Appendix C
Two-Mode Approximate and Computer Solution Methods of Analysis for a Free-Standing Intake Tower

C-1. General

The example problem presented in this appendix illustrates the procedure used to determine the inertial forces, shears, and moments acting on an intake tower when it is subjected to a given design earthquake. Those forces are then used to illustrate the procedure used to select the reinforcement required for shear and bending resistance and the procedure used to check for potential brittle failure modes. The approximate lumped-mass method described in Appendix B is used to determine the inertial forces, shears, and moments for a maximum design earthquake (MDE) acting in the long (longitudinal) direction of the tower. Those results are compared with the results obtained from a structural analysis computer program with dynamic analysis capability. The structural analysis program was also used to obtain results for the MDE in the transverse direction, and to obtain results for the operational basis earthquake (OBE) in both the longitudinal and transverse directions. The results obtained from the structural analysis program are used to design the reinforcement for the tower and to investigate all potential modes of failure. To illustrate the process, reinforcing steel design and failure mode investigations were performed for the MDE and OBE. This example problem does not account for the presence of an access bridge.

C-2. Discretized Lumped-Mass Model

a. A cantilever tower having five steps in size throughout its height is chosen for the example. Figure C-1 is a sketch of such a tower. The tower has a 0.61-m- (2.0-ft-) thick concrete slab at the top and a heavy 1.83-m- (6.0-ft-) thick slab at its base. The unit weight of concrete (γ_{conc}) is equal to 2,402.7 kg/m^3 (150 lb/ft^3).

b. The tower is discretized into 13 lumped masses, as shown in Figure C-2. The masses are lumped at the two ends and middle of each step, except for the top and bottom slabs where the masses are lumped at the top and bottom of the slabs.

c. The tower is investigated for forces, shears, and moments resulting from earthquakes in each principal direction. The results are combined in accordance with the provisions of Chapter 4 and Appendix B to account for multidirectional ground motion effects.

C-3. Design Earthquake

The tower is located in seismic zone 2B in Oregon. Since no site-specific earthquake has been developed for this particular tower, a standard design response spectrum for a rock site is used. Standard spectra are intended to be used only for preliminary structural evaluations when site-specific response spectra are not yet available for the site. The standard design response spectrum representing the MDE, developed using the spectral acceleration maps of ER 1110-2-1806, is shown in Figure C-3. The peak ground accelerations for the MDE and OBE are 0.25g and 0.12g, respectively.

C-4. Inside and Outside Added Hydrodynamic Masses

a. *Approximate method*. The hydrodynamic masses are approximated using the following simplifying assumptions:

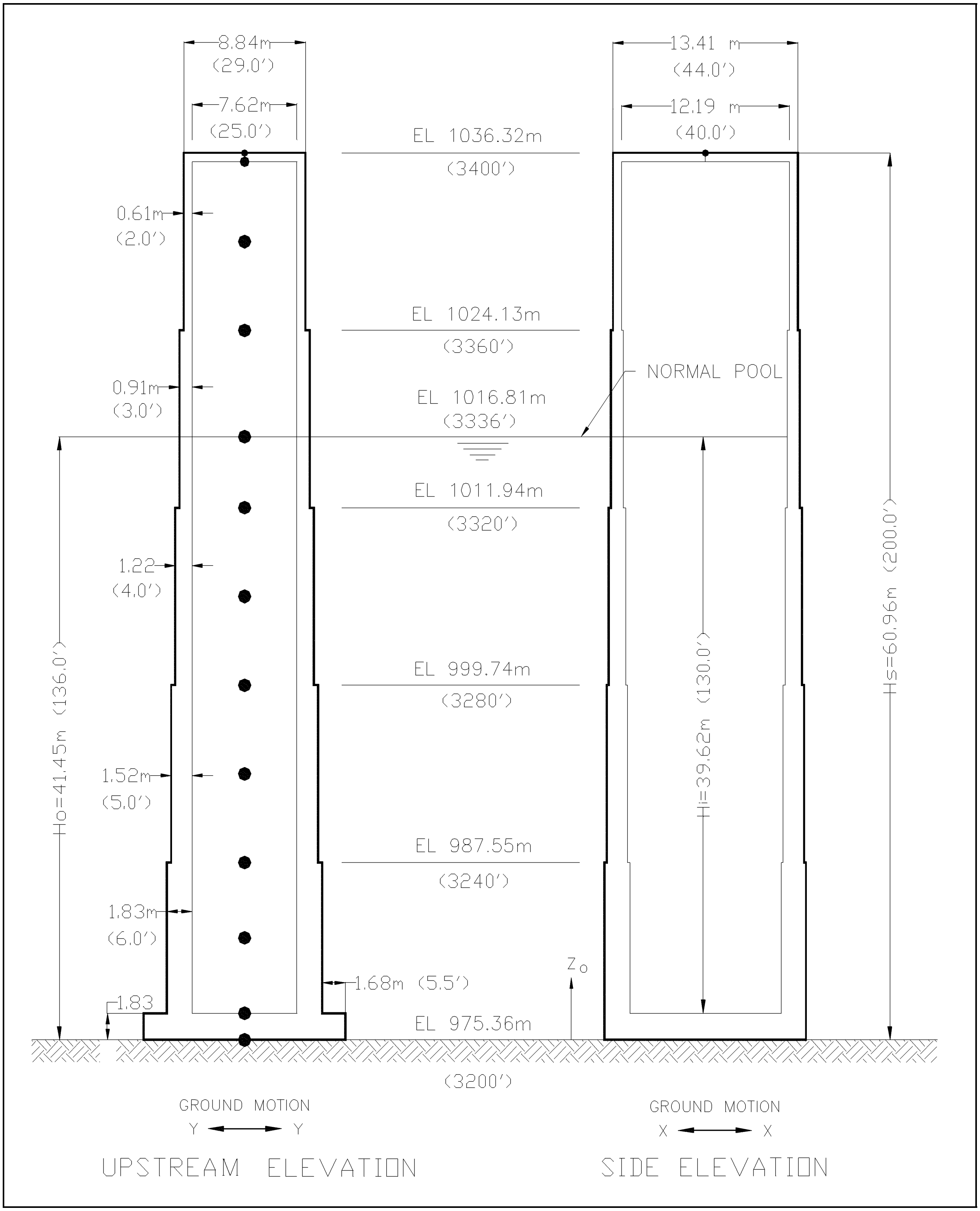

Figure C-1. Tower geometry

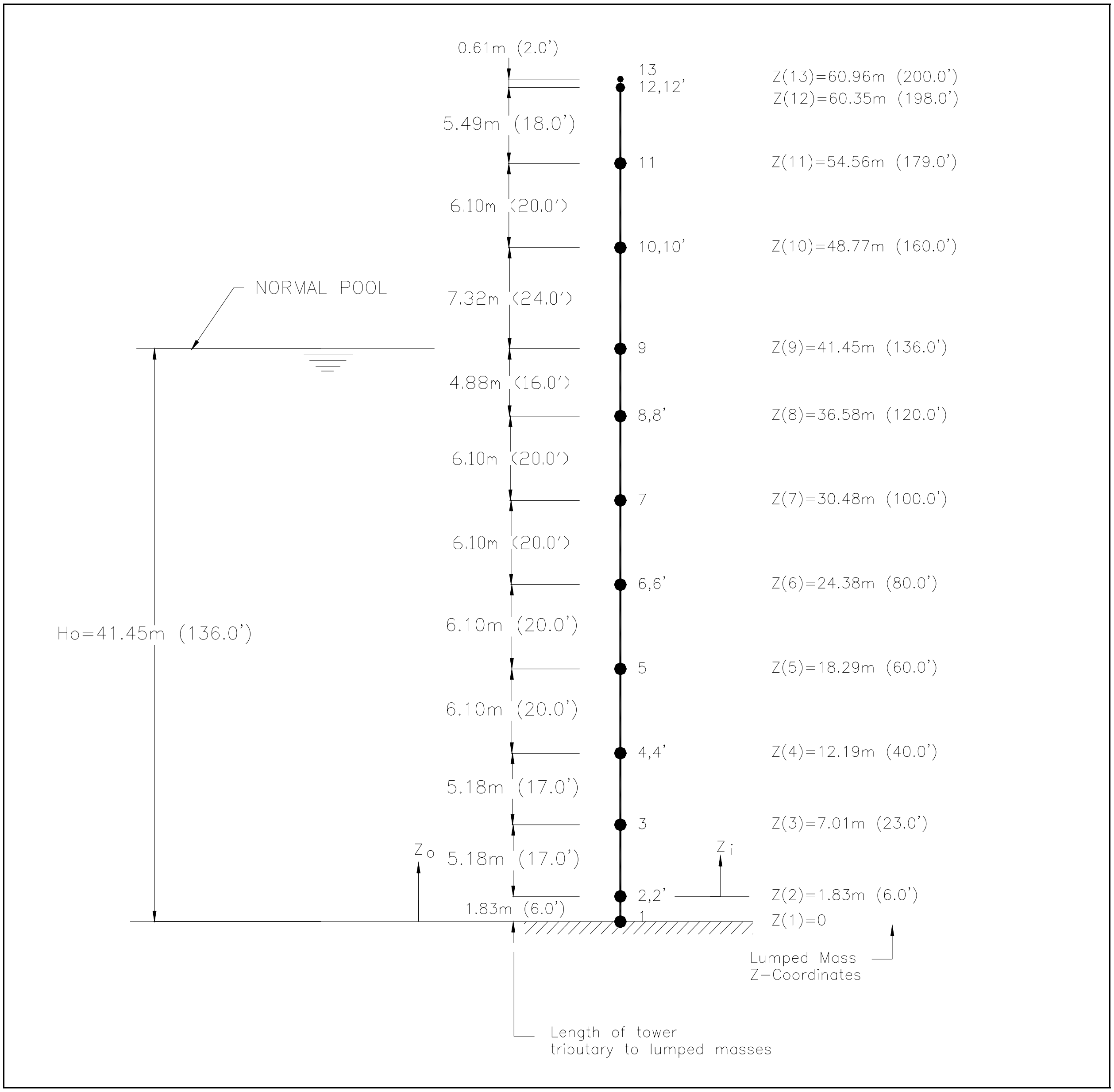

Figure C-2. Tower lumped-mass beam element idealization

(1) Constant average section dimensions such as that between elevations 999.74 m and 1011.94 m are assumed for the entire submerged height of H_o (Figure C-2). Dimensions for these typical sections are shown in Figure C-5.

(2) The area of the tower average section both inside (A_i) and outside (A_o) is converted directly to equivalent circular areas (πr_i^2 and πr_o^2) for the added-mass calculations.

(3) The normalized hydrodynamic added mass $m_\infty^o / \rho_w A_o$ for the infinitely long tower associated with the outside water is obtained from Figure C-6 for the width to depth ratio (a_o/b_o) of the average cross-section.

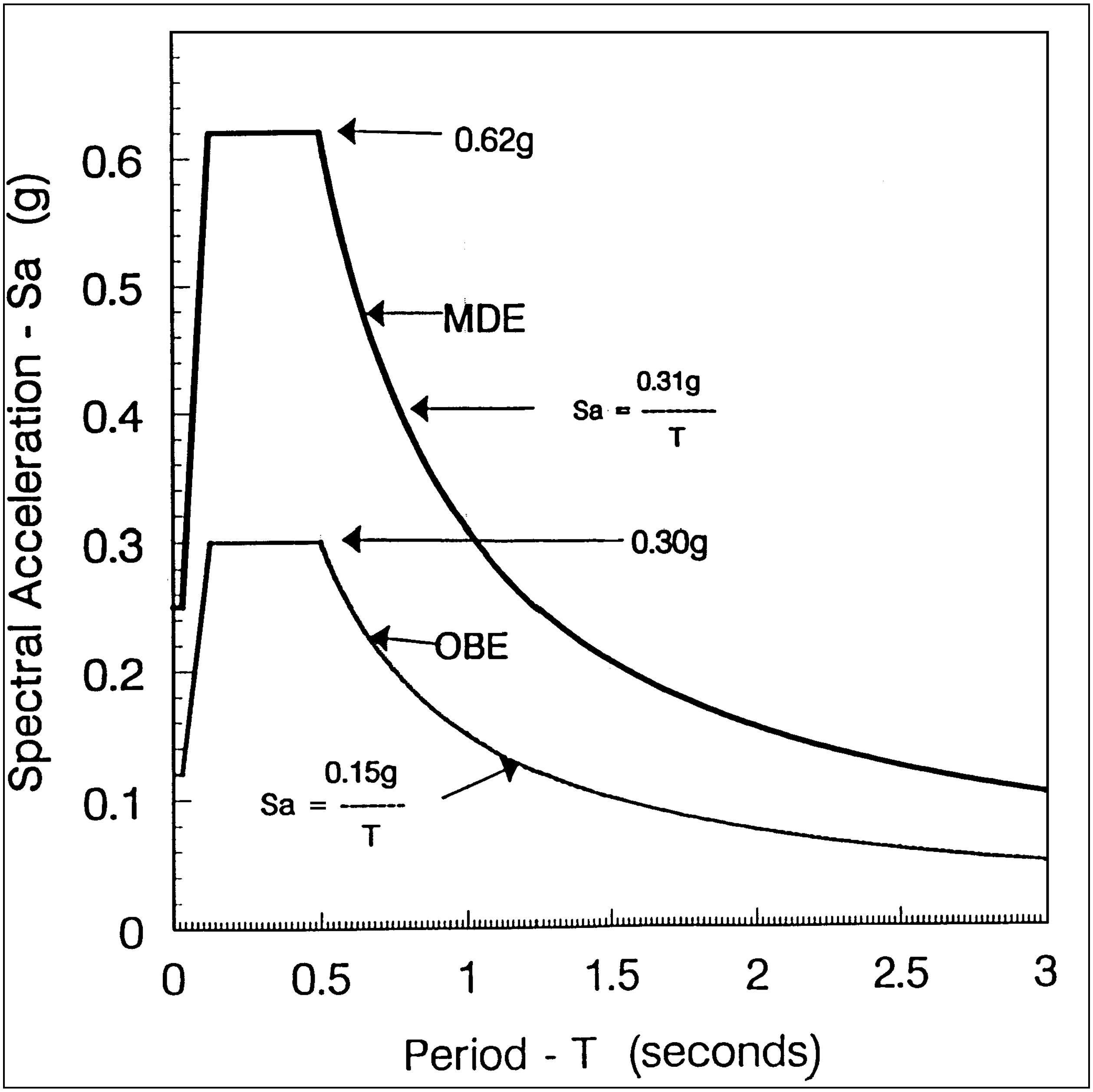

Figure C-3. Standard response spectra for example problem

(4) The normalized added mass is multiplied by $\rho_w A_o$ to obtain the absolute added mass m_∞^o, where ρ_w is the water mass density and A_o is the outside area of the average section.

(5) Knowing r_o/H_o and m_∞^o, the outside added mass m_a^o at any distance z above the base is obtained from Figure C-7. The inside added mass is obtained similarly from Figure C-8, except that no added mass for the infinitely long tower is involved. Tables C-1 and C-2 provide summary calculations for the approximate added hydrodynamic masses for the outside and inside water in transverse direction, respectively.

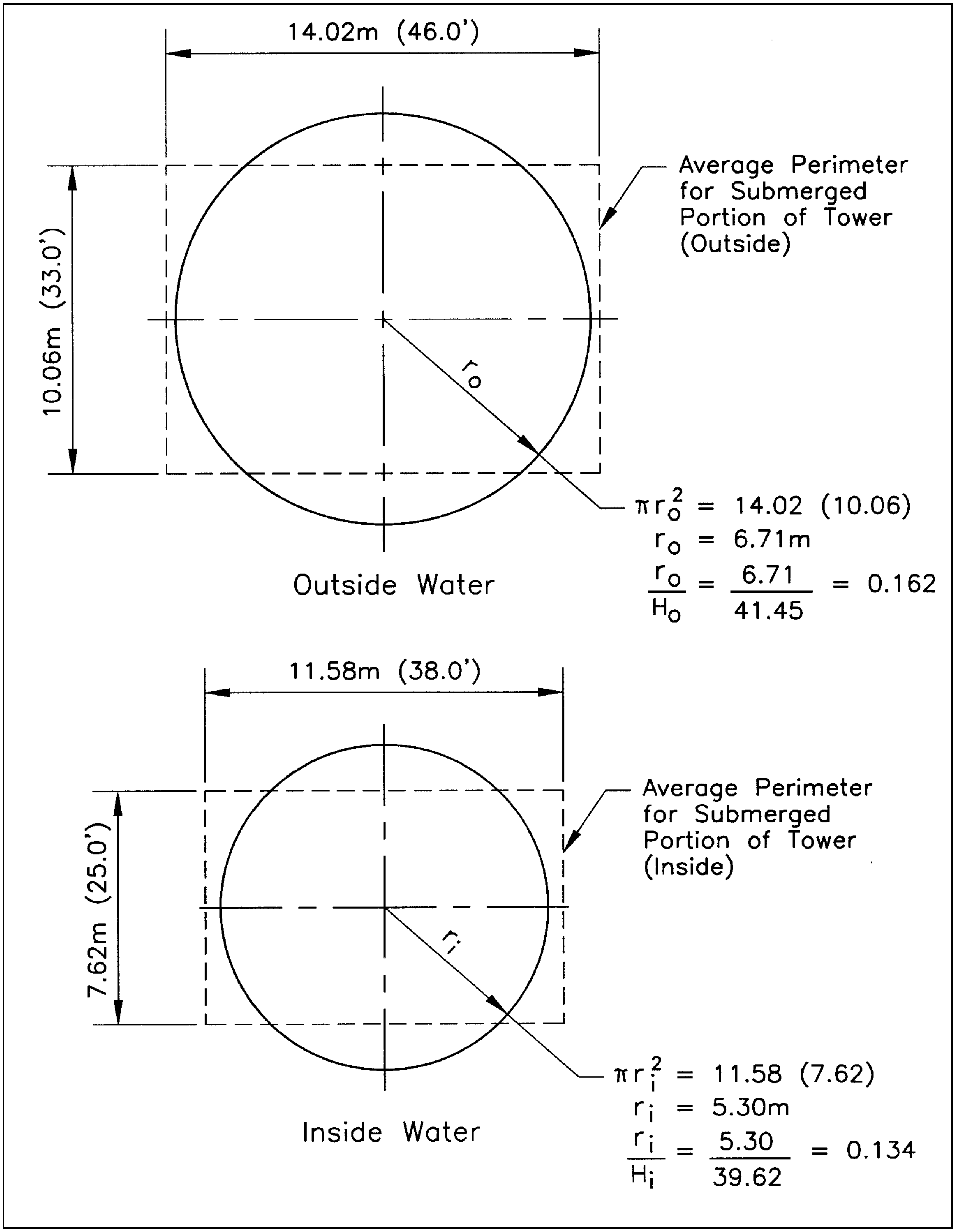

Figure C-4. Added hydrodynamic mass, outside and inside water, circular areas equivalent to average tower dimensions

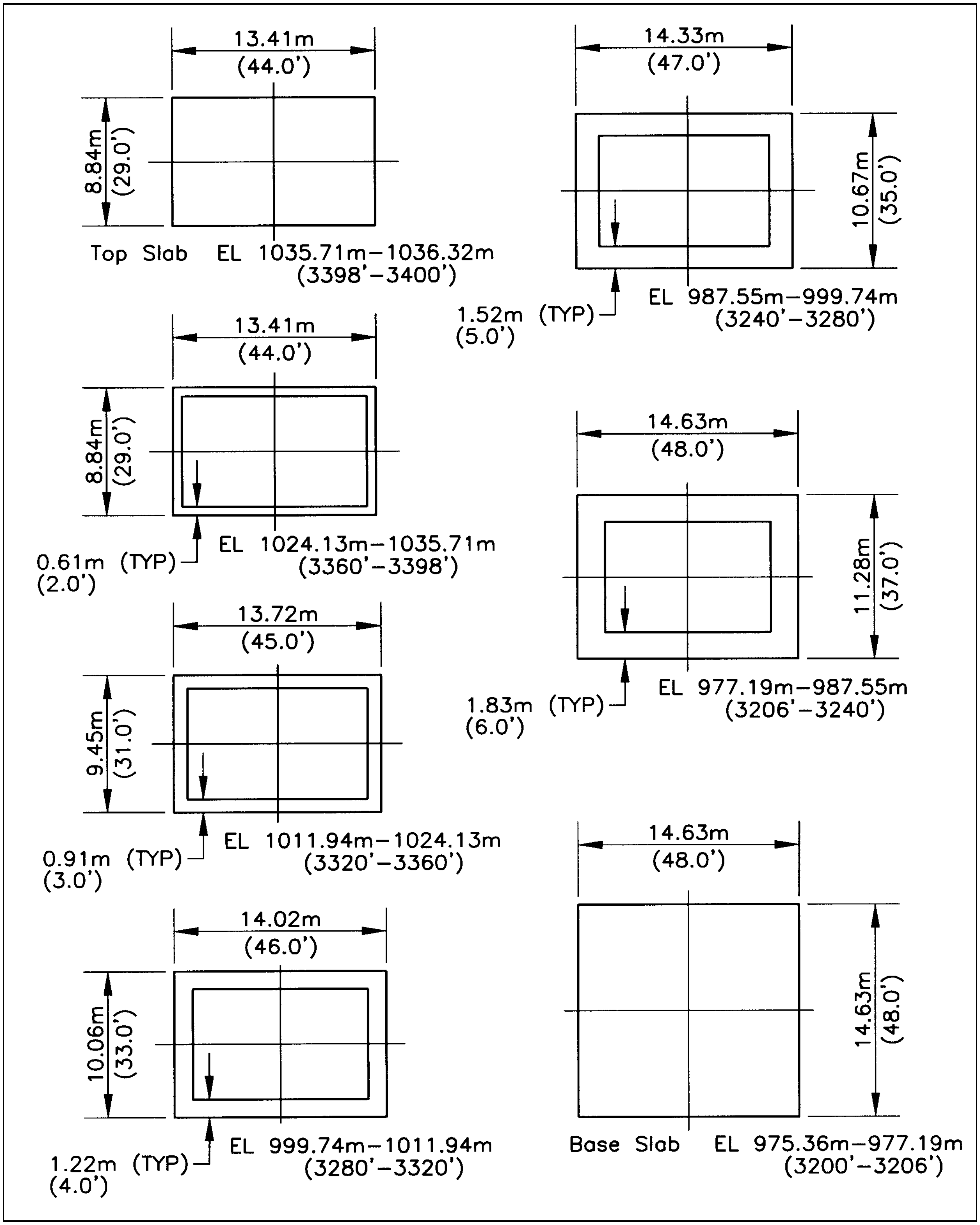

Figure C-5. Tower typical sections and dimensions

a_o/b_o	$\frac{m_\infty^o}{\rho_w A_o}$
X	Y
1/5	0.311
1/4	0.377
1/3	0.480
1/2	0.667
2/3	0.853
1	1.186
3/2	1.661
2	2.136
3	3.038
4	3.896
5	4.752

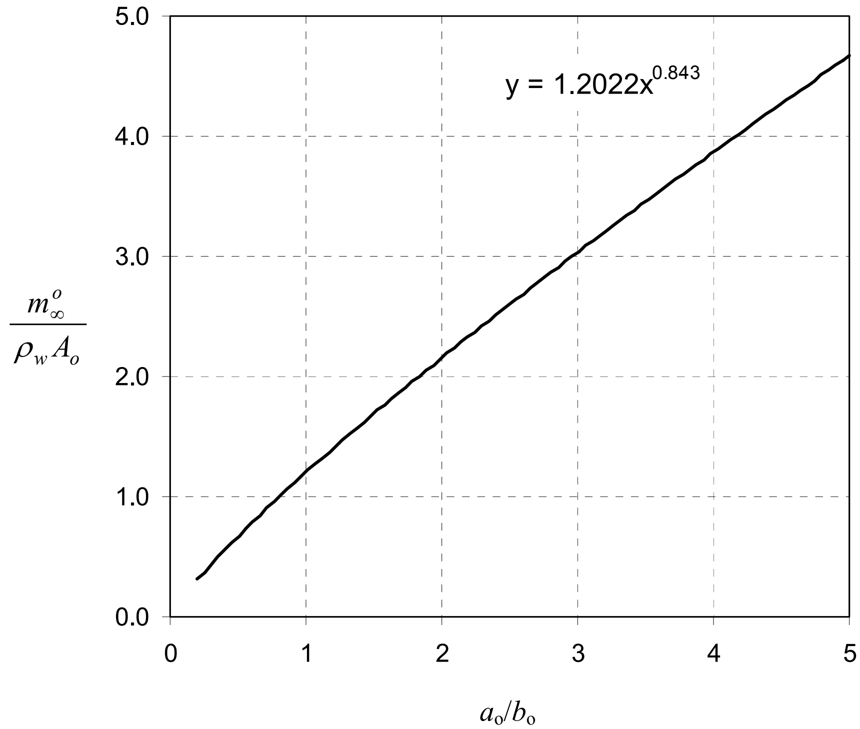

Figure C-6. Added mass m_∞^o for infinitely long tower associated with outside water

b. Refined method.

