to placing of concrete. Drying of the surface shall be complete and may be accomplished by air jet. In the process of wet sandblasting construction joints, care shall be taken to prevent undercutting of aggregate in the concrete.

The surfaces of all contraction joints shall be cleaned thoroughly of accretions of concrete or other foreign material by scraping, chipping, or other means approved by the Authority.

H- ***18. Placing.*** -(a) ***Transporting.*** -The methods and equipment used for transporting concrete and the time that elapses during transportation shall be such as will not cause appreciable segregation of coarse aggregate, or slump loss in excess of 1 inch, in the concrete as it is delivered into the work. The use of aluminum pipe for delivery of pumped concrete will not be permitted.

(b) ***Placing.*** -The contractor shall keep the Contracting Authority advised as to when placing of concrete will be performed. Unless inspection is waived in each specific case, placing of concrete shall be performed only in the presence of a duly authorized Authority inspector.

The surfaces of all rock against which concrete is to be placed shall be cleaned and, except in those cases where seepage or other water precludes drying of the rock face, shall be dampened and brought to a surface-dry condition. Except for tunnels, surfaces of highly porous or absorptive horizontal or nearly horizontal rock foundations to which concrete is to be bonded shall be covered with a layer of mortar approximately three-eighths of an inch thick prior to placement of the concrete. The mortar shall have the same proportions of water, air-entraining agent, cement, pozzolan, and sand as the regular concrete mixture, unless otherwise directed. The water-cement plus pozzolan ratio of the mortar in place shall not exceed that of the concrete to be placed upon it, and the consistency of the mortar shall be suitable for placing and working in the manner hereinafter specified. The mortar shall be spread and shall be worked thoroughly into all irregularities of the surface. Concrete shall be placed immediately upon the fresh mortar.

A mortar layer shall not be used on concrete construction joints. Unless otherwise directed in formed work, structural concrete placements shall be started with an oversanded mix containing %-inch maximum-size aggregate; a maximum net water-cement plus pozzolan ratio of 0.47, by weight; 6 percent air, by volume of concrete; and having a maximum slump of 4 inches. This mix shall be placed approximately 3 inches deep on the joint at the bottom of the placement.

Retempering of concrete will not be permitted. Any concrete which has become so stiff that proper placing cannot be assured shall be wasted. Concrete shall be deposited in all cases as nearly as practicable directly in its final position and shall not be caused to flow such that the lateral movement will permit or cause segregation of the coarse aggregate from the concrete mass. Methods and equipment employed in depositing concrete in forms shall be such as will not result in clusters or groups of coarse aggregate particles being separated from the concrete mass, but if clusters do occur they shall be scattered before the concrete is vibrated. Where there are a few scattered individual pieces of coarse aggregate that can be restored into the mass by vibration, this will not be objectionable and should be done.

Concrete in tunnel lining may be placed by pumping or any other approved method. Where the concrete in the invert is placed separately from the concrete in the arch and without inside forms, it shall not be placed by pneumatic placing equipment unless an approved type of discharge box which prevents segregation is provided and used. The equipment used in placing the concrete and the method of its operation shall be such as will permit introduction of the concrete into the forms without high-velocity discharge and resultant separation. After the concrete has been built up over the arch at the start of a placement, the end of the discharge line shall be kept well buried in the concrete during placement of the arch and sidewalls to assure complete filling. The end of the discharge line shall be marked so as to indicate the depth of burial at any time. Special care shall be taken to force concrete into all irregularities in the rock surfaces and to completely fill the tunnel arch. Placing equipment shall be operated by experienced operators only.

Where tunnel lining placements are terminated with sloping joints, the contractor shall thoroughly consolidate the concrete at such joints to a reasonably uniform and stable slope while the concrete is plastic. If thorough consolidation at the sloping joints is not obtained, as determined by the Authority, the Authority reserves the right to require the use of bulkheaded construction joints. The concrete at the surface of such sloping joints shall be clean and surface dry before being covered with fresh concrete. The cleaning of such sloping joints shall consist of the removal of all loose and foreign material.

Except as intercepted by joints, all formed concrete other than concrete in tunnel lining, including mass concrete in the dam, shall be placed in continuous approximately horizontal layers. The depth of layers for mass concrete shall generally not exceed 18 inches, and the depth for all other concrete shall generally not exceed 20 inches. The Authority reserves the right to require lesser depths of layers where concrete in 20-inch layers cannot be placed in accordance with the requirements of these specifications. Except where joints are specified herein or on the drawings, care shall be taken to prevent cold joints when placing concrete in any portion of the work. The concrete placing rate shall be such as to ensure that each layer is placed while the previous layer is soft or plastic, so that the two layers can be made monolithic by penetration of the vibrators. To prevent featheredges, construction joints that are located at the tops of horizontal lifts near sloping exposed concrete surfaces shall be inclined near the exposed surface, so that the angle between such inclined surfaces and the exposed concrete surface will be not less than 50°.

