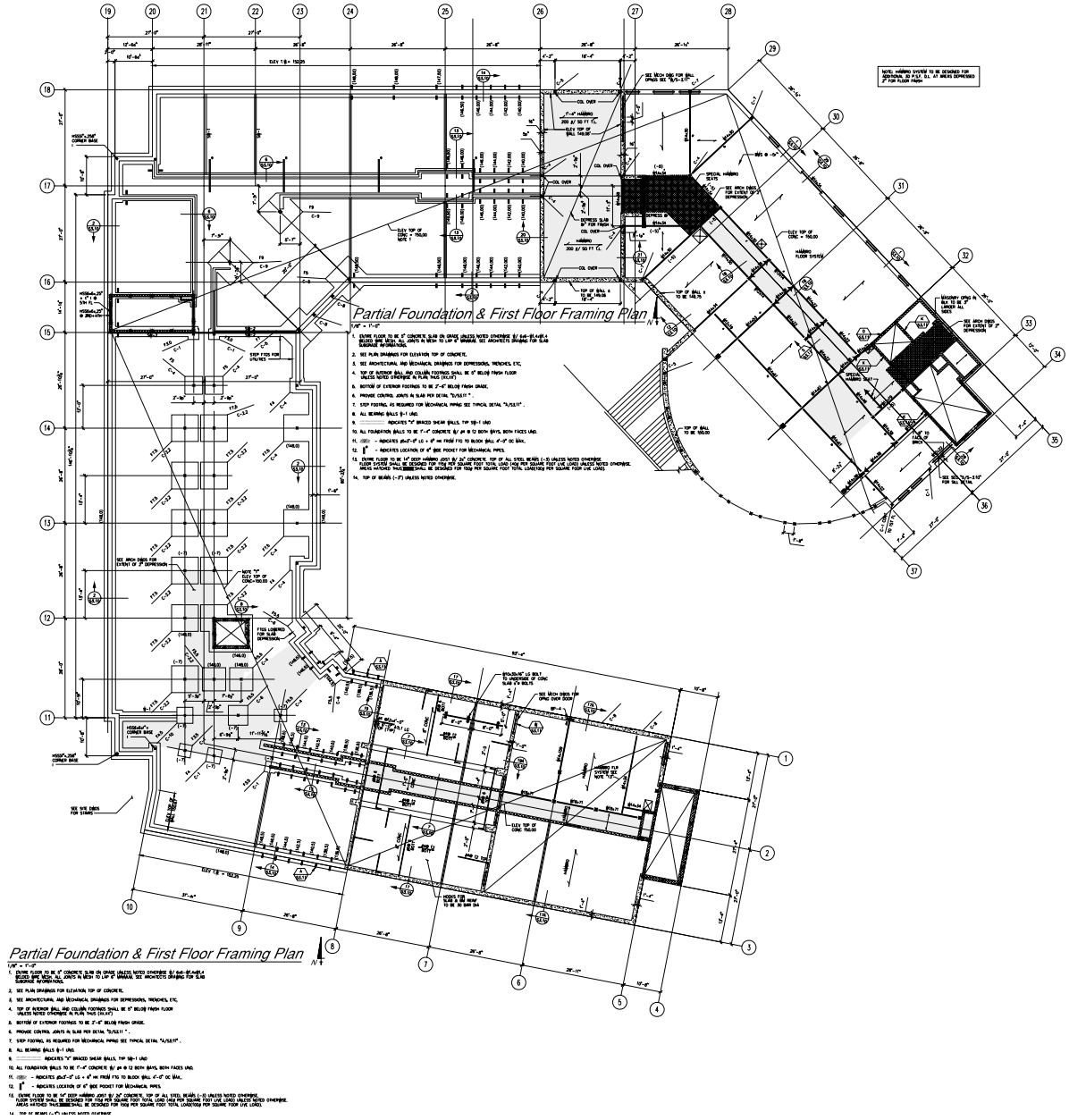