(1) Refined added-mass analysis is carried out by converting each uniform section of the tower, first into an "equivalent" uniform elliptical section, and then into a corresponding "equivalent" circular section. The calculation steps are generally similar to those described in C-4*a*, except that they are performed separately for each uniform section of the tower and involve conversion to both elliptic and circular sections, as described in Appendix D. Tables C-3 and C-4 provide a summary of the refined added-mass calculations for the outside and inside water, respectively. Figure C-9 provides a comparison between the total (inside + outside) refined and approximate added masses. The results show a close agreement between the refined and approximate added masses for the example tower. The approximate added mass values are therefore used in all succeeding calculations.

(2) Appendix D provides a step-by-step procedure for computation of the refined added-mass of water applied to longitudinal excitation. Note that Tables C-3 and C-4 were developed using the same procedure. However, since locations of lumped masses in this example somewhat differ from those selected in Appendix D (compare Figures C-1 and C-2 with similar figures in Appendix D), the unit added-mass values in Tables C-3 and C-4 do not match the corresponding values for transverse excitation given in the last two columns of Table D-7.

C-5. Structural Mass and Moments of Inertia

a. The area, mass, and gross moments of inertia at each level of discontinuity are now computed, with the results presented in Table C-5. Dimensions are taken from Figure C-5. I_{yy} is the larger moment of inertia for bending in the longer (longitudinal) direction, and I_{xx} is the smaller moment of inertia for bending in the shorter (transverse) direction. Mass m_o is mass/unit length.

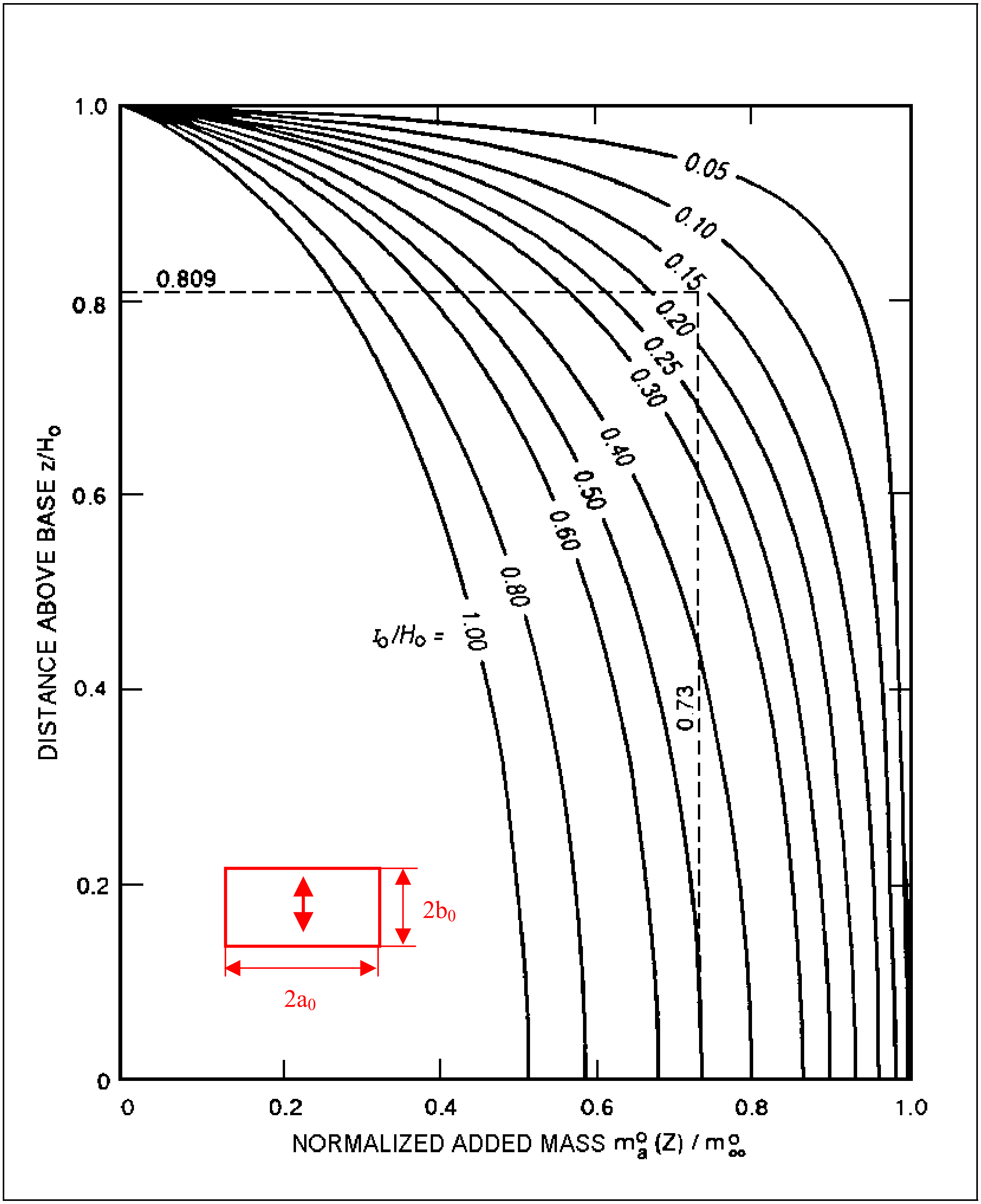

Figure C-7. Normalized outside hydrodynamic added mass

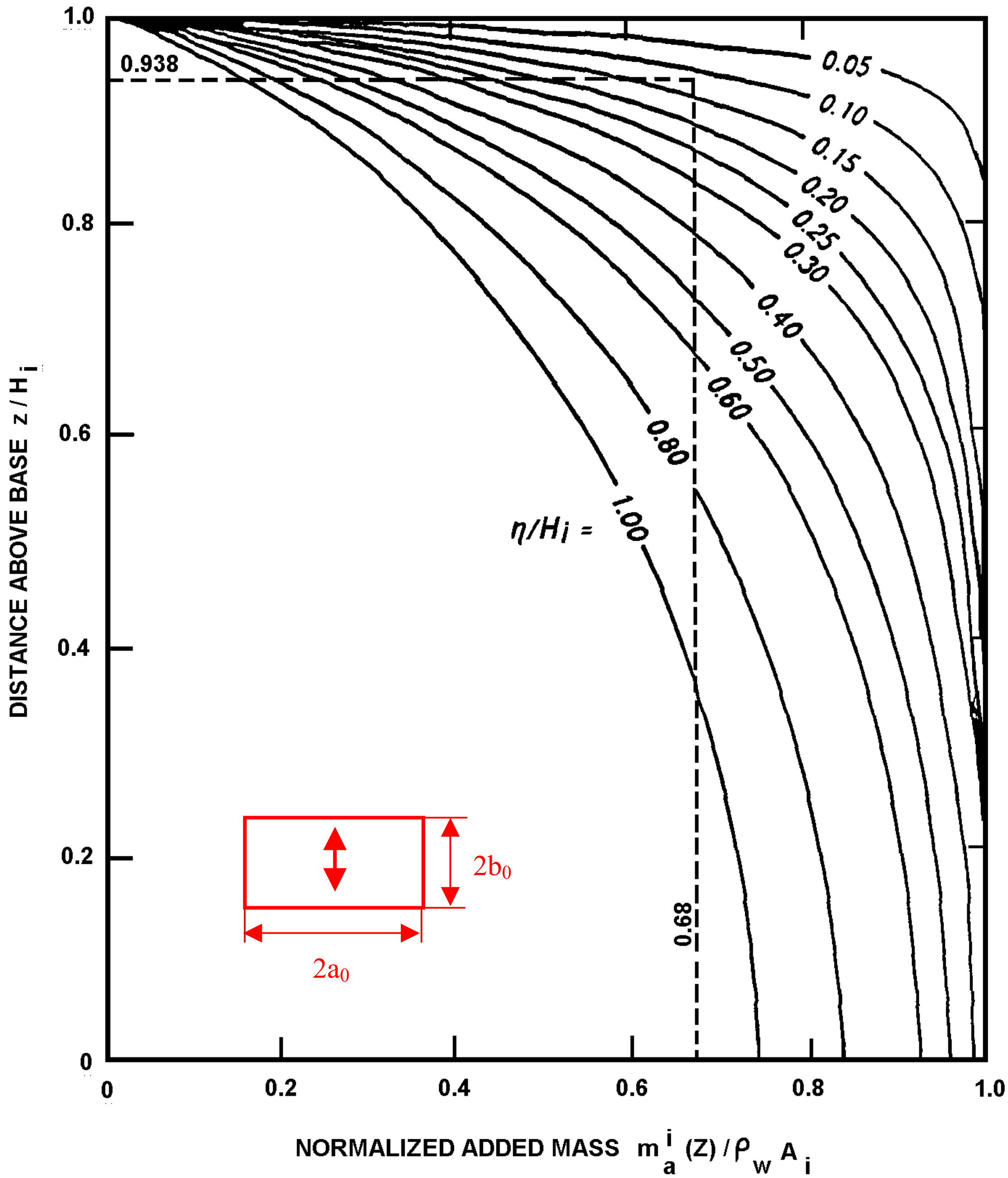

Figure C-8. Normalized inside hydrodynamic added mass

Table C-1
Approximate Added Hydrodynamic Mass Calculations, Outside Water (Transverse Direction)

<table>
<tr><th>Node #</th><th>Distance from the Base z m (ft)</th><th>z / H_0</th><th>Elem #</th><th>Depth $2a_0$ m (ft)</th><th>Width $2b_0$ m (ft)</th><th>a_0/b_0</th><th>r_0 m (ft)</th><th>r_0/H_0</th><th>$\frac{m_a^o}{m_\infty^a}$</th><th>$\rho_w A_0$ kN-s^2/m^2 (k-s^2/ft^2)</th><th>$\frac{m_\infty^a}{\rho_w A_o}$</th><th>$m_\infty^a$ kN-s^2/m^2 (k-s^2/ft^2)</th><th>Length m (ft)</th><th>Distributed Hydrodynamic Added Mass m_a^o kN-s^2/m^2 (k-s^2/ft^2)</th><th>Lumped Hydrodynamic Added Mass m_a^o kN-s2/m2 (k-s^2/ft^2)</th></tr>
<tr><td rowspan="2">9</td><td rowspan="2">41.45
(136)</td><td rowspan="2">1.000</td><td rowspan="3">8</td><td rowspan="3">14.02
(46)</td><td rowspan="3">10.06
(33)</td><td rowspan="3">1.394</td><td rowspan="3">6.700
(21.98)</td><td rowspan="3">0.366</td><td rowspan="3">0.610</td><td rowspan="3">141.027
(2.94)</td><td rowspan="3">1.563</td><td rowspan="3">220.407
(4.60)</td><td rowspan="3">4.88
(16)</td><td rowspan="3">134.423
(2.81)</td><td rowspan="2">327.777
(22.47)</td></tr>
<tr></tr>
<tr><td rowspan="2">8</td><td rowspan="2">36.58
(120)</td><td rowspan="2">0.882</td><td rowspan="2">867.719
(59.47)</td></tr>
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(46)</td><td rowspan="2">10.06
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(2.94)</td><td rowspan="2">1.563</td><td rowspan="2">220.407
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(21.98)</td><td rowspan="2">0.366</td><td rowspan="2">0.919</td><td rowspan="2">141.027
(2.94)</td><td rowspan="2">1.563</td><td rowspan="2">220.407
(4.60)</td><td rowspan="2">6.10
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Table C-2
Approximate Added Hydrodynamic Mass Calculations, Inside Water (Transverse Direction)

<table>
<tr><th>Node #</th><th>Distance from the Base z m (ft)</th><th>z / H_i</th><th>Elem #</th><th>Depth $2a_i$ m (ft)</th><th>Width $2b_i$ m (ft)</th><th>a_i/b_i</th><th>r_i m (ft)</th><th>r_i/H_i</th><th>$\frac{m_a^i}{m_\infty^i}$</th><th>m_∞^i kN-s^2/m^2 (k-s^2/ft^2)</th><th>Length m (ft)</th><th>Distributed Hydrodynamic Added Mass m_a^i kN-s^2/m^2 (k-s^2/ft^2)</th><th>Lumped Hydrodynamic Added Mass m_a^i kN-s2/m2 (k-s^2/ft^2)</th></tr>
<tr><td rowspan="2">9</td><td rowspan="2">41.45
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(38)</td><td rowspan="3">7.62
(25)</td><td rowspan="3">1.520</td><td rowspan="3">5.300
(17.39)</td><td rowspan="3">0.134</td><td rowspan="3">0.705</td><td rowspan="3">88.258
(1.84)</td><td rowspan="3">4.88
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(17.39)</td><td rowspan="2">0.134</td><td rowspan="2">0.998</td><td rowspan="2">88.258
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(6)</td><td rowspan="3"></td></tr>
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<tr></tr>
</table>

Table C-3
Refined Added Hydrodynamic Mass Calculations, Outside Water (Transverse Direction)

Node #	Distance from the Base z m (ft)	z / H_0	Elem #	Depth $2a_0$ m (ft)	Width $2b_0$ m (ft)	a_0/b_0	$\tilde{a}_o$ m (ft)	$\tilde{b}_o$ m (ft)	$\frac{\tilde{a}_o}{H_o}$	r_0/H_0	r_0 m (ft)	$\frac{m_a^o}{m_\infty^o}$	$\rho_w A_0$ kN-s²/m² (k-s²/ft²)	$\frac{m_\infty^o}{\rho_w A_o}$	m_∞^o kN-s²/m² (k-s²/ft²)	Length m (ft)	Distributed Hydrodynamic Added Mass m_a^o kN-s²/m² (k-s²/ft²)	Lumped Hydrodynamic Added Mass m_a^o kN-s2/m2 (k-s ²/ft²)
9	41.45 (136)	1.000																310.283 (21.27)
			8	13.72 (45)	9.45 (31)	1.452	7.738 (25.39)	5.331 (17.49)	0.423	0.372	6.798 (22.30)	0.607	129.600 (2.70)	1.618	209.653 (4.37)	4.88 (16)	127.249 (2.66)	
8	36.58 (120)	0.882																843.909 (57.84)
			7	14.02 (46)	10.06 (33)	1.394	7.910 (25.95)	5.675 (18.62)	0.433	0.385	7.044 (23.11)	0.794	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	175.074 (3.66)	
7	30.48 (100)	0.735																1121.204 (76.85)
			6	14.02 (46)	10.06 (33)	1.394	7.910 (25.95)	5.675 (18.62)	0.433	0.385	7.044 (23.11)	0.875	141.027 (2.94)	1.563	220.407 (4.60)	6.10 (20)	192.775 (4.03)	
6	24.38 (80)	0.588																1229.038 (84.24)
			5	14.33 (47)	10.67 (35)	1.343	8.082 (26.52)	6.019 (19.75)	0.442	0.399	7.290 (23.92)	0.909	152.826 (3.19)	1.514	231.414 (4.83)	6.10 (20)	210.453 (4.40)	
5	18.29 (60)	0.441																1297.771 (88.95)
			4	14.33 (47)	10.67 (35)	1.343	8.082 (26.52)	6.019 (19.75)	0.442	0.399	7.290 (23.92)	0.930	152.826 (3.19)	1.514	231.414 (4.83)	6.10 (20)	215.325 (4.50)	
4	12.19 (40)	0.294																1244.985 (85.33)
			3	14.63 (48)	11.28 (37)	1.297	8.254 (27.08)	6.363 (20.88)	0.451	0.412	7.534 (24.72)	0.936	164.996 (3.44)	1.471	242.675 (5.06)	5.18 (17)	227.217 (4.75)	
3	7.01 (23)	0.169																1179.899 (80.87)
			2	14.63 (48)	11.28 (37)	1.297	8.254 (27.08)	6.363 (20.88)	0.451	0.412	7.534 (24.72)	0.940	164.996 (3.44)	1.471	242.675 (5.06)	5.18 (17)	228.202 (4.77)	
2	1.83 (6)	0.044																806.719 (55.29)
			1	14.63 (48)	14.63 (48)	1.000	8.254 (27.08)	8.254 (27.08)	0.451	0.451	8.248 (27.06)	0.929	214.049 (4.46)	1.185	253.562 (5.29)	1.83 (6)	235.666 (4.92)	
1	0.00 (0)	0.000																215.493 (14.77)

Table C-4
Refined Added Hydrodynamic Mass Calculations, Inside Water (Transverse Direction)

Node #	Distance from the Base z m (ft)	z / H_i	Elem #	Depth $2a_i$ m (ft)	Width $2b_i$ m (ft)	a_i/b_i	$\tilde{a}_i$ m (ft)	$\tilde{b}_i$ m (ft)	$\frac{\tilde{a}_i}{H_i}$	r_i/H_i	r_i m (ft)	$\frac{m_a^i}{m_\infty^i}$	m_∞^i kN-s²/m² (k-s²/ft²)	Length m (ft)	Distributed Hydrodynamic Added Mass m_i kN-s²/m² (k-s²/ft²)	Lumped Hydrodynamic Added Mass m_i kN-s2/m2 (k-s²/ft²)
9	41.45 (136)	1.046														169.505 (11.62)
			8	11.89 (39)	7.62 (25)	1.560	6.707 (22.00)	4.299 (14.10)	0.169	0.110	4.339 (14.24)	0.767	90.580 (1.89)	4.88 (16)	69.515 (1.45)	
8	36.58 (120)	0.923														433.422 (29.71)
			7	11.58 (38)	7.62 (25)	1.520	6.535 (21.44)	4.299 (14.10)	0.165	0.110	4.346 (14.26)	0.981	88.258 (1.84)	6.10 (20)	86.587 (1.81)	
7	30.48 (100)	0.769														532.166 (36.48)
			6	11.58 (38)	7.62 (25)	1.520	6.535 (21.44)	4.299 (14.10)	0.165	0.110	4.346 (14.26)	0.997	88.258 (1.84)	6.10 (20)	88.008 (1.84)	
6	24.38 (80)	0.615														529.744 (36.31)
			5	11.28 (37)	7.62 (25)	1.480	6.363 (20.88)	4.299 (14.10)	0.161	0.110	4.352 (14.28)	0.998	85.935 (1.79)	6.10 (20)	85.792 (1.79)	
5	18.29 (60)	0.462														523.033 (35.85)
			4	11.28 (37)	7.62 (25)	1.480	6.363 (20.88)	4.299 (14.10)	0.161	0.110	4.352 (14.28)	0.999	85.935 (1.79)	6.10 (20)	85.807 (1.79)	
4	12.19 (40)	0.308														477.819 (32.75)
			3	10.97 (36)	7.62 (25)	1.440	6.191 (20.31)	4.299 (14.10)	0.156	0.110	4.357 (14.29)	0.998	83.613 (1.74)	5.18 (17)	83.480 (1.74)	
3	7.01 (23)	0.177														432.590 (29.65)
			2	10.97 (36)	7.62 (25)	1.440	6.191 (20.31)	4.299 (14.10)	0.156	0.110	4.357 (14.29)	0.999	83.613 (1.74)	5.18 (17)	83.491 (1.74)	
2	1.83 (6)	0.046														216.295 (14.83)
			1	0.00 (0)	0.00 (0)	1.000	0.000 (0.00)	0.000 (0.00)	0.000	0.000	0.000 (0.00)	0.000	0.000 (0.00)	1.83 (6)		
1	0.00 (0)	0.000														

C-6. Total Lumped-Mass Values

a. As indicated in Figure C-2, the lumped masses 1 through 13 are positioned at convenient points along the height of the tower. The magnitude of each of the lumped masses can be found by summing the mass of the structure with that of the inside and outside added hydrodynamic masses. The computed results are shown in Table C-6 for the longitudinal direction and in Table C-7 for the transverse direction.

b. Calculations of the total mass for each direction follow the same procedure, except that the values of the hydrodynamic added masses are different. In Tables C-6 and C-7, the structural mass m_0 is the same for each direction. However, the added hydrodynamic masses for the transverse direction are larger than for the longitudinal direction. This is because for the earthquake motions in the transverse direction, the wider face of the tower is accelerated into the surrounding water. Therefore, the added masses are commensurately higher.

