Appendix

Appendix	Description
A	**Building Loads**
	Dead Loads
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	Snow Loads
	Wind Loads
	Seismic Loads
B	**Depth Analysis**
	MDOF System Stiffness Matrices
	Drilled Pier Design Procedure
	Intact Rock Contour Drawings
C	**Breadth Analyses**
	Bracing Connections
	Partial HVAC Ductwork Plans

Appendix A

Dead & Live Load Requirements / Weight of Building Calculations

GRAVITY LOADS (ASCE 7-02, IBC 2000, + SOME EDUCATED GUESSES)

- DEAD LOADS
  - 6 ½" NML. WT. CONCRETE SLAB : 12 PSF/IN × 6 ½ = 78 PSF
  - METAL DECK : 3 PSF
  - FRAMING MEMBERS : 10 PSF
  - MEP EQUIPMENT : 10 PSF
  - EXT WALLS : 45 PSF
  - CARPET : 1 PSF
- PARTITIONS : 20 PSF
- LIVE LOADS
  - OFFICES : 50 + 20 PSF
  - LABORATORIES : 60 PSF
  - STAIRS / CORRIDORS : 100 PSF
- SNOW : 30 PSF (GROUND)
- ROOF DEAD : 60 PSF

FLOOR AREAS

114' × 260' → 30,000 SF PER FLOOR

→ 748'

STRUCTURE WEIGHT (FOR SEISMIC)

$W_{ROOF} = (30000)(60) + (14/2)(45)(748) = 2010\ k$

$W_1 = (30000)(122) + (15/2)(45)(748) = 3885\ k$

$W_2 = (30000)(122) + (15/2 + 14/2)(45)(748) = 4094\ k$

$W_3 = (30000)(122) + (14/2 + 14/2)(45)(748) = 4079\ k$

$W = W_{ROOF} + W_1 + W_2 + W_3 = 14{,}100\ k$

Appendix A

Snow Load Analysis

SNOW LOAD (ASCE 7-02)

$p_g = 30$ PSF GROUND SNOW LOAD

$C_e = 1.0$ PARTIALLY EXPOSED

$C_t = 1.0$ FOR FLAT ROOF
$= 1.2$ FOR SCREEN ROOFS

$I = 1.1$ CATEGORY III BUILDING

$p_f = 0.7 C_e C_t I p_g = 0.7(1.0)(1.0)(1.1)(30) = \boxed{23.1 \text{ psf}}$

* A VALUE OF 25 PSF WAS USED FOR DESIGN

$p_s = C_s p_f = (1.0)[(23.1)(1.2)] = \boxed{27.7 \text{ psf}}$

* A VALUE OF 28 PSF WAS USED FOR DESIGN

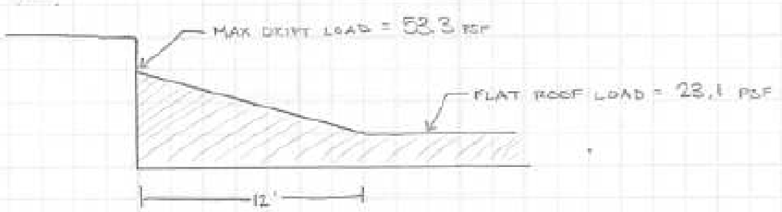
DRIFT — SCREEN ROOF – FLAT ROOF PROJECTION (SEC. 7.8)

$\gamma = 0.13(30) + 14 = 17.9$ pcf ≤ 30 pcf

$h_d = \begin{cases} 1.5' & \text{FIGURE 7-9} \\ W/4 = 12/4 = 3' & \leftarrow \text{CONTROLS} \end{cases}$

$h_b = (23.1)/(17.9) = 1.3'$

$h_d / h_b = 2.3$



* THE DESIGNED LOADS ARE
25 PSF FLAT ROOF LOAD
75 PSF MAX. DRIFT LOAD

Appendix A

Wind Load Analysis

WIND LOAD CALCULATIONS : NORTH-SOUTH DIRECTION [EAST-WEST]

- DESIGNED VALUES FROM GENERAL NOTES OF STRUCTURAL DRAWINGS
  - BASIC WIND SPEED, V_{3s} : 90 MPH
  - WIND IMPORTANCE FACTOR, I_w : 1.15
  - WIND EXPOSURE : B
  - HEIGHT & EXPOSURE ADJUSTMENT FACTOR : 1.19
  - P_{ROOF} : +15.9/-17.3 PSF, FIELD
  - P_{ROOF} : +15.9/-20.3 PSF, EDGE
  - P_{ROOF} : +15.9/-20.3 PSF, CORNER
  - P_{WALL} : +17.4/-18.8 PSF, FIELD
  - P_{WALL} : +17.4/-23.3 PSF, CORNER

$K_d = 0.85$ (TABLE 6-4)

C_p : WINDWARD → $C_p = 0.8$
LEEWARD → $C_p = -0.5$ [-0.3]

$K_{zt} = 1.0$

$G = 0.830$ [0.799]