In placing unformed concrete on slopes so steep as to make internal vibration of the concrete impracticable without forming, the concrete shall be placed ahead of a nonvibrated slip-form screed extending approximately 2% feet back from its leading edge. Concrete ahead of the slip-form screed shall be consolidated by internal vibrators so as to ensure complete tilling under the slip-form.

In placing mass concrete in the dam, the contractor shall, when required, maintain the exposed area of fresh concrete at the practical minimum, by first building up the concrete in successive approximately horizontal layers to the full width of the block and to full height of the lift over a restricted area at the downstream end of the block, and then continuing upstream in similar progressive stages to the full area of the block. The slope formed by the unconfined upstream edges of the successive layers of concrete shall be kept as steep as practicable in order to keep its area to a minimum. Concrete along these edges shall not be vibrated until adjacent concrete in the layer is placed, except that it shall be vibrated immediately when weather conditions are such that the concrete will harden to the extent that it is doubtful whether later vibration will fully consolidate and integrate it with more recently placed adjacent concrete. Clusters of large aggregate shall be scattered before new concrete is placed over them. Each deposit of concrete shall be vibrated completely before another deposit of concrete is placed over it.

Concrete shall not be placed during rains sufficiently heavy or prolonged to wash mortar from coarse aggregate on the forward slopes of the placement. Once placement of concrete has commenced in a block, placement shall not be interrupted by diverting the placing equipment to other uses.

Concrete buckets shall be capable of promptly discharging the low slump, 6-inch mass concrete mixes specified, and the dumping mechanism shall be designed to permit the discharge of as little as a %-cubic-yard portion of the load in one place. Buckets shall be suitable for attachment and use of drop chutes where required in confined locations.

Construction joints shall be approximately horizontal unless otherwise shown on the drawings or prescribed by the Authority, and shall be given the prescribed shape by the use of forms, where required, or other means that will ensure suitable joining with subsequent work. All intersections of construction joints with concrete surfaces which will be exposed to view shall be made straight and level or plumb.

If concrete is placed monolithically around openings having vertical dimensions greater than 2 feet, or if concrete in decks, top slabs, beams, or other similar parts of structures is placed monolithically with supporting concrete, the following instructions shall be strictly observed:

(1) Placing of concrete shall be delayed from 1 to 3 hours at the top of openings and at the bottoms of bevels under decks, top slabs, beams, or other similar parts of structures when bevels are specified, and at the bottom of such structure members when bevels are not specified; but in no case shall the placing be delayed so long that the vibrating unit will not readily penetrate of its own weight the concrete placed before the delay. When consolidating concrete placed after the delay, the vibrating unit shall penetrate and revibrate the concrete placed before the delay.

(2) The last 2 feet or more of concrete placed immediately before the delay shall be placed with as low a slump as practicable, and special care shall be exercised to effect thorough consolidation of the concrete.

(3) The surfaces of concrete where delays are made shall be clean and free from loose and foreign material when concrete placing is started after the delay.

(4) Concrete placed over openings and in decks, top slabs, beams, and other similar parts of structures shall be placed with as low a slump as practicable and special care shall be exercised to effect thorough consolidation of the concrete.

(c) Consolidation. -Concrete shall be consolidated to the maximum practicable density, so that it is free from pockets of coarse aggregate and entrapped air, and closes snugly against all surfaces of forms and embedded materials. Consolidation of concrete in structures shall be by electric- or pneumatic-drive, immersion-type vibrators. Vibrators having vibrating heads 4 inches or more in diameter shall be operated at speeds of at least 6,000 revolutions per minute when immersed in the concrete. Vibrators having vibrating heads less than 4 inches in diameter shall be operated at speeds of at least 7,000 revolutions per minute when immersed in the concrete. Immersion-type vibrators used in mass concrete shall be heavy duty, two-man vibrators capable of readily consolidating mass concrete of the consistency specified: ***Provided,*** that heavy-duty, one-man vibrators may be used if they are operated in sufficient number, and in a manner and under conditions as to produce equivalent results to that specified for two-man vibrators: ***Provided further,*** that where practicable in vibrating mass concrete, the contractor may employ gang vibrators, satisfactory to the Authority, mounted on self-propelled equipment in such a manner that they can be readily raised and lowered to eliminate dragging through the fresh concrete, and provided all other requirements of these specifications with respect to placing and control of concrete are met.

Consolidation of concrete in the sidewalls and arch of tunnel lining shall be by electric- or pneumatic-driven form vibrators supplemented where practicable by immersion-type vibrators. Form vibrators shall be rigidly attached to the forms and shall operate at speeds of at least 8,000 revolutions per minute when vibrating concrete.