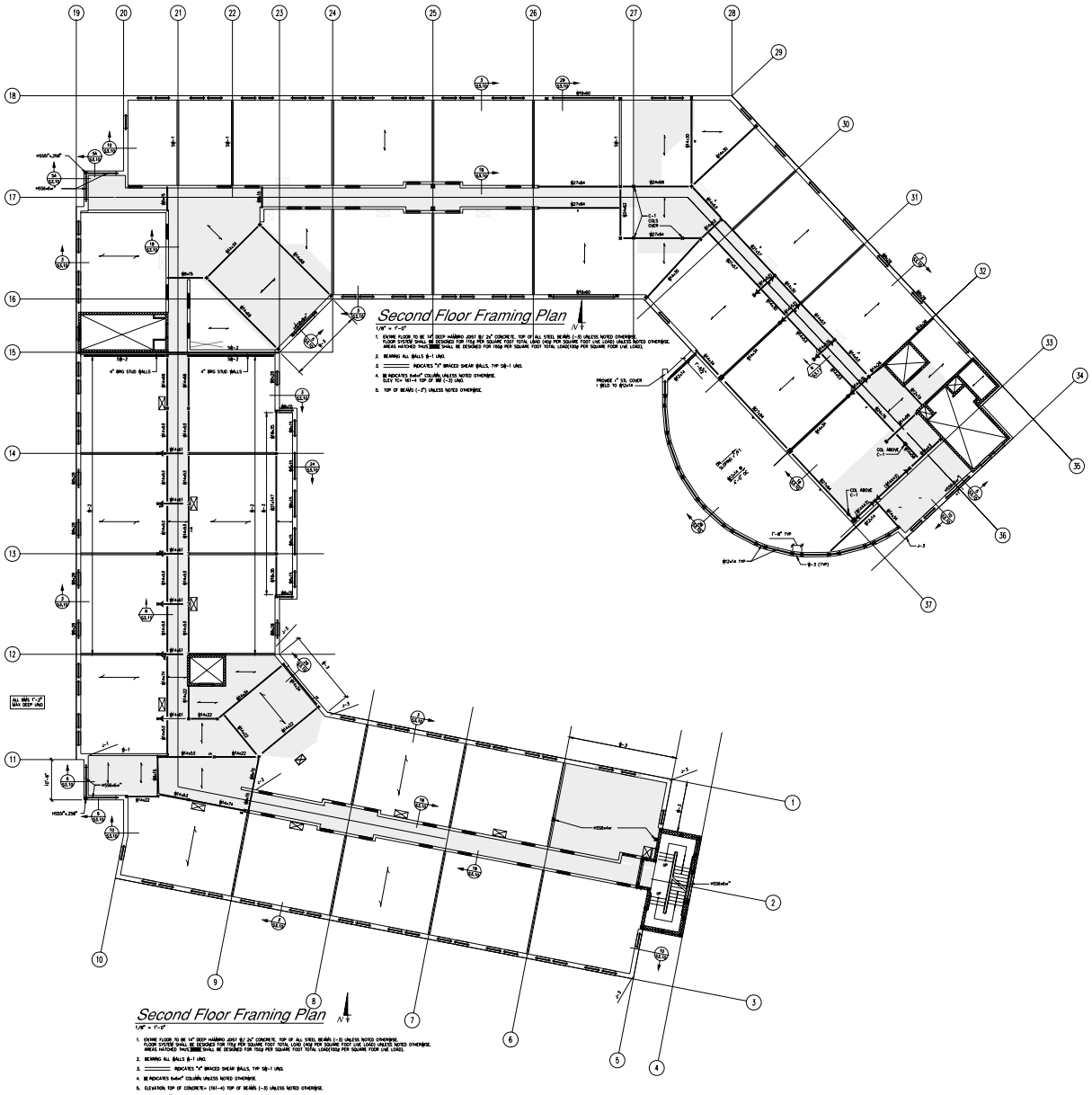
Appendix