K_z : (TABLE 6-3)

0'-15'	0.57
20'	0.62
25'	0.66
30'	0.70
40'	0.76
50'	0.81
60'	0.85

K_h : 0.83 FOR $h = 55'$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

0'-15'	11.6 PSF
20'	12.6
25'	13.4
30'	14.2
40'	15.4
50'	16.4
60'	17.2

$q_h = 0.00256 K_h K_{ht} K_d V^2 I = 16.8$ PSF

Appendix A

Wind Load Analysis (cont'd)

WIND LOAD CALCS (CONT'D) [EAST-WEST VALUES]

$P_w = qGC_p = q(0.83)(0.8)$

0-15'	7.7 PSF	[7.4]
20'	8.3	[8.0]
25'	8.9	[8.6]
30'	9.4	[9.0]
40'	10.2	[9.8]
50'	10.9	[10.5]
60'	11.4	[11.0]

$P_e = (16.8)(0.83)(-0.5) = -7.0$ [-6.7]

NET PRESSURE (P_{NET})

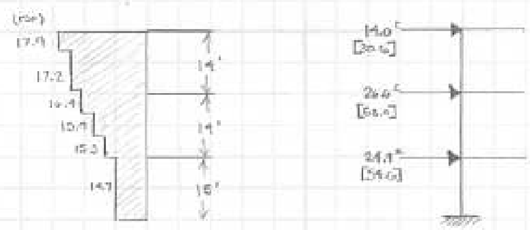
0-15'	(7.7) + (7.0) = 14.7 PSF	[14.1]
-20'	= 15.3	[14.7]
-25'	= 15.9	[15.3]
-30'	= 16.4	[15.7]
-40'	= 17.2	[16.5]
-50'	= 17.9	[17.2]
-60'	= 18.4	[17.7]

• FLOOR SHEAR LOADS

$F_1 = [(14.7)(7.5) + (15.3)(5) + (15.9)(2)](114) = 24.9\ k$ [54.6k]

$F_2 = [(15.9)(3) + (16.4)(5) + (17.2)(6)](114) = 26.6\ k$ [58.0k]

$F_3 = [(17.2)(4) + (17.9)(3)](114) = 14.0\ k$ [30.6k]



BASE SHEAR
65.5 k [143.2k]

BASE MOMENT
1746.9'k [3816.8'k]

Appendix A

Seismic Load Analysis

SEISMIC LOAD CALCULATIONS (ASCE 7-02)

- DESIGN VALUES FROM GENERAL NOTES OF STRUCTURAL DRAWINGS

  SEISMIC USE GROUP: II

  SEISMIC DESIGN CATEGORY: B

  $S_{DS} = 0.19$

  $S_{D1} = 0.05$

  SITE CLASS: B

  DESIGN BASE SHEAR: 895 KIPS

  SEISMIC RESISTING SYSTEM: CONCENTRICALLY BRACED FRAMES (STRUCTURAL STEEL SYSTEM NOT SPECIFICALLY DESIGNED FOR SEISMIC RESISTANCE.)

  ANALYSIS PROCEDURE: EQUIVALENT LATERAL FORCE PROCEDURE

$I = 1.25$ (TABLE 9.1.4)

$S_{MS} = 25\ \%g$ (FIGURE 9.4.1.1 (a))

$S_{M1} = 6\ \%g$ (FIGURE 9.4.1.1 (b))

$F_a = F_v = 1.0$ (TABLE 9.4.1.2.4)

$S_{DS} = \frac{2}{3} S_{MS} = 0.167\ g$

$S_{D1} = \frac{2}{3} S_{M1} = 0.04\ g$

$T_o = 0.2\ S_{D1}/S_{DS} = 0.048\ s$

$T_s = S_{D1}/S_{DS} = 0.24\ s$

$R = 5$ RESPONSE MOD. FACTOR

$W_o = 2$ SYSTEM OVERSTRENGTH FACTOR

$C_d = 4.5$ DEFLECTION AMP. FACTOR

(TABLE 9.5.2.2 ORDINARY STEEL CONCENTRICALLY BRACED FRAMES

$C_s = \frac{S_{DS}}{R/I} = 0.06$

$T = T_a = C_t h_n^x = (0.02)(43)^{(0.75)} = 0.336 < C_u(0.1N) = 0.51$

$V = C_s W$

Appendix A

Seismic Load Analysis (cont'd)

SEISMIC LOAD CALCS (CONT'D)