In consolidating each layer of concrete the vibrator shall be operated in a near-vertical position and the vibrating head shall be allowed to penetrate and revibrate the concrete in the upper portion of the underlying layer. In the area where newly placed concrete in each layer joins previously placed concrete, particularly in mass concrete, more than usual vibration shall be performed, the vibrator penetrating deeply and at close intervals into the upper portion of the previously placed layer along these contacts. In all vibration of mass concrete, vibration shall continue until bubbles of entrapped air have generally ceased to escape. Additional layers of concrete shall not be superimposed on concrete previously placed until the previously placed concrete has been vibrated thoroughly as specified. Care shall be exercised to avoid contact of the vibrating head with surfaces of the forms.

H-19. ***Repair*** of Concrete. -Concrete shall be repaired in accordance with the Bureau of Reclamation "Standard Specifications for Repair of Concrete," dated November 15, 1970. Imperfections and irregularities on concrete surfaces shall be corrected in accordance with section H-20 (Finishes and Finishing).

H-20. ***Finishes and Finishing.***-(a) General. -Allowable deviations from plumb or level and from the alinement, profile grades, and dimensions shown on the drawings are specified in section H-15 (Tolerances for Concrete Construction): these are defined as "tolerances" and are to be distinguished from irregularities in finish as described herein. The classes of finish and the requirements for finishing of concrete surfaces shall be as specified in this section or as indicated on the drawings. The contractor shall keep the Contracting Authority advised as to when finishing of concrete will be performed. Unless inspection is waived in each specific case, finishing of concrete shall be performed only in the presence of an Authority inspector. Concrete surfaces will be tested by the Authority where necessary to determine whether surface irregularities are within the limits hereinafter specified.

Surf ace irregularities are classified as "abrupt" or "gradual." Offsets caused by displaced or misplaced form sheathing or lining or form sections, or by loose knots in forms or otherwise defective form lumber, will be considered as abrupt irregularities and will be tested by direct measurements. All other irregularities will be considered as gradual irregularities and will be tested by use of a template, consisting of a straightedge or the equivalent thereof for curved surfaces. The length of the template will be 5 feet for testing of formed surfaces and 10 feet for testing of unformed surfaces.

(b) *Formed* Surfaces.-The classes of finish for formed concrete surfaces are designated by use of symbols F 1, F2, F3, and F4. No sack rubbing or sandblasting will be required on formed surfaces. No grinding will be required on formed surfaces, other than that necessary for repair of surface imperfections. Unless otherwise specified or indicated on the drawings, the classes of finish shall apply as follows:

Fl. -Finish F1 applies to formed surfaces upon or against which fill material or concrete is to be placed, to formed surfaces of contraction joints, and to the upstream face of the dam below the minimum water pool elevation. The surfaces require no treatment after form removal except for repair of defective concrete and filling of holes left by the removal of fasteners from the ends of tie rods as required in section H-l 9 (Repair of Concrete), and the specified curing. Correction of surface irregularities will be required for depressions only, and only for those which, when measured as described in subsection (a) above, exceed 1 inch.

F2. -Finish F2 applies to all formed surfaces not permanently concealed by fill material or concrete, or not required to receive finishes F 1, F3, or F4. Surface irregularities, measured as described in subsection (a) above, shall not exceed one-fourth of an inch for abrupt irregularities and one-half of an inch for gradual irregularities: *Provided,* that surfaces over which radial gate seals will operate without sill or wall plates shall be free from abrupt irregularities.

F3. -Finish F3 applies to formed surfaces, the appearance of which is considered by the Authority to be of special importance, such as surfaces of structures prominently exposed to public inspection. Included in this category are superstructures of large powerplants and pumping plants, parapets, railings, and decorative features on dams and bridges and permanent buildings. Surface irregularities, measured as described in subsection (a) above shall not exceed one-fourth of an inch for gradual irregularities and one-eighth of an inch for abrupt irregularities, except that abrupt irregularities will not be permitted at construction joints.

F4.–Finish F4 applies to formed surfaces for which accurate alinement and evenness of surface are of paramount importance from the standpoint of eliminating destructive effects of water action. When measured as described in subsection (a) above, abrupt irregularities shall not exceed one-fourth of an inch for irregularities parallel to the direction of flow, and one-eighth of an inch for irregularities not parallel to the direction of flow. Gradual irregularities shall not exceed one-fourth of an inch. *(Note:* When waterflow velocities on formed concrete surfaces of outlet works, spillways, etc., are calculated to exceed 40 feet per second, further limitations should be considered for the allowable irregularities to prevent cavitation.)