*** Calculations not shown in this appendix are available upon request**

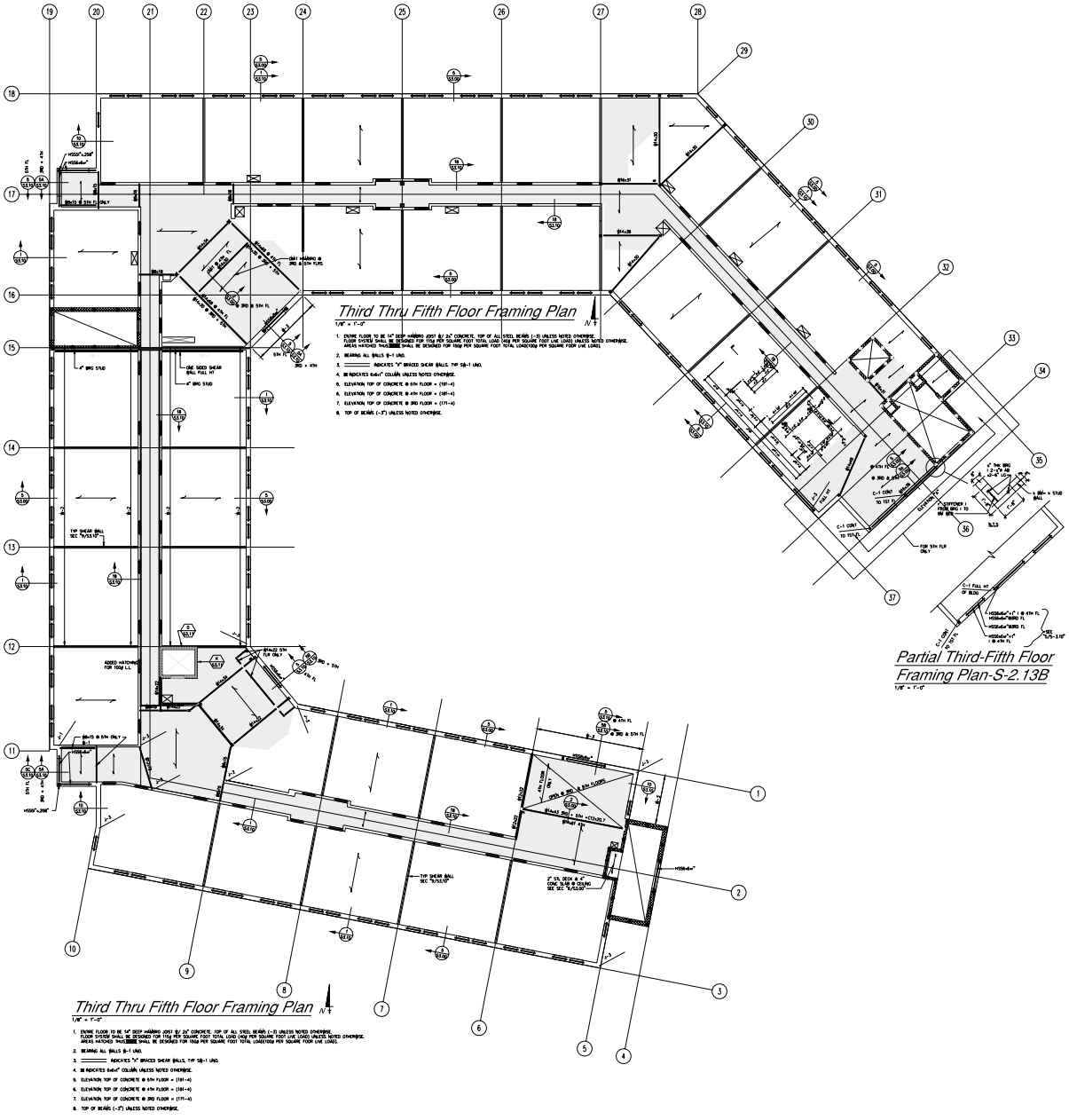
First Floor Plan:



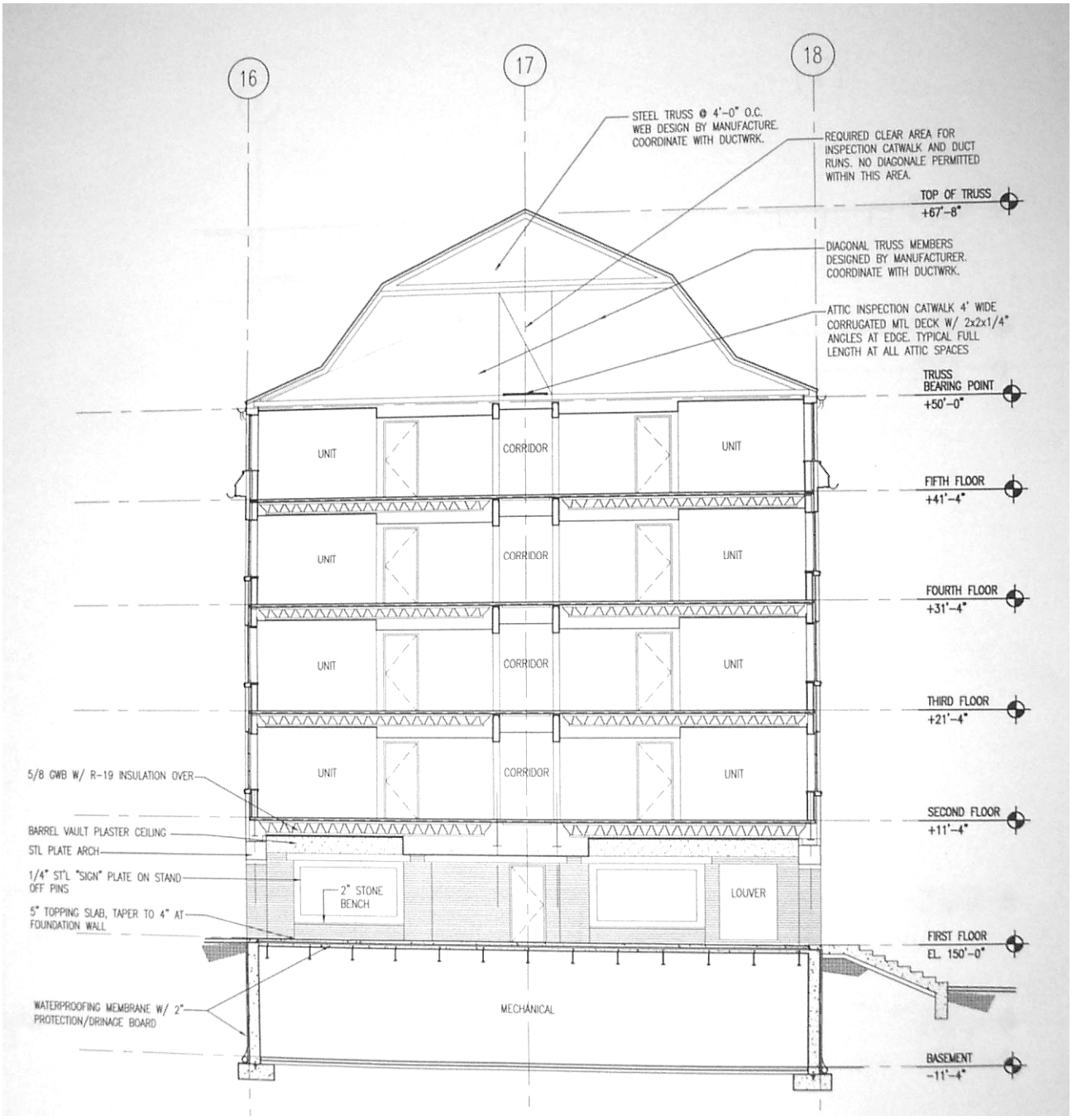
Second Floor Plan:



Third through Fifth Floor Plan:



Building Section:



Seismic Calculations:

Seismic Analysis for New Structure

- Too tall for simplified method
- Use Equivalent Lateral Force Method

$S_s = 0.225$
$R = 3.5$ → intermediate reinforced masonry shear walls
$I = 1.0$
$S_1 = 0.07$
$F_a = 1.2$, $F_v = 1.7$ } Site class C

$S_{MS} = F_a S_s = 1.2(0.225) = 0.27$
$S_{M1} = F_v S_1 = 1.7(0.07) = 0.119$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.27) = 0.18$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.119) = 0.079$