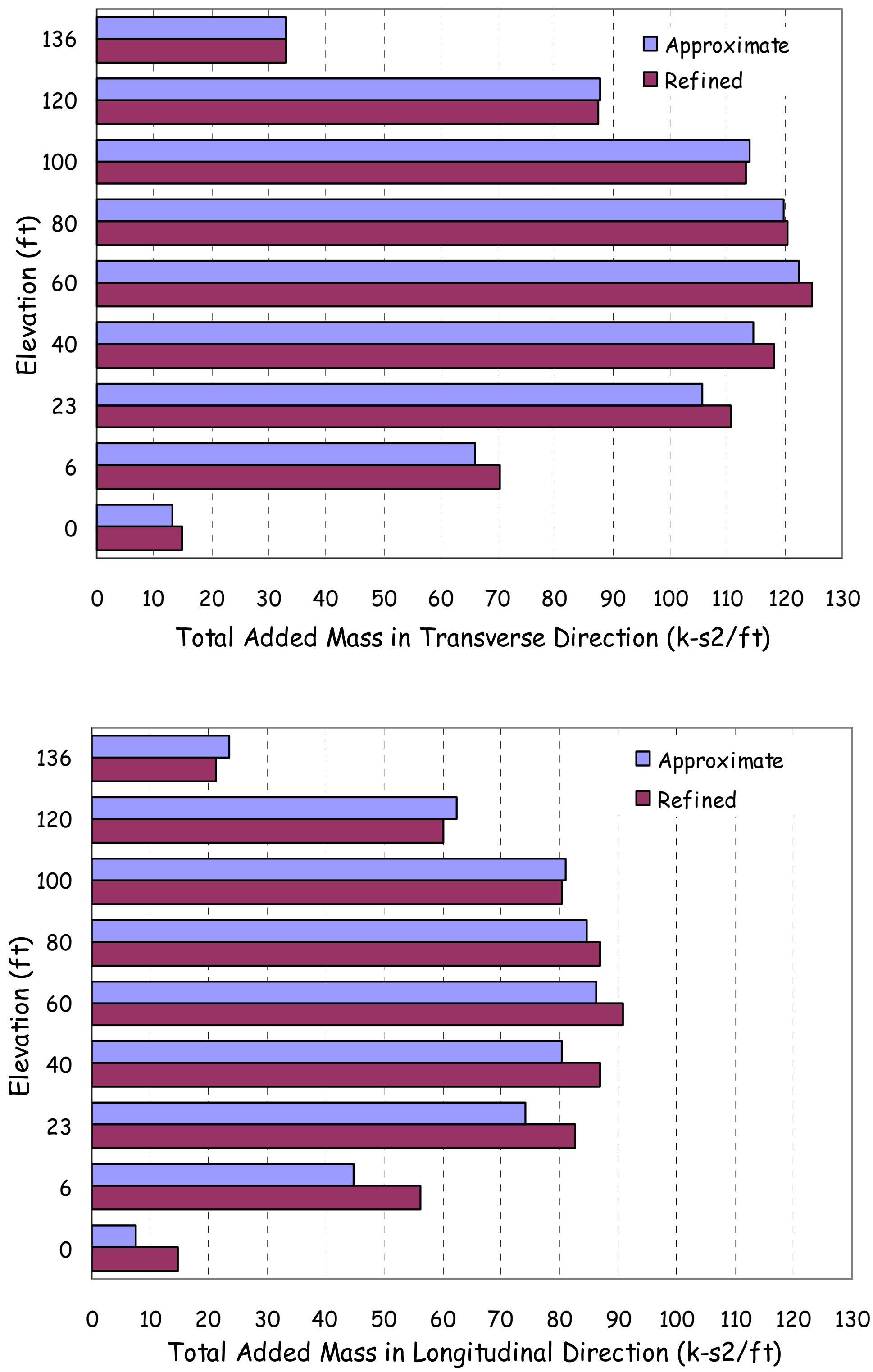


Figure C-9. Comparison of approximate and refined total added hydrodynamic masses

Table C-5
Tower Section Properties and Mass

Node #	Distance from the Base m (ft)	Elem #	Length m (ft)	Section Area m^2 (ft^2)	Ixx m^4 (ft^4)	Iyy m^4 (ft^4)	Distributed Mass Due to Self Weight kN-s^2/m^2 (k-s^2/ft^2)	Mass - m_0 Due to Self Weight kN-s^2/m^2 (k-s^2/ft)
13	60.96 (200)							86.72 (5.94)
		12	0.61 (2)	118.54 (1276)	772 (89,426)	1,777 (205,861)	284.53 (5.94)	
12	60.35 (198)							264.93 (18.16)
		11	5.79 (19)	25.64 (276)	322 (37,343)	626 (72,528)	61.54 (1.29)	
11	54.56 (179)							356.41 (24.43)
		10	5.79 (19)	25.64 (276)	322 (37,343)	626 (72,528)	61.54 (1.29)	
10	48.77 (160)							520.75 (35.69)
		9	7.32 (24)	39.02 (420)	526 (60,935)	965 (111,825)	93.65 (1.96)	
9	41.45 (136)							570.91 (39.13)
		8	4.88 (16)	39.02 (420)	526 (60,935)	965 (111,825)	93.65 (1.96)	
8	36.58 (120)							614.41 (42.11)
		7	6.10 (20)	52.77 (568)	762 (88,279)	1,324 (153,357)	126.65 (2.65)	
7	30.48 (100)							772.09 (52.92)
		6	6.10 (20)	52.77 (568)	762 (88,279)	1,324 (153,357)	126.65 (2.65)	
6	24.38 (80)							875.39 (60.00)
		5	6.10 (20)	66.89 (720)	1,034 (119,750)	1,703 (197,290)	160.55 (3.35)	
5	18.29 (60)							978.70 (67.08)
		4	6.10 (20)	66.89 (720)	1,034 (119,750)	1,703 (197,290)	160.55 (3.35)	
4	12.19 (40)							995.42 (68.23)
		3	5.18 (17)	81.38 (876)	1,344 (155,737)	2,104 (243,792)	195.33 (4.08)	
3	7.01 (23)							1012.14 (69.37)
		2	5.18 (17)	81.38 (876)	1,344 (155,737)	2,104 (243,792)	195.33 (4.08)	
2	1.83 (6)							975.85 (66.89)
		1	1.83 (6)	214.05 (2304)	3,818 (442,368)	3,818 (442,368)	513.75 (10.73)	
1	0.00 (0)							469.78 (32.20)

C-7. Inertial Forces by the Approximate Lumped-Mass Method (Longitudinal Direction)

a. The physical constants computed in paragraphs C-5 and C-6 are now used to compute the dynamic response of the tower to earthquake excitation. The dynamic analysis follows five steps:

(1) Using the moments of inertia, determine the normalized displacements of the first and second shape functions.

(2) Using the normalized displacements and the physical constants, determine the first and second natural periods of vibration and the mass participation factors. Compute natural periods using the effective moments of inertia. For the MDE excitation, the effective moment of inertia is taken equal to 80 percent of the gross moments of inertia. For the OBE excitation, the effective moment of inertia is taken the same as the gross values.

(3) Using these natural periods and the standard response spectrum, determine the first and second spectral accelerations.

Table C-6
Total Mass for Earthquake Motions in Longitudinal Direction

Node #	Distance from the Base m (ft)	Mass - m_0 Due to Self Weight kN-s^2/m (k-s^2/ft)	Inside Hydrodynamic Added Mass Longitudinal Dir. kN-s^2/m (k-s^2/ft)	Outside Hydrodynamic Added Mass Longitudinal Dir. kN-s^2/m (k-s^2/ft)	Total Lumped Mass Longitudinal Dir. kN-s^2/m (k-s^2/ft)
13	60.96 (200)	86.72 (5.94)			86.72 (5.94)
12	60.35 (198)	264.93 (18.16)			264.93 (18.16)
11	54.56 (179)	356.41 (24.43)			356.41 (24.43)
10	48.77 (160)	520.75 (35.69)			520.75 (35.69)
9	41.45 (136)	570.91 (39.13)	138.27 (9.48)	173.33 (11.88)	882.51 (60.49)
8	36.58 (120)	614.41 (42.11)	389.32 (26.68)	487.26 (33.40)	1490.98 (102.19)
7	30.48 (100)	772.09 (52.92)	516.45 (35.40)	657.68 (45.08)	1946.22 (133.40)
6	24.38 (80)	875.39 (60.00)	526.45 (36.08)	741.40 (50.82)	2143.24 (146.90)
5	18.29 (60)	978.70 (67.08)	522.49 (35.81)	803.83 (55.10)	2305.02 (157.99)
4	12.19 (40)	995.42 (68.23)	477.73 (32.74)	791.49 (54.25)	2264.64 (155.22)
3	7.01 (23)	1012.14 (69.37)	432.58 (29.65)	772.18 (52.93)	2216.90 (151.95)
2	1.83 (6)	975.85 (66.89)	216.30 (14.83)	602.36 (41.29)	1794.50 (123.00)
1	0.00 (0)	469.78 (32.20)		215.49 (14.77)	685.27 (46.97)

(4) Using the normalization ratios and the spectral acceleration of each lumped mass, determine the inertia forces at each lumped mass, as well as the corresponding shears and moments at those points with computations including both shape functions.

(5) Combine the two sets of forces, shears, and moments to obtain the final design values.

b. For the example tower, the foregoing steps are presented in sequence in the following discussions.

(1) The first step in the approximate analysis is to determine the first and second shape functions and their normalized displacements. Figure C-10 shows plots of the first and second shape functions interpolated from the normalized shape function values given in Tables B-1 and B-2. The ratio of the moment of inertia of the base to that of the top step I_{BASE}/I_{TOP} is computed using the earlier calculations for moment of inertia. The section just above the base is used for the base moment of inertia, and the section just below the top slab is used for the top moment of inertia. Using the moments of inertia for the stiff bottom and top slabs would introduce additional inaccuracies because the bottom and top slabs are not representative of the stiffness of the rest of the tower. Also, it should be remembered that the stepped taper assumed in creating the tables may or may not agree exactly with the actual stepped taper. For these reasons, the shape functions are considered approximate. The following calculations illustrate the use of the two-mode approximate method for the longitudinal MDE excitation. Similar calculations are required for the transverse MDE and for both the longitudinal and transverse OBE excitations.

$$\frac{I_{\text{BASE}}}{\text{I}_{\text{TOP}}} = \frac{2096.61}{623.74} = 3.361 \qquad \text{(C-1)}$$

Values of the interpolated normalized displacements are listed in Figure C-10 for the appropriate values of I_{BASE}/I_{TOP}.

Table C-7
Total Mass for Earthquake Motions in Transverse Direction

Node #	Distance from the Base m (ft)	Mass - m_0 Due to Self Weight kN-s²/m (k-s²/ft)	Inside Hydrodynamic Added Mass Transverse Dir. kN-s²/m (k-s²/ft)	Outside Hydrodynamic Added Mass Transverse Dir. kN-s²/m (k-s²/ft)	Total Lumped Mass Transverse Dir. kN-s²/m (k-s²/ft)
13	60.96 (200)	86.72 (5.94)			86.72 (5.94)
12	60.35 (198)	264.93 (18.16)			264.93 (18.16)
11	54.56 (179)	356.41 (24.43)			356.41 (24.43)
10	48.77 (160)	520.75 (35.69)			520.75 (35.69)
9	41.45 (136)	570.91 (39.13)	169.51 (11.62)	310.28 (21.27)	1050.70 (72.02)
8	36.58 (120)	614.41 (42.11)	433.42 (29.71)	843.91 (57.84)	1891.74 (129.66)
7	30.48 (100)	772.09 (52.92)	532.17 (36.48)	1121.20 (76.85)	2425.46 (166.24)
6	24.38 (80)	875.39 (60.00)	529.74 (36.31)	1229.04 (84.24)	2634.18 (180.55)
5	18.29 (60)	978.70 (67.08)	523.03 (35.85)	1297.77 (88.95)	2799.50 (191.88)
4	12.19 (40)	995.42 (68.23)	477.82 (32.75)	1244.98 (85.33)	2718.22 (186.31)
3	7.01 (23)	1012.14 (69.37)	432.59 (29.65)	1179.90 (80.87)	2624.63 (179.89)
2	1.83 (6)	975.85 (66.89)	216.30 (14.83)	806.72 (55.29)	1998.86 (137.00)
1	0.00 (0)	469.78 (32.20)		215.49 (14.77)	685.27 (46.97)

(2) The second step in the analysis is to determine the first and second natural periods of vibration. If there were no added mass of water associated with the tower, the natural periods could be obtained directly from Figure C-11. Nevertheless, it is good practice to compute these periods for reference and comparison. Figure C-11 is entered with a parameter:

$$\sqrt{\frac{m_{\mathrm{TOP}}\mathrm{L}^4}{\mathrm{EI}_{\mathrm{TOP}}}} = \sqrt{\frac{61.54 \times 60.96^4}{21{,}525{,}000 \times 626 \times 0.8}} = 0.281 \qquad \text{(C-2)}$$

where

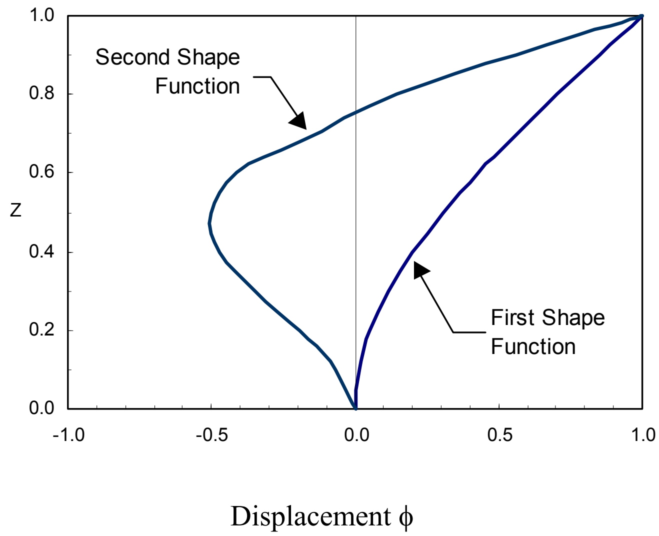
m_{top} = mass of top step

L = overall height of tower

E = modulus of elasticity

For this value of the parameter and $(I_{\mathrm{BASE}}/I_{\mathrm{TOP}}) = 3.361$, Figure C-11 results in a first period of $T_1 = 0.32$ sec, and a second period of $T_2 = 0.06$ sec for a case with no added mass.

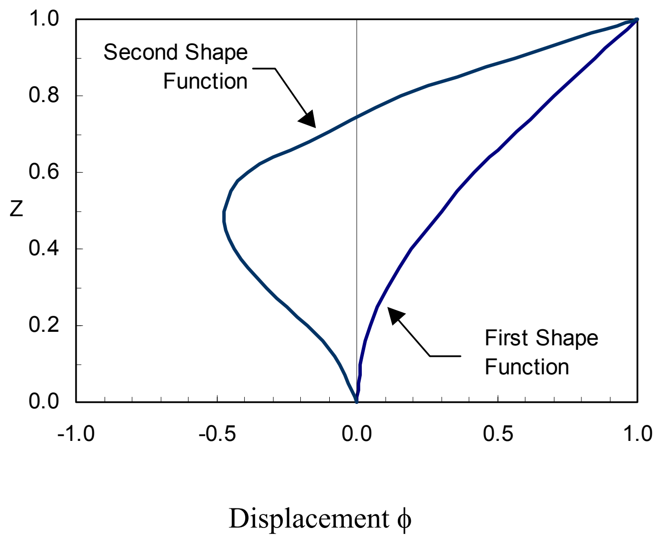
(a) Since the example tower carries added mass, the actual calculations for the natural periods must be made using calculated values for the stiffness k^* and the actual effective mass m^*. The stiffness k^* is independent of mass and is determined from the coefficients given in Tables B-1 and B-2. The coefficients of k^* are interpolated for a value of $(I_{\mathrm{BASE}}/I_{\mathrm{TOP}}) = 3.361$.



$$I_{BASE}/I_{TOP} = 3.361$$

Longitudinal Directon

Z	ϕ1	ϕ2
1.00	1.000	1.000
0.99	0.985	0.956
0.90	0.852	0.559
0.80	0.704	0.146
0.68	0.538	-0.191
0.60	0.428	-0.415
0.50	0.306	-0.504
0.40	0.201	-0.470
0.30	0.116	-0.348
0.20	0.053	-0.192
0.12	0.022	-0.085
0.03	0.006	-0.021
0.00	0.000	0.000



$$I_{BASE}/I_{TOP} = 4.170$$

Transverse Direction

Z	ϕ1	ϕ2
1.00	1.000	1.000
0.99	0.985	0.956
0.90	0.849	0.564
0.80	0.700	0.158
0.68	0.533	-0.170
0.60	0.421	-0.388
0.50	0.300	-0.472
0.40	0.196	-0.438
0.30	0.112	-0.322
0.20	0.051	-0.177
0.12	0.021	-0.078
0.03	0.005	-0.020
0.00	0.000	0.000

Figure C-10. Approximate shape functions

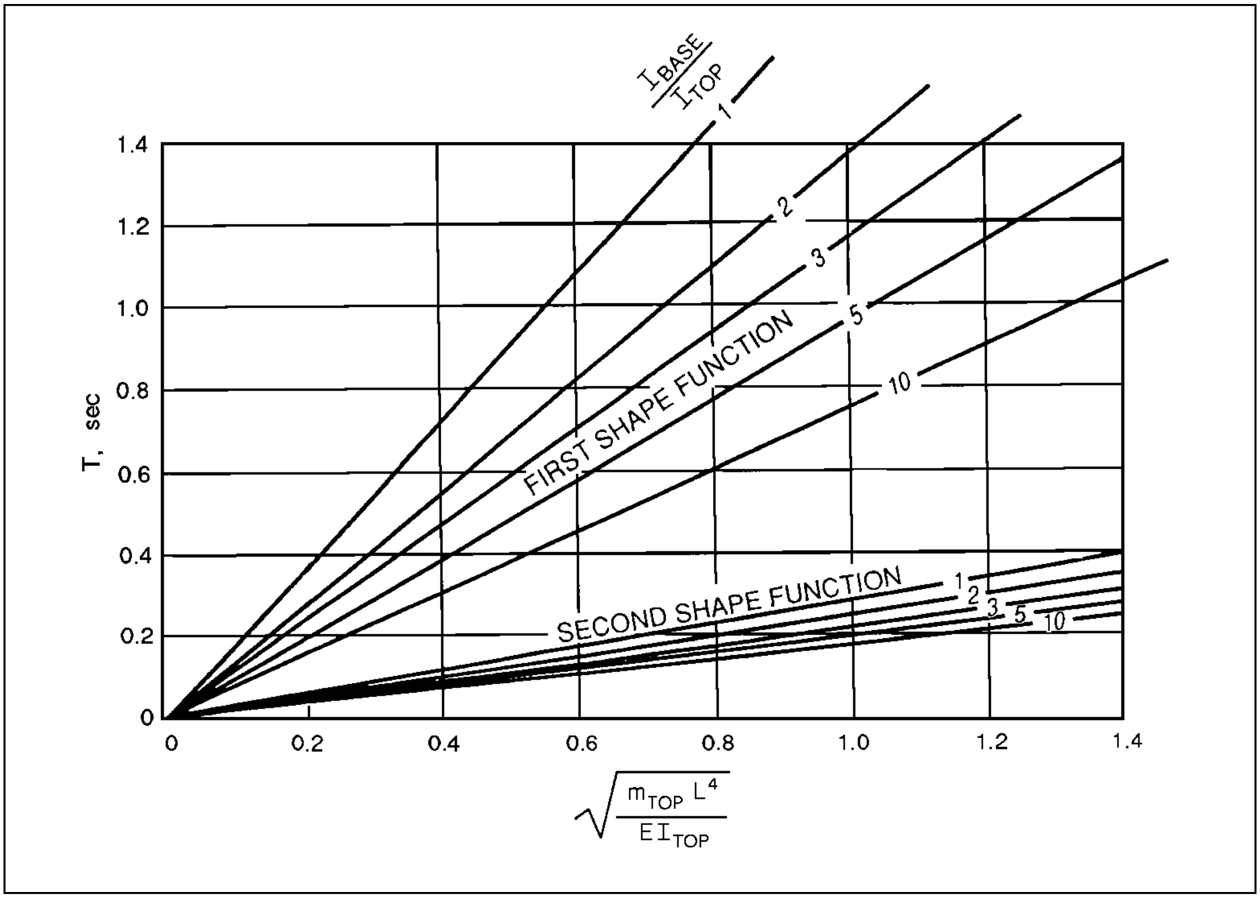


Figure C-11. Approximate natural periods of intake towers having no added mass

- For the first shape function:

$$k^* = 8.042 \frac{EI_{\mathrm{TOP}}}{L^3}$$

$$= \frac{8.042 \times 21{,}525{,}000 \times 626 \times 0.8}{60.69^3}$$

$$= 387{,}811 \text{ kN/m} \,(26{,}566 \text{ kips/ft})$$

- For the second shape function:

$$k^* = 149.97 \frac{EI_{\mathrm{TOP}}}{L^3} = 7{,}232{,}034 \,\text{kN/m} \,(495{,}413 \,\text{kips/ft})$$

The effective masses m^* and the normalization ratio L_n/m^* are computed using the total lumped mass $M(z)$ at each discretized point.

(b) The values of L_n and m* are defined mathematically as

$$L_n = \Sigma M(z)\phi(z)$$

$$m^* = \Sigma M(z)\phi(z)^2$$

where z is the height of the lumped mass $M(z)$ above the base and $\phi(z)$ is shape function, as defined in Figure C-10. Tables C-8 and C-9 provide the values of L_n and m^* for the two shape functions. Values of $\phi(z)$ are obtained from the values given in Figure C-10.

Table C-8
***L_n, M^**, Forces, Shears, and Moments for Longitudinal MDE Excitation, First Shape Function**

Node #	Height h_j m (ft)	Mode Shape ϕ_{j1}	Lumped Mass m_j kN-s^2/m (k-s^2/ft)	ϕ_{j1} X m_j kN-s^2/m (k-s^2/ft)	ϕ_{j1}^2X m_j kN-s^2/m (k-s^2/ft)	Lateral Displacement ux_{j1} mm (in)	Elastic Force fx_{j1} kN (kips)	Shear Vx_{j1} kN (kips)	h_j X $f_{j,1}$ kN-m (k-ft)	Moment My_{j1} kN-m (k-ft)
13	60.96 (200)	1.000	86.72 (5.94)	86.724 (5.94)	86.724 (5.94)	56.656 (2.23)	1,114 (251)	1,114 (251)	67926.486 (50152.41)	0 (0)
12	60.35 (198)	0.985	264.93 (18.16)	261.008 (17.89)	257.145 (17.62)	55.817 (2.20)	3,354 (755)	4,468 (1,005)	202390.398 (149431.63)	679 (502)
11	54.56 (179)	0.852	356.41 (24.43)	303.662 (20.81)	258.720 (17.73)	48.271 (1.90)	3,902 (878)	8,369 (1,884)	212869.669 (157168.83)	26,554 (19,605)
10	48.77 (160)	0.704	520.75 (35.69)	366.608 (25.13)	258.092 (17.69)	39.886 (1.57)	4,710 (1060)	13,080 (2,944)	229717.068 (169607.83)	75,023 (55,392)
9	41.45 (136)	0.538	911.98 (62.51)	491.008 (33.65)	264.359 (18.12)	30.503 (1.20)	6,309 (1420)	19,389 (4,363)	261516.277 (193086.25)	170,705 (126,037)
8	36.58 (120)	0.428	1526.57 (104.63)	653.371 (44.78)	279.643 (19.17)	24.249 (0.96)	8,395 (1889)	27,784 (6,253)	307051.911 (226706.74)	265,260 (195,850)
7	30.48 (100)	0.306	1952.82 (133.85)	597.562 (40.96)	182.854 (12.53)	17.337 (0.68)	7,678 (1728)	35,461 (7,980)	234020.543 (172785.23)	434,628 (320,901)
6	24.38 (80)	0.201	2110.27 (144.64)	424.164 (29.07)	85.257 (5.84)	11.388 (0.45)	5,450 (1226)	40,911 (9,207)	132890.686 (98117.66)	650,801 (480,508)
5	18.29 (60)	0.116	2236.77 (153.31)	259.466 (17.78)	30.098 (2.06)	6.572 (0.26)	3,334 (750)	44,245 (9,957)	60968.020 (45014.74)	900,197 (664,645)
4	12.19 (40)	0.053	2168.97 (148.66)	114.956 (7.88)	6.093 (0.42)	3.003 (0.12)	1,477 (332)	45,722 (10,289)	18007.814 (13295.77)	1,169,915 (863,787)
3	7.01 (23)	0.022	2094.71 (143.57)	46.084 (3.16)	1.014 (0.07)	1.246 (0.05)	592 (133)	46,314 (10,423)	4150.930 (3064.77)	1,406,829 (1,038,709)
2	1.83 (6)	0.006	1628.41 (111.61)	8.956 (0.61)	0.049 (0.00)	0.312 (0.01)	115 (26)	46,429 (10,449)	210.450 (155.38)	1,646,810 (1,215,895)
1	0.00 (0)	0.000	580.47 (39.79)	0.000 (0.00)	0.000 (0.00)	0.000 (0.00)	0 (0)	46,429 (10,449)	0.000 (0.00)	1,731,720 (1,278,587)

NOTES:

$L_1 = \Sigma\phi_{j1} X m_j =$ 247.676 (k-s^2/ft) 3613.567 (kN-s^2/m)

$M_1 = \Sigma\phi_{j1}^2 X m_j =$ 117.208 (k-s^2/ft) 1710.047 (kN-s^2/m)

$L_1 / M_1 =$ 2.113

$\omega_1 = 2\pi / T_1 =$ 15.059 rad/sec ($T_1 =$ 0.42 sec)

S_a(MDE) = 0.620 g = 19.96 ft/sec^2 6.080 m/sec^2

Table C-9
L_n*, *m*, **Forces, Shears, and Moments for Longitudinal MDE Excitation, Second Shape Function**

Node #	Height h_j m (ft)	Mode Shape ϕ_{j2}	Lumped Mass m_j kN-s^2/m (k-s^2/ft)	ϕ_{j2} X m_j kN-s^2/m (k-s^2/ft)	ϕ_{j2}^2X m_j kN-s^2/m (k-s^2/ft)	Displacement ux_{j2} mm (in)	Elastic Force fx_{j2} kN (kips)	Shear Vx_{j2} kN (kips)	h_j X $f_{j,2}$ MDE kN-m (k-ft)	Moment My_{j2} kN-m (k-ft)
13	60.96 (200)	1.000	86.72 (5.94)	86.724 (5.94)	86.724 (5.94)	-3.006 -(0.12)	-908 -(204)	-908 -(204)	-55359.368 -(40873.68)	0 (0)
12	60.35 (198)	0.956	264.93 (18.16)	253.246 (17.36)	242.077 (16.59)	-2.874 -(0.11)	-2,652 -(597)	-3,560 -(801)	-160040.515 -(118163.29)	-554 -(409)
11	54.56 (179)	0.559	356.41 (24.43)	199.233 (13.66)	111.371 (7.63)	-1.680 -(0.07)	-2,086 -(470)	-5,646 -(1,271)	-113825.083 -(84040.88)	-21,170 -(15,631)
10	48.77 (160)	0.146	520.75 (35.69)	76.030 (5.21)	11.100 (0.76)	-0.439 -(0.02)	-796 -(179)	-6,442 -(1,450)	-38826.249 -(28666.72)	-53,869 -(39,773)
9	41.45 (136)	-0.191	911.98 (62.51)	-173.823 -(11.91)	33.131 (2.27)	0.573 (0.02)	1,820 (410)	-4,622 -(1,040)	75451.616 (55708.46)	-100,996 -(74,569)
8	36.58 (120)	-0.415	1526.57 (104.63)	-633.525 -(43.42)	262.913 (18.02)	1.248 (0.05)	6,634 (1493)	2,012 (453)	242643.192 (179151.62)	-123,538 -(91,212)
7	30.48 (100)	-0.504	1952.82 (133.85)	-984.220 -(67.46)	496.047 (34.00)	1.515 (0.06)	10,306 (2319)	12,318 (2,772)	314134.091 (231935.75)	-111,274 -(82,157)
6	24.38 (80)	-0.470	2110.27 (144.64)	-991.826 -(67.98)	466.158 (31.95)	1.413 (0.06)	10,386 (2337)	22,704 (5,109)	253249.340 (186982.49)	-36,184 -(26,716)
5	18.29 (60)	-0.348	2236.77 (153.31)	-778.397 -(53.35)	270.882 (18.57)	1.046 (0.04)	8,151 (1834)	30,855 (6,944)	149064.876 (110059.60)	102,219 (75,472)
4	12.19 (40)	-0.192	2168.97 (148.66)	-416.443 -(28.54)	79.957 (5.48)	0.577 (0.02)	4,361 (981)	35,216 (7,925)	53166.531 (39254.64)	290,310 (214,346)
3	7.01 (23)	-0.085	2094.71 (143.57)	-178.050 -(12.20)	15.134 (1.04)	0.256 (0.01)	1,864 (420)	37,080 (8,345)	13070.542 (9650.42)	472,783 (349,072)
2	1.83 (6)	-0.021	1628.41 (111.61)	-34.604 -(2.37)	0.735 (0.05)	0.064 (0.00)	362 (82)	37,442 (8,426)	662.669 (489.27)	664,917 (490,931)
1	0.00 (0)	0.000	580.47 (39.79)	0.000 (0.00)	0.000 (0.00)	0.000 (0.00)	0 (0)	37,442 (8,426)	0.000 (0.00)	733,392 (541,488)

NOTES:

$L_2 = \Sigma\phi_{j2} X m_j =$ -245.078 (k-s^2/ft) -3575.655 (kN-s^2/m)

$M_2 = \Sigma\phi_{j2}^2 X m_j =$ 142.306 (k-s^2/ft) 2076.230 (kN-s^2/m)

$L_2 / M_2 =$ -1.722

$\omega_2 = 2\pi / T_2 =$ 59.019 rad/sec (T_2 = 0.106 sec.)

S_a(MDE) = 0.620 g = 19.96 ft/sec^2 6.080 m/sec^2

(c) The natural periods are calculated using the values of k* and m* defined above, where :

$$T = 2\pi\sqrt{m^* / k^*}$$

- First Shape Function

$$T_1 = 2\pi\sqrt{\frac{1{,}710.047}{387{,}811}}$$
$$T_1 = 0.42 \text{ sec}$$
$$\omega_1 = 14.96 \ rad/\text{sec}$$

- Second Shape Function

$$T_2 = 2\pi\sqrt{\frac{2{,}076.23}{7{,}232{,}034}}$$
$$T_2 = 0.106 \text{ sec}$$
$$\omega_2 = 59.275 \ rad/\text{sec}$$

where ω is the natural frequency.

(d) The mass participation factors are similarly calculated using the sums defined above.

- First Shape Function

$$\frac{L_1}{m^*} = \frac{3{,}631.567}{1{,}710.047}$$
$$= 2.113$$

- Second Shape Function

$$\frac{L_2}{m^*} = \frac{-3{,}575.655}{2{,}076.230}$$
$$= -1.722$$

(3) The third step in the analysis is to determine the first and second spectral accelerations S_A. Given the two natural periods calculated in the preceding section the corresponding spectral accelerations are determined using the standard response spectrum of Figure C-3.

- First Shape Function

$T_1 = 0.42$
$S_A = 0.62\text{g} = 6.08 \text{ m/sec}^2$

- Second Shape Function

$T_2 = 0.106$
$S_A = 0.62\text{g} = 6.08 \text{ m/sec}^2$

The absolute displacement of the top mass in the longitudinal direction is computed as

$$\Delta_{TOP} = \frac{L_n}{m^*} \times \frac{S_A}{\omega^2}$$

- First Shape Function

Δ_{TOP} = 57.40 mm (2.26 in)

- Second Shape Function

Δ_{TOP} = 2.98 mm (0.12 in)

(4) The fourth step in the analysis is to determine the forces, shears, and moments at the level of each lumped mass. The forces acting on each mass consist of longitudinal and transverse forces from each of the two shape functions, producing four sets of forces, shears, and moments. Tables C-8 and C-9 summarize the computations of these values for the longitudinal direction. The lateral force F_n acting on any mass n is computed by:

$$F_n = (L_n/m^*)\ S_{An}\ M_n\ \phi_n$$

The shear at any level n is the sum of forces F_n above that level. The moment at any level n is given by

$$V_n = \sum_{i=n}^{L} F_i$$

$$M_n = M_{n+1} + V_{n+1} \times \Delta h$$

where Δh = height of mass M_{n+1} - height of mass M_n

(5) The fifth step in the analysis is to combine the shears and moments for the two shape functions into a single resultant. In this example, the combination is computed using the square-root-of-the-sum-of-the-squares (SRSS) method. The absolute displacement of the top mass, for the combined first and second shape functions is

Δ_{TOP} = 57.49 mm (2.27 in)

The base shears V_b and base moments M_b are, for the first and second shape functions combined

V_b = 59,646 kN (13,423 kips)
M_b = 1,880,616 kN-m (1,388,522 kip-ft)

C-8. Comparison of Results From Two-mode Approximate Method With Those From Intake Tower Analysis Program (ITAP)

a. Table C-10 compares results for the MDE longitudinal excitation from the two-mode approximate procedure with those obtained from the Intake Tower Analysis Program (Quest Structures 2000). The ITAP model of the example tower consisted of 12 beam elements with shear deformation, as shown in Figure C-2. The first 10 modes of vibration and 5 percent modal damping were used to compute the dynamic response. Some of the unique capabilities of the program ITAP developed by QUEST Structures for Structures Laboratory at the U.S. Army Engineer Research and Development Center include the following:

Table C-10
Approximate 2-Mode Method versus Intake Tower Analysis Program (ITAP) -- MDE Longitudinal Excitation

Item	2-Mode Approximate Method	Multi-mode Computer Solution
Period of first mode of vibration	0.42 sec	0.45 sec
Period of second mode of vibration	0.106 sec	0.134 sec
Top displacement	57.49 mm (2.27 in)	62.43 mm (2.46 in)
Base shear	59,646 kN (13,423 kips)	57,855 kN (13,020 kips)
Base moment	1,880,616 kN-m (1,388,522 kip-ft)	1,783,747 kN-m (1,317,000 kip-ft)

- Automatic computation of section properties for solid and hollow sections from limited input geometry data.
- Automatic evaluation of the inside and outside added-mass of water from input water levels.
- Static, response spectrum, and time history response analyses.
- Pre- and post-processor capabilities with graphical presentation of the model and results.

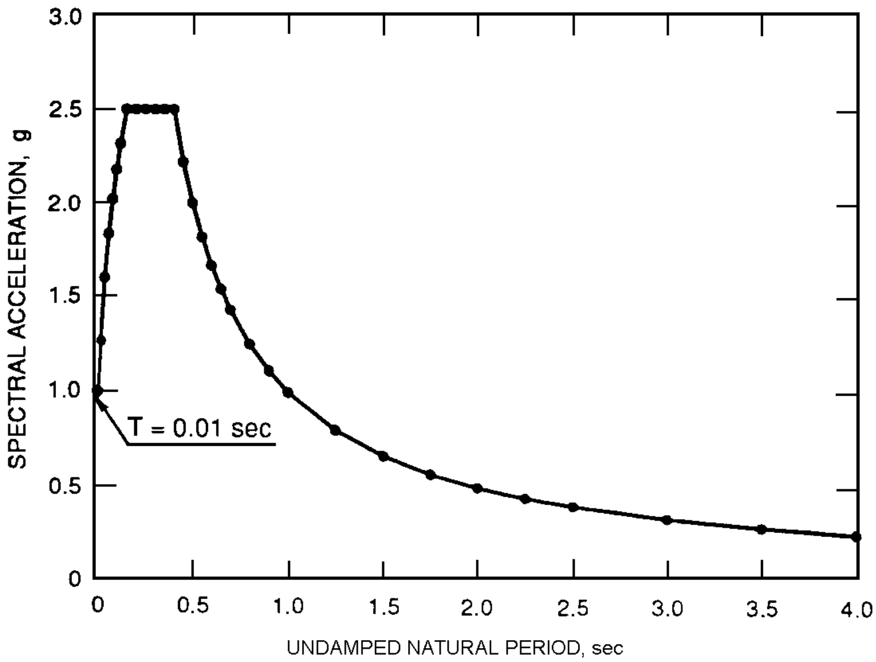
These unique capabilities listed above make ITAP a more cost-effective and efficient program. It runs with simple input data requiring the least amount of input data, yet automatically produces all pertinent results needed for performance evaluation of the tower. Generally, input for computer analysis includes joint designations and coordinates with member designations and properties. In addition, joint masses and the seismic input in the form of response spectrum (Figure C-12) or acceleration time-histories are needed for the evaluation of earthquake response. Input to ITAP for the longitudinal MDE Excitation is shown in Figure C-13.

b. The results from the two-mode approximate procedure are generally in good agreement with the results from the computer solution. The periods and displacements for the approximate method are smaller than those for the computer solution, but the base shear and bending moment are slightly higher (Table C-10). The complete results for the MDE and OBE excitations (Figures C-14 and C-15) show slightly higher base shear and moments for the approximate method than the computer solution, but displacements may be higher or lower. Overall, the two-mode approximation provides reasonable results for the example tower. In other situations, with significant irregularities, a computer solution should be considered for the final design.

c. The computer analysis results for both directions of earthquake ground motion and for both the MDE and OBE are presented in Tables C-11 and C-12. These results will be used to demonstrate the methods for calculating reinforcing steel requirements and for investigating all potential modes of failure.

C-9. Intake Tower Reinforced Concrete Design - General

a. According to ER 1110-2-1806, hydraulic structures should have adequate earthquake resistance to sustain the MDE without collapsing and to survive the OBE with only cosmetic damage. It is generally uneconomical to design intake towers to remain elastic during the MDE, so the structure must dissipate energy through postelastic deformation. The structure is therefore designed to undergo inelastic deformation under MDE but remain elastic under OBE, whichever controls.



SPECTRAL ACCELERATION VERSUS PERIOD (5% DAMPED)

T, sec	S_A, g (5% DAMPED)
0.002	1.0000
0.005	1.0000
0.008	1.0000
0.010	1.0000
0.020	1.2643
0.040	1.5985
0.060	1.8335
0.080	2.0210
0.100	2.1795
0.120	2.3182
0.150	2.5000
0.200	2.5000
0.250	2.5000
0.300	2.5000
0.350	2.5000
0.400	2.5000
0.450	2.2222
0.500	2.0000
0.550	1.8182
0.600	1.6667
0.650	1.5385
0.700	1.4286
0.800	1.2500
0.900	1.1111
1.000	1.0000
1.250	0.8000
1.500	0.6667
1.750	0.5714
2.000	0.5000
2.250	0.4444
2.500	0.4000
3.000	0.3333
3.500	0.2857
4.000	0.2500

Figure C-12. Response spectrum seismic input for computer analysis

b. Intake structures will undergo failure mechanisms similar to cantilever shear walls and cantilever bridge piers. The general procedures and design philosophy outlined in Pauley (1980) and Federal Highway Administration (1995) are usually applicable to the design of intake towers. The assumptions made in this manual are that intake tower walls, unless specially designed with ductility in mind, will be structures with limited ductility. Ductility can be enhanced with confinement reinforcement and specially reinforced boundary elements. However, the guidance provided herein pertains to structures without special ductility enhancement, which includes all existing intake towers designed and constructed by the Corps to date.

c. Structures with limited ductility will have displacement ductility ratios between 1.5 and 3.5. A displacement ductility of 2.5 will provide energy dissipation equivalent to a fully elastic structure designed for a moment reduction factor of two (Figure C-16). The structure, however, can undergo repeated cycles of inelastic displacement only if brittle modes of failure do not occur. The seismic design approach for intake towers is therefore to

(1) Permit limited inelastic bending displacements to occur under the MDE to reduce seismic bending moment demands and bending reinforcing steel requirements.

```
$ ITAP (Intake Tower Analysis Program, developed by QUEST Structures)
$ Intake Tower Example Problem
$ Earthquake in X-direction (longitudinal)
$ Maximum Design Earthquake (MDE)
$ Effective stiffness equal to 0.8 gross stiffness
$ Metric Units (kN-m)
$ Title Card
 TOWER WITH APPROXIMATE MASS
$
$ Master Control Card
$ NumElms NumMode lCases iShea WatDen  WatEo   WatEi  Gravity  Mass Sec
    12      20      1      1    9.807 41.4528 41.4528   9.807         4
$
$ Nodal Point and Element Data
$ Elm  Z-bot   Z-top   SecID MatID
  1   0.0      1.8288    1     1
  2   1.8288   7.0104    2     1
  3   7.0104   12.192    2     1
  4   12.192   18.288    3     1
  5   18.288   24.384    3     1
  6   24.384   30.48     4     1
  7   30.48    36.576    4     1
  8   36.576   41.4528   5     1
  9   41.4528  48.768    5     1
 10   48.768   54.5592   6     1
 11   54.5592  60.3504   6     1
 12   60.3504  60.96     7     1
$
$ Material Properties
$ iMatID       E          nu   weight density
    1     0.2153E+08     0.2     0.2400E+02
$
$ Section Properties
$ iSecID iSecType   a       b       c     d         shear coeffs
stiff.Red
    1       1    14.6304 14.6304              0.666651093 0.666651093  0.8
    2      11    14.6304 11.2776 10.9728 7.62 0.505525540 0.394891945  0.8
    3      11    14.3256 10.6680 11.2776 7.62 0.508635682 0.386206897  0.8
    4      11    14.0208 10.0584 11.5824 7.62 0.512310606 0.377518939  0.8
    5      11    13.7160 9.44880 11.8872 7.62 0.516349125 0.369078270  0.8
    6      11    13.4112 8.83920 12.1920 7.62 0.521940764 0.360950896  0.8
    7       1    13.4112 8.83920              0.666686351 0.666686351  0.8
$
$ Type of Load Case
$(0:eigen, 1:gravity, 2:hydrostatic, 3:static, 4:RSA, 5:TH)
                                                     4
$
$ Response Spectrum Analysis
$ Spectrum Heading
Maximum Design Earthquake (MDE) with PGA=0.62g
$
$  npts   FX     FY     FZ  0=Displ,  1=Acc. vs. period
    34   9.807  0.00  0.00            1
```

Figure C-13. Input to ITAP for the longitudinal MDE excitation (Continued)

```
$
$ Period      X-Spec
  0.0010      0.2517
  0.0020      0.2554
  0.0050      0.2666
  0.0080      0.2778
  0.0100      0.2852
  0.0200      0.3224
  0.0400      0.3968
  0.0600      0.4712
  0.0800      0.5456
  0.1000      0.6200
  0.1200      0.6200
  0.1500      0.6200
  0.2500      0.6200
  0.3000      0.6200
  0.3500      0.6200
  0.4000      0.6200
  0.4500      0.6200
  0.5000      0.6200
  0.5500      0.5636
  0.6000      0.5167
  0.6500      0.4769
  0.7000      0.4429
  0.8000      0.3875
  0.9000      0.3444
  1.0000      0.3100
  1.2500      0.2480
  1.5000      0.2067
  1.7500      0.1771
  2.0000      0.1550
  2.2500      0.1378
  2.5000      0.1240
  3.0000      0.1033
  3.5000      0.0886
  4.0000      0.0775
$
$ Load Combinations for various load cases
$ Number of Combinations
  0
$ Combination Factors
$ When several load cases are considered in a single computer run, they can be
$ combined by assigning a none-zero load factor here
```

Figure C-13. (Concluded)

(2) Prevent shear failure by providing strengths equal to the full elastic demand of the MDE.

(3) Check other potential brittle modes of failure.

d. A moment reduction factor R_M has been developed for intake tower sections to represent the acceptable ratio of seismic moment that could develop in the tower if the structure behaved entirely elastically under the ground motion associated with the MDE to the prescribed design seismic moment considered acceptable if some limited inelastic displacement is permitted.

e. The design calculations for the example intake tower illustrate the approach used for new towers without special ductility reinforcement. The design will investigate the conditions necessary to prevent the following failure mechanisms from occurring during the MDE:

Table C-11
Maximum Design Earthquake Results

Item	Longitudinal Direction	Transverse Direction
Period of first mode of vibration	0.45 sec	0.58 sec
Period of second mode of vibration	0.134 sec	0.18 sec
Top displacement	62.43 mm (2.46 in)	88.44 m (3.48 in)
Base shear	57,855 kN (13,020 kips)	57,233 kN (12,880 kips)
Base moment	1,783,747 kN-m (1,317,000 kip-ft)	1,925,959 kN-m (1,422,000 kip-ft)

Table C-12
Operational Basis Earthquake Results

Item	Longitudinal Direction	Transverse Direction
Period of first mode of vibration	0.406 sec	0.529 sec
Period of second mode of vibration	0.126 sec	0.165 sec
Top displacement	30.21 mm (1.19 in)	42.80 mm (1.69 in)
Base shear	27,995 kN (6,300 kips)	27,701 kN (6,234 kips)
Base moment	863,025 kN-m (637,200 kip-ft)	931,693 kN-m (687,900 kip-ft)

(1) Moment failure (flexure).

(2) Shear failure (diagonal tension).

(3) Sliding shear failure.

(4) Rebar splice failure.

(5) Rebar anchorage failure.

(6) Compressive spalling failure.

(7) Fracture of tensile reinforcement (minimum tensile reinforcement requirements).

f. The example problem inertia forces, shears, and moments from the ITAP lumped-mass computer solution are used as the basis for design. The section at the base of the tower, where maximum shears and moments occur, is used to illustrate the design process.

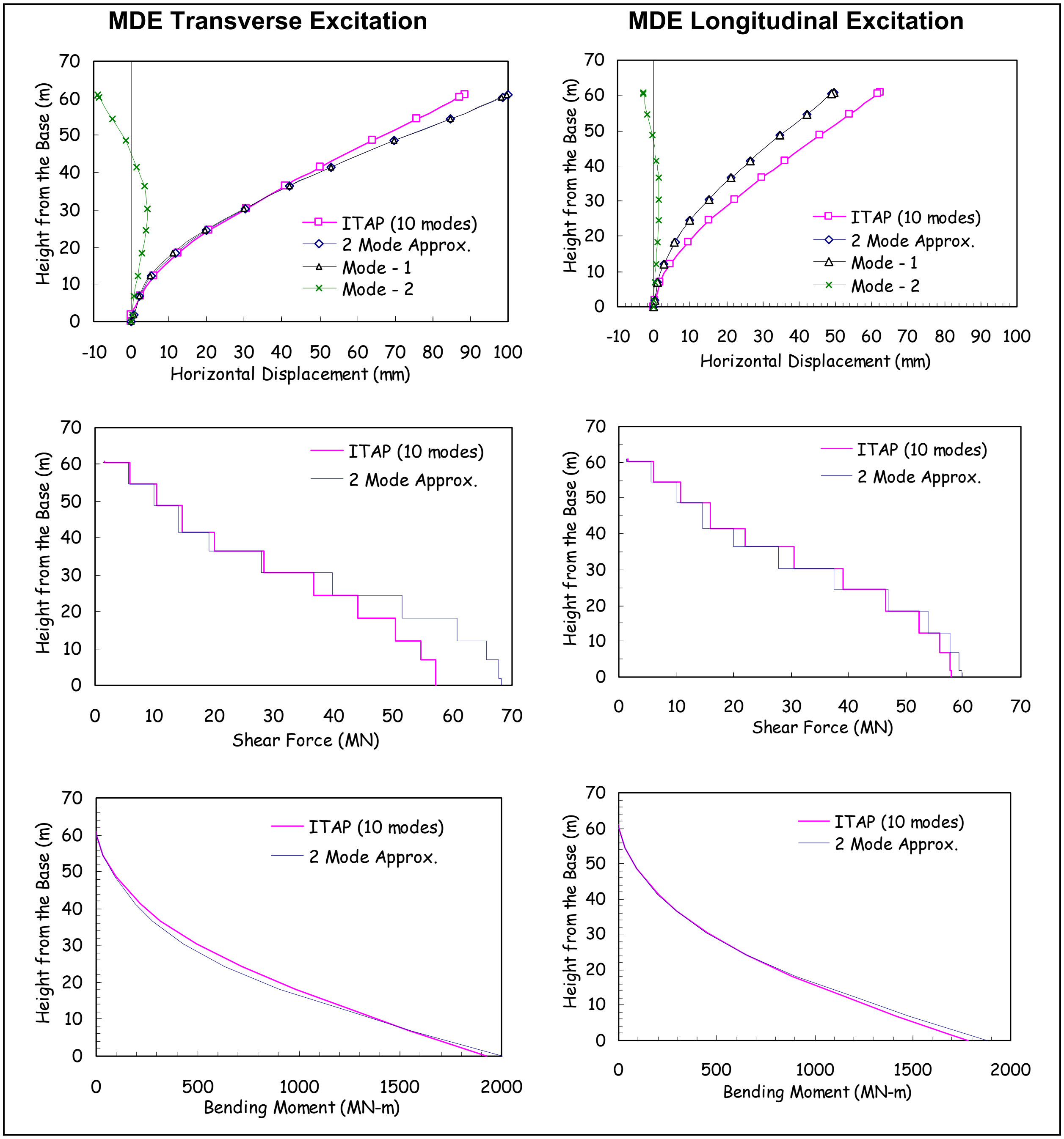


Figure C-14. Comparison of MDE results for the two-mode approximate procedure and computer dynamic analysis

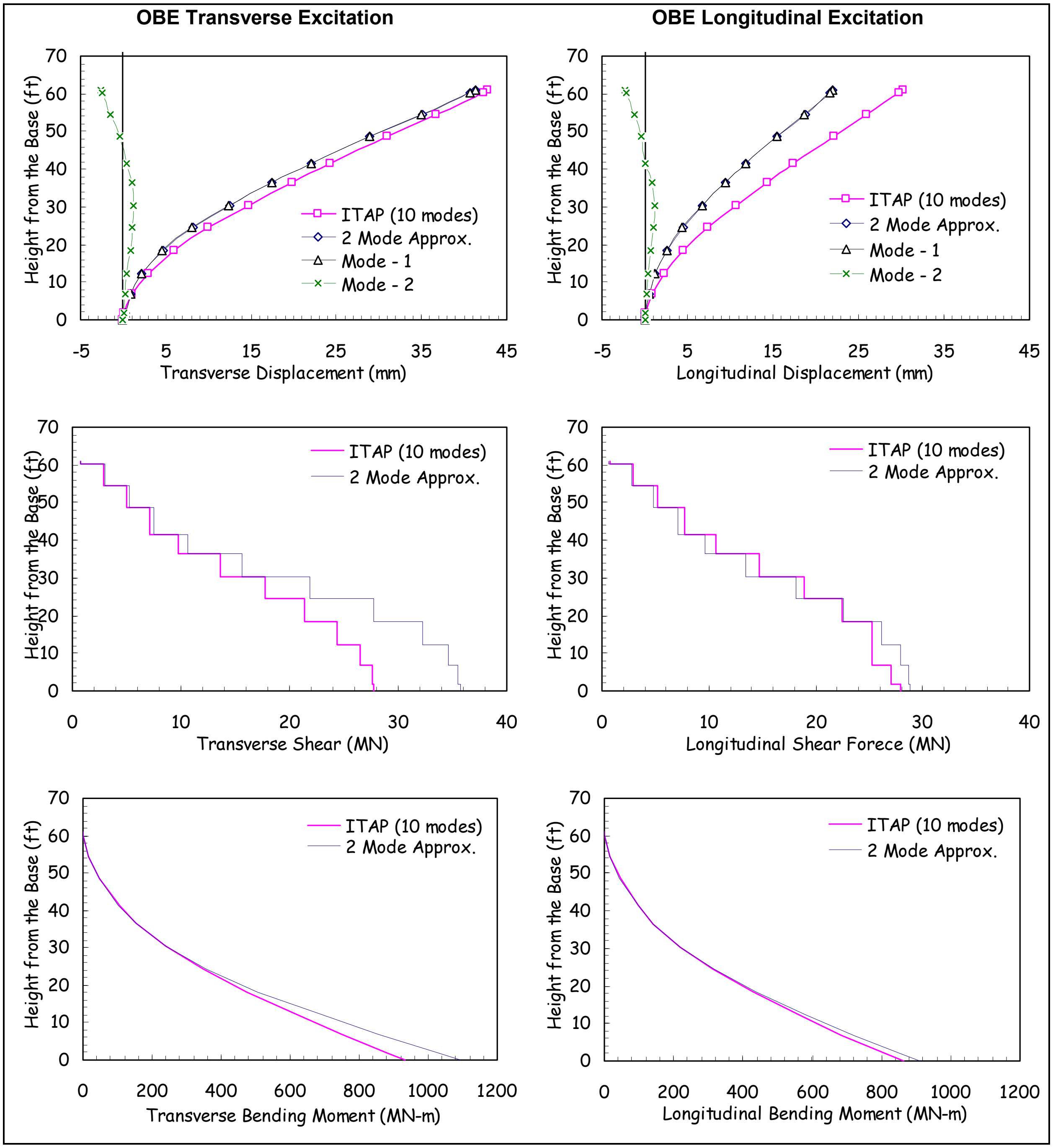
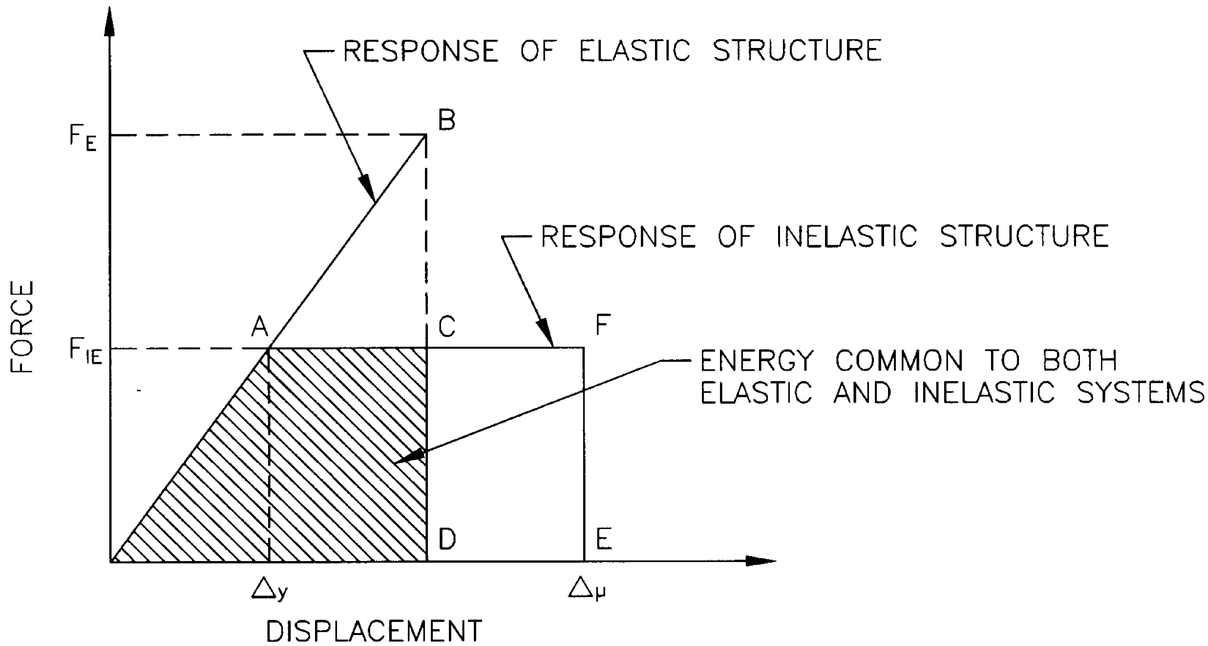


Figure C-15. Comparison of OBE results for the two-mode approximate procedure and computer dynamic analysis



μ = displacement ductility factor
R_M = moment reduction factor
F_E = force experienced by elastic structure
F_{IE} = force experienced by inelastic structure
Δy = displacement of inelastic structure at yield
$\Delta \mu$ = displacement of inelastic structure at ultimate

Assuming an equal maximum potential energy response for the elastic and inelastic structures, the area ABC must equal the area CDEF.

With ductility equal to $\Delta \mu / \Delta y$ and
the moment reduction factor equal to F_E/F_{IE}
then: $R_M = \sqrt{2\mu - 1}$
A μ factor of 2.5 will provide a R_M factor of 2.0

Figure C-16. Relationship between displacement ductility and moment reduction factor using equal maximum potential energy response

C-10. Procedures for Seismic Design of Intake Towers

a. In most cases intake towers are subjected to the gravity, hydrostatic, and earthquake loading. Since the tower represents a cantilever structure, the applied loads are balanced by reaction moment (M_u), shear (V_u), and axial force (P_u), as shown in Figure C-17. The basic objective of the design is to ensure that the safety and serviceability of the structure under these loads are maintained.

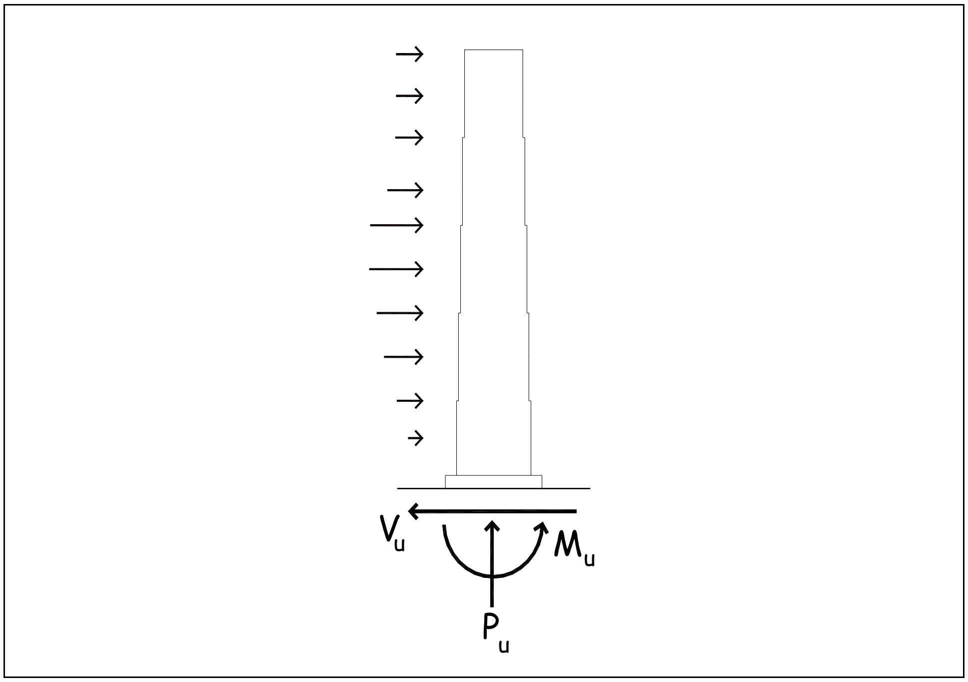


Figure C-17. Loads acting on tower during an earthquake

b. Design provisions outlined in this example are applicable to structures with capacities well below the balanced point of the axial-force bending-moment interaction curve (Figure C-18). Since the vertical load in towers is generally the result of self-weight, which is not very large, most intake towers will have capacities well below the balanced point.

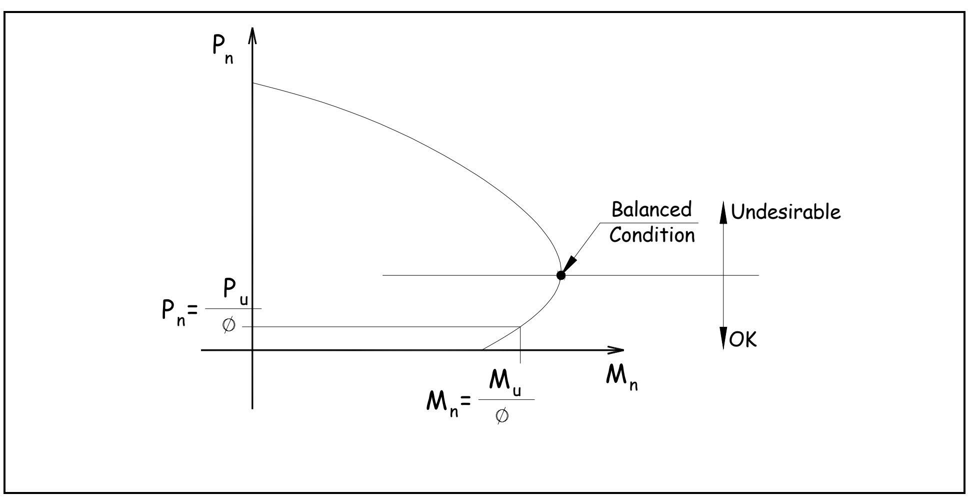


Figure C-18. Axial force and bending moment interaction diagram

c. The basic seismic design of the reinforced concrete intake towers should follow the following steps:

(1) Select desirable material properties (usually known or assumed).

(2) Select overall geometry of the tower cross sections (usually known before the design).

(3) Determine demands M_u, P_u, and V_u (from the analysis) and apply appropriate load factors.

(4) Determine the approximate amount of reinforcement steel.

(5) Perform shear design with $\phi = 0.85$ to obtain V_n.

(6) Perform bending design including vertical load to obtain M_n.

(7) Check ultimate-state failure modes and redo shear design if necessary:

- Tower will fail in bending. Since the bending is a ductile failure mode, the design is OK if the ductility requirement is met and the shear strength is not exceeded.

- Tower will fail in shear. Since the shear is a brittle failure, the failure under shear should be avoided. This is done by increasing shear strength of the tower by redoing shear design with $\phi =$ 0.60.

(8) Perform sliding shear failure check.

(9) Check anchorage for the moment reinforcing bars at the base.

(10) Design reinforcing bar splices.

(11) Perform comprehensive spalling check.

(12) Perform minimum tensile reinforcement requirements check.

(13) Sketch final reinforcement arrangement and detailing.

(14) Repeat the design for other cross sections.

Note: since Microsoft Excel spreadsheet was used in developing this design example, the numbers given may not exactly match the numbers obtained by hand calculations. This is because hand calculations usually consider fewer decimal digits than the spreadsheet.

C-11. Material Properties

The material properties assumed in this example are summarized in Table C-13.

Table C-13
Assumed Material Properties

Parameter		Value	
		Non-SI Units	Metric Units
Re-bar Material Properties			
Modulus of Elasticity	(E_s)	29,000.00 ksi	199,947.95 MPa
Nominal Yield Strength	(f_y)	60.000 ksi	413.69 MPa
Strain Hardening		0.80 %	0.80 %
Steel Ultimate Stress		75.000 ksi	517.11 MPa
Ultimate Strain		5.00 %	5.00 %
Concrete Material Properties			
Modulus of Elasticity	(E_c)	3,122.00 ksi	21,525.43 MPa
Shear Modulus	(G)	1,300.83 ksi	8,986.93 MPa
Poisson's Ratio	(ν)	0.20	0.20
Compressive Strength	(f'_c)	3.00 ksi	20.68 MPa
Modulus of Rupture	(F_r)	0.41 ksi	2.82 MPa
Concrete Ultimate Strain	(e_c)	0.30 %	0.30 %

C-12. Section Properties

In order to perform a design, the overall dimensions of the structure should be known. In most cases geometry of the tower is known prior to the design process. Figure C-19 shows dimensions of the most critical or bottom section of the tower, where the largest bending moment, shear, and axial forces occur. The critical section can also be determined on the basis of the demand-capacity ratios, if approximate capacities of the sections are known.

C-13. Determination of Nominal Loading Capacity

The computed moment and force demands (Table C-14) after application of load factors should be less than or equal to the factored uni-axial nominal capacities:

$$M_u \leq \phi \cdot M_n$$
$$V_u \leq \phi \cdot V_n$$
$$P_u \leq \phi \cdot P_n$$

From Table C-15 since:

$$P_u = Both\,(52{,}422.00)\; and\; (73{,}391) < 0.1 \cdot A_g \cdot f_c' =$$
$$0.1 \cdot (14.63 \times 11.28 - 7.62 \times 10.97) \times 20{,}684.00 =$$
$$168{,}440.00$$

where A_g is the gross section area and f_c' is the concrete compression strength, the bending moment strength reduction factor of $\phi = 0.90$ applies. Otherwise a $\phi = 0.70$ should be employed in accordance with American Concrete Institute (ACI) 318-95, sec. 9.3.2.2.

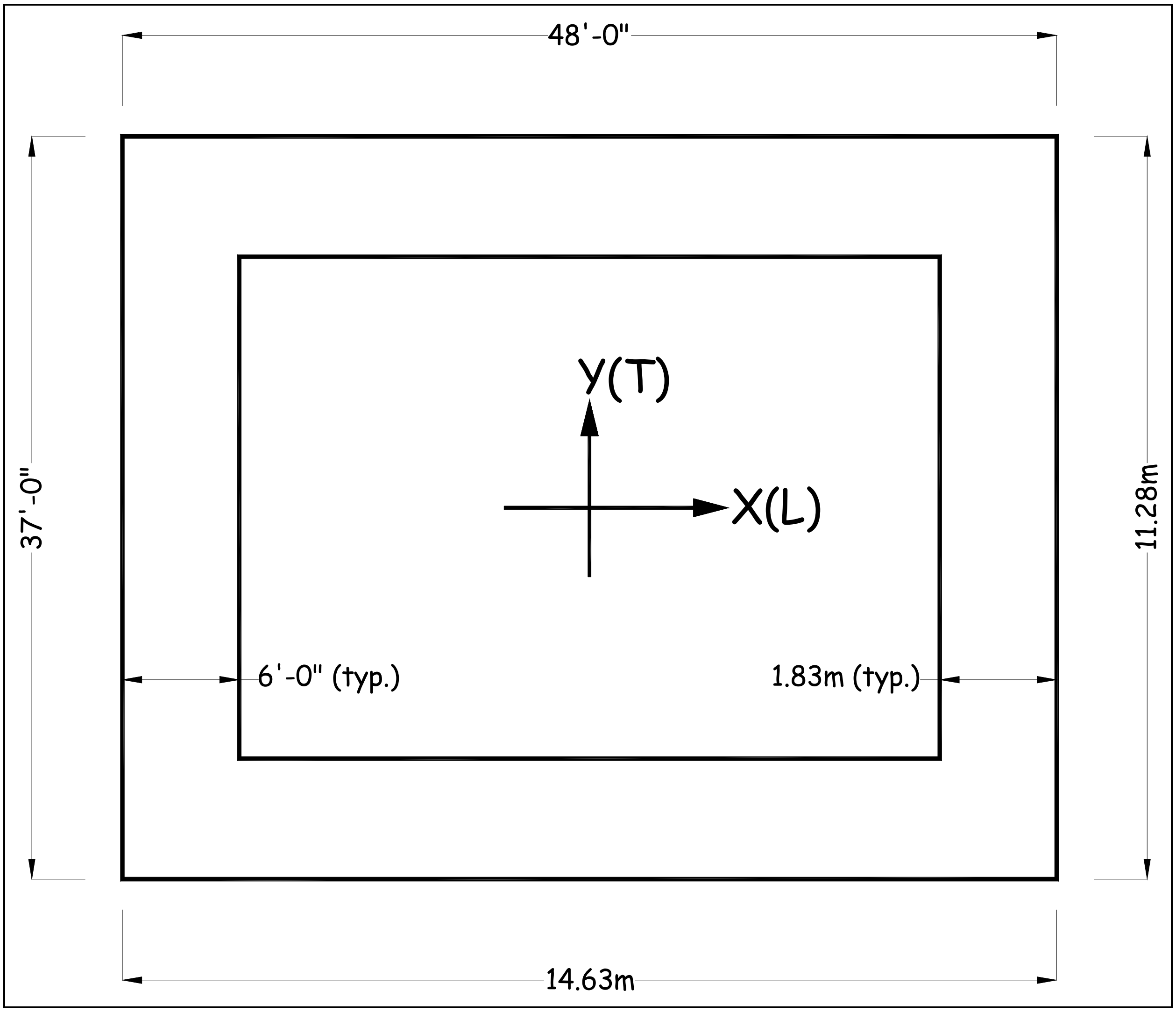


Figure C-19. Overall dimensions of critical section (section-1)

Table C-14
Summary of Computed Maximum Loads and Applicable Load Factors

Calculated Loads from Analyses		Longitudinal Direction		Transverse Direction		Load Factor Equations 4-4 & 4-5
			MDE			
Bending Moment	M	1,783,747 kN-m	(1,317,000 k-ft)	1,925,959 kN-m	(1,422,000 k-ft)	1.1
Shear Force	V	57,855 kN	(13,020 kips)	57,233 kN	(12,880 kips)	1.1
Axial Load	P	52,422 kN	(11,784 kips)	52,422 kN	(11,784 kips)	1
			OBE			
Bending Moment	M	863,025 kN-m	(637,200 k-ft)	931,693 kN-m	(687,900 k-ft)	1.5
Shear Force	V	27,995 kN	(6,300 kips)	27,701 kN	(6,234 kips)	1.5
Axial Load	P	52,422 kN	(11,784 kips)	52,422 kN	(11,784 kips)	1.4

Table C-15
Summary of Factored Maximum Loads and Preliminary Strength Reduction Factors

Factored Loads from Table C-14		Longitudinal Direction		Transverse Direction		Strength Reduction Factors (ACI 318-95 9.3.2)
MDE						
Bending Moment	M_u	1,962,122 kN-m	(1,448,700 k-ft)	2,118,555 kN-m	(1,564,200 k-ft)	0.9
Shear Force	V_u	63,641 kN	(14,322 kips)	62,956 kN	(14,168 kips)	0.85
Axial Load	P_u	52,422 kN	(11,784 kips)	52,422 kN	(11,784 kips)	0.7
OBE						
Bending Moment	M_u	1,294,538 kN-m	(955,800 k-ft)	1,397,540 kN-m	(1,031,850 k-ft)	0.9
Shear Force	V_u	41,993 kN	(9,450 kips)	41,552 kN	(9,351 kips)	0.85
Axial Load	P_u	73,391 kN	(16,498 kips)	73,391 kN	(16,498 kips)	0.7

If the tower is expected to fail in bending, the shear force strength reduction factor shall be taken as 0.85. While for the failure in shear, the shear force strength reduction factor shall be taken as 0.60 (See ACI 318-95 sec. 9.3.4.1).

C-14. Preliminary Selection of Reinforcement

a. Minimum reinforcement ratio per ACI 318-95 (sec. 11.10.9) is

Horizontal: $\rho_{h,\min} = 0.0025$

Vertical: $\rho_{v,\min} = 0.0025$

b. Maximum vertical reinforcement should not be taken more than 75 percent of the balanced vertical reinforcement ratio ρ_b:

$$\rho_{v,\max} = 0.75 \cdot \rho_b = 0.75 \times (0.0214\,) = 0.0160$$

Where ρ_b is given by ACI 318-95 sec. 8.4.3 in psi units:

$$\rho_b = \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87{,}000}{87{,}000 + f_y} \right) =$$

$$\frac{0.85 \times (0.85) \times (3{,}000)}{60{,}000} \cdot \left(\frac{87{,}000}{87{,}000 + 60{,}000} \right) = 0.0214$$

in which $\beta_1 = 0.85$ for concrete with $f'_c = 3{,}000$ psi

and f_y is the yield stress of the longitudinal reinforcement

c. According to the calculated reinforcement ratio limits, and from statistical information (Dove 1998) on existing towers it is reasonable to assume a reinforcement ratio of

$$\rho_a = 0.0035$$

d. The total area of the assumed reinforcement (A_s) will be:

$$A_s = 0.0035 \cdot A_g =$$
$$0.0035 \times (14.63 \times 11.28 - 7.62 \times 10.97) =$$
$$0.0035 \times (81.383)$$

$$A_s = 2848.40 \text{ cm}^2 \quad (441.50 \text{ in}^2)$$

e. At this point, the units for design calculations should be selected. Since the standard reinforcing bar sizes are in non-SI units, the design will be carried out in non-SI units and then converted into metric. Note that there is no guarantee that the standard non-SI-unit re-bars will be easily available in SI standards.

f. The minimum reinforcing spacing s_1 according to ACI 318-95 (sec. 11.10.9) is the minimum of $l_w/3$, $3h$ and 18 in., where l_w is horizontal length and h is thickness of the tower.

(1) Assuming reinforcing bar #11 with $A_{\#11} = 1.56 \text{ in}^2 = (10.06 \text{ cm}^2)$, the approximate required number of reinforcing bars is obtained from

$$n = A_s / A_{\#11} = 441.5 / 1.56 = 283$$

Approximately 283 reinforcing bars should be fitted into the cross section shown in Figure C-19. The reinforcing bars should be arranged in not less than two rows. The approximate spacing is calculated as

$$s = \frac{2 \cdot (48 - 1 + 48 - 1 + 37 - 1 + 37 - 1)}{283} = 1.17 \text{ ft}$$

(2) The spacing is assumed as 12 in. (approximately 30 cm).

g. Provide two layers of reinforcement close to the wall surfaces. Provide horizontal and vertical reinforcement for confining the core, usually the same amount in each direction.

h. The preliminary arrangement of the reinforcement is given in Figure C-20.

C-15. Design for Seismic Shears (Diagonal Tension)

a. *General.* The tower capacity in shear is least along the diagonal direction of earthquake attack. The shear reinforcing steel requirements can be determined using Equations 4-14 through 4-17. The shear strength shall equal, or exceed, the shear demands as represented by Equations 4-4 to 4-8 and given by

$$U_{long} = D_{long} + L_{long} \pm E_{long} \pm 0.30\, E_{trans}$$
$$U_{trans} = D_{trans} + L_{trans} \pm E_{trans} \pm 0.30\, E_{long}$$

The basic design equation according to ACI 318-95 is

$$V_u / \phi \le V_n = V_c + V_s$$

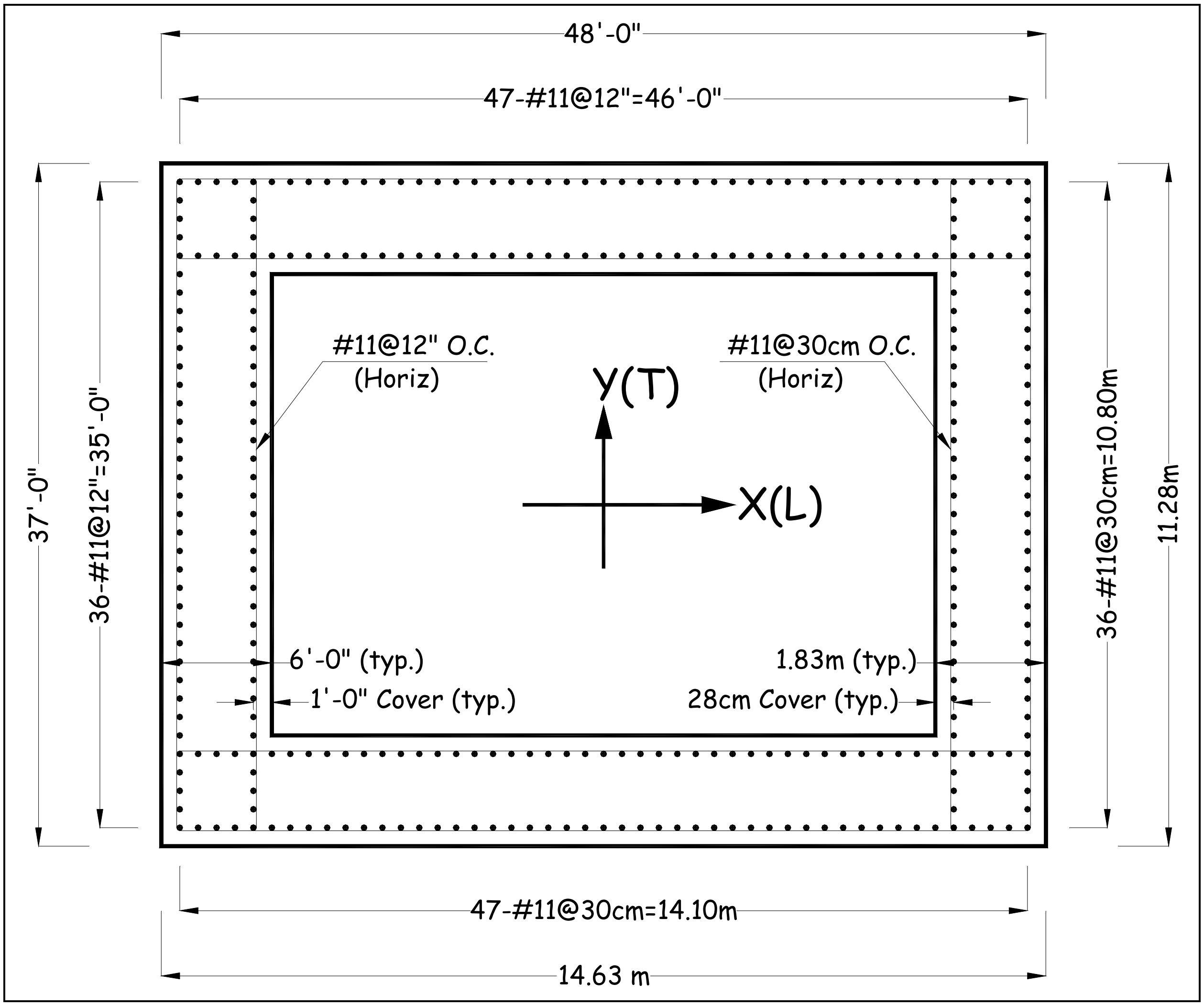


Figure C-20. Preliminary reinforcement arrangement

where

V_n = nominal shear resistance

V_c = nominal shear resistance provided by concrete

V_s = nominal shear resistance provided by shear reinforcement

b. Concrete nominal shear strength V_c. Using Equation 4-15, the nominal shear strength of concrete is calculated as follows:

(1) In MPa units:

$$V_c = 2\left(K + \frac{P_u}{13.8 \cdot A_g}\right) 0.083 \sqrt{f'_{CA}} \cdot A_E$$

$$V_c = 2\left(0.5 + \frac{(52{,}422)/1{,}000}{13.8(81.383)}\right)0.083 \times$$
$$\sqrt{1.5(20.68)}(0.8)(81.383) =$$
$$= 32.910 \text{ MN} = 32{,}910 \text{ kN}$$

(2) In psi units:

$$V_c = 2\left(K + \frac{P_u}{2{,}000 \cdot A_g}\right)\sqrt{f'_{CA}} \cdot A_E$$
$$V_c = 2\left(0.5 + \frac{11{,}784(1{,}000)}{2{,}000(876)144}\right) \times$$
$$\sqrt{1.5(3{,}000)}(0.8)(876)144 =$$
$$= 7{,}402{,}014 \text{ lb} = 7{,}402 \text{ kips}$$

c. Shear steel requirements. Nominal shear strength provided by the reinforcement is calculated as:

$$V_s = \frac{A_v \cdot f_y (0.8 \cdot d)}{s}$$

(1) In the longitudinal direction:

$$V_{s,long} = \frac{A_v \cdot f_y (0.8 \cdot d)}{s}$$
$$V_{s,long} = \frac{4(10.06)(413.69 \times 1{,}000)(0.8)(14.63)}{0.3(10{,}000)}$$
$$V_{s,long} = 64{,}975 \text{ kN } (14{,}606 \text{ kips})$$

(2) In the transverse direction:

$$V_{s,trans} = \frac{A_v \cdot f_y (0.8 \cdot d)}{s}$$
$$V_{s,trans} = \frac{4(10.06)(413.69 \times 1{,}000)(0.8)(11.28)}{0.3(10{,}000)}$$
$$V_{s,trans} = 50{,}085 \text{ kN } (11{,}259 \text{ kips})$$

d. Ultimate shear capacity.

(1) Using Equation 4-14, the ultimate shear capacity is calculated as follows:

(a) In the longitudinal direction:

$$\begin{aligned} V_{u,long} &= \phi \cdot (V_c + V_{s,long}) \\ &= 0.85(32{,}910 + 64{,}975) \\ &= 83{,}202 \text{ kN } (18{,}707 \text{ kips}) \end{aligned}$$

(b) In the transverse direction:

$$\begin{aligned} V_{u,trans} &= \phi \cdot (V_c + V_{s,trans}) \\ &= 0.85(32{,}910 + 50{,}085) \\ &= 70{,}546 \text{ kN } (15{,}862 \text{ kips}) \end{aligned}$$

(2) The multidirectional shear demands are compared to the shear capacities following a principle similar to that for Equations 4-7 and 4-8, where $\alpha = 0.3$ is used for rectangular towers:

(a) For the longitudinal direction:

$$\frac{V_{E,long}}{V_{u,long}} + 0.30 \cdot \frac{V_{E,trans}}{V_{u,trans}} \leq 1.0$$

$$\frac{63{,}641}{83{,}202} + 0.30 \frac{62{,}956}{70{,}546} = 1.033 > 1.0 \quad \Rightarrow \quad \text{NOT Okay}$$

(b) For the transverse direction:

$$\frac{V_{E,trans}}{V_{u,trans}} + 0.30 \cdot \frac{V_{E,long}}{V_{u,long}} \leq 1.0$$

$$\frac{62{,}956}{70{,}546} + 0.30 \frac{63{,}641}{83{,}202} = 1.122 > 1.0 \Rightarrow \text{NOT Okay}$$

(3) Since shear capacity is not adequate, the amount of preliminary reinforcement should be increased. This can be achieved by reducing the re-bar spacing. Set new spacing to s = 25.0 cm (10.0 in).

(a) In the longitudinal direction:

$$\begin{aligned} V_{s,long} &= \frac{4(10.06)(413.69 \times 1{,}000)(0.8)(14.63)}{0.25(10{,}000)} \\ &= 77{,}970 \text{ kN } (17{,}527 \text{ kips}) \end{aligned}$$

$$\begin{aligned} V_{u,long} &= \phi \cdot (V_c + V_{s,long}) \\ &= 0.85(32{,}910 + 77{,}970) \\ &= 94{,}248 \text{ kN } (21{,}190 \text{ kips}) \end{aligned}$$

$$\frac{63{,}641}{94{,}248} + 0.30 \frac{62{,}956}{79{,}060} = 0.914 < 1.0 \quad \Rightarrow \quad \text{Okay}$$

(b) In the transverse direction:

$$V_{s,trans} = \frac{4\,(10.06)\,(413.69 \times 1{,}000)(0.8)(11.28)}{0.25(10{,}000)}$$
$$= 60{,}102 \text{ kN } (13{,}511 \text{ kips})$$

$$V_{u,trans} = \phi \cdot (V_c + V_{s,trans})$$
$$= 0.85\,(32{,}910 + 60{,}102)$$
$$= 79{,}060 \text{ kN } (17{,}776 \text{ kips})$$
$$\frac{62{,}956}{79{,}060} + 0.30\frac{63{,}641}{94{,}248} = 0.999 < 1.0 \quad \Rightarrow \quad \text{Okay}$$

Shear failure should be prevented whenever possible due to its brittle mode. Thus, it is important that the bending strength is reached before shear failure. In this manner, if the tower is designed properly for bending, the desirable ductility could be achieved. If the bending failure cannot be achieved before shear failure, then shear strength reduction factor ϕ should be taken as 0.6 to provide some additional shear strength and shear design repeated. Figure C-21 shows the reinforcing bar arrangement after the design for shear resistance.

C-16. Design for Seismic Bending Moments

a. General. The bending capacity of the tower shall be computed similar to cantilever columns with distributed reinforcement. Approximate hand calculations should be carried out in order to obtain approximate required reinforcement and to verify the final design. The final design in this example is performed by the computer program CGSI.

b. Design for MDE. The values of nominal loading should be established first, in order to perform a design.

(1) Nominal axial load equals:

$$P_n = \frac{P_u}{\phi} = \frac{52{,}422}{0.7} = 74{,}889 \text{ kN} \quad (16{,}835 \text{ kips})$$

(2) The nominal design value of the bending moment in the longitudinal direction (Y-Y) is:

$$M_u = \frac{M_{calculated}}{R_M}$$
$$= \frac{1{,}962{,}122}{2.0} = 981{,}061 \text{ kN - m } (723{,}555 \text{ k - ft})$$

where R_M is the moment reduction factor. R_M should not exceed two.

(3) The nominal moment capacity M_n is obtained as follows:

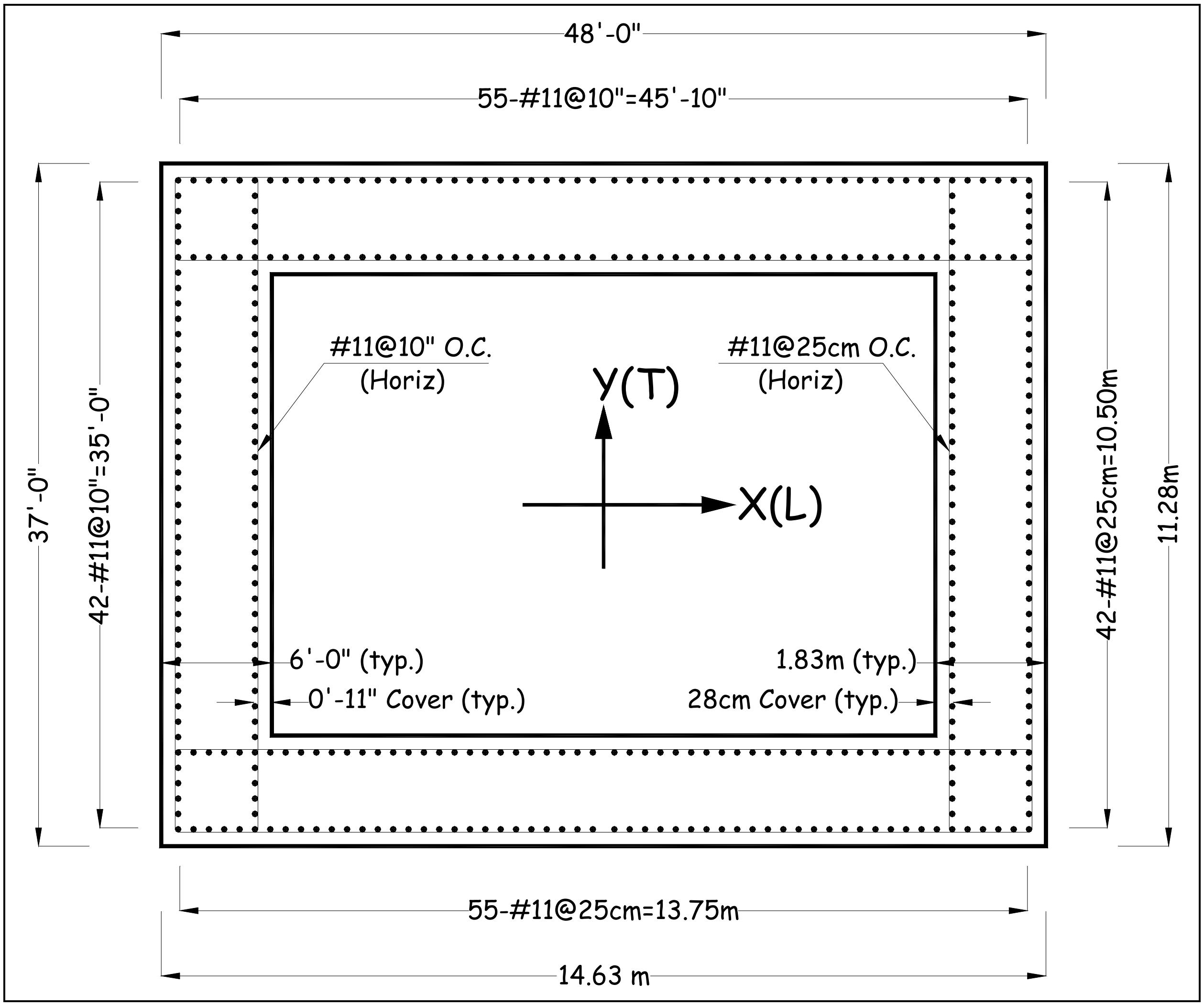


Figure C-21. Reinforcing bar arrangement after the design for shear resistance

$$\phi \cdot M_n \geq M_u$$

or

$$M_n = M_u / \phi = 981{,}061 / 0.9 =$$
$$= 1{,}090{,}068 \text{ kN - m } (803{,}950 \text{ k - ft})$$

(4) The eccentricity is calculated as indicated in Figure C-22:

$$e = \frac{M_n}{P_n} = \frac{1{,}090{,}068}{74{,}889} = 14.56 \text{ m} \quad (573.07 \text{ in})$$

(5) The compressive force in the concrete C_c can be obtained from force equilibrium in the vertical direction, as shown in Figure C-22:

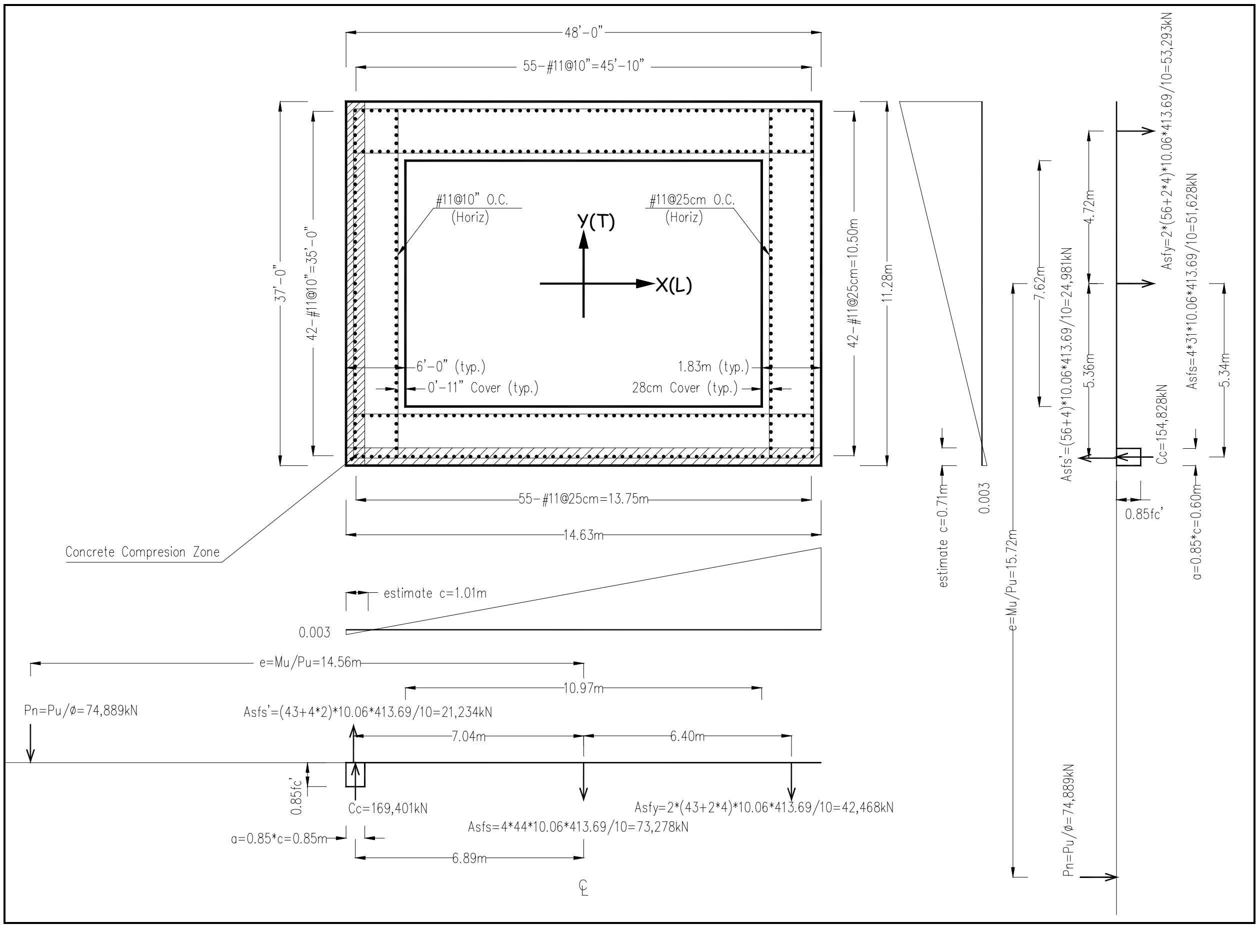


Figure C-22. Approximate design of the section-1 for bending moment resistance

$$\sum F_z = C_c + A_s f_s' + A_s f_s + A_s f_y + P_n = 0$$

$$\sum F_z = C_c + 21{,}234 - 73{,}278 - 42{,}468 - 74{,}889 = 0$$

$$C_c = 169{,}401 \text{ kN} \quad (38{,}081 \text{ kips})$$

(6) The depth of the concrete in compression can be obtained as

$$a = \frac{C_c}{0.85 \cdot f_c' \cdot 11.28}$$

$$= \frac{171{,}066}{0.85(20.68)(11.28)} = 0.85 \text{ m} \quad (33.64 \text{ in})$$

while the distance from extreme compression fiber to neutral axis is:

$$c = a / 0.85 = 0.85 / 0.85 = 1.01 \text{ m} \quad (39.57 \text{ in})$$

(7) The moment capacity of the section is obtained from the moment equilibrium with respect to the center line of the section:

$$\sum M_y = 0$$

$$\sum M_y = 42{,}468(6.40) + 73{,}278(0.0) +$$

$$21{,}234(7.04) + 169{,}401(6.89) = P_n \cdot e$$

$$P_n \cdot e = 271{,}830 + 0.0 + 149{,}398 + 1{,}166{,}836$$
$$= 1{,}588{,}064 \text{ kN-m} \quad (1{,}171{,}235 \text{ kip-ft})$$

$$1{,}588{,}064 \text{ kN - m} > 1{,}090{,}068 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$$

(8) Similarly, in the transverse direction (X-X):

$$M_n = \frac{M_u}{\phi \cdot R_M} = \frac{2{,}118{,}555}{0.9\ (2.0)}$$
$$= 1{,}176{,}975 \text{ kN-m} \quad (868{,}047 \text{ kip-ft})$$

$$e = \frac{M_n}{P_n} = \frac{1{,}176{,}975}{74{,}889} = 15.72 \text{ m} \quad (618.75 \text{ in})$$

As shown in Figure C-22:

$$\sum F_y = C_c + A_s f_s' + A_s f_s + A_s f_y + P_n = 0$$

$$\sum F_y = C_c + 24{,}981 - 51{,}628 - 53{,}293 - 74{,}889 = 0$$

$$C_c = 154{,}828 \text{ kN} \quad (34{,}805 \text{ kips})$$

$$a = \frac{C_c}{0.85 \cdot f_c' \cdot 14.63}$$
$$= \frac{154{,}828}{0.85(20.68)(14.63)} = 0.60 \text{ m} \quad (23.70 \text{ in})$$

$$c = a / 0.85 = 0.60 / 0.85 = 0.71 \text{ m} \quad (27.88 \text{ in})$$

$$\sum M_x = 0$$

$$\sum M_x = 53{,}293\ (4.72) + 51{,}628\ (0.0) + 24{,}981(5.36) + 154{,}828(5.34) = P_n \cdot e$$

$P_n \cdot e = 251{,}779 + 0.0 + 133{,}884 + 826{,}452$
$= 1{,}212{,}113$ kN-m (893,962 kip-ft)

$1{,}212{,}113 \text{ kN - m} > 1{,}176{,}975 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$

(9) Checking bi-directional criteria. Equation 4-6 should be used to account for earthquake bi-directional effects, as follows:

(a) In longitudinal direction:

$$\frac{M_{E,long}}{M_{u,long}} + 0.3 \cdot \frac{M_{E,trans}}{M_{u,trans}} \leq 1.0$$

$$\frac{1{,}090{,}068}{1{,}588{,}068} + 0.3 \cdot \frac{1{,}176{,}975}{1{,}212{,}113} = 0.978 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{M_{E,trans}}{M_{u,trans}} + 0.3 \cdot \frac{M_{E,long}}{M_{u,long}} \leq 1.0$$

$$\frac{1{,}176{,}975}{1{,}212{,}133} + 0.3 \frac{1{,}090{,}068}{1{,}588{,}064} = 1.177 > 1.0 \Rightarrow \text{NOT OK}$$

Since the strength criteria are not satisfied, the reinforcing bar spacing is reduced from 25 cm (10 in) to 20 cm (8 in) and moment calculations repeated.

c. Redesign for MDE Load Case. Repetition of these calculations for the 20-cm spacing results in the following section forces and moment capacities for MDE load case.

(1) Longitudinal direction

$C_c = 193{,}966$ kN (43,603 kips)
$a = 0.98$ m (38.51 in)
$c = 1.15$ m (45.31 in)
$P_n \cdot e = 1{,}871{,}237$ kN-m (1,380,081 kip-ft)
$1{,}871{,}237 \text{ kN - m} > 1{,}090{,}068 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$

(2) Transverse direction

$C_c = 174{,}397$ kN (39,204 kips)
$a = 0.68$ m (26.69 in)
$c = 0.80$ m (31.40 in)
$P_n \cdot e = 1{,}429{,}583$ kN-m (1,054,351 kip-ft)
$1{,}429{,}583 \text{ kN - m} > 1{,}176{,}975 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$

(3) Checking bidirectional strength criteria

(a) In longitudinal direction:

$$\frac{M_{E,long}}{M_{u,long}} + 0.3 \cdot \frac{M_{E,trans}}{M_{u,trans}} \leq 1.0$$

$$\frac{1{,}090{,}068}{1{,}871{,}237} + 0.3 \cdot \frac{1{,}176{,}975}{1{,}429{,}583} = 0.829 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{M_{E,trans}}{M_{u,trans}} + 0.3 \cdot \frac{M_{E,long}}{M_{u,long}} \leq 1.0$$

$$\frac{1{,}176{,}975}{1{,}429{,}583} + 0.3 \frac{1{,}090{,}068}{1{,}871{,}237} = 0.998 < 1.0 \Rightarrow \text{OK}$$

d. Check for OBE load case. The design now is checked for the OBE to ensure that the serviceability requirements are met. Note that the section capacity for the OBE condition should be computed since it differs from that for the MDE. The results are summarized below.

(1) Longitudinal direction:

$C_c = 222{,}256$ kN (49,962 kips)
$a = 1.12$ m (44.13 in)
$c = 1.32$ m (51.92 in)
$P_n \cdot e = 2{,}060{,}336$ kN-m (1,519,546 kip-ft)
$2{,}060{,}336 \text{ kN - m} > 1{,}438{,}375 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$

(2) Transverse direction:

$C_c = 204{,}352$ kN (45,938 kips)
$a = 0.79$ m (31.28 in)
$c = 0.93$ m (36,80 in)
$P_n \cdot e = 1{,}576{,}442$ kN-m (1,162,663 kip-ft)
$1{,}576{,}442 \text{ kN - m} > 1{,}552{,}822 \text{ kN - m} \quad \Rightarrow \quad \text{Okay}$

(3) Check for bidirectional strength criteria.

(a) In longitudinal direction:

$$\frac{1{,}438{,}375}{2{,}060{,}336} + 0.3 \cdot \frac{1{,}552{,}822}{1{,}576{,}442} = 0.994 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{1{,}552{,}822}{1{,}576{,}442} + 0.3\frac{1{,}438{,}375}{2{,}060{,}336} = 1.19 > 1.0 \Rightarrow \text{NOT OK}$$

Since the strength criteria are not satisfied, the section should be redesigned with 15 cm (6 in) of spacing.

e. Redesign for OBE load case.

(1) Longitudinal direction:

$$\begin{aligned}
&C_c = 261{,}809\ \text{kN} \quad (58{,}854\ \text{kips})\\
&a = 1.32\ \text{m} \quad (51.98\ \text{in})\\
&c = 1.55\ \text{m} \quad (61.16\ \text{in})\\
&P_n \cdot e = 2{,}539{,}236\ \text{kN-m} \quad (1{,}872{,}746\ \text{kip-ft})\\
&2{,}539{,}236\ \text{kN - m} > 1{,}438{,}375\ \text{kN - m} \quad \Rightarrow \quad \text{Okay}
\end{aligned}$$

(2) Transverse direction:

$$\begin{aligned}
&C_c = 237{,}661\ \text{kN} \quad (53{,}425\ \text{kips})\\
&a = 0.92\ \text{m} \quad (36.38\ \text{in})\\
&c = 1.09\ \text{m} \quad (42.79\ \text{in})\\
&P_n \cdot e = 1{,}945{,}437\ \text{kN-m} \quad (1{,}434{,}805\ \text{kip-ft})\\
&1{,}945{,}437\ \text{kN - m} > 1{,}552{,}822\ \text{kN - m} \quad \Rightarrow \quad \text{Okay}
\end{aligned}$$

(3) Check for bidirectional strength criteria.

(a) In longitudinal direction:

$$\frac{1{,}438{,}375}{2{,}539{,}236} + 0.3 \cdot \frac{1{,}552{,}822}{1{,}945{,}437} = 0.81 < 1.0 \Rightarrow \text{OK}$$

(b) In transverse direction:

$$\frac{1{,}552{,}822}{1{,}945{,}437} + 0.3\frac{1{,}438{,}375}{2{,}539{,}236} = 0.97 < 1.0 \Rightarrow \text{OK}$$

f. Final Design Check. Since the design was changed to satisfy moment requirement, one last check is required to determine whether the tower fails in shear or in bending. This can be achieved by comparing the ratios of nominal and demand forces:

If $\dfrac{M_n}{V_n} < \dfrac{M_u}{V_u}$ then the tower will fail in bending.

In order to evaluate this inequality, shear capacities of the structure should be recomputed with the new re-bar arrangement.

(1) Final check for MDE.

(a) In longitudinal direction:

$$V_{n,long} = V_c + V_{s,long} = 32{,}910 + 127{,}904 =$$
$$= 136{,}692 \text{ kN } (30{,}731 \text{ kips})$$

$$\frac{2{,}355{,}697}{136{,}692} = 17.23 < \frac{1{,}962{,}122}{63{,}641} = 30.83 \quad \Rightarrow \quad \text{Okay}$$

(b) In transverse direction:

$$V_{n,trans} = V_c + V_{s,trans} = 32{,}910 + 98{,}593 =$$
$$= 111{,}777 \text{ kN } (25{,}131 \text{ kips})$$

$$\frac{1{,}801{,}361}{111{,}777} = 16.12 < \frac{2{,}118{,}555}{62{,}956} = 33.65 \quad \Rightarrow \quad \text{Okay}$$

(2) Final check for OBE.

(a) In longitudinal direction:

$$\frac{2{,}539{,}236}{136{,}692} = 18.58 < \frac{1{,}294{,}538}{63{,}641} = 20.34 \quad \Rightarrow \quad \text{Okay}$$

(b) In the transverse direction:

$$\frac{1{,}945{,}437}{111{,}777} = 17.40 < \frac{1{,}397{,}540}{62{,}956} = 22.20 \quad \Rightarrow \quad \text{Okay}$$

C-17. Final Design Using a Computer Program

Due to the relative complexity of the section, performing the final design using a computer program is recommended. In this example, the Concrete General Strength Investigation (CGSI) is used (Figure C-23 and Table C-16).

a. The bending moment strength shall equal, or exceed, the bending moment demands by the following equation:

(1) In longitudinal direction:

$$\mathrm{U}_{\mathrm{long}} = D_{long} + L_{long} \pm \frac{\mathrm{M}_{\mathrm{long}}}{\mathrm{R}_{\mathrm{M}}} \pm 0.3 \cdot \frac{M_{trans}}{R_M} \qquad \text{Load case 1 (Longitudinal)}$$

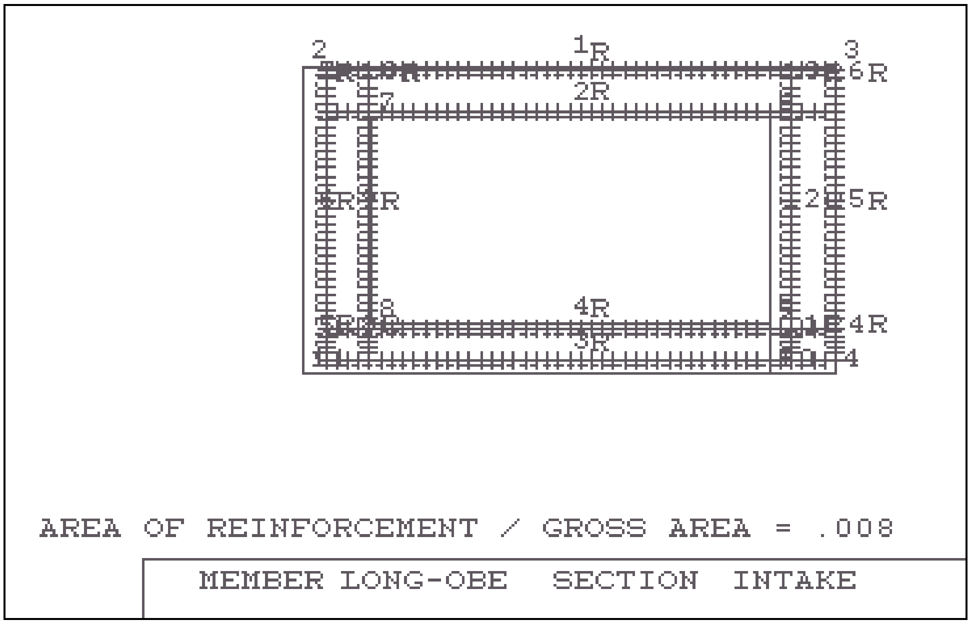


Figure C-23. Input geometry for CGSI program

Table C-16
Input Data for CGSI Analysis

In Longitudinal Direction	In Transverse Direction
1000 OBE-LONGITUDINAL	1000 OBE-TRANSVERSE
1010 MEMBER, LONG-OBE	1010 MEMBER, TRANS-OBE
1020 SECTION, INTAKE	1020 SECTION, INTAKE
1030 MATERIALS, 3000.0000, 60000.0000	1030 MATERIALS, 3000.0000, 60000.0000
1040 DIMENSION, FT	1040 DIMENSION, FT
1050 POINTS, 11	1050 POINTS, 11
1060 0.0000, 0.0000	1060 0.0000, 0.0000
1070 0.0000, 37.0000	1070 0.0000, 37.0000
1080 48.0000, 37.0000	1080 48.0000, 37.0000
1090 48.0000, 0.0000	1090 48.0000, 0.0000
1100 42.0000, 0.0000	1100 42.0000, 0.0000
1110 42.0000, 31.0000	1110 42.0000, 31.0000
1120 6.0000, 31.0000	1120 6.0000, 31.0000
1130 6.0000, 6.0000	1130 6.0000, 6.0000
1140 42.000, 6.0000	1140 42.000, 6.0000
1150 42.000, 0.0000	1150 42.000, 0.0000
1160 0.0000, 0.0000	1160 0.0000, 0.0000
1170 REINFORCE, 16	1170 REINFORCE, 16
1180 47, 3.12, 1.0, 36.0, 47.0, 36.0	1180 47, 3.12, 1.0, 36.0, 47.0, 36.0
1190 47, 3.12, 1.0, 31.0, 47.0, 31.0	1190 47, 3.12, 1.0, 31.0, 47.0, 31.0
1200 47, 3.12, 1.0, 1.0, 47.0, 1.0	1200 47, 3.12, 1.0, 1.0, 47.0, 1.0
1210 47, 3.12, 1.0, 5.0, 47.0, 5.0	1210 47, 3.12, 1.0, 5.0, 47.0, 5.0
1220 4, 3.12, 1.0, 2.0, 1.0, 4.0	1220 4, 3.12, 1.0, 2.0, 1.0, 4.0
1230 26, 3.12, 1.0, 6.0, 1.0, 30.0	1230 26, 3.12, 1.0, 6.0, 1.0, 30.0
1240 4, 3.12, 1.0, 32.0, 1.0, 35.0	1240 4, 3.12, 1.0, 32.0, 1.0, 35.0
1250 4, 3.12, 5.0, 2.0, 5.0, 4.0	1250 4, 3.12, 5.0, 2.0, 5.0, 4.0
1260 26, 3.12, 5.0, 6.0, 5.0, 30.0	1260 26, 3.12, 5.0, 6.0, 5.0, 30.0
1270 4, 3.12, 5.0, 32.0, 5.0, 35.0	1270 4, 3.12, 5.0, 32.0, 5.0, 35.0
1280 4, 3.12, 43.0, 2.0, 43.0, 4.0	1280 4, 3.12, 43.0, 2.0, 43.0, 4.0
1290 26, 3.12, 43.0, 6.0, 43.0, 30.0	1290 26, 3.12, 43.0, 6.0, 43.0, 30.0
1300 4, 3.12, 43.0, 32.0, 43.0, 35.0	1300 4, 3.12, 43.0, 32.0, 43.0, 35.0
1310 4, 3.12, 47.0, 2.0, 47.0, 4.0	1310 4, 3.12, 47.0, 2.0, 47.0, 4.0
1320 26, 3.12, 47.0, 6.0, 47.0, 30.0	1320 26, 3.12, 47.0, 6.0, 47.0, 30.0
1330 4, 3.12, 47.0, 32.0, 47.0, 35.0	1330 4, 3.12, 47.0, 32.0, 47.0, 35.0
1340 A77	1340 A77
1350 PLOT	1350 PLOT
1410 LOAD, LONG-OBE, 1	1410 LOAD, TRAN-OBE, 1
1420 1060835.000, 343573.0000, -16498.0000, 24.0000, 18.5000	1420 318251.0000, 1145242.000, -16498.0000, 24,0000, 18.5000
1430 EXIT	1430 EXIT

(2) In transverse direction:

$$\mathrm{U}_{\mathrm{trans}} = D_{trans} + L_{trans} \pm \frac{\mathrm{M}_{\mathrm{trans}}}{\mathrm{R}_{\mathrm{M}}} \pm 0.3 \cdot \frac{M_{long}}{R_M} \qquad \text{Load case 2 (Transverse)}$$

b. The examination of the results from CGSI (Figures C-24 and C-25) shows that the approximately designed section is adequate to carry the demand loads. Thus, it is confirmed that the section requires #11 reinforcing bar spaced at 6 in in horizontal and vertical directions, as shown in Figure C-26.

C-18. Sliding Shear Failure Check

A total number of 4*(95+73-4)=656 #11 bars (#11 @ 15cm (6 in.) center space) cross the potential horizontal sliding shear plane of the base of the tower. The nominal sliding shear resistance is determined using Equation 4-19:

$$\begin{aligned} V_{SL} &= P_u + 0.25 \cdot f_y \cdot A_{VF} \\ &= 52{,}422 + 0.25(413.69)(656)(10.06/10) \end{aligned}$$

$$\begin{aligned} V_{SL} &= 120{,}704\ \text{kN} \quad (27{,}134\ \text{kips}) \\ &> 74{,}871\ \text{kN} \quad (16{,}831\ \text{kips}) \quad \Rightarrow \quad \text{Okay} \end{aligned}$$

Since the nominal sliding shear resistance is significantly greater than the sliding shear demand, sliding shear is not a problem.

C-19. Anchorage of Moment Steel in Base Slab

a. Equation 4-11 can be used to determine the required anchorage length. The minimum length, however, should be at least 30 bar diameters.

$$l_a = \frac{k_s \cdot d_b}{\sqrt{f_c'}\left(1 + 2.5 \cdot \dfrac{c}{d_b}\right)} \qquad \text{(in psi units)}$$

where k_s is the re-bar constant calculated by

$$k_s = \frac{f_y - 11{,}000}{4.8} = \frac{60{,}000 - 11{,}000}{4.8} = 10{,}208$$

d_b = #11 bar diameter =3.58 cm (1.41 in)

$$c = \min(13.45, 5.83) = 5.83\ \text{cm} \quad (2.3\ \text{in})$$

which is lesser of the clear cover over the bar or bars or half of the clear spacing between adjacent bars.

$$2.5\left(\frac{c}{d_b}\right) = 2.5\left(\frac{5.83}{3.58}\right) = 4.07$$

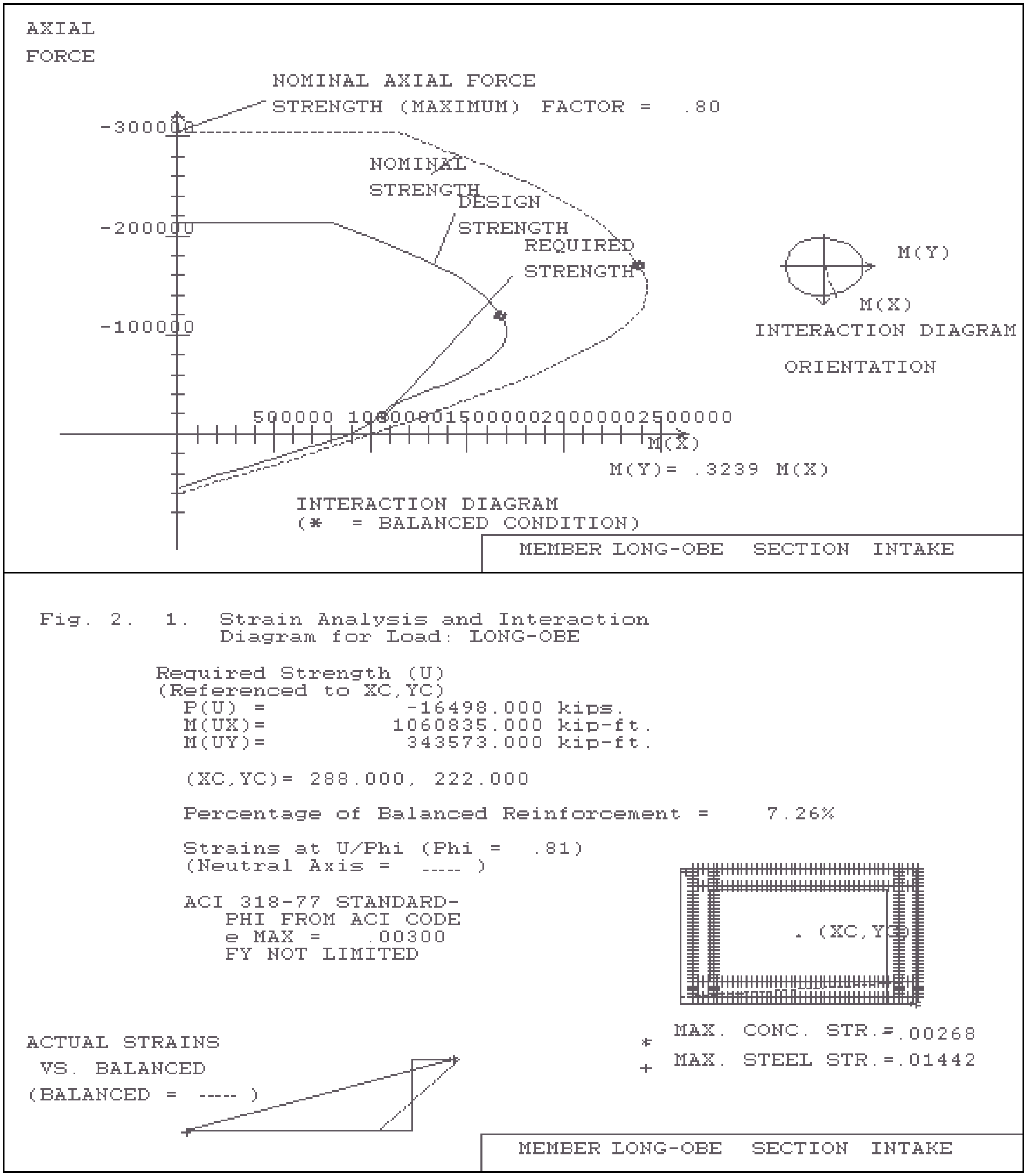


Figure C-24. CGSI analysis results in longitudinal direction

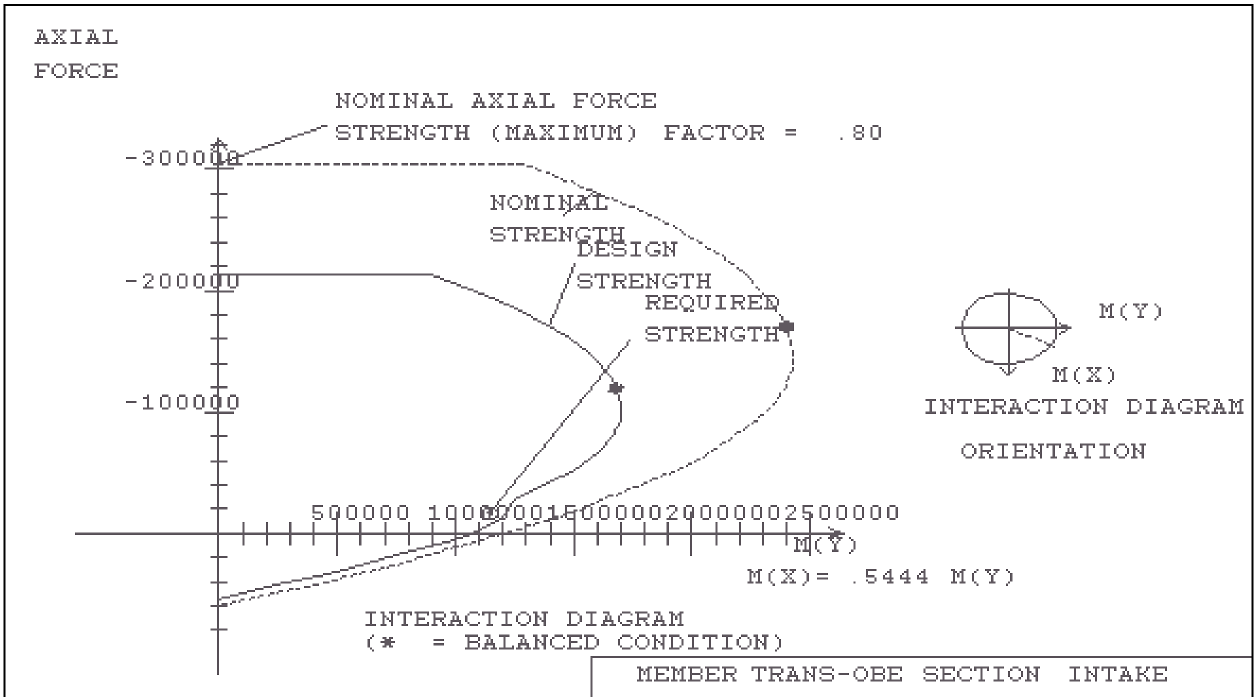


Fig. 2. 1. Strain Analysis and Interaction
Diagram for Load: TRAN-OBE

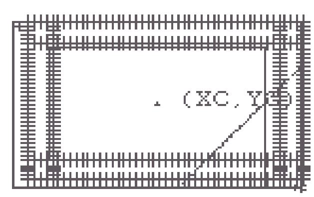
Required Strength (U)
(Referenced to XC,YC)
P(U) = -16498.000 kips.
M(UX)= 623464.000 kip-ft.
M(UY)= 1145242.000 kip-ft.

(XC,YC)= 288.000, 222.000

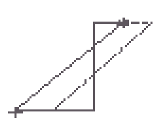
Percentage of Balanced Reinforcement = 6.10%

Strains at U/Phi (Phi = .81)
(Neutral Axis = -----)

ACI 318-77 STANDARD-
PHI FROM ACI CODE
e MAX = .00300
FY NOT LIMITED



ACTUAL STRAINS
VS. BALANCED
(BALANCED = -----)



* MAX. CONC. STR.= .00154
\+ MAX. STEEL STR.=.00407

MEMBER TRANS-OBE SECTION INTAKE

Figure C-25. CGSI analysis results in transverse direction

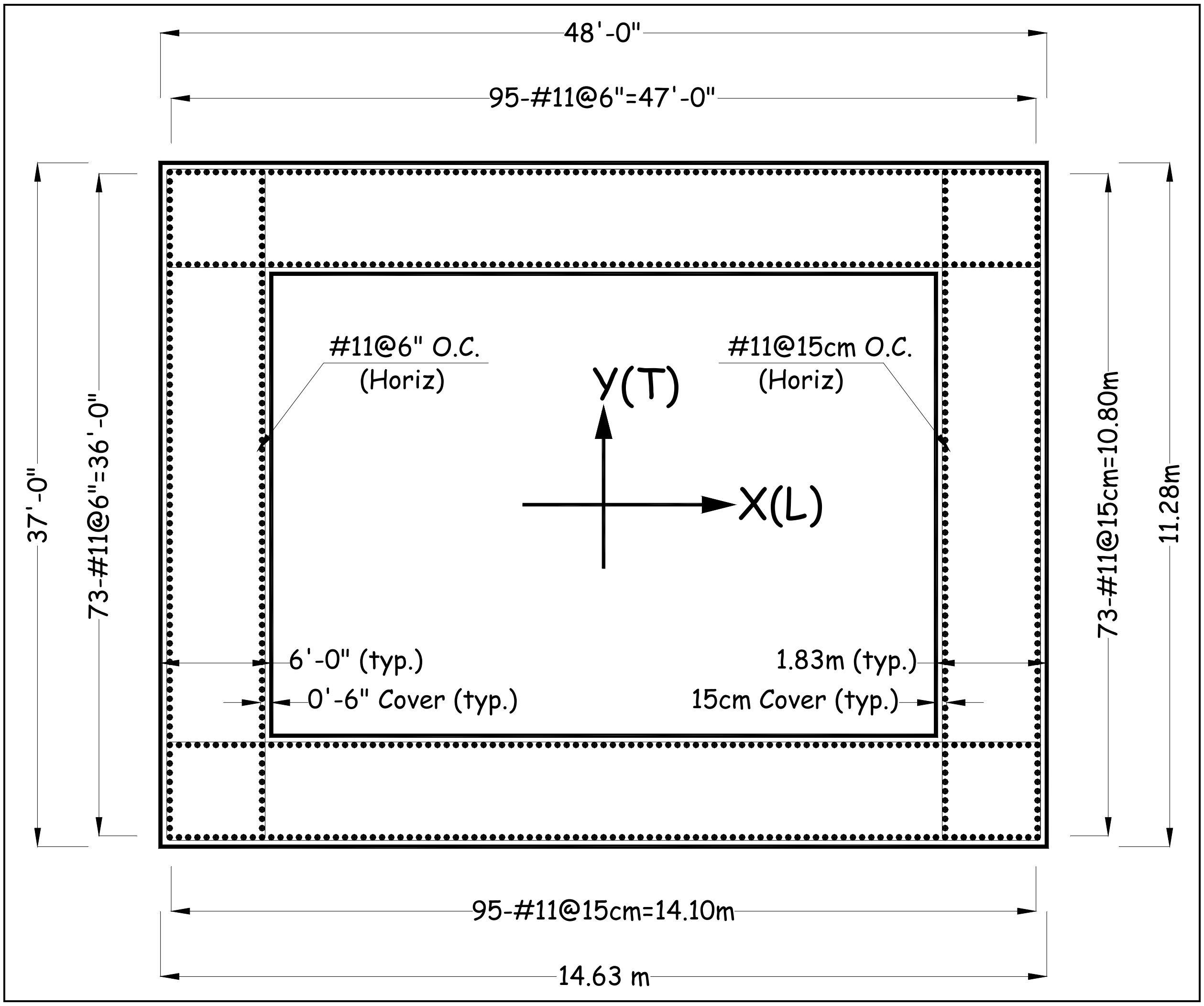


Figure C-26. Final reinforcing bar arrangement after designing for bending resistance

(ratio of (c/d_b)=1.63 should not exceed 2.5)

$$l_a = \frac{10{,}208(1.41)}{\sqrt{3{,}000}\,(1.0+4.07+0.0)} = 51.83\,\text{in}\ (131.65\,\text{cm})$$

l_a cannot be less than

$$l_a = 30 \cdot d_b = 30 \cdot 3.58 = 107.44 \text{ cm (42.3 in)}$$

The final length for straight anchorage is

$$l_a = 1.10 \text{ m (43.00 in)}$$

b. For 90-degree standard hooks, the effective anchorage length in inches is:

$$l_a = 1{,}200 \cdot \mathrm{d_b} \cdot \frac{f_y}{60{,}000\sqrt{f_c'}}$$
$$= 1{,}200(1.41)\frac{60{,}000}{60{,}000\sqrt{3{,}000}} = 30.89\,\text{in}\;(78.46\,\text{cm})$$

l_a cannot be less than

$$l_a = 15 \cdot d_b = 15 \cdot 3.58 = 53.72\ \text{cm}\ (21.15\ \ \text{in})$$

Final hooked anchorage length is

$$l_a = 0.80\ \text{m}\ (31.00\ \ \text{in})$$

C-20. Design of Reinforcing Steel Splices

The lap splice length required for new and existing towers should not be less than:

$$l_{s,\min} = 1{,}860\frac{d_b}{\sqrt{f_c'}} = 1{,}860\frac{1.41}{\sqrt{3{,}000}} = 48\,\text{in}\quad(1.25\,\text{m})$$

When concrete compressive strains exceed 0.002 in/in, transverse confinement steel should be provided at splice locations. The minimum area of transverse reinforcement necessary to prevent bond deterioration is given by Equation 4-13:

$$A_{tr} = \frac{s}{l_s} \cdot \frac{f_y}{f_{yt}} \cdot A_b = \frac{15}{125} \cdot \frac{413.69}{413.69} \cdot 10.06$$
$$= 1.21\,\text{cm}^2\ (0.19\,\text{in}^2) < A_{s,\#11} = 10.06\,\text{cm}^2\ (1.56\,\text{in}^2)$$

Since the required transverse reinforcing bar area is less than what is provided by #11 horizontal bars, the additional transverse reinforcement is not necessary.

C-21. Comprehensive Spalling Check

It can be assumed that the comprehensive spalling failures will not occur, when at ultimate load conditions, the ratio of the neutral axis depth to the total depth of the tower cross section is less than 15 percent (Equation 4-18):

a. In the longitudinal direction:

$$\frac{c}{d} = \frac{1.55}{14.63 - 0.28} = 0.140 < 0.15 \quad \Rightarrow \text{Okay}$$

where c = 1.55 is the location of neutral axis.

b. In the transverse direction:

$$\frac{c}{d} = \frac{1.09}{11.28 - 0.28} = 0.075 < 0.15 \quad \Rightarrow \text{Okay}$$

where c = 1.09 is the location of neutral axis.

C-22. Minimum Tensile Reinforcement Requirements

In the design of new intake towers, the nominal moment capacity should equal or exceed 120 percent of the uncracked moment capacity, or "cracking moment." This ensures that there is an adequate reinforcement to handle the energy, which is suddenly transferred from the concrete to the reinforcing steel. Assuming that the axial load is concentric, the cracking moment is determined by Equation 4-9:

$$M_{CR} = \left(\frac{I_g}{C}\right) \cdot \left(\frac{P_u}{A_G} + f_r\right)$$

In longitudinal direction:

$$I_g = \frac{11.28(14.63)^3 - 7.62(10.97)^3}{12}$$

$$I_g = 2104.16 \text{ m}^4 \quad (243{,}792 \text{ ft}^4)$$

$$C = \frac{14.63}{2} = 7.32 \text{ m} \quad (24 \text{ ft})$$

$$P_u = 52{,}422 \text{ kN} \quad (11{,}784 \text{ kips})$$

$$A_G = 14.63(11.28) - 10.97(7.62)$$

$$= 81.38 \text{ m}^2 \quad (876 \text{ ft}^2)$$

$$f_r = 0.62\sqrt{f'_c} = 0.62\sqrt{20.68} = 2.82 \text{ MPa}$$

$$f_r = 7.50\sqrt{f'_c} = 7.50\sqrt{3{,}000} = 410.79 \text{ psi}$$

$$M_{CR} = \left(\frac{2{,}104.16}{7.32}\right) \cdot \left(\frac{52{,}422}{81.38} + 2{,}820\right)$$

$$M_{CR} = 996{,}365 \text{ kN-m} \quad (734{,}843 \text{ kip-ft})$$

$$M_n = 1{,}945{,}437 \text{ kN-m} \quad (1{,}434{,}805 \text{ kip-ft})$$

$$\frac{M_n}{M_{CR}} = \frac{1{,}945{,}437}{996{,}365} = 1.95 > 1.20 \qquad \text{Okay}$$

C-23. Conclusions and Recommendations

a. Application of load and moment reduction factors and the use of different performance criteria for MDE and OBE make it difficult to determine the critical load case in advance. The design was therefore carried out for both MDE and OBE load cases. During the design it became necessary to increase the amount of reinforcing bars several times.

b. Since strength and serviceability performance criteria must be satisfied, the following design steps are recommended:

- Carry out design for both the MDE and OBE load cases.

- Check the design to ensure it meets the MDE performance criteria.

- Check the design to ensure it meets the OBE performance criteria.

- Use a computer program such as CGSI to supplement hand calculations and check the final design.