(c) Unformed Surfaces. -The classes of finish for unformed concrete surfaces are designated by the symbols U1, U2, and U3. Interior surfaces shall be sloped for drainage where shown on the drawings or directed. Surfaces which will be exposed to the weather and which would normally be level, shall be sloped for drainage. Unless the use of other slopes or level surfaces is indicated on the drawings or directed, narrow surfaces such as tops of walls and curbs, shall be sloped approximately three-eighths of an inch per foot of width; broader surfaces such as walks, roadways, platforms, and decks shall be sloped approximately one-fourth of an inch per foot. Unless otherwise specified or indicated on the drawings, these classes of finish shall apply as follows:

U1.--Finish U1 (screeded finish) applies to unformed surfaces that will be covered by fill material or by concrete. Finish U1 is also used as the first stage of finishes U2 and U3. Finishing operations shall consist of sufficient leveling and screeding to produce even, uniform surfaces. Surface irregularities measured as described in subsection (a) above, shall not exceed three-eighths of an inch.

U2. -Finish U2 (floated finish) applies to unformed surfaces not permanently concealed by fill material or concrete, or not required to receive finish U1 or U3. U2 is also used as the second stage of finish U3. Floating may be performed by use of hand- or power-driven equipment. Floating shall be started as soon as the screeded surface has stiffened sufficiently, and shall be the minimum necessary to produce a surface that is free from screed marks and is uniform in texture. If finish U3 is to be applied, floating shall be continued until a small amount of mortar without excess water is brought to the surface, so as to permit effective troweling. Surface irregularities, measured as described in subsection (a) above, shall not exceed one-fourth of an inch. Joints and edges of gutters, sidewalks, and entrance slabs, and other joints and edges shall be tooled where shown on the drawings or directed.

U3. -Finish U3 (troweled finish) applies to the inside floors of buildings, except floors requiring a bonded-concrete finish or a terrazzo finish, and to inverts of draft tubes and tunnel spillways. When the floated surface has hardened sufficiently to prevent an excess of fine material from being drawn to the surface, steel troweling shall be started. Steel troweling shall be performed with firm pressure so as to flatten the sandy texture of the floated surface and produce a dense uniform surface, free from blemishes and trowel marks. Surface irregularities, measured as described in subsection (a) above, shall not exceed one-fourth of an inch.

(Note: When waterflow velocities on unformed concrete surfaces of outlet works, spillways, etc., are calculated to exceed 40 feet per second, further limitations on U2 and/or U3 finishes should be considered for the allowable irregularities to prevent cavitation.)

H-2 1. ***Protection.*** -The contractor shall protect all concrete against injury until final acceptance by the Contracting Authority. Fresh concrete shall be protected from damage due to rain, hail, sleet, or snow. The contractor shall provide such protection while the concrete is still plastic and whenever such precipitation, either periodic or sustaining, is imminent or occurring, as determined by the Authority.

Immediately following the first frost in the fall the contractor shall be prepared to protect all concrete against freezing. After the first frost, and until the mean daily temperature in the vicinity of the worksite falls below 40° F. for more than 1 day, the concrete shall be protected against freezing temperatures for not less than 48 hours after it is placed.

After the mean daily temperature in the vicinity of the worksite falls below 40° F. for more than 1 day, the following requirements shall apply :

(a) ***Mass Concrete.*** -Mass concrete shall be maintained at a temperature not lower than 40° F. for at least 96 hours after it is placed. Mass concrete cured by application of curing compound will require no additional protection from freezing if the protection at 40° F. for 96 hours is obtained by means of approved insulation in contact with the forms or concrete surfaces; otherwise, the concrete shall be protected against freezing temperatures for 96 hours immediately following the 96 hours protection at 40° F. Mass concrete cured by water curing shall be protected against freezing temperatures for 96 hours immediately following the 96 hours of protection at 40° F. Discontinuance of protection of mass concrete against freezing temperatures shall be such that the drop in temperature of any portion of the concrete will be gradual and will not exceed 20° F. in 24 hours. After March 15, when the mean daily temperature rises above 40° F. for more than 3 successive days, the specified 96-hour protection at a temperature not lower than 40° F. for mass concrete may be discontinued for as long as the mean daily temperature remains above 40° F.: ***Provided,*** that the specified drop in temperature limitation is met, and that the concrete is protected against freezing temperatures for not less than 48 hours after placement.