Base Shear

$V = C_s W$

$A_{floor} = 27,418.5\ ft^2$
$A_{roof} = 27,418.5\ ft^2$
$W = 4(27,418.5)(93) + 27,418.5(15) = 10,611.0^k$

- Assume 8" grouted shear walls
$W = 24.5(22.67)(10)(80)(5) = 2,221.6^k$

- Hollow 12" Bearing walls
$W = 1151(55.5)(44.33) + 1074(55.5)(38.67) = 5137^k$

$W_{TOTAL} = 10,611.0 + 2221.6 + 5137 = 17,970.0^k$

$T = C_t h_n^x = 0.02(68)^{0.75} = 0.474$ sec.

$C_s = \frac{S_{D1}}{R/I} = \frac{0.18}{3.5} = 0.051$

- Max. allowable period $= 1.7(0.474) = 0.806$ sec.

$C_{s\,max} = \frac{S_{D1}}{T(R/I)} = \frac{0.079}{0.806(3.5)} = 0.028$

$V = C_s w = 0.028(17,970) = 503.2^k$

- How many walls needed to reach this period?

$(\#\text{ of walls})(8")(22.67')(12^{in}/_{ft})(0.8)\,1.5\sqrt{1500} \geq 503.2$

# of walls $= 4.97 \approx 5$ walls in each direction

Check Period

$$T = \frac{2\pi}{3.52}\sqrt{\frac{\mu H^4}{EI}}$$

- $\mu = 0.057$ k-sec²/in²
- H = 68 ft.
- E = 1350 ksi

- Try 4 walls in each wing

$$I = 4\left(\frac{0.66(22.67)^3}{12}\right) + 4\left(\frac{0.66(22.25)^3}{12}\right) + 4\left(\frac{0.66(15.46)^3}{12}\right) = 5852.9 \text{ ft}^4$$

$T = 0.99$ sec. > 0.806

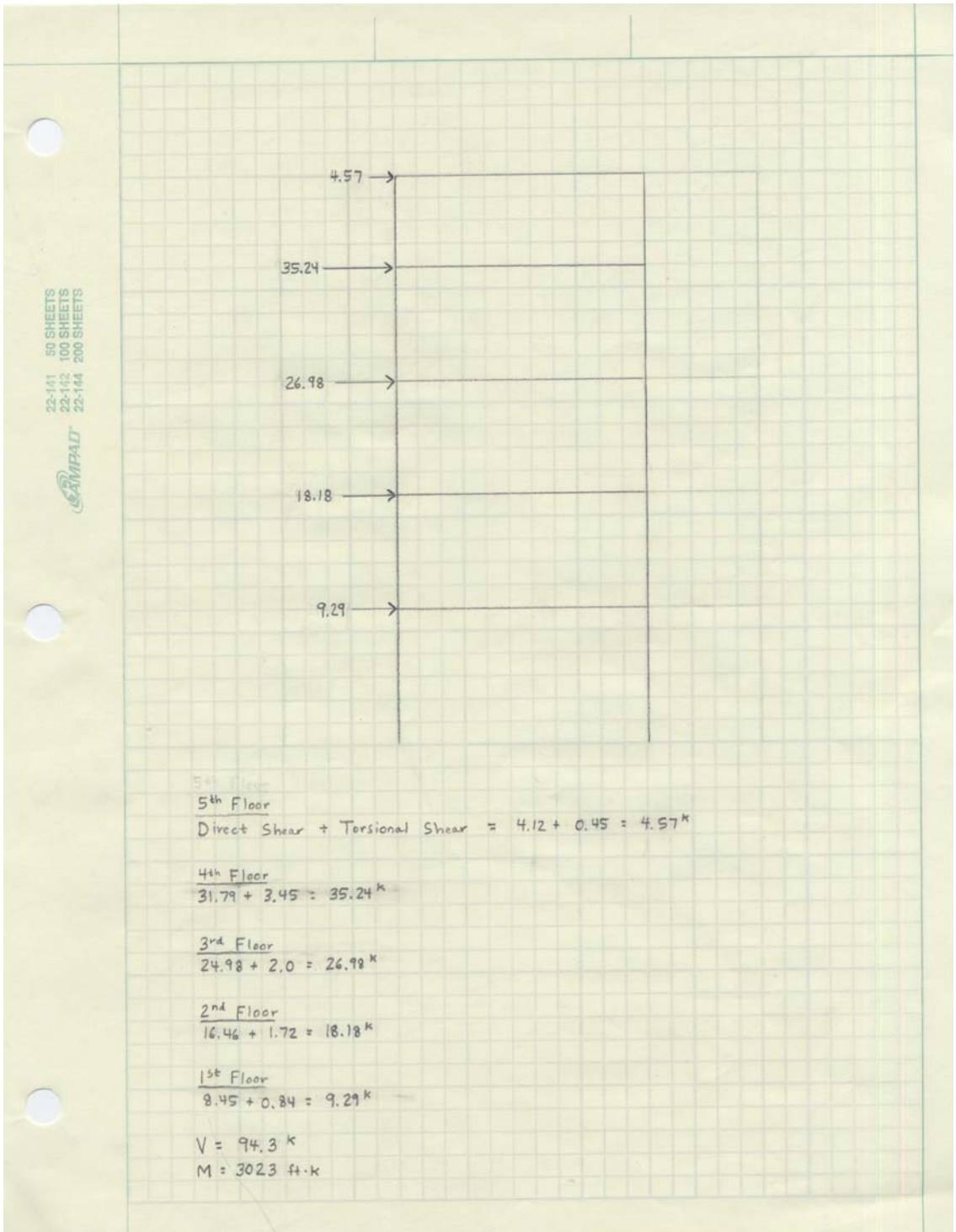
$$C_{s,max} = \frac{0.079}{0.81(3.5)} = 0.028$$