$V = C_s W = (0.06)(14100) = 846\ k$ BASE SHEAR

* VERY COMPARABLE TO DESIGN VALUE OF 895^k

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ $k = 1.0$ FOR $T \le 0.5s$

$C_{ROOF} = \frac{(2010)(43)}{(266131)} = 0.325$

$C_3 = \frac{(4079)(29)}{(266131)} = 0.444$

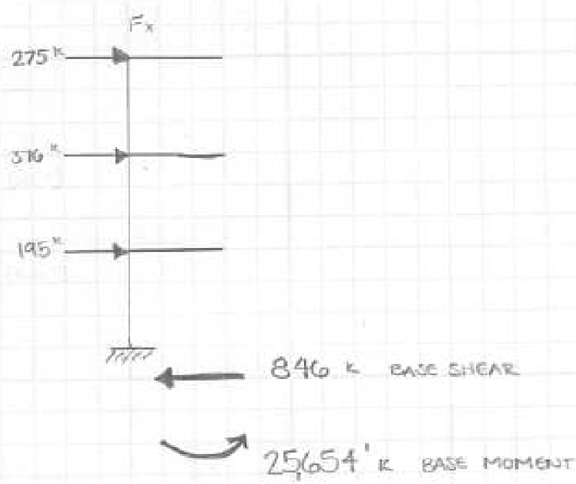
$C_2 = \frac{(4094)(15)}{(266131)} = 0.231$

$F_x = C_{vx} V$

$F_{ROOF} = 275\ k$

$F_3 = 376\ k$

$F_2 = 195\ k$



Appendix B

MDOF System Stiffness Matrices

Existing System

1

2679	-1079	0
-1079	1983	-904
0	-904	904

6

1250	-625	0
-625	1170	-546
0	-546	546

2

2162	-906	0
-906	1810	-904
0	-904	904

7

1250	-625	0
-625	866	-241
0	-241	241

3

1985	-906	0
-906	1810	-904
0	-904	904

8

2335	-1079	0
-1079	1983	-904
0	-904	904

4

2335	-1079	0
-1079	1983	-904
0	-904	904

9

1812	-906	0
-906	1810	-904
0	-904	904

5

2162	-906	0
-906	1810	-904
0	-904	904

10

1812	-906	0
-906	1810	-904
0	-904	904

Revised System

1

1367	-550	0
-550	952	-402
0	-402	402

6

1435	-717	0
-717	1263	-546
0	-546	546

2

1103	-462	0
-462	864	-402
0	-402	402

7

1898	-835	0
-835	1380	-545
0	-545	545

3

1632	-816	0
-816	1526	-709
0	-709	709

8

1632	-816	0
-816	1526	-709
0	-709	709

4

[redacted]

9

[redacted]

5

[redacted]

10

[redacted]

Appendix B

Drilled Pier Design Procedure

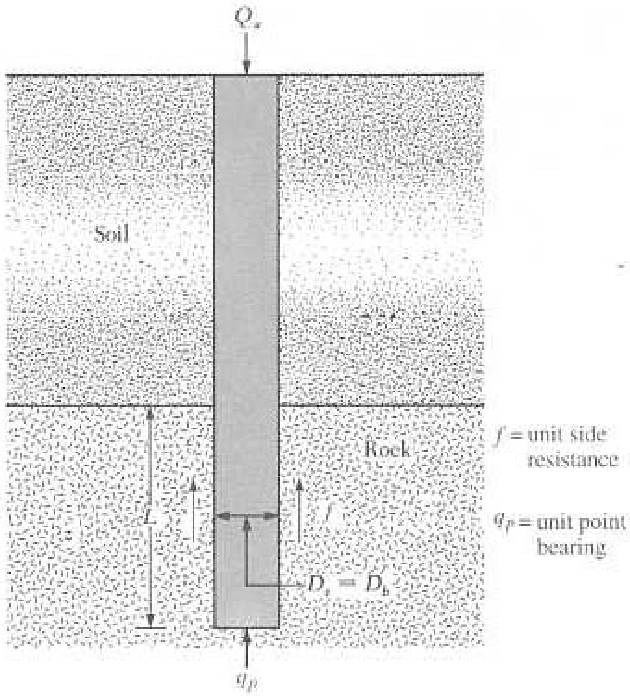


Figure 12.27 Drilled shaft socketed into rock

1. Calculate the ultimate unit side resistance as

$$f\,(\text{lb/in}^2) = 2.5q_u^{0.5} \leq 0.15q_u \tag{12.62}$$

where q_u = unconfined compression strength of a rock core of NW size or larger, or of the drilled shaft concrete, whichever is smaller (in lb/in^2)

In SI units, Eq (12.62) can be expressed as

$$f\,(\text{kN/m}^2) = 6.564q_u^{0.5}(\text{kN/m}^2) \leq 0.15q_u\,(\text{kN/m}^2) \tag{12.63}$$

2. Calculate the ultimate capacity based on side resistance only, or

$$Q_u = \pi D_s L f \tag{12.64}$$

3. Calculate the settlement s_e of the shaft at the top of the rock socket, or

$$s_e = s_{e(s)} + s_{e(b)} \tag{12.65}$$

where $s_{e(s)}$ = elastic compression of the drilled shaft within the socket, assuming no side resistance
$s_{e(b)}$ = settlement of the base

However,

$$s_{e(s)} = \frac{Q_u L}{A_c E_c} \tag{12.66}$$

Das, Braja M.. <u>Principles of Foundation Engineering</u>. 5th Edition.