(b) ***Concrete Other Than Mass Concrete.*** -All concrete other than mass concrete shall be maintained at a temperature not lower than 50° F. for at least 72 hours after it is placed. Such concrete cured by application of curing compound will require no additional protection from freezing if the protection at 50° F. for 72 hours is obtained by means of approved insulation in contact with the forms of concrete surfaces; otherwise, the concrete shall be protected against freezing temperatures for 72 hours immediately following the 72 hours protection at 50° F. Concrete other than mass concrete cured by water curing shall be protected against freezing temperatures for 72 hours immediately following the 72 hours protection at 50° F. Discontinuance of protection of such concrete against freezing temperatures shall be such that the drop in temperature of any portion of the concrete will be gradual and will not exceed 40° F. in 24 hours. After March 15, when the mean daily temperature rises above 40° F. for more than 3 successive days, the specified 72-hour protection at a temperature not lower than 50° F. may be discontinued for as long as the mean daily temperature remains above 40° F.: ***Provided,*** that the specified drop in temperature limitation is met, and that the concrete is protected against freezing temperatures for not less than 48 hours after placement.

(c) Use ***of*** *Unvented* ***Heaters.*** -Where artificial heat is employed, special care shall be taken to prevent the concrete from drying. Use of unvented heaters will be permitted only when unformed surfaces of concrete adjacent to the heaters are protected for the first 24 hours from an excessive carbon dioxide atmosphere by application of curing compound: ***Provided,*** that the use of curing compound on such surfaces for curing of the concrete is permitted by and the compound is applied in accordance with section H-22 (Curing). (Include this proviso only when the use of sealing compound is not permitted on some concrete surfaces.)

H-22. ***Curing.*** -(a) ***General.*** -Concrete shall be cured either by water curing in accordance with subsection (b) or by application of wax base curing compound in accordance with subsection (c), except as otherwise hereinafter provided.

The unformed top surfaces of walls and piers shall be moistened by covering with water-saturated material or by other effective means as soon as the concrete has hardened sufficiently to prevent damage by water. These surfaces and steeply sloping and vertical formed surfaces shall be kept completely and continually moist, prior to and during form removal, by water applied on the unformed top surfaces and allowed to pass down between the forms and the formed concrete faces. This procedure shall be followed by the specified water curing or by application of curing compound.

(b) ***Water Curing.*** -Concrete cured with water shall be kept wet for at least 21 days for concrete containing pozzolan and for at least 14 days for concrete not containing pozzolan. Water curing shall start as soon as the concrete has hardened sufficiently to prevent damage by moistening the surface, and shall continue until completion of the specified curing period or until covered with fresh concrete: ***Provided,*** that water curing of concrete may be reduced to 6 days during periods when the mean daily temperature in the vicinity of the worksite is less than 40° F.: *Provided further,* that during the prescribed period of water curing, when temperatures are such that concrete surfaces may freeze, water curing shall be temporarily discontinued. The concrete shall be kept wet by covering with water-saturated material or by a system of perforated pipes, mechanical sprinklers, or porous hose, or by any other approved method which will keep all surfaces to be cured continuously (not periodically) wet. Water used for curing shall be furnished by the contractor and shall meet the requirements of these specifications for water used for mixing concrete in accordance with section H-7 (Water).

(c) Wax Base Curing Compound. -Wax base curing compound shall be applied to surfaces to form a water-retaining film on exposed surfaces of concrete, on concrete joints, and where specified, to prevent bonding of concrete placed on or against such joints. The curing compound shall be white pigmented and shall conform to Bureau of Reclamation "Specifications for Wax-Base Curing Compound," dated May 1, 1973. The compound shall be of uniform consistency and quality within each container and from shipment to shipment.

Curing compound shall be mixed thoroughly and applied to the concrete surfaces by spraying in one coat to provide a continuous, uniform membrane over all areas. Coverage shall not exceed 150 square feet per gallon, and on rough surfaces coverage shall be decreased as necessary to obtain the required continuous membrane. Mortar encrustations and fins on surfaces designated to receive finish F3 or F4 shall be removed prior to application of curing compound. The repair of all other surface imperfections shall not be made until after application of curing compound.

When curing compound is used on unformed concrete surfaces, application of the compound shall commence immediately after finishing operations are completed. When curing compound is to be used on formed concrete surfaces, the surfaces shall be moistened with a light spray of water immediately after the forms are removed and shall be kept wet until the surfaces will not absorb more moisture. As soon as the surface film of moisture disappears but while the surface still has a damp appearance, the curing compound shall be applied. Special care shall be taken to insure ample coverage with the compound at edges, corners, and rough spots of formed surfaces. After application of the curing compound has been completed and the coating is dry to touch, any required repair of concrete surfaces shall be performed. Each repair, after being finished, shall be moistened and coated with curing compound in accordance with the foregoing requirements.