Weight of Shear walls $= 16(22.67)(10)(80)(5) = 1451^{k}$

$W = 10,611 + 1451 + 5137 = 17,199^{k}$

$V = 0.028(17,199) = 481.6^{k}$

Seismic Forces:



Shear Wall Design:

Shear Wall Design

$f_v = \frac{94,300}{8(22.67)(12)} = 43.33/1.33 = 32.5$ psi

M = 3023 k·ft
V = 94.3 k

$M/Vd = 3023/((94.3)(22.67)) = 1.41 > 1.0 \rightarrow F_v = 35$ psi

∴ NO SHEAR REINFORCEMENT REQUIRED

$f_s = \frac{M}{A_s j d}$; Assume $j = 0.85$, $d = 0.8\ell$

$32,000 = \frac{3023(12000)}{A_s(0.85)(0.8)(22.67)(12)}$

$A_s = 6.13\ in^2 \rightarrow$ Try 8-#8 Bars; $A_s = 6.32\ in^2$

#8 @ 8, d = 240 in.

$\rho = \frac{6.32}{7.625(240)} = 0.0035$, $n = 21.5$

$k = -\rho n + \sqrt{(\rho n)^2 + 2\rho n} = -0.0035(21.5) + \sqrt{(0.0035)(21.5))^2 + 2(0.0035)(21.5)} = 0.32$

$j = (1 - k/3) = 1 - 0.32/3 = 0.893$

$f_s = \frac{3023(12000)}{6.32(0.893)(240)} = 26,782 < 32,000 \rightarrow$ OK

$f_m = \frac{2(3023)(12000)}{7.625(0.893)(0.32)(240)^2} = 578/1.33 = 434$ psi $< 500 \rightarrow$ OK

∴ USE 8-#8 REINFORCING BARS @ BASE

2nd Story

V = 85.01 k
M = 1955.3 ft·k

$32,000 = \frac{1955.3(12000)}{A_s(0.85)(0.8)(22.67)(12)}$

$A_s = 3.96\ in^2 \rightarrow$ Try 5-#8 Bars; $A_s = 3.95\ in^2$, d = 252 in.