Appendix B

Drilled Pier Design Procedure (cont'd)

and

$$s_{e(b)} = \frac{Q_u I_f}{D_s E_{mass}} \quad (12.67)$$

where Q_u = ultimate load obtained from Eq. (12.62) or Eq. (12.63) (this assumes that the contribution of the overburden to the side shear is negligible)

A_c = cross-sectional area of the drilled shaft in the socket (12.68)

$= \frac{\pi}{4} D_s^2$

E_c = Young's modulus of the concrete and reinforcing steel in the shaft

E_{mass} = Young's modulus of the rock mass into which the socket is drilled

I_f = elastic influence coefficient (see Figure 12.28)

The magnitude of E_{mass} can be determined from the average plot shown in Figure 12.29. In this figure, E_{core} is the Young's modulus of intact specimens of rock cores of NW size or larger. However, unless the socket is very long (O'Neill, 1997),

$$s_e \approx s_{e(b)} = \frac{Q_u I_f}{D_s E_{mass}} \quad (12.69)$$

4. If s_e is less than 10 mm ($\approx$0.4 in.), then the ultimate load-carrying capacity is that calculated by Eq. (12.64). If $s_e \geq$ 10 mm. (0.4 in.), then go to Step 5.

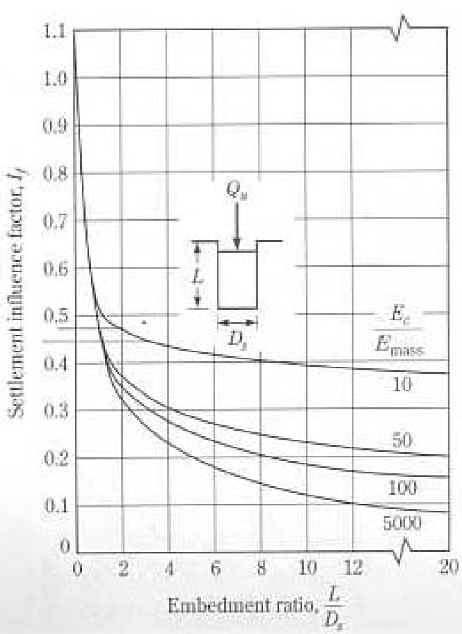


Figure 12.28 Variation of I_f (after Reese and O'Neill, 1989)

Das, Braja M.. Principles of Foundation Engineering. 5th Edition.

Appendix B

Drilled Pier Design Procedure (cont'd)

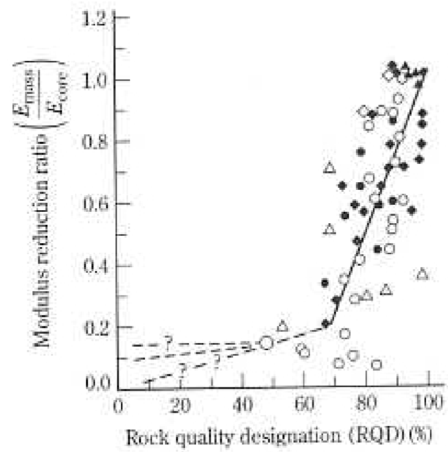


Figure 12.29 Plot of $E_{\text{mass}}/E_{\text{core}}$ vs. RQD (after Reese and O'Neill, 1989)

5. If $s_e \geqq 10$ mm (0.4 in.), there may be rapid, progressive side shear failure in the rock socket, resulting in a complete loss of side resistance. In that case, the ultimate capacity is equal to the point resistance, or

$$Q_u = 3A_p\left[\frac{3 + \dfrac{c_s}{D_s}}{10\left(1 + 300\dfrac{\delta}{c_s}\right)^{0.5}}\right]q_u \qquad (12.70)$$

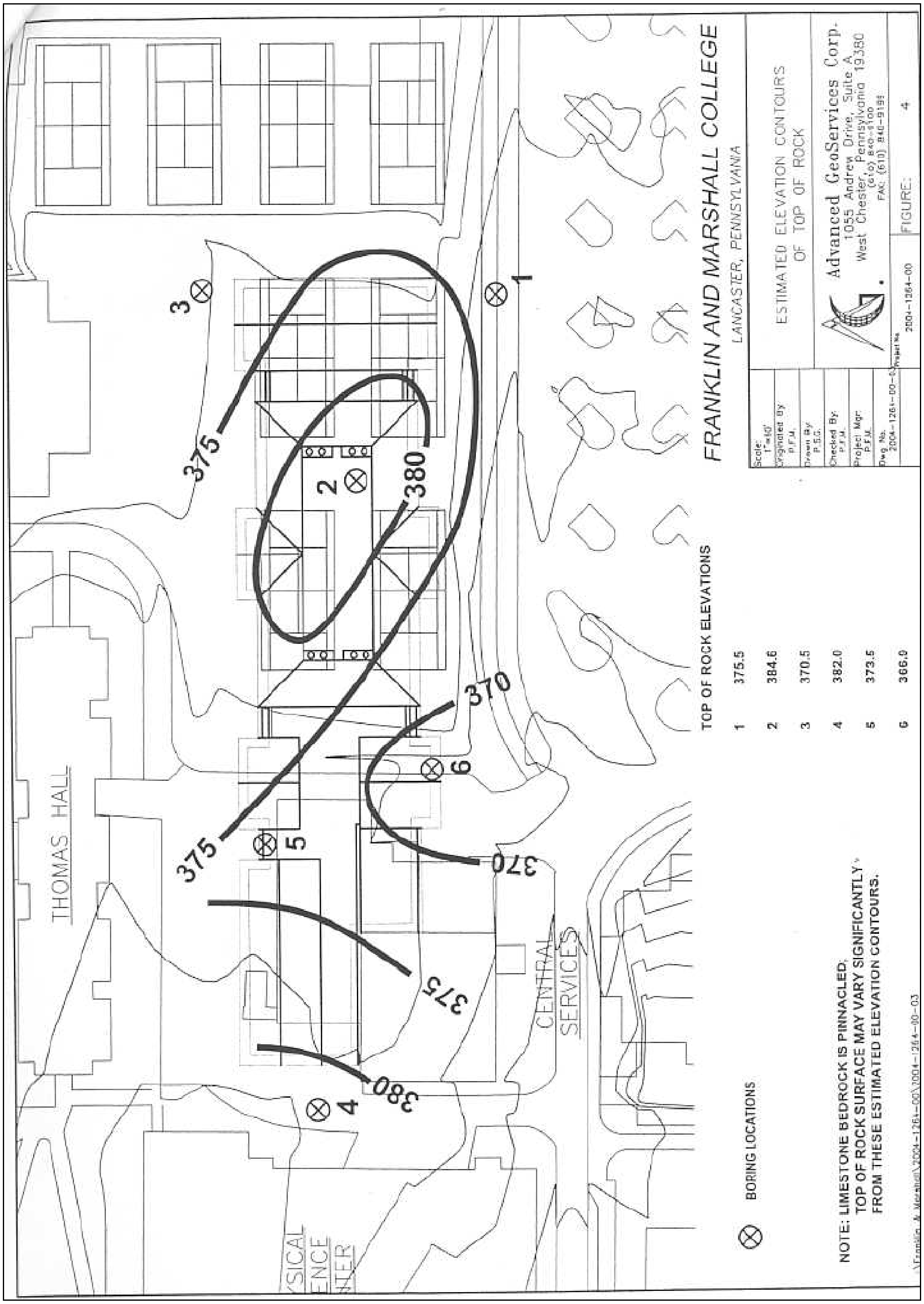
where c_s = spacing of discontinuities (same unit as D_s)
δ = thickness of individual discontinuity (same unit as D_s)
q_u = unconfined compression strength of the rock beneath the base of the socket, or the drilled shaft concrete, whichever is smaller

Note that Eq. (12.70) applies for horizontally stratified discontinuities with $c_s > 305$ mm (12 in.) and $\delta < 5$ mm (0.2 in.).

Das, Braja M.. Principles of Foundation Engineering. 5th Edition.

Appendix B

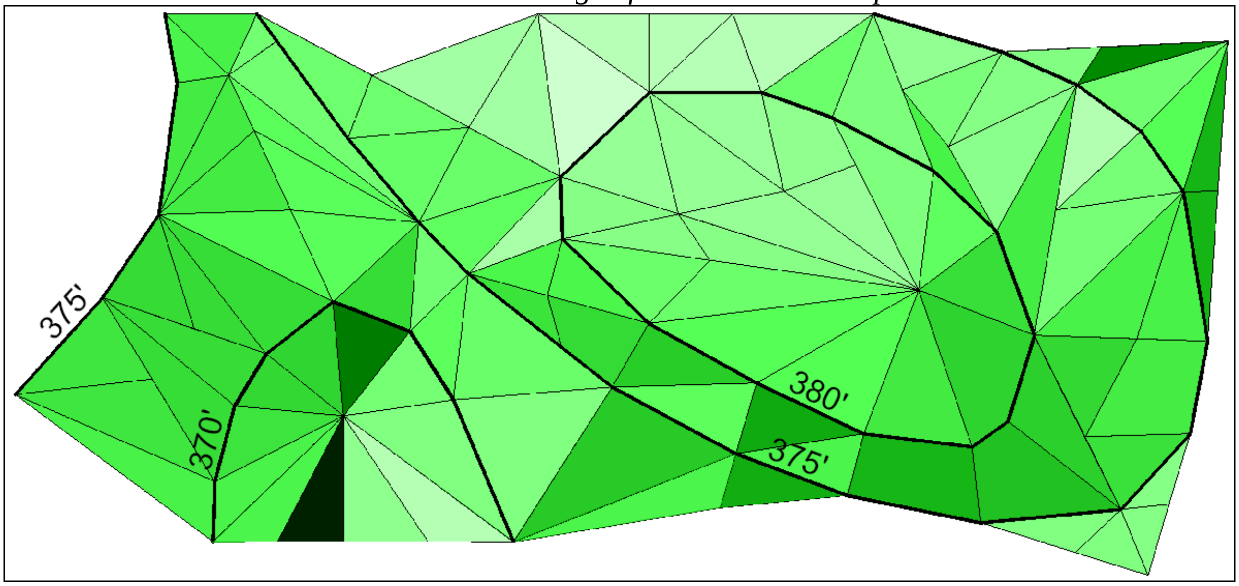
Intact Rock Contour Drawings



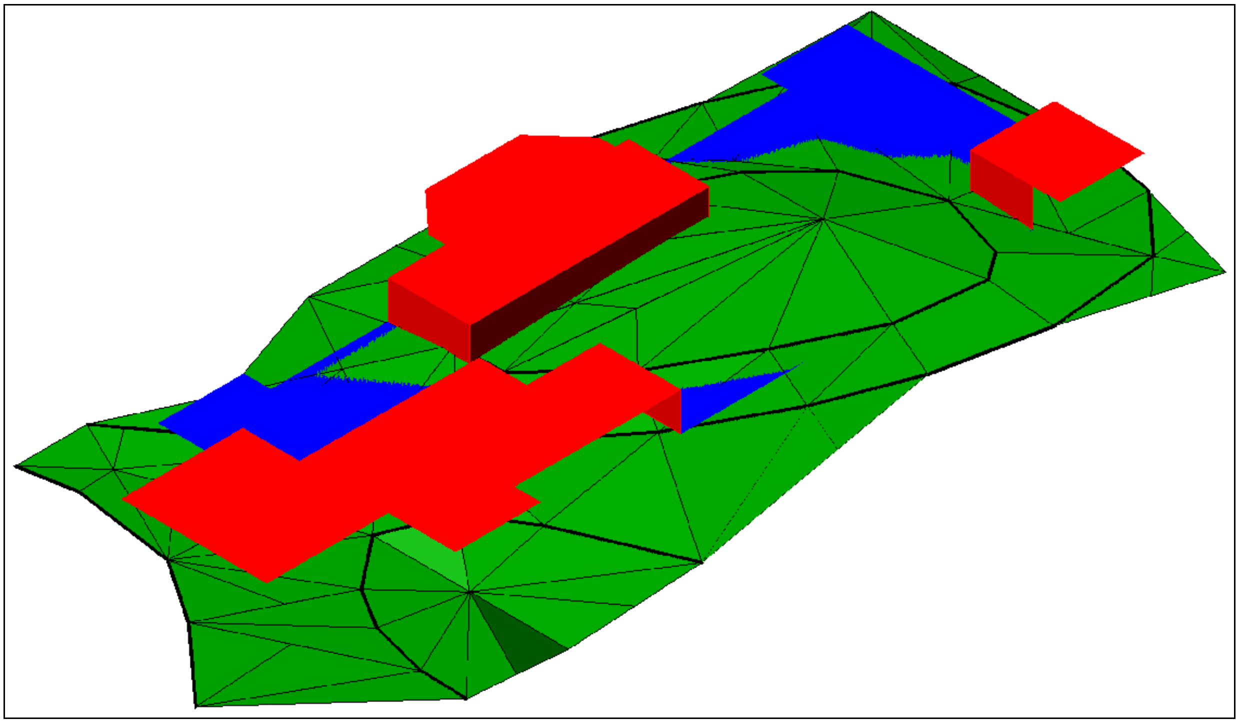
Appendix B

Intact Rock Contour Drawings (cont'd)

3-D CAD Drawings of AGC Contour Map



Plan View



SW Isometric View

Appendix C

Bracing Connections

Weld Size	4/16	*Assumes >= 1/2" Thick Connector Plates		Brace End Req'd Weld Area (in^2)	Beam Bottom Req'd Weld Area (in^2)	Column Req'd Weld Area (in^2)
Frame	(theta)	Story Shear (k)	Brace Axial Force (k)			
1	0.951	22.3	83.4	20.0	10.0	20.0
	0.951	43.1	64.2	20.0	10.0	20.0
	0.983	31.5	28.4	20.0	10.0	20.0
2	0.951	20.9	78.2	20.0	10.0	20.0
	0.951	40.4	60.2	20.0	10.0	20.0
	0.983	29.5	26.6	20.0	10.0	20.0
3	0.951	62.5	233.1	20.0	10.0	20.0
	0.951	120.4	179.4	20.0	10.0	20.0
	0.983	88.1	79.4	20.0	10.0	20.0
4	0.951	48.2	179.8	20.0	10.0	20.0
	0.951	92.9	138.4	20.0	10.0	20.0
	0.983	67.9	61.2	20.0	10.0	20.0
5	0.951	47.5	177.1	20.0	10.0	20.0
	0.951	91.5	136.3	20.0	10.0	20.0
	0.983	66.9	60.3	20.0	10.0	20.0
6	0.757	51.1	152.6	20.0	10.0	20.0
	0.757	98.6	117.4	20.0	10.0	20.0
	0.791	72.1	51.3	20.0	10.0	20.0
7	0.757	76.6	228.8	20.0	10.0	20.0
	0.757	147.8	176.0	20.0	10.0	20.0
	1.112	108.1	243.8	20.0	10.0	20.0
8	0.951	48.2	179.8	20.0	10.0	20.0
	0.951	92.9	138.4	20.0	10.0	20.0
	0.983	67.9	61.2	20.0	10.0	20.0
9	0.951	24.5	91.5	20.0	10.0	20.0
	0.951	47.3	70.4	20.0	10.0	20.0
	0.983	34.6	31.2	20.0	10.0	20.0
10	0.951	10.7	40.1	20.0	10.0	20.0
	0.951	20.7	30.8	20.0	10.0	20.0
	0.983	15.1	13.7	20.0	10.0	20.0
	# of Connections Per Story			4	1	2
	TOTAL WELD AREA (in^2)			3900.0		
	TOTAL WELD VOLUME (in^3)			1950.0		
	TOTAL WELD MATERIAL (lbs)			**107.8**		

Welded Connection Design for the Existing Bracing System

Appendix C

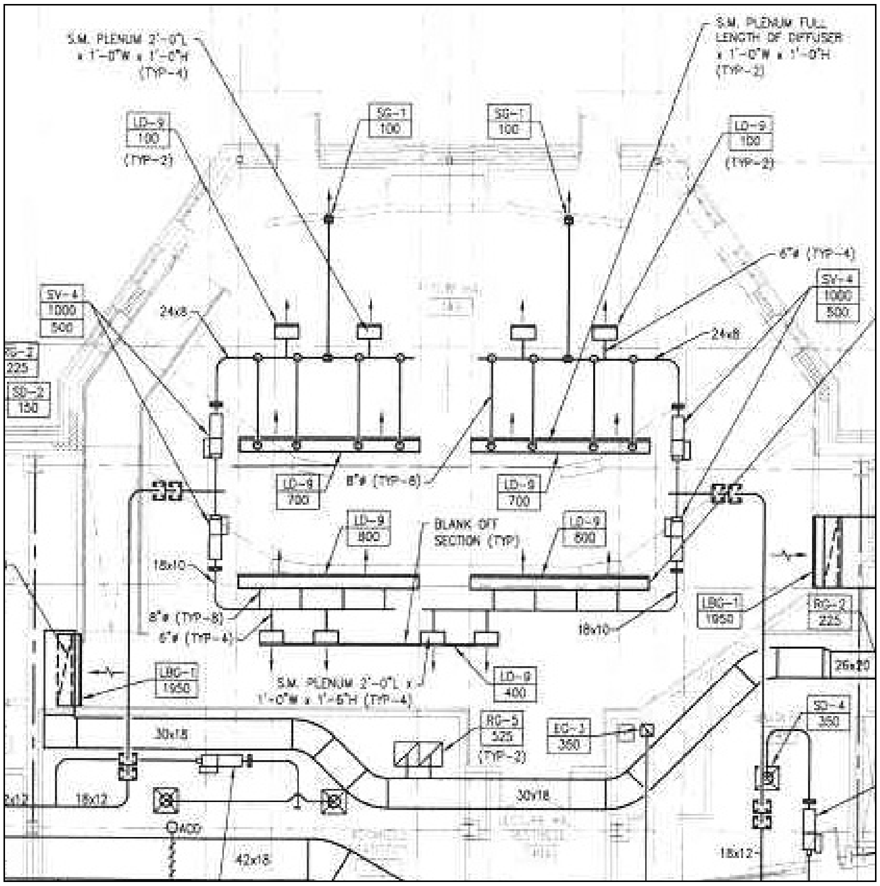
Bracing Connections (cont'd)

Weld Size	4/16	*Assumes >1/2" Thick Connector Plates*		Brace End Req'd Weld Area (in^2)	Beam Bottom Req'd Weld Area (in^2)	Column Req'd Weld Area (in^2)
Frame	(theta)	Story Shear (k)	Brace Axial Force (k)			
1	0.951	30.5	114.0	20.0	10.0	20.0
	0.951	58.9	87.7	20.0	10.0	20.0
	0.983	43.1	38.8	20.0	10.0	20.0
2	0.951	28.3	105.8	20.0	10.0	20.0
	0.951	54.7	81.4	20.0	10.0	20.0
	0.983	40.0	36.0	20.0	10.0	20.0
3	0.951	112.0	418.1	20.0	10.0	20.0
	0.951	216.0	321.7	20.0	10.0	20.0
	0.983	158.0	142.4	20.0	10.0	20.0
6	0.757	64.8	193.5	20.0	10.0	20.0
	0.757	125.0	148.9	20.0	10.0	20.0
	0.791	91.4	65.0	20.0	10.0	20.0
7	0.757	96.1	286.9	20.0	10.0	20.0
	0.757	185.3	220.8	20.0	10.0	20.0
	0.791	135.6	96.4	20.0	10.0	20.0
8	0.951	94.3	351.8	20.0	10.0	20.0
	0.951	181.8	270.7	20.0	10.0	20.0
	0.983	132.9	119.8	20.0	10.0	20.0
	# of Connections Per Story			4	1	2
	TOTAL WELD AREA (in^2)				2340.0	
	TOTAL WELD VOLUME (in^3)				1170.0	
	TOTAL WELD MATERIAL (lbs)				**64.7**	

Welded Connection Design for Revised Bracing System

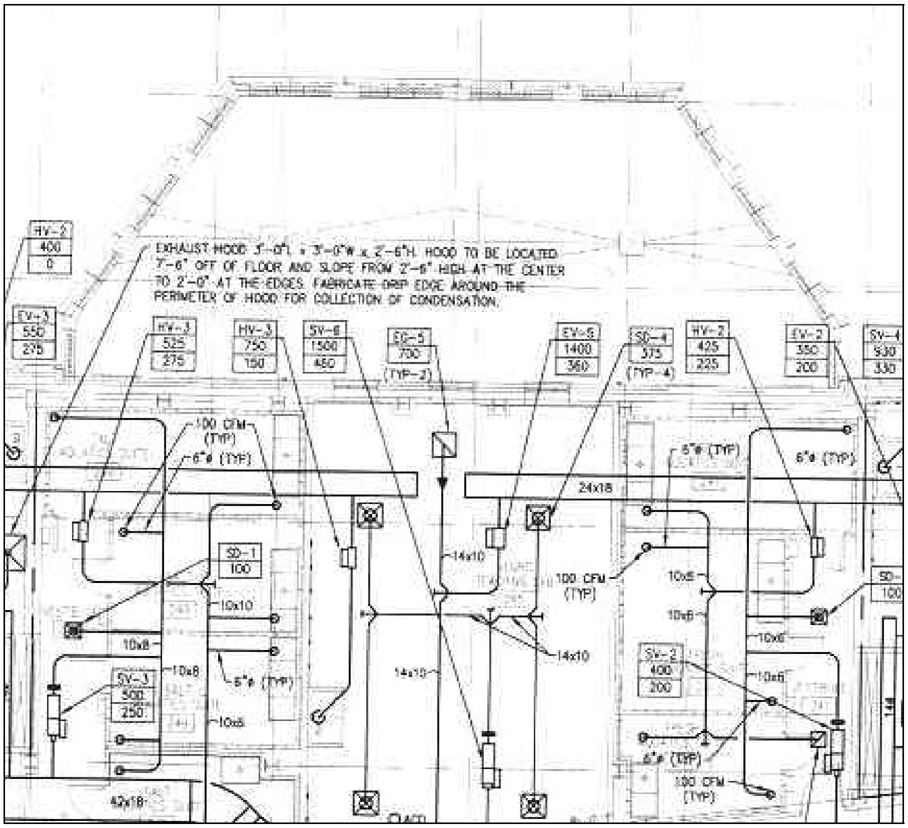
Appendix C

Partial HVAC Ductwork Plans

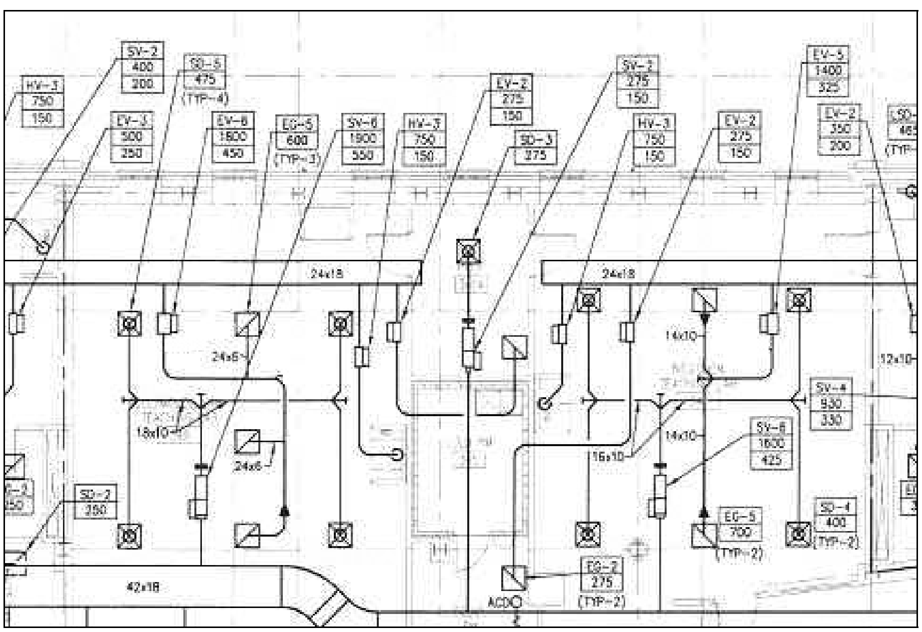


First Floor

Appendix C
Partial HVAC Ductwork Plans (cont'd)



Second Floor



Third Floor

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