Equipment for applying curing compound and the method of application shall be in accordance with the provisions of chapter VI of the eighth edition of the Bureau of Reclamation Concrete Manual [1]. Traffic and other operations by the contractor shall be such as to avoid damage to coatings of curing compound for a period of not less than 28 days. Where it is impossible because of construction operations to avoid traffic over surfaces coated with curing compound, the film shall be protected by a covering of sand or earth not less than 1 inch in thickness or by other effective means. The protective covering shall not be placed until the applied compound is completely dry. Before final acceptance of the work, the contractor shall remove all sand or earth covering in an approved manner. Any curing compound that is damaged or that peels from concrete surfaces within 28 days after application, shall be repaired without delay and in an approved manner.

(d) *Costs.* -The costs of furnishing and applying all materials used for curing concrete shall be included in the price bid in the schedule for the concrete on which the curing materials are used.

H-23. ***Measurement of Concrete.*** -Measurement, for payment, of concrete required to be placed directly upon or against surfaces of excavation will be made to the lines for which payment for excavation is made. Measurement, for payment, of all other concrete will be made to the neatlines of the structures, unless otherwise specifically shown on the drawings or prescribed in these specifications. In the event cavities resulting from careless excavation, as determined by the Contracting Authority, are required to be filled with concrete, the materials furnished by the Authority and used for such refilling will be charged to the contractor at their cost to the Authority at the point of delivery to the contractor. In measuring concrete for payment, the volume of all openings, recesses, ducts, embedded pipes, woodwork, and metalwork, each of which is larger than 100 square inches in cross section will be deducted.

H-24. ***Payment for Concrete.*** -Payment for concrete in the various parts of the work will be made at the unit prices per cubic yard bid therefor in the schedule, which unit prices shall include the cost of all labor and materials required in the concrete construction, except that payment for furnishing and handling cement, and payment for furnishing and placing reinforcing bars will be made at the unit prices bid therefor in the schedule.

H-25. ***Bibliography.***

Bureau of Reclamation

[1] "Concrete Manual," eighth edition, 1975.

American Society for Testing and Materials

[2] ASTM Designation: A 185, "Welded Steel Wire Fabric for Concrete Reinforcement."
[3] ASTM Designation: A 615, "Deformed Billet-Steel Bars for Concrete Reinforcement."
[4] ASTM Designation: A 617, "Axle-Steel Deformed Bars for Concrete Reinforcement."
[5] ASTM Designation: C 184, "Standard Method of Test for Fineness of Hydraulic Cement by the No. 100 and 200 Sieves."
[6] ASTM Designation: C 260, "Standard Specifications for Air-Entraining Admixtures for Concrete."
[7] ASTM Designation: E-l 1, "Standard Specifications for Wire-Cloth Sieves for Testing Purposes."

General Services Administration (Federal Supply Service)

[8] Federal Specification AAA-S-121d, "Scale (weighing; General Specifications for)."
[9] Federal Specification SS-C-192G (Including Amendment 3), "Portland Cement."
[10] Federal Specification SS-P-570B, "Pozzolan (for Use in Portland Cement Concrete)."
[11] Federal Test Method Standard No. 158A, 'Cements, Hydraulic; Sampling, Inspection, and Testing."

U.S. Department of Commerce, Bureau of Standards

[12] Product Standard PS l-66, "Softwood Plywood, Construction and Industrial."

Sample Specifications for Controlling Water and Air Pollution

I-1. ***Scope.*** -The following sample specifications prescribe water quality controls and preventive measures for discharge of wastes and/or pollution into a river, lake, or estuary due to construction operations; and the prevention and control of air pollution. They are written in the form of mandatory provisions which should be required of the contractor.

A. PREVENTION OF WATER POLLUTION

I-2. General. -The contractor shall comply with applicable Federal and State laws, orders, and regulations concerning the prevention, control, and abatement of water pollution. Permits to discharge wastes into receiving waters shall be obtained by the contractor either from the State water pollution control agency or from the Environmental Protection Agency.

The contractor's construction activities shall be performed by methods that will prevent entrance or accidental spillage of solid matter, contaminants, debris, and other objectionable pollutants and wastes into streams, flowing or dry watercourses, lakes, and underground water sources. Such pollutants and wastes include but are not restricted to refuse, garbage, cement, concrete, sewage effluent, industrial waste, radioactive substances, mercury, oil and other petroleum products, aggregate processing tailings, mineral salts, and thermal pollution. Pollutants and wastes shall be disposed of at sites approved by the Contracting Authority.

The contractor shall control his construction activities so that turbidity resulting from his operations shall not exist in concentrations that will impair natural or developed water supplies, fisheries, or recreational facilities downstream from the construction area.

At least 40 days prior to beginning of construction of each phase of work, the contractor shall submit for approval two copies of his plans for the treatment and disposal of all waste and for control of turbidity in the ___________ River which may result from his operations. The plans shall be submitted to the Construction Engineer, Post Office Box, ______ __________, ___________. The plans shall include complete design and construction details of turbidity control features. Such plans shall also show the methods of handling and disposal of oils or other petroleum products, chemicals, and similar industrial wastes.

