Appendix C
Design Example-Chopra's Simplified Method

C-1. General

a. The design example problem described in Appendix B will be analyzed using Chopra's simplified design response spectrum method. All hand calculations are included.

b. Definitions of symbols and notations used in this appendix can be found in the Glossary. Refer to Appendix B where the values of several parameters required for the analysis were derived.

C-2. Fundamental Natural Period (Empty Reservoir/Rigid Foundation)

$$T_1 = 1.4\ H_s/\sqrt{E_s} = 1.4\ (600)/\sqrt{3.59\times 10^6}$$

$$T_1 = 0.443 \text{ sec}$$

C-3. Reservoir Effect on Natural Period

$$\tilde{T}_r = R_r T_1$$

$H_s = 600$ ft $\qquad$ R_r from Figure C-1

	POOL ELEVATION	
	NORMAL	LOW
H	495 ft	270 ft
H/H_s	0.825	0.450
R_r	1.110	1.000
$\tilde{T}_r$	0.492 sec	0.443 sec

C-4. Ratio of Resonant Period of Dam To Fundamental Period of the Reservoir

$$R_w = T_1^r/\tilde{T}_r$$

$$T_1^r = 4H/C$$

$$C = 4720 \text{ ft/sec}$$

	POOL ELEVATION	
	NORMAL	LOW
H	495 ft	270 ft
T_1^r	0.420 sec	0.229 sec
R_w	0.85	0.52

C-5. Foundation Effect On Natural Period

$E_s/E_f = 3.59 \times 10^6/3.50 \times 10^6 = 1.025$

$R_f = 1.190$ $\qquad$ from Figure C-2

$\tilde{T}_1 = R_r R_f T_1$

	POOL ELEVATION	
	NORMAL	LOW
R_r	1.110	1.000
R_f	1.190	1.190
T_1	0.443 sec	0.443 sec
$\tilde{T}_1$	0.585 sec	0.527 sec

C-6. Effective Damping Factor

$$\tilde{\varepsilon}_1 = \frac{1}{R_r}\frac{1}{(R_f)^3}\varepsilon_1 + \varepsilon_r + \varepsilon_f$$

$\varepsilon_f = 0.0701$ $\qquad$ from Figure C-3

$\varepsilon_r =$ values $\qquad$ from Figure C-4

	POOL ELEVATION	
	NORMAL	LOW
R_r	1.110	1.000
R_f	1.190	1.190
ε_1	7.00 %	7.00 %
H/H_s	0.825	0.450
ε_r	0.0158	0
$\tilde{\varepsilon}_1$	12.33 %	11.16 %

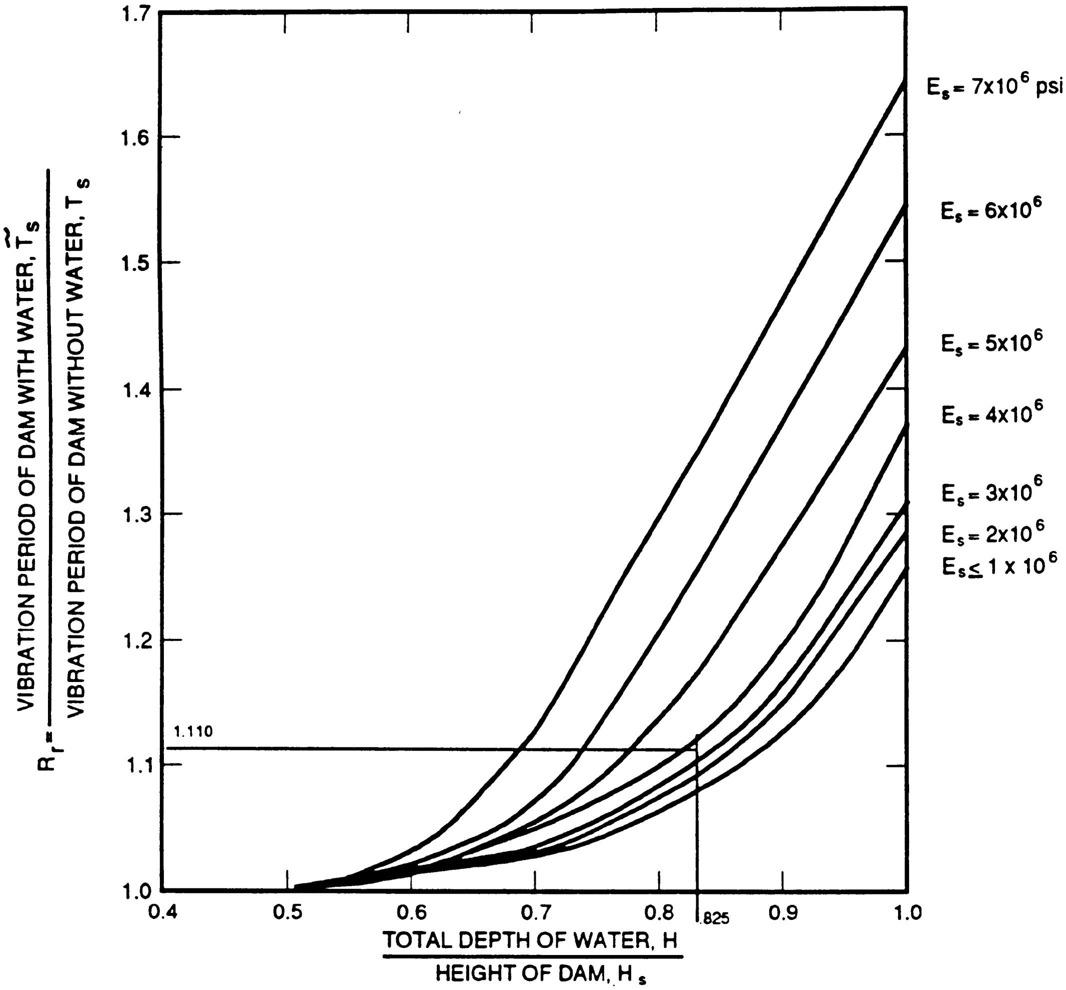
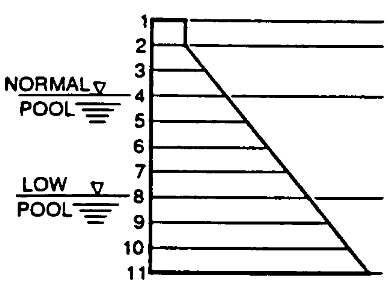