$\rho = \frac{3.95}{7.625(252)} = 0.0021$
k = 0.256
j = 0.914

$f_s = \frac{1955.3(12000)}{3.95(0.914)(252)} = 25,790$ psi $< 32,000 \rightarrow$ OK

$f_m = \frac{2(1955.3)(12000)}{7.625(0.256)(0.914)(252)^2} = 414.2/1.33 = 310.7$ psi $< 500 \rightarrow$ OK

∴ USE 5-#8 BARS @ 2nd FLOOR

3rd Story

$V = 66.79^{K}$

$M = 1105.6\ ft\cdot K$

$$32{,}000 = \frac{1105.6(12000)}{A_s(0.85)(0.8)(22.47)(12)}$$

$A_s = 2.24\ in^2 \rightarrow$ Try 3 - #8 Bars; $A_s = 2.37\ in^2$, $d = 260$ in.

$$\rho = \frac{2.37}{7.625(260)} = 0.0012$$

$K = 0.202$

$j = 0.932$

$$f_s = \frac{1105.6(12000)}{2.37(0.932)(260)} = 23{,}102\ psi < 32{,}000 \rightarrow OK$$

$$f_m = \frac{2(1105.6)(12000)}{7.625(0.202)(0.932)(260)^2} = 273.4\ psi < 500 \rightarrow OK$$

$\therefore$ USE 3 - #8 BARS @ 3rd FLOOR

4th Story

$V = 39.81^{K}$

$M = 437.7\ ft\cdot K$

$$32{,}000 = \frac{437.7(12000)}{A_s(0.85)(0.8)(22.47)(12)}$$

$A_s = 0.89\ in^2 \rightarrow$ Try 1 - #8 Bar; $A_s = 0.79\ in^2$, $d = 268$ in.

$$\rho = \frac{0.79}{7.625(268)} = 0.00039$$

$K = 0.121$

$j = 0.96$

$$f_s = \frac{437.7(12000)}{0.79(0.96)(268)} = 25{,}842\ psi < 32{,}000 \rightarrow OK$$

$$f_m = \frac{2(437.7)(12000)}{7.625(0.121)(0.96)(268)^2} = 165.1\ psi < 500 \rightarrow OK$$

$\therefore$ USE 1 - #8 BAR @ 4th FLOOR

5th Story

$V = 4.57^{K}$

$M = 39.6\ ft\cdot K$

$\therefore$ USE 1 - #8 BAR @ 5th FLOOR

Drift Calculations:

Story Drift

94.3^k → 11.33'

$$\Delta = \frac{Ph^3}{12EI}$$

- $E = 1350$ ksi
- $I = \frac{7.625(22.67(12))^3}{12} = 1.28 \times 10^7$ in^4

$$\Delta_1 = \frac{94.3(11.33(12))^3}{12(1350)(1.28\times10^7)} = 0.001 \text{ in.}$$

$$\Delta_2 = \frac{85.01(10(12))^3}{12(1350)(1.28\times10^7)} = 0.0007 \text{ in.}$$

$$\Delta_3 = \frac{66.79(10(12))^3}{12(1350)(1.28\times10^7)} = 0.0006 \text{ in.}$$

$$\Delta_4 = \frac{39.81(10(12))^3}{12(1350)(1.28\times10^7)} = 0.0003 \text{ in.}$$

$$\Delta_5 = \frac{4.57(8.67(12))^3}{3(1350)(1.28\times10^7)} = 0.0001 \text{ in.}$$

$\Delta_{1\,allow} = 0.007h_x = 0.079$ in.

$\Delta_{2\text{-}4\,allow} = 0.007h_x = 0.07$ in.

$\Delta_{5\,allow} = 0.01h_x = 0.0867$ in.

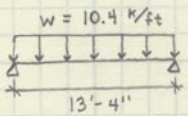
$\therefore$ OK

$\Delta_{TOTAL} = 0.0027 \text{ in.} < \frac{h}{400} = 2.04 \text{ in.}$

$\therefore$ DRIFT IS OK

Interior Beam Calculations:

Beam @ 2nd Floor Corridor



$M_{max} = \frac{wl^2}{8} = \frac{10.4(13.33)^2}{8} = 231 \text{ k·ft}$

$S_{req} = \frac{231(12)}{0.66(50)} = 84 \text{ in}^3$

- Using $L/360$ deflection criteria, $\Delta_{allow} = \frac{13.33(12)}{360} = 0.44 \text{ in.}$

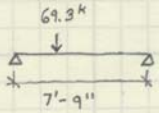
$0.44 = \frac{5(10.4)(13.33)^4(1728)}{384(29000)I_{req}}$

$I_{req} = 579.0 \text{ in}^4$

Acceptable Choices: W12 x 72
W14 x 61
W21 x 48

While the W21 x 48 is the lightest and probably most economical, it is a fairly deep member. Therefore, the W14x61 is a better choice.