Except as otherwise provided in section I-4(a) below, approval of the contractor's plans shall not relieve the contractor of the responsibility for designing, constructing, operating, and maintaining pollution and turbidity control features in a safe and systematic manner, and for repairing at his expense any damage to, or failure of, the pollution and turbidity control structures and equipment caused by floods or storm runoff.

I-3. ***Control of*** Turbidity.-Turbidity increases above the natural turbidities in the ____________ River that are caused by construction activities shall be limited to those increases resulting from performance of required construction work in the river channel and will be permitted only for the shortest practicable period required to complete such work and as approved by the Contracting Authority. This required construction work will- include such work as diversion of the river, construction or removal of cofferdams and other specified earthwork in or adjacent to the river channel, pile driving, and construction of turbidity control structures.

The spawning period for trout (or other game fish) in the ____________ River is normally during the period __________ through ________ Accordingly, no change in the diversion or channelization of the river will be permitted during this particularly sensitive period.

Mechanized equipment shall not be operated in flowing water except as necessary to construct approved crossings or to perform the required construction, as outlined above.

The contractor's methods of unwatering, of excavating foundations, of operating in the borrow areas, and of stockpiling earth and rock materials shall include preventive measures to control siltation and erosion, and to intercept and settle any runoff of muddy waters. Waste waters from construction of dam and appurtenances, aggregate processing, concrete batching and curing, drilling, grouting, and similar construction operations shall not enter flowing or dry watercourses without the use of special approved turbidity control methods.

I-4. ***Turbidity Control Methods.*** -(a) General.-Turbidity control shall be accomplished through the use of plans approved by the Contracting Authority in accordance with section N-2 above.

The Bureau of Reclamation's methods for control of turbidity during construction at the damsite as set forth in (c) below are acceptable methods. The contractor may adopt these methods or he may submit for approval alternative methods of equivalent adequacy. If the contractor elects to utilize the Bureau's methods and his plans for implementation are approved by the Contracting Authority, and if such approved plans do not effectively control turbidity due to no fault of the contractor, additional work will be directed for which payment will be made in accordance with the "General Provisions" portion of the specifications. If the contractor elects to propose for approval different methods of turbidity control, the contractor shall bear the full responsibility for their satisfactory operation in controlling turbidity. The approval of the contractor's alternate proposals by the Contracting Authority shall not be construed to relieve the contractor from his responsibility.

The contractor's plans, submitted in accordance with section I-2 above, shall show complete design and construction details for implementing either the Bureau's methods or the contractor's alternative methods.

(b) *Requirements for Turbidity Control During Construction at the Damsite.-The* turbidity control method to be used during construction at the damsite shall: (1) Provide for treatment of all turbid water at the damsite resulting from construction of dam and appurtenances; washing of aggregate obtained from approved sources, if such washing is performed at the damsite; drilling; grouting; or similar construction operations: Provided, that the Contracting Authority may direct that clear water removed from foundations be discharged directly to the river without treatment. The treatment plant shall have a capacity to treat 0 to ________ gallons of turbid water per minute so that the turbidity of any effluent discharged to the river does not exceed -Jackson turbidity units.

(2) Include bypass and control equipment suitable for blending treated and untreated waste waters and obtaining effluents of varying degrees of turbidity. The decision to discharge to the river completely treated effluent or a blend of treated and untreated effluent will be the responsibility of the Contracting

Authority, and will depend on the natural turbidity existing in the river at any particular time.

(3) Have a capability of adjusting the *pH* and alkalinity values of any effluent discharged to the river.

(4) Use only chemicals which have been approved by the Environmental Protection Agency for use in potable water and which have been proven to be harmless to terrestrial wildlife and aquatic life.

(5) Have provisions for accumulating, transporting, and depositing sludge in disposal areas so that the material will not wash into the river by high flows or storm runoff, as approved by the Contracting Authority.

(6) Provide for removal of the treatment plant, cleanup of the site, and restoration of the site to its original condition as approved by the Contracting Authority. All materials, plant, and appurtenances used for turbidity control shall remain the property of the contractor.

(c) Bureau's Methods of Turbidity Control at the Damsite.-The Bureau of Reclamation's methods for controlling turbidity during construction at the damsite are based on collecting turbid waters in sumps, and pumping from the sumps to: (1) A water clarification plant, Dorr-Oliver[1] Pretreater (--foot diameter by--foot water depth), or equal, with automatic chemical dosage feeders for hydrated lime, alum, and an acid or coagulant aid if needed; or

(2) A treatment plant consisting of equalizing tanks, sedimentation flumes, settling tanks, and ponds combined with innocuous stabilizing and flocculating chemicals as required. Such a treatment plant shall be the Dow Turbidity Control System, as proposed by Dow Chemical U.S.A.,' or equal.

(d) *Sampling and Testing of Water Quality.* -The Contracting Authority will do such water quality sampling and testing in connection with construction operations as is necessary to insure compliance with the water quality standards of the State of __________ and the Environmental Protection Agency.

Turbidities of all effluents discharged to the river from the contractor's construction operations shall be monitored by continuous recorders such as the HACH 6491 or 7855 strip chart recorder provided with CR Surface Scatter Turbidimeter Model 2411 or 2426, [1] or equal, which shall be furnished, installed, and operated by the contractor. Locations of the recorders shall be as approved by the Contracting Authority.

Copies of the recordings shall be submitted daily to the Contracting Authority and shall include the date, time of day, and name of person or persons responsible for operation of the equipment and recorder.

Sampling and testing by the Contracting Authority in no way relieves the contractor of the responsibility for doing such monitoring as is necessary for the controlling of his operations to prevent violation of the water quality standards.

I-5. Payment.-Payment for control of turbidity during construction at the damsite will be made at the applicable lump-sum price bid therefor in the schedule, which lump-sum price shall include the cost of furnishing all labor, equipment, and materials for designing, constructing, operating, maintaining, and removing all features necessary for control of turbidity in accordance with these sections.

Payment of percentages of the lump-sum price for control of turbidity during construction at the damsite will be made as follows:

(1) Fifty percent of the lump sum in the first monthly progress estimate after completion of the initial installation of the approved plant for treatment of the turbid water.

(2) Twenty-five percent of the lump sum in the first monthly progress estimate after completion of all concrete placement in the dam.

(3) Twenty-five percent of the lump sum in the first monthly progress estimate after completion of the turbidity control operation at the damsite, and removal of equipment.

The costs of all other labor, equipment, and materials necessary for control of turbidity at

[1] **Mention of these firms should not be construed as an indication that they are the only suppliers of these or similar products nor as an endorsement by the Bureau of Reclamation.**

locations other than the damsite and for prevention of water pollution for compliance with these sections shall be included in the prices bid in the schedule for other items of work.

B. ABATEMENT OF AIR POLLUTION

I-6. ***General.*** -The contractor shall comply with applicable Federal, State, and local laws and regulations concerning the prevention and control of air pollution.

In his conduct of construction activities and operation of equipment, the contractor shall utilize such practicable methods and devices as are reasonably available to control, prevent, and otherwise minimize atmospheric emissions or discharges of air contaminants.

The emission of dust into the atmosphere will not be permitted during the manufacture, handling, and storage of concrete aggregates, and the contractor shall use such methods and equipment as are necessary for the collection and disposal, or prevention, of dust during these operations. The contractor's methods of storing and handling cement and pozzolans shall also include means of eliminating atmospheric discharges of dust.

Equipment and vehicles that show excessive emissions of exhaust gases due to poor engine adjustments, or other inefficient operating conditions, shall not be operated until corrective repairs or adjustments are made.

Burning shall be accomplished only at times and at locations approved by the Contracting Authority. Burning of materials resulting from clearing of trees and brush, combustible construction materials, and rubbish will be permitted only when atmospheric conditions for burning are considered favorable by appropriate State or local air pollution or fire authorities. In lieu of burning, such combustible materials may be removed from the site, chipped, or buried as provided in section ______

Where open burning is permitted, the burn piles shall be properly constructed to minimize smoke, and in no case shall unapproved materials such as tires, plastics, rubber products, asphalt products, or other materials that create heavy black smoke or nuisance odors be burned.

Storage and handling of flammable and combustible materials, provisions for fire prevention, and control of dust resulting from drilling operations shall be done in accordance with the applicable provisions of the Department of Labor "Safety and Health Regulation for Construction" and the Bureau of Reclamation Supplement thereto.

Dust nuisance resulting from construction activities shall be prevented in accordance with section ______

The costs of complying with this section shall be included in the prices bid in the schedule for the various items of work.

I- 7. ***Dust Abatement.*** -During the performance of the work required by these specifications or any operations appurtenant thereto, whether on right-of-way provided by the Contracting Authority or elsewhere, the contractor shall furnish all the labor, equipment, materials, and means required, and shall carry out proper and efficient measures wherever and as often as necessary to reduce the dust nuisance, and to prevent dust which has originated from his operations from damaging crops, orchards, cultivated fields, and dwellings, or causing a nuisance to persons. The contractor will be held liable for any damage resulting from dust originating from his operations under these specifications on Authority right-of-way or elsewhere.

The cost of sprinkling or of other methods of reducing formation of dust shall be included in the prices bid in the schedule for other items of work.

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