Figure C-1. Standard values for R_r, the ratio of fundamental vibration periods of the dam with and without water. Chopra (1978)

7. Key Dimensions



y	y/H_s	Section Width
600	1.000	20.0
570	0.950	20.0
533	0.888	53.8
495	0.825	88.4
460	0.767	120.4
420	0.700	156.8
345	0.575	225.3
270	0.450	293.7
180	0.300	375.8
90	0.150	457.9
0	0.000	540.0

8. Properties of Concrete Mass

For sections *y* distance above the foundation,

w_s = (section width) × 0.155 k/ft^3

ϕ = value from Figure C-5 (based on y/H_s)

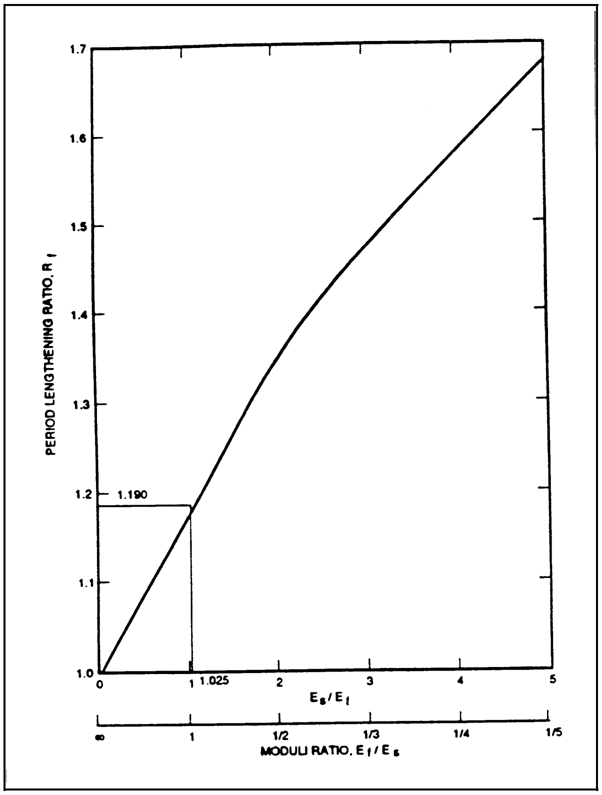


Figure C-2. Standard values for R_f , the period lengthening ratio due to dam-foundation rock interaction. Fenves and Chopra (1986)

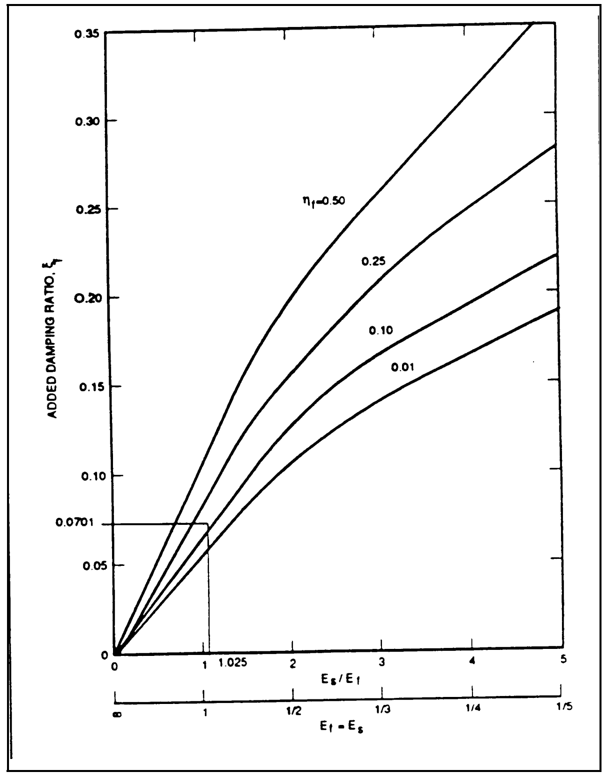
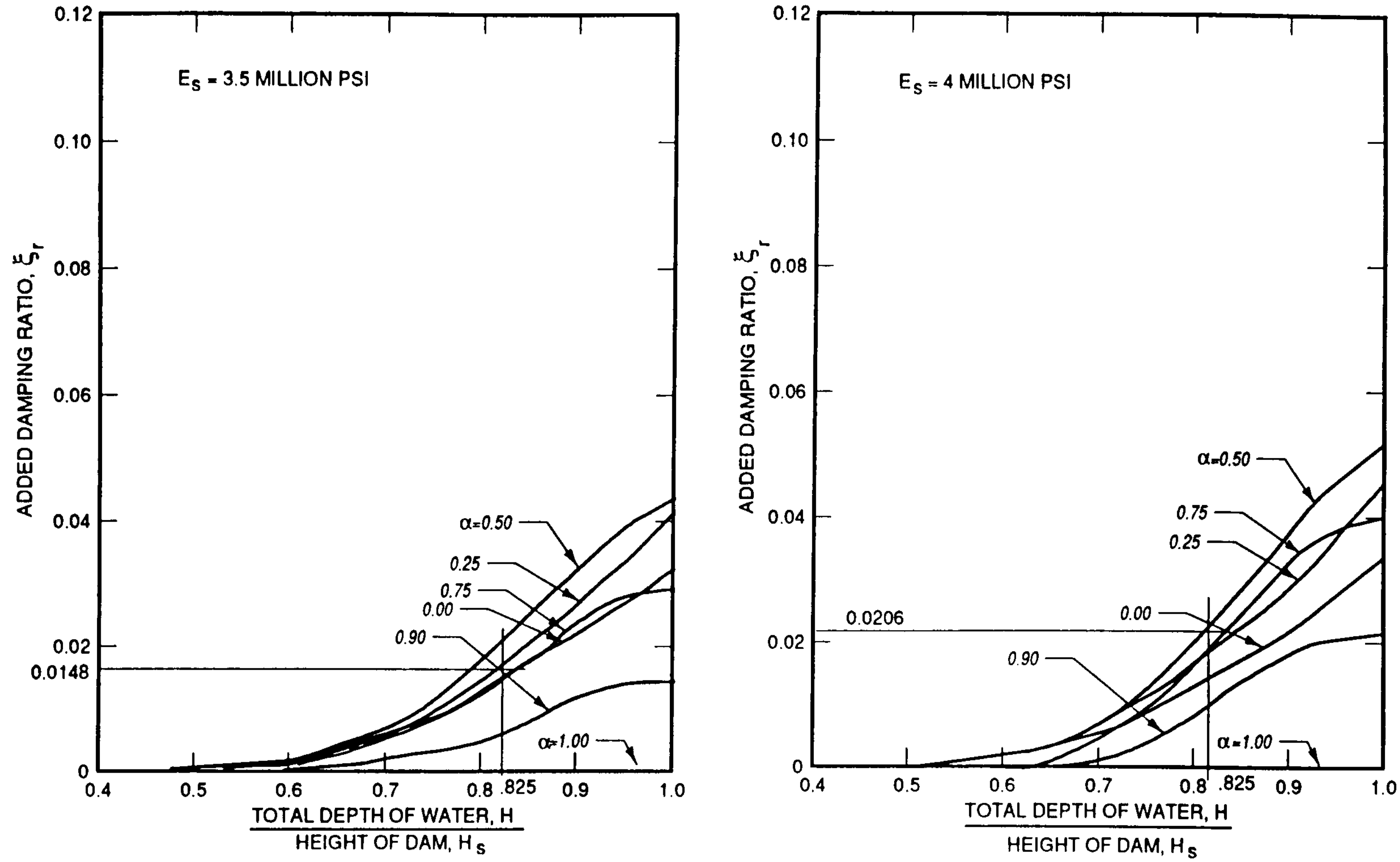


Figure C-3. Standard values for ε_f , the added damping due to dam-foundation rock interaction. Fenves and Chopra (1986)

y	w_s (k/ft)	ϕ	$w_s\phi$	$w_s\phi^2$	*dy* (Segment Height)	$w_s\phi dy$	$w_s\phi^2 dy$
600	3.10	1.000	3.10	3.10	30	84.9	78.5
570	3.10	0.829	2.56	2.13	37	151.9	110.3
533	8.34	0.678	5.65	3.83	38	251.0	152.0
495	13.70	0.552	7.56	4.17	35	282.1	141.8
460	18.66	0.459	8.56	3.93	40	350.6	144.8
420	24.30	0.369	8.97	3.31	75	666.4	207.4
345	34.92	0.252	8.80	2.22	75	606.4	127.9
270	45.52	0.162	7.37	1.19	90	538.6	69.8
180	58.25	0.079	4.60	0.36	90	299.7	18.9
90	70.97	0.029	2.06	0.06	90	92.7	2.7
0	83.70	0	0	0			
					600 ft	3,324.3	1,054.1



NOTE: PLOTS OF THE STANDARD VALUES FOR ξ_r, THE ADDED DAMPING RATIO DUE TO HYDRODYNAMIC EFFECTS; OTHER VALUES OF E_S ARE CONTAINED IN REPORT NO. UCB/EERC-85/10, JUNE 1986, EARTHQUAKE ENGINEERING CENTER, UNIVERSITY OF CALIFORNIA, BERKELEY, CALIF.

Figure C-4. Values for ε_r, the added damping ratio due to hydrodynamic effects; E_s = 3.5 and 4.0 million psi

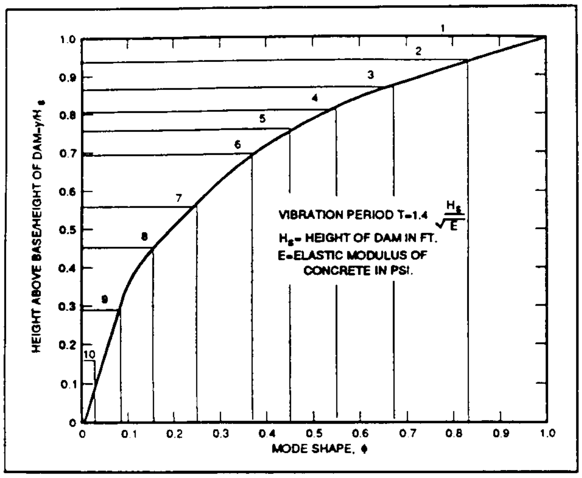


Figure C-5. Standard mode shape and fundamental period for the dam on a rigid foundation and empty reservoir. Chopra (1978)

C-9. Hydrodynamic Influence

For sections y distance above the foundation,

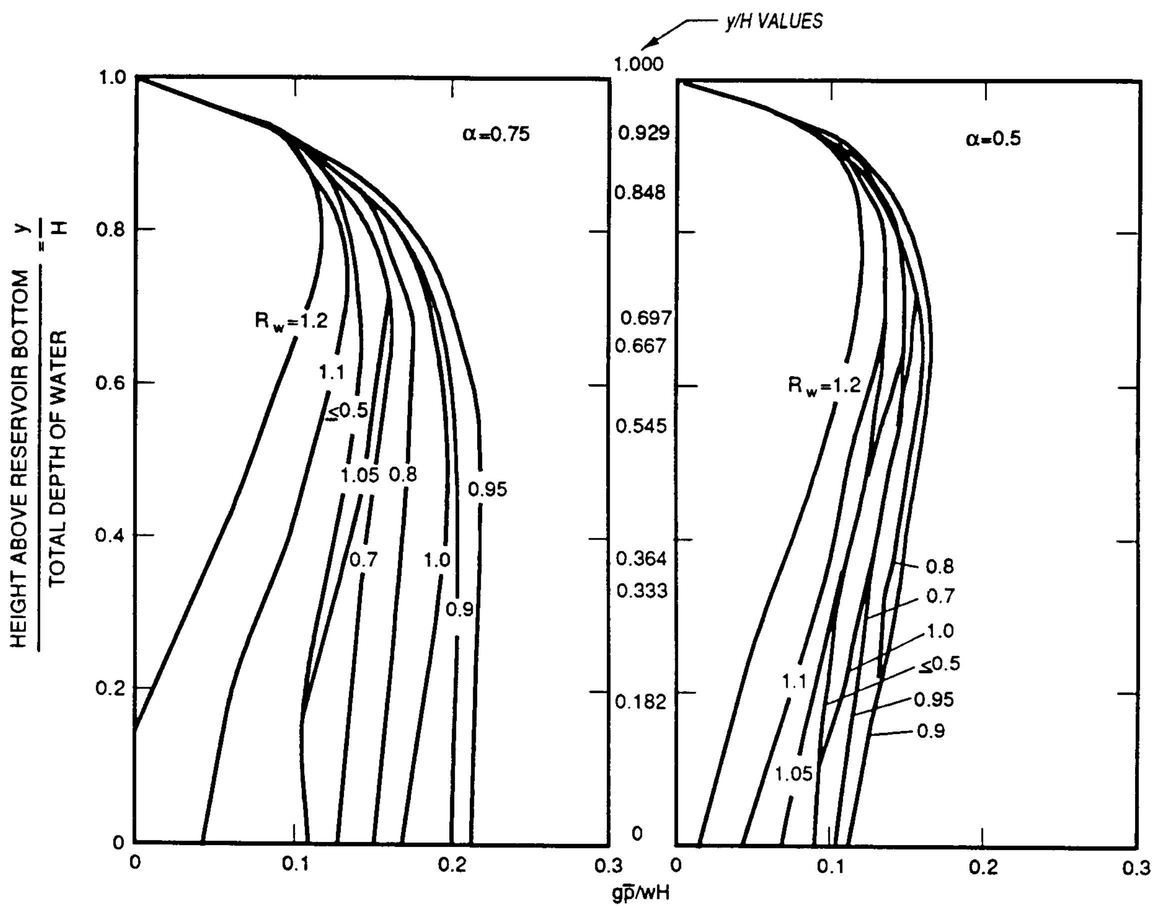
$g\bar{p}/wH$ = value from Figure C-6 (based on y/H and R_w) interpolate Figure C-6 plots for α = 0.75 and α = 0.50 for the required α = 0.69 as calculated in Appendix B

w = 0.0624 k/ft^3
H_s = 600 ft

$gp = [wH(H/H_s)^2]\ (g\bar{p}/wH)$ = CONSTANT × $(g\bar{p}/wH)$

	POOL ELEVATION	
	NORMAL	LOW
H	495	270
R_w	0.85	0.52
CONSTANT	21.02	3.41

NORMAL POOL						LOW POOL			
y	dy (Seg Ht)	$\frac{y}{H}$	$\frac{g\bar{p}}{wH}$	gp	$\frac{g\bar{p}}{wH}dy$	$\frac{y}{H}$	$\frac{g\bar{p}}{wH}$	gp	$\frac{g\bar{p}}{wH}dy$
495		1.000	0	0					
	35				1.75				
460		0.929	0.100	2.10					
	40				4.88				
420		0.848	0.144	3.03					
	75				11.89				
345		0.697	0.173	3.64					
	75				13.24				
270		0.545	0.180	3.78		1.000	0	0	
	90				16.06				6.39
180		0.364	0.177	3.72		0.667	0.142	0.48	
	90				15.57				11.66
90		0.182	0.169	3.55		0.333	0.117	0.40	
	90				14.90				9.77
0		0	0.162	3.41		0	0.100	0.34	
					78.29				27.82



NOTE: PLOTS OF THE STANDARD VALUES FOR THE HYDRODYNAMIC PRESSURE FUNCTION $\bar{p}$: OTHER VALUES OF α ARE CONTAINED IN REPORT NO. UCB/EERC-85/10, JUNE 1986, EARTHQUAKE ENGINEERING RESEARCH CENTER, UNIVERSITY OF CALIFORNIA, BERKELEY, CALIF.

Figure C-6. Standard values for the hydrodynamic pressure function $\bar{p}$ for full reservoir; i.e., H/H_s = 1, α = 0.75 and 0.50

C-10. Generalized Mass, $\tilde{M}_1$

$$\tilde{M}_1 = (R_r)^2 M_1$$

$$M_1 = (1/g) \int_0^{H_s} w_s \phi^2 dy = (1/32.2)\ (1{,}054.1)$$

$$= 32.74\ \text{k-sec}^2/\text{ft}$$

	POOL ELEVATION	
	NORMAL	LOW
R_r	1.110	1.000
M_1	40.34	32.74

C-11. Generalized Earthquake Force Coefficient, $\tilde{L}_1$

$$\tilde{L}_1 = L_1 + (1/g)\ F_{st}\ (H/H_s)^2 A_p$$

$$L_1 = (1/g) \int_0^{H_s} w_s \phi dy = (1/32.2)\ (3{,}324.3)$$

$$= 103.2\ \text{k-sec}^2/\text{ft}$$

$$F_{st} = wH^2/2 = (0.0624/2)\ H^2 = 0.0312H^2$$

$$A_p = \frac{2}{H} \int_0^H \frac{g\bar{p}}{wH}\ dy$$

	POOL ELEVATION	
	NORMAL	LOW
H	495	270
F_{st}	7,645	2,274
$\int_0^H \frac{g\bar{p}}{wH}$	78.29	27.82
A_p	0.316	0.206
$\tilde{L}_1$	154.26	106.15

C-12. Response Spectrum Acceleration, $\tilde{S}_a$

a. As discussed in Appendix B, the conditions for this example problem require site specific design response spectra. However, since this is only for the purpose of demonstrating Chopra's simpified method, the standard design response spectra shown in Figure 5-2 and Table 5-1 will be assumed to be the site-specific design response spectra.

b. For both the earthquake load cases, the fundamental period T_1 is greater than 0.4 sec; therefore:

$$S_a = K_2 S_{a(5\%)}$$

where

$$K_2 = 1.466 - 0.2895\mathbf{ln}(\beta)$$

$S_{a(5\%)}$ = value at period $\tilde{T}_1$ obtained by interpolating Table 6-1 between the appropriate values of T.

The spectral ordinates S_a are for the design response spectrum normalized to a PGA = 1 g. These values must be scaled by the PGA factor shown in Table 5-2 for an MCE occurring in seismic Zone 3. The scaling factor is 0.550 g; therefore:

$$\tilde{S}_a = 0.550 \times K_2 S_{a(5\%)} \times 32.2\ \text{ft/sec}^2$$

	POOL ELEVATION	
	NORMAL	LOW
T_1	0.585 sec	0.527 sec
β	12.33%	11.16%
K_2	0.7388	0.7676
$S_{a(5\%)}$	1.7094 g	1.8975 g
$\tilde{S}_a$	22.37 ft/sec2	25.80 ft/sec2

C-13. Equivalent Lateral Earthquake Force for Fundamental Mode, f_1

$$f_1 = \frac{\tilde{L}_1 \tilde{S}_a}{\tilde{M}_1 g}\ (w_s\phi + gp) = \text{constant} \times (w_s\phi + gp)$$

	POOL ELEVATION	
	NORMAL	LOW
$\tilde{L}_1$	154.26	106.15
$\tilde{M}_1$	40.34	32.74
$\tilde{S}_a$	22.37 ft/sec^2	25.80 ft/sec^2
constant	2.657	2.598

for a section y-distance above the foundation, values of f_1 in (k/ft) are as follows:

		gp		f_1	
y	$ws\phi$	NORMAL	LOW	NORMAL	LOW
600	3.10			8.24	8.05
570	2.56			6.80	6.65
533	5.65			15.01	14.68
495	7.56	0		20.09	19.64
460	8.56	2.10		28.32	22.24
420	8.97	3.30		31.88	23.30
345	8.80	3.64		33.05	22.86
270	7.37	3.78	0	29.63	19.15
180	4.60	3.72	0.48	22.11	13.20
90	2.06	3.55	0.40	14.91	6.39
0	0	3.41	0.34	9.06	0.88

C-14. Equivalent Lateral Earthquake Force for the Higher Modes, f_{sc}

$$f_{sc} = \frac{1}{g}\left[w_s\left(1 - \frac{L_1}{M_1}\,\phi\right) + \left(gp_0 - \frac{B_1}{M_1}\,w_s\phi\right)\right]a_g$$

$$a_g/g = \text{PGA} = 0.550\ g$$

$$f_{sc} = 0.550 \times \left[w_s\left(1 - \frac{L_1}{M_1}\phi\right) + \left(gp_0 - \frac{B_1}{M_1}\ w_s\phi\right)\right]$$

$$gp_0 = \left(\frac{g\bar{p}_0}{wH}\right) \times wH\left(\frac{H}{H_s}\right)^2 \qquad B_1 = 0.2\ \frac{F_{st}}{g}\left(\frac{H}{H_s}\right)^2$$

$$\left(\frac{g\bar{p}_0}{wH}\right) = \text{value from Figure C-7 (for values of } y/H)$$

	POOL ELEVATION	
	NORMAL	LOW
L_1	103.2	103.2
M_1	32.74	32.74
$\left(\frac{H}{H_s}\right)^2$	0.681	0.202
$wH\left(\frac{H}{H_s}\right)^2$	21.03	3.40
F_{st}	7,645	2,274
B_1	32.34	2.85

NORMAL POOL

y	W_S	ϕ	$W_S(1 - \frac{L_1}{M_1}\phi)$	y/H	$\frac{g\bar{P}_O}{wH}$	gp_O	$W_S\phi$	$(gp_O - \frac{B_1}{M_1}W_S\phi)$	f_{SC}
600	3.10	1.000	-6.67				3.10	-3.06	-5.35
570	3.10	0.829	-5.00				2.56	-2.53	-4.14
533	8.34	0.678	-9.48				5.65	-5.58	-8.28
495	13.70	0.552	-10.14	1.000	0.000	0.00	7.56	-7.47	-9.69
460	18.66	0.459	-8.34	0.929	0.187	3.93	8.56	-4.53	-7.08
420	24.30	0.369	-3.96	0.848	0.312	6.56	8.97	-2.30	-3.44
345	34.92	0.252	7.18	0.697	0.473	9.95	8.80	1.26	4.64
270	45.52	0.162	22.28	0.545	0.587	12.34	7.37	5.06	15.04
180	58.25	0.079	43.74	0.364	0.677	14.24	4.60	9.70	29.39
90	70.97	0.029	64.48	0.182	0.727	15.29	2.06	13.26	42.76
0	83.70	0.000	83.70	0.000	0.746	15.69	0.00	15.69	54.66

LOW POOL

y	W_S	ϕ	$W_S(1 - \frac{L_1}{M_1}\phi)$	y/H	$\frac{g\bar{P}_O}{wH}$	gp_O	$W_S\phi$	$(gp_O - \frac{B_1}{M_1}W_S\phi)$	f_{SC}
600	3.10	1.000	-6.67				3.10	-0.27	-3.82
570	3.10	0.829	-5.00				2.56	-0.22	-2.87
533	8.34	0.678	-9.48				5.65	-0.49	-5.48
495	13.70	0.552	-10.14				7.56	-0.66	-5.94
460	18.66	0.459	-8.34				8.56	-0.75	-5.00
420	24.30	0.369	-3.96				8.97	-0.78	-2.61
345	34.92	0.252	7.18				8.80	-0.77	3.52
270	45.52	0.162	22.28	1.000	0.000	0.00	7.37	-0.64	11.90
180	58.25	0.079	43.74	0.667	0.500	1.70	4.60	1.30	23.34
90	70.97	0.029	64.48	0.333	0.693	2.36	2.06	2.18	24.77
0	83.70	0.000	83.70	0.000	0.746	2.54	0.00	2.54	47.43

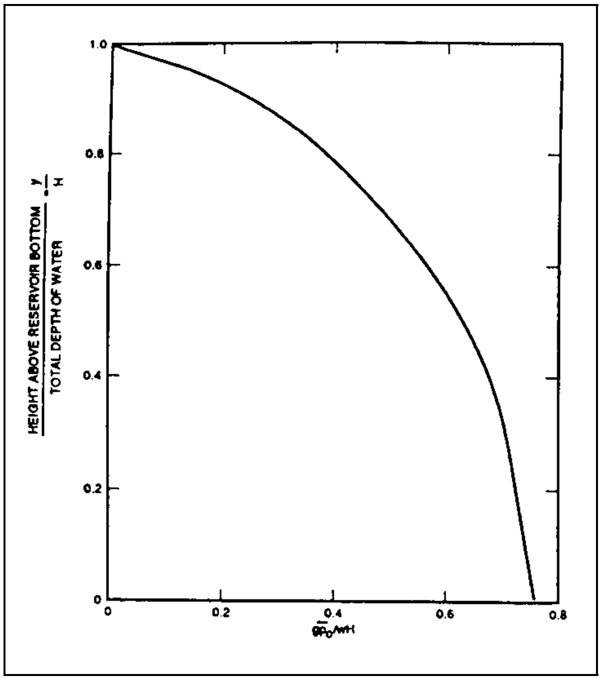


Figure C-7. Standard mode shape and fundamental period for the dam on a rigid foundation and empty reservoir. Chopra (1978)

C-15. Allowable Tensile Stress

Appendix B established the direct tensile strength of the basic RCC mix to be:

f'_t = 290 psi (for the parent concrete)

f'_t = 205 psi (for the lift joints)

Because of the high strain rates associated with a seismic event, the dynamic tensile strength is greater than the direct tensile strength obtained from the lab tests:

DTS = 1.5 f'_t = 1.5 × 290 = 435 psi (for the parent concrete)

DTS = 1.5 f'_t = 1.5 × 205 = 307 psi (for the lift joints)

In accordance with paragraph 4-3*c*, the allowable tensile stress for a new RCC dam in seismic Zone 3 for the MCE load condition is:

$f_{t(allowable)}$ = 1.33 × 435 = 579 psi (for the parent concrete)

$f_{t(allowable)}$ = 1.33 × 307 = 408 psi (for the lift joints)

C-16. Determining Stresses for the Earthquake Load Cases

a. The response of the dam to the design earthquake ground motion is obtained by applying the equivalent lateral forces f_1 and f_{sc} to the dam as static loads, and performing a static analysis to determine the tensile tresses. The lateral forces f_1 and f_{sc} are distributed forces in kips/ft. They are treated as individual loading cases in the static analysis. As discussed in paragraph 7-7, the stresses produced by these forces represent maximum modal responses. Thus, they must be combined by a statistical method. The square root of the sum of the squares (SRSS) method is used, and the maximum tensile stresses are as follows:

$$f_t = \sqrt{\sigma_1^2 + \sigma_{sc}^2}$$

where

f_t = maximum tensile stress due only to design earthquake loading in a direction normal to the lift joints (does not include hydrostatic or dead load)

σ_1 = tensile stress contribution of the fundamental mode as produced by the statically applied load f_1

σ_{sc} = tensile stress contribution of the higher modes as produced by the statically applied load f_{sc}

b. Two static analysis options are available for determining the tensile stresses σ_1 and σ_{sc}. The first option is to consider the dam as a simple vertical cantilever fixed at the foundation and loaded laterally by the loads f_1 and f_{sc}. The stresses σ_1 and σ_{sc} are simple bending stresses that can be hand calculated by the beam bending formula Mc/I. The maximum principal tensile stresses occurring at the downstream face are then approximated by the following:

$$f_{t(\max)} = \sqrt{\sigma_1^2 + \sigma_{sc}^2}\ \sec^2\Theta + \sigma_{st}\sec^2\Theta$$

where

$f_{t(max)}$ = the maximum principal tensile stress

Θ = the angle of the downstream face measured from vertical

σ_{st} = stress normal to the lift joints caused by static loads (hydrostatic, dead load of dam, etc.)

The second option is to apply the lateral loads f_1 and f_{sc} to a finite element model of the dam, fixed at the dam base, and perform a static analysis to obtain the tensile stresses normal to the lift joints σ_1 and σ_{sc}, and also the corresponding maximum principal tensile stresses $\sigma_{1(max)}$ and $\sigma_{sc(max)}$. The maximum tensile stress normal to the lift joints is calculated by the formula for f_1 described above, and the maximum principal tensile stress, $f_{t(max)}$, is calculated as follows:

$$f_{t(\max)} = \sqrt{\sigma^2_{1(\max)} + \sigma^2_{sc(\max)}}$$

c. The second option will be used for this example problem. The same finite element model of the dam will be used as formulated in Appendix D to analyze the example problem by the composite finite element-equivalent mass system method. This will allow a good comparison of the two different methods. Node points at the dam base, nodes 56 through 62, will be fully fixed for this analysis since Chopra's simplified method does not include modeling of the dam foundation. To load the finite element model, the distributed lateral forces f_1 and f_{sc} were converted to concentrated lateral joint loads applied to the appropriate upstream face node points. Note that y = the distance above the foundation in feet coincides with the node point locations of the finite element model shown in Figures D-1 and D-2 of Appendix D.

d. The static loads accompanying the earthquake ground motion loading consist of the hydrostatic load of the forebay on the upstream face and the dead load weight of the dam. These loads are identical to those calculated in Table D-2 and discussed in paragraph D-12 of Appendix D for the composite finite element analysis. However, since Chopra's simplified method does not account for deformations in the foundation, the static stresses σ_{st} and $\sigma_{st(max)}$ for Chopra's method are different than those derived by the composite finite element analysis.

e. The tabulations on the following pages show the critical tensile stresses for the MCE load cases.

MCE Normal Pool Load Case:
Critical Tensile Stresses Normal to the Lift Joints at the Upstream Face

	Stress Normal to the Lift Joint (ksf, tension is +)					
y	σ_1	σ_{sc}	Dynamic Response f_1	Static Stress σ_{st}	Critical Tensile Stress (psi)	Percent Overstressed
590	24.96	-16.16	29.73	-1.55	196	----
570	35.99	-23.49	42.98	-5.32	262	----
533	34.04	-20.77	39.88	-11.39	198	----
495	37.21	-21.16	42.81	-15.94	187	----
460	44.47	-25.23	51.13	-21.63	205	----
420	50.97	-25.38	56.94	-27.61	204	----
345	60.14	-25.69	65.40	-35.89	205	----
270	72.93	-26.86	77.72	-45.49	224	----
180	90.22	-26.54	94.04	-53.62	281	----
90	122.20	-5.22	122.31	-51.20	494	21
0	49.19	7.57	49.76	-14.20	247	----

MCE Low Pool Load Case:
Critical Tensile Stresses Normal to the Lift Joints at the Upstream Face

y	Stress Normal to the Lift Joint (ksf, tension is +) σ_1	σ_{sc}	Dynamic Response f_1	Static Stress σ_{st}	Critical Tensile Stress (psi)	Percent Overstressed
590	24.39	-11.51	26.97	-1.55	176	----
570	35.20	-16.78	38.99	-5.31	234	----
533	32.93	-14.61	36.03	-11.54	170	----
495	35.46	-14.81	38.43	-16.47	152	----
460	43.95	-16.85	47.07	-22.23	172	----
420	49.12	-16.60	51.85	-28.80	160	----
345	54.77	-17.22	57.41	-37.81	136	----
270	63.01	-18.85	65.77	-49.40	114	----
180	73.96	-18.21	76.17	-63.43	88	----
90	94.48	-0.38	94.48	-75.90	129	----
0	36.71	6.17	37.22	-26.95	71	----

MCE Normal Pool Load Case:
Critical Tensile Stresses Normal to the Lift Joints at the Downstream Face

y	Stress Normal to the Lift Joint (ksf, tension is +) σ_1	σ_{sc}	Dynamic Response f_1	Static Stress σ_{st}	Critical Tensile Stress (psi)	Percent Overstressed
590	-12.57	8.22	15.02	-1.53	94	----
570	-66.14	42.51	78.62	-2.77	527	----
533	-28.88	18.29	34.18	0.20	239	----
495	-37.32	22.07	43.36	0.55	305	----
460	-48.38	24.90	54.41	-0.12	377	----
420	-58.90	22.75	63.14	-3.02	418	2
345	-67.59	15.47	69.34	-7.89	427	5
270	-69.36	5.99	69.62	-12.26	398	----
180	-58.51	-2.58	58.57	-14.43	307	----
90	-39.08	-5.49	39.46	-12.46	188	----
62	-12.65	-1.76	12.77	-6.37	44	----

MCE Low Pool Load Case:
Critical Tensile Stresses Normal to the Lift Joints at the Downstream Face

y	Stress Normal to the Lift Joint (ksf, tension is +)				Critical Tensile Stress (psi)	Percent Overstressed
	σ_1	σ_{sc}	Dynamic Response f_1	Static Stress σ_{st}		
590	-12.29	5.88	13.62	-1.53	84	----
570	-64.53	30.26	71.27	-2.72	476	17
533	-28.87	12.50	31.46	-0.19	217	----
495	-36.55	14.91	39.47	0.07	275	----
460	-45.13	16.86	48.18	0.36	337	----
420	-51.52	15.28	53.74	0.57	377	----
345	-55.61	9.96	56.49	0.49	396	----
270	-54.69	3.17	54.78	0.03	381	----
180	-44.66	-2.71	44.74	-1.38	301	----
90	-29.18	-4.43	29.51	-2.48	187	----
0	-9.45	-1.43	9.56	-3.14	45	----

MCE Normal Pool Load Case:
Critical Principal Tensile Stresses at the Downstream Face

y	Principal Tensile Stress (ksf, tension is +)				Critical Tensile Stress (psi)	Percent Overstressed
	$\sigma_{1(max)}$	$\sigma_{sc(max)}$	Dynamic Response $f_{1(max)}$	Static Stress $\sigma_{st(max)}$		
590	13.09	8.56	15.64	-1.53	98	----
570	76.19	48.90	90.53	-3.15	607	5
533	49.78	31.11	58.70	-1.73	396	----
495	63.24	36.74	73.14	-2.14	493	----
460	79.09	39.67	88.48	-3.16	593	2
420	94.33	35.75	100.88	-6.48	656	13
345	111.10	25.71	114.04	-13.16	701	21
270	112.70	9.90	113.13	-20.42	643	11
180	95.51	7.40	95.80	-24.02	498	----
90	64.46	9.96	65.22	-21.27	305	----
0	16.50	2.91	16.75	-7.06	67	----

MCE Low Pool Load Case: Critical Principal Tensile Stresses at the Downstream Face						
	Principal Tensile Stress (ksf, tension is +)					
y	$\sigma_{1(max)}$	$\sigma_{sc(max)}$	Dynamic Response $f_{1(max)}$	Static Stress $\sigma_{st(max)}$	Critical Tensile Stress (psi)	Percent Overstressed
590	-12.80	6.12	14.19	-1.53	88	----
570	-74.33	34.80	82.07	-3.11	548	----
533	-49.43	21.34	53.84	-1.82	361	----
495	-61.73	24.79	66.52	-1.74	450	----
460	-73.50	26.79	78.23	-1.55	533	----
420	-82.26	24.03	85.70	-2.04	581	1
345	-91.19	16.71	92.71	-3.45	620	7
270	-88.52	5.70	88.70	-4.45	585	1
180	-72.64	-6.29	72.91	-6.12	464	----
90	-48.07	-7.89	48.71	-6.63	292	----
0	-12.24	-2.28	12.45	-3.15	65	----

C-17. Conclusions

The Chopra simplified method is used only for preliminary design of new dams. Preliminary design progresses to the point where it becomes apparent that, with limited refinement, the final design will be satisfactory. Refer to Figure C-8 which shows zones where the basic RCC mix is overstressed. It appears that use of superior mixes in these areas will lead to a satisfactory final design. Final design uses a more refined method such as the composite finite element method demonstrated in Appendix D for this example problem. The more refined methods allow for modeling and verifying the zones of superior concrete, where this is not possible with the simplified method.

C-18. Comparison of Results

Paragraph D-16 compares the results of this analysis using Chopra's simplified method with the results of the same example problem analyzed by the composite finite element method.

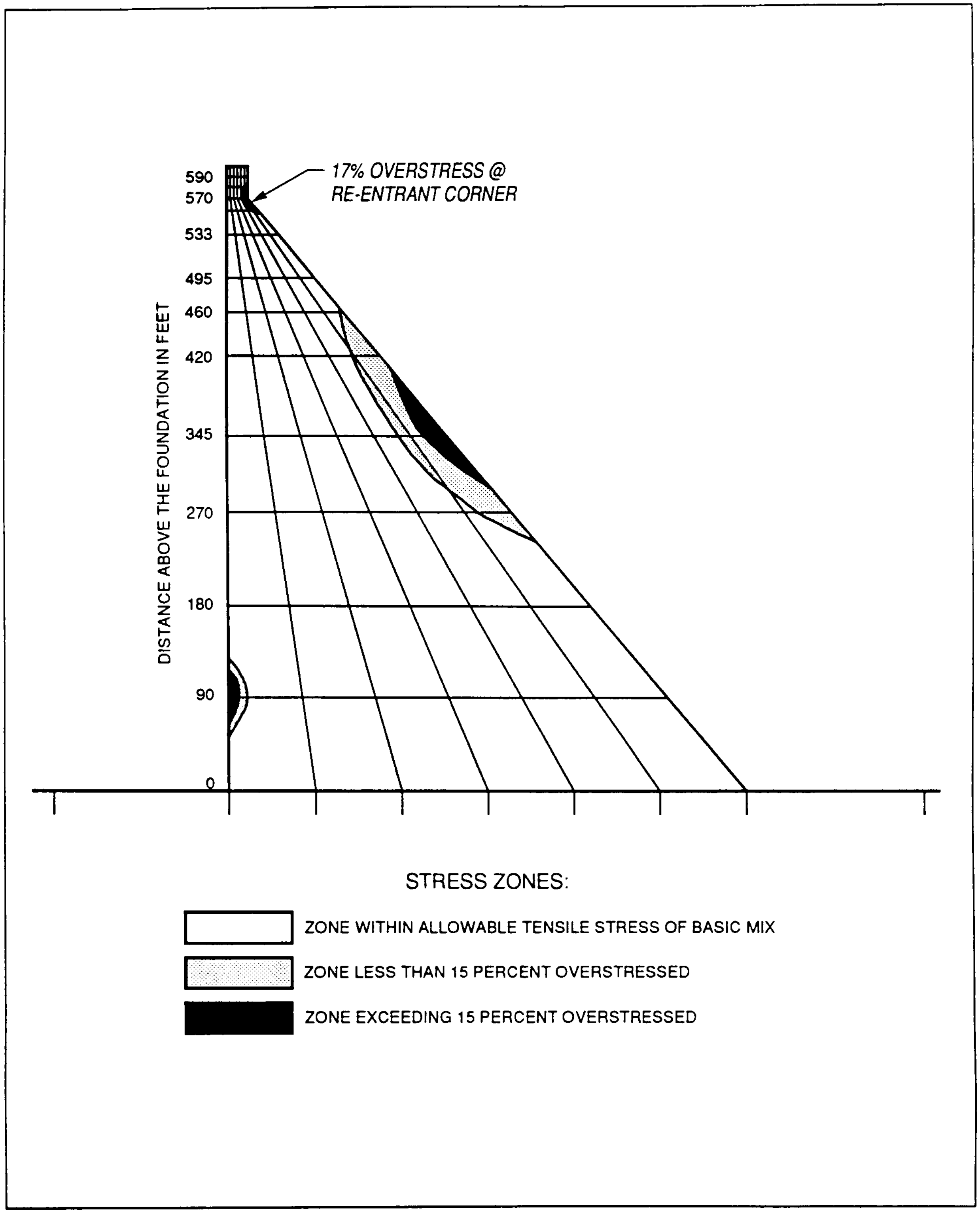


Figure C-8. Zones exceeding the allowable tensile stress for the basic RCC mix