2nd Floor Support Beam



$M = \frac{Pab}{l} = \frac{69.3(2.25)(5.5)}{7.75} = 110.7 \text{ k·ft}$

$S_{req} = \frac{110.7(12)}{0.66(50)} = 40.3 \text{ in}^3$

$0.26 = \frac{69.3(2.25)^2(5.5)^2(1728)}{3(29000)(7.75)I_{req}}$

$I_{req} = 104.6 \text{ in}^4$

∴ USE W14x61

Cost Analysis:

Cost Estimate

Bearing Walls:

$1151(50) + 363(50) + 711(38.67) - 16,175 = 87,019.5\ ft^2$

$(10.65\ \$/ft^2)(87,019.5 ft^2) =$ **$926,758**

Mortar for Bearing Walls:

7.5 bags / 300 blocks

$\frac{103,194.4}{0.89} = 116,095$ blocks → 2900 bags of mortar

$(2900\ bags)(7.10\ \$/bag) =$ **$20,590**

1 ton of sand / 300 blocks → 387 tons of sand

$(387\ tons)(19\ \$/ton) =$ **$7353**

Shear / Interior Walls:

$14.5(22.67)(50) + 10(22.67)(38.67) = 25,202\ ft^2$

$(25,202\ ft^2)(7.15\ \$/ft^2) =$ **$180,196**

Mortar for Shear / Interior Walls:

$\frac{25,202}{0.89} = 28,317$ blocks → 708 bags of mortar

$(708\ bags)(7.10\ \$/bag) =$ **$5027**

$\frac{28,317}{300}$ → 95 tons of sand

$(95\ tons)(19\ \$/ton) =$ **$1805**

Grout for Shear Walls:

- 8" Thk. pumped

$(16)(22.67)(50) = 18,136\ ft^2$

$(18,136\ ft^2)(3.53\ \$/ft^2) =$ **$64,020**

Scaffolding:

$(320 + 212)(11.33) = 6027.6 / 100 = 60.3$ C.S.F $(81\ \$/month)(7\ months) =$ **$34,190**

Reinforcing for Shear Walls:

16 bars (11.33 ft.) + 18(10) + 2(8.67) = 378.62 ft (16 walls) = 6058 ft. of bars

Area of #8 bar = 0.0055 ft^2

6058(0.0055) = 33.23 ft^3

Weight of Steel = 490 lb/ft^3 ⇒ 490(33.23) = 16,285 lb.

(16,285 lb)(1.16 $/lb.) = $18,891

Precast Planks:

4(27,418.5) + 4478 + 7280 = 121,432 ft^2

(121,432 ft^2)(8.40 $/$ft^2$) = $1,020,029

Concrete Topping:

(121,432 ft^2)(2.11 $/$ft^2$) = $256,222

Bond Beams:

1151(4) + 363(4) + 711(3) = 8189 ft.

(8189 ft)(14.90 $/ft) = $122,016

Waste:

(1,099,020 + 180,196 + 122,016)(0.03) = $42,037

(20,590 + 5027 + 7353 + 1805)(0.25) = $8694

Wide Flange Beams:

W14x22 – 115 ft.(35.50 $/ft) = $4082

W14x30 – 81 ft(40.50 $/ft) = $3281

W14x34 – 170 ft.(45.50 $/ft) = $7735

W14x61 – 264 ft.(75.67 $/ft) = $19,976

W14x68 – 81 ft(83.83 $/ft) = $6791

Light Gauge Metal Trusses:

(277 Trusses)(400 $/Truss) = $110,800

Pre-cast Lintels:

(856 lintels)(369 $/lintel) = $315,864

Total Cost = $3,176,357

Construction Schedule: