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Structural Design Option
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Spring Run Assisted Living
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Structural Technical Report 1

Structural Concepts / Structural Existing Conditions Report

1 Introduction

This technical report is a detailed breakdown of the building systems of Spring Run Assisted Living, which is the newest building in Willow Valley Retirement Community. It will begin to analyze the building but is only a preliminary analysis.

Like most of the other buildings the newly formed retirement community, Willow Valley, this Y shaped building utilizes load bearing masonry walls and precast concrete plank. Steel columns and lintels were used on a limited basis along with cast in place concrete for the footings and slab-on-grade. The masonry walls act as not only the main gravity load resistance but also it is the sole lateral resistance system. This 118,4000 square foot building is intended for the elderly who are nearly self-sufficient but may need help with a few everyday things. The architect wanted the feeling of an elite community so when you pull up to the building you enter under a Porte Cochere. Next, as you walk into the building is the "Grand Hotel" entrance. The corridor leading off of the grand entry is meant to give the feeling of a "Main Street". This is acquired through the presence of such things as a gift shop, dining area, café, administration, library, mail room, lounge areas and rest rooms.

This report is designed to give the basic overall structural design of the building along with the loadings which went into design. You will find a detailed description of what materials were used, the use for these materials and the extent of their use. Placed throughout this report are some of the AutoCAD drafts of sections and floor plans to help provide a better understanding of their assembly. A spot check of a typical floor framing as well as a preliminary lateral analysis of a random shear wall will be found after seismic and wind analyses. Hand calculations used during the analyses and spot checks can be found at the end of the report.

2 Building Overview

2.1 Materials

2.1.1 Cast in Place Concrete

The cast in place concrete is used for the footings and slab on grades. There are spread footings for the few columns used on this project along with wall footings for the reinforced load bearing masonry shear walls. All reinforcing in the concrete is to be of Grade 60. The following compressive strengths for the cast in place concrete are specified:

Fig 2.1.1
Typical bays in Spring Run

- Spread/wall footings   3000 psi
- Piers   4000 psi
- Slab on grade (interior)   3500 psi
- Slab on grade (exterior)   4500 psi
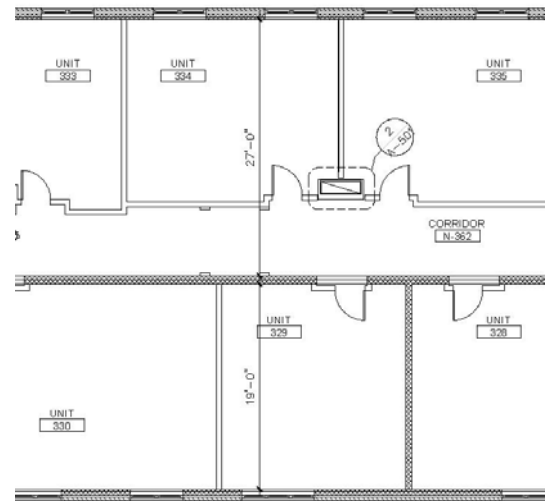
2.1.2 Masonry

Masonry walls, which range between 8" and 14", are used as the main vertical support for the building. They also are the sole method of shear resistance. The concrete masonry is specified to be $f'_m = 1500$ psi; the mortar needs to be of types M or S and the grout needs to have a compressive strength of 3000 psi. All reinforcing is to be of grade 60 steel.

2.1.3 Steel

Between several of the openings, the engineer found it to be unfeasible to use masonry piers as load bearing, therefore he utilized steel columns. The steel columns range in size from W8x31's thru W10x68's. One HSS6x6x1/2 was used on the south side of the building. Tube steel was chosen over a strong wide flange shape for aesthetic purposes. The following steel types are specified for structural steel:
- WF shapes   ASTM 992
- Square and rectangular tubing   ASTM 500, Grade B
- All other structural steel   ASTM 36

2.1.4 Precast Concrete

While the SOG's are cast in place concrete, all elevated floors including the roof are made of precast planks. The hollowcore precast planks have two average spans. The first average span is 27' and includes the depth of a typical living unit and the width of the hallway. The second average span is 19' and is only the depth of a living unit. On the upper floors, above the grand entry way, the precast planks needed to span distances of up to 31'.

2.2 Building Systems

2.2.1 Foundations

The building will rest on concrete footings. Below these footings is to be 1' of compacted structural fill and then bedrock. All concrete used in these footings is to reach a compressive strength equal to or exceeding 3000 psi after 28 days of curing. Grade 60 reinforcing was specified for all bars including longitudinal, transverse, and dowels. These dowels will either go into the piers or may continue through to the masonry walls. The piers are specified to have concrete of compressive strength of 4000psi.

2.2.2 Columns

When the architect was designing the building, he felt that it was a good idea to make the first floor have a public thoroughfare which was to resemble a retail "main street." Because of this, Masonry walls were not a feasible idea and so the plan to use steel columns in their place was implemented.

With the exception of the individual HSS column used; all the columns are to be rolled wide flange shapes. These columns are going to be connected to the footings or piers using 4 anchor bolts and base plate. The anchor bolts need to be ¾ø with 9" of embedment into the pier or footing. The base plate is specified for

a

Fig 2.2.1
Baseplate and anchor rod detail

ASTM 36 steel. The location of the anchor bolts is to be 1 at each corner of the base plate to be certain that it meets OSHA's requirements for constructability, with exception of the HSS column which is not to be used for a steel erector to climb.
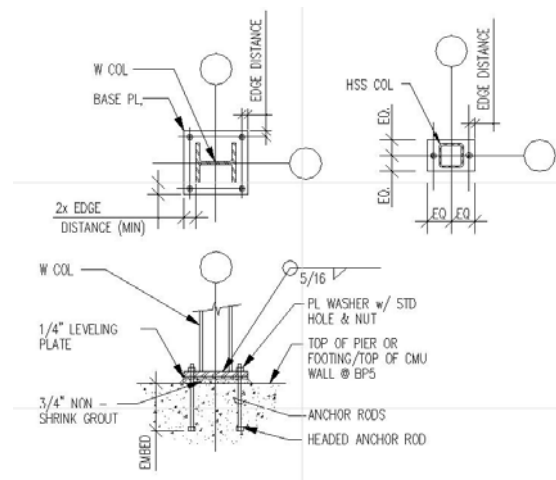
2.2.3 Masonry Walls

The masonry walls serve two purposes in this structure. They are not only for gravity loads but they are also utilized for shear resistance. Below grade, 14" CMU's are to be used. The dowels from the footings or piers are to extend into these initial courses and are called out to match the vertical reinforcing which is specified on the plans.
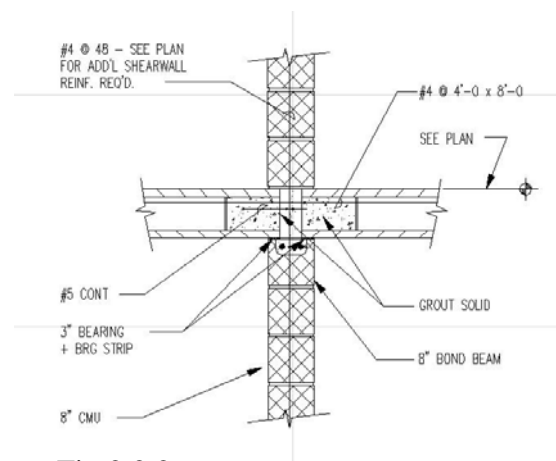
Fig 2.2.2
Grouting of planks to bearing walls

Solomon
Spring Run Assisted Living

For all the masonry walls used for shear resistance, the structural engineer used 2-#5 bars in the last two cells of the wall. Where the precast floor planks are to meet with these load bearing masonry walls, they are to be integrally grouted thus to act as one system. The typical connection for the plank to wall is a #4 which has its long leg of 4' extending into the plank. These are to be spaced at 48" O.C.

2.2.4 Structural Floor

The floor system is to be of 8" precast hollow core planks. The fabricator is responsible for the selection of plank to be used based on some criteria. The planks are to be fabricated of normal weight concrete with a compressive strength of no less than 5000 psi after 28 days of curing. Precast plank joints shall be filled with 3000 psi flowable grout. All superimposed design loads for design are placed on the structural notes sheet and are as follows:

- Floor Framing   15 PSF + Topping
- Roof Framing   25 PSF
   *Any plank member located over a corridor needs an additional 10 PSF of load.

Where the precast plank bears on a masonry wall, the engineer is asking for the cores of the planks to be grouted solid along with the cavities of the masonry walls as to make one rigid diaphragm. Rebar will also be used with the grout to increase the strength capacity.

3 Design Codes and Standards

3.1 Design Codes
International Building Code 2003 (IBC 2003)

3.2 Design Standards and Specifications
ASCE 7-02 "Minimum Design Loads for Buildings and Other Structures"
1999 Masonry Standards Joint Committee Code, Specifications and Commentaries (MSJC 1999)
ACI 318-02 "Building Code Requirements for Reinforced Concrete"

4 Design Live Loads   *Loads acquired from ASCE 7-02 Table 4.1

Roof live load...........................30	PSF
Corridors – First floor 100	PSF
Others....................40	PSF
Lobbies...................................100	PSF
Mechanical Rooms.................150	PSF
Storage (Light).......................125	PSF
Dwelling Units........................40	PSF

5 Wind Analysis

Spring Run Assisted Living meets all the necessary requirements for method 1 analysis, components and cladding.

Exposure Category	C
$\lambda =$	1.58
$V =$	90 mph
$I_W =$	1.15
$P_{s30} =$	14.8
Area 1 $P_{net30} =$	5.1 psf / -13.7 psf
Area 2 $P_{net30} =$	5.1 psf / -18.4 psf
Area 3 $P_{net30} =$	5.1 psf / -22.1 psf
$P_s = \lambda \, I \, P_{s30} =$	26.2 psf
Area 1$P_{net} = \lambda \, I \, P_{net30} =$	9.3 psf/ -24.9 psf
Area 2$P_{net} = \lambda \, I \, P_{net30} =$	9.3 psf/ -33.4 psf
Area 3$P_{net} = \lambda \, I \, P_{net30} =$	9.3 psf/ -40.2 psf

(For additional calculations see Appendix B)

5 Seismic Analysis

Seismic Design Category	A
$S_s =$	0.279 g
$S_1 =$	0.075 g
$S_{MS} =$	0.223 g
$S_{M1} =$	0.06 g
$S_{DS} =$	0.1487 g
$S_{D1} =$	0.04 g
$I_E =$	1.25
No. Stories = N =	4
$C_u =$	1.7
R =	3.5
$h_n =$	43 ft
$C_T =$	0.02
x=	0.75
$T_a = C_T (h_n)^x =$	0.3358 sec
$T_{max} = C_u T_a =$	0.571 sec
T =	0.3358 sec
$C_s = S_{DS} / (R / I_E) =$	0.0531
$C_{s\,max} = S_{D1} / (R / I_E) / T =$	0.0425
$C_{s\,min} = 0.044 S_{DS} I_E =$	0.0082
$C_{s\,min} = 0.5 S_1 / (R / I_E) =$	N/A
$C_s =$	0.0425

$F_x = C_{vx} V$ k = 1.00

$C_{vx} = w_x h_x^k / Sum (w_i h_i^k)$ (from i to n)

x	w_x		h_x		$w_x h_x^k$	C_{vx}	F_x
x = 1	$w_x =$ 4837	kips	$h_x =$ 11	ft	$w_x h_x^k =$ 53207	$C_{vx} =$ 0.12742	$F_x =$ 90.4 kips
x = 2	$w_x =$ 4837	kips	$h_x =$ 23	ft	$w_x h_x^k =$ 111251	$C_{vx} =$ 0.26643	$F_x =$ 189.1 kips
x = 3	$w_x =$ 4837	kips	$h_x =$ 33	ft	$w_x h_x^k =$ 159621	$C_{vx} =$ 0.38227	$F_x =$ 271.3 kips
x = 4	$w_x =$ 2174	kips	$h_x =$ 43	ft	$w_x h_x^k =$ 93482	$C_{vx} =$ 0.22388	$F_x =$ 158.9 kips

Sum $(w_i h_i^k) =$ 417561   V = Sum $(F_i) =$ 710 kips

Structure Weight	W = Sum $(w_i) =$	16685 kips
Base Shear	V = Cs W =	710 kips

Per table 1617.4.2				
S_{D1} >=0.4	0.3	0.2	0.15	<=0.1
C_u 1.2	1.3	1.4	1.5	1.7

$F_4 =$ 158.9 kips $h_4 =$ 43.00 ft

$F_3 =$ 271.3 kips $h_3 =$ 33.00 ft

$F_2 =$ 189.1 kips $h_2 =$ 23.00 ft

$F_1 =$ 90.4 kips $h_1 =$ 11.00 ft

V = 710 kips
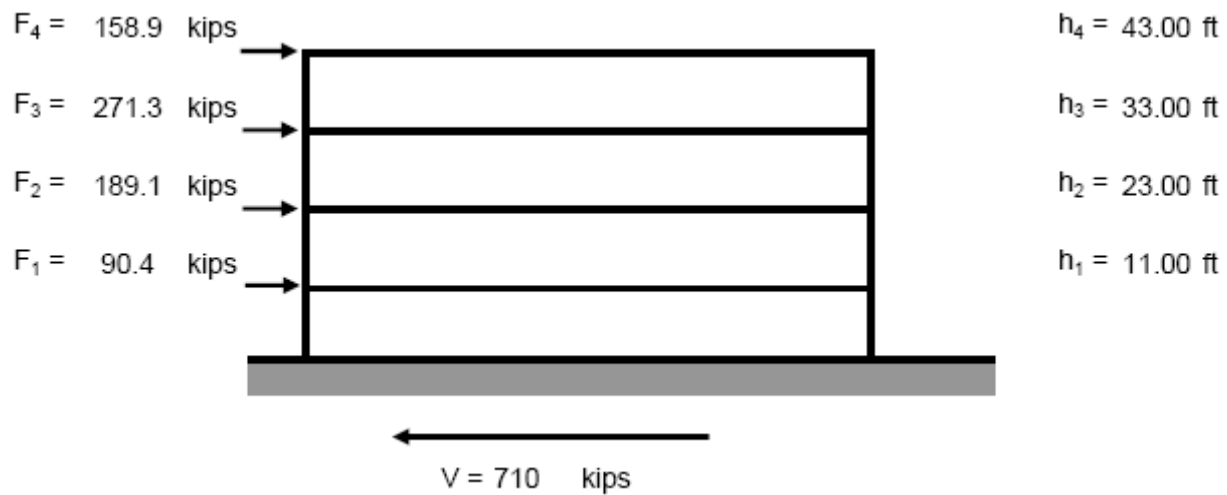
(For additional calculations see Appendix C)

7 Spot Check Results

7.1 Hollow Core Floor Plank Spot Check

The engineer asked for 8" hollow core plank to be sure that it lines up with the masonry bearing walls. I calculated the superimposed loads on the plank in the bay which appeared to be the longest span and the highest loads. Upon completion of that, I referenced PCI to confirm that there was a member which could support those loads (total superimposed load was 150 psf). Next, I contacted Conewago Precast who assured me that there were still members being produced which could support this.

7.1 Shear Wall/Lateral Spot Check

Since the lateral load distribution is to be completed later in the semester, I used an assumption that all the walls were taking an amount of shear solely determined on the length of the wall. Based on the total length of walls in the "X" direction and the percent that the wall I chose makes up of that total, the fraction of the total shear was 0.6%.

Through analysis based on the axial loads from the floors above, wind load, and seismic loading, I concluded that the masonry wall is sufficiently reinforced. In fact, the wall need not be reinforced at all because the axial force was so great that when P/A was subtracted from M/S, I concluded that the wall was not actually in then but rather compression. I further checked the wall for compression and it surpassed that test as well.

(For additional calculations on the previous 2 spot checks, see Appendix D and E respectively)

ROOF DL   - PLANK FLOORING   63 PSF
    3,174^k SUPER IMPOSED DL   25 PSF
                                          88 PSF

          AREA - 24,700 SQ. FT.

FLOOR DL   - PLANK FLOORING   63 PSF
1-4 SAME   TOPPING   15 PSF
2,545^k SUPERIMPOSED DL   25 PSF
+2,292^k AREA - 24,700 SQ. FT   103 PSF
4,837^k

          EXT WALLS (1200')   105 PSF
          INT WALLS (1000')   65 PSF
          12' TALL

                     * NOTES
                        MASONRY WALLS - 65 PSF
                        BRICK VENEER - 40 PSF

WIND CALCS

*THIS BUILDING MEETS THE REQUIREMENTS FOR COMPONENTS AND CLADDING.

$I = 1.15$ Tbl. 1.1
$V = 90$ MPH   Fig. 6.1
EXPOSURE CATEGORY C
$\lambda = 1.58$ Fig 6.2
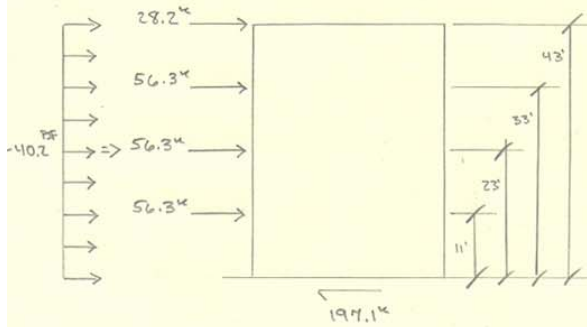$P_{s30} = 14.4$ Fig 6.2
$P_s = \lambda I P_{s30}$

$\qquad P_s = 26.2$ PSF

$P_{net30} \Rightarrow$ AREA 1 $\rightarrow$ $^+5.1$; $^-13.7$ } THESE VALUES ARE DIFFERENT THAN THE ENG.
$\quad$ (PSF) AREA 2 $\rightarrow$ $^+5.1$; $^-18.4$ } POSSIBLY BECAUSE OF DIFFERENT AMOUNTS
$\qquad$ AREA 3 $\rightarrow^+5.1$; $^-22.1$ } OF AREA INCLUDED [HE WAS CONSERVATIVE]

$P_{net} = \lambda I P_{net30}$

$\quad P_{net} \Rightarrow$ AREA 1 $\quad ^+9.3 \quad ^-24.9$
$\quad$ (PSF) $\quad$ AREA 2 $\quad ^+9.3 \quad ^-33.4$
$\qquad$ AREA 3 $\quad ^+9.3 \quad ^-40.2$

LENGTH $= 140' \rightarrow ^-40.2 \cdot 140 = 5.628^k$

SEISMIC CALCS.

BASIC STRUC. SYSTEM - BEARING WALLS

SEISMIC RESISTING SYSTEM - INTERMEDIATE REINF SHEAR WALLS

SEISMIC USE GROUP —————— II
         Tbl. 9.1.3

OCCUPANCY IMPORTANCE FACTOR —— $I = 1.25$
         Tbl. 9.1.4

$S_s = 0.279g$ (INTERPOLATED)
         Fig. 9.4.1.1 (a)

$S_1 = 0.075g$ (INTERPOLATED)
         Fig. 9.4.1.1 (b)

SITE CLASS ——————————— A
         GEOTECH REPORT

$F_a = 0.8$
         Tbl. 9.4.1.2.4a

$F_v = 0.8$
         Tbl. 9.4.2.4b

$S_{MS} = F_a S_s$ & $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.8)(0.279g)$

         $S_{DS} = 0.149g$ - THIS CONFIRMS S_{DS} ON GENERAL NOTES ON
                              STRUC. DWGS.

$S_{M_1} = F_v S_1$ & $S_{D1} = \frac{2}{3} S_{M_1} = \frac{2}{3}(0.8)(0.075g)$

         $S_{D1} = 0.04g$ - THIS CONFIRMS S_{D1}

SEISMIC USE GROUP ——————— A
         Tbl. 9.4.2.1a & Tbl. 9.4.2.1b

$R = 3.5$ - THE ENG. USED 3.0 WHICH IS MORE CONSERVATIVE OF
                 A VALUE
         Tbl. 9.5.2.2

$C_v = 1.7$
         Tbl. 9.5.5.3.1

$C_t = 0.02$ & $x = 0.75$

     REMAINDER OF CALCULATIONS PERFORMED USING EXCEL.
         FILE AVAILABLE UPON REQUEST

$T_a = C_T(h_n)^x = 0.3358$ SEC

$T_{MAX} = C_u T_a = 0.571$ SEC

$T \longrightarrow$ $T = 0.3358$ SEC

$C_S = \dfrac{S_{DS}}{(R/I)} = 0.0531$

$C_{SMAX} = \dfrac{S_{D1} T}{(R/I)} = 0.0425$

$C_{SMIN} = 0.044 S_{DS} I = 0.0082$

$\vdash_S \longrightarrow$ $C_S = 0.0425$

$W_{x_1} = 4837^k$, $h_x = 11'$ $W_x h_x^k = 53204$
$C_{vx} = 0.12742$ $F_x = 90.4^k$

$W_{x_2} = 4837^k$, $h_x = 23'$ $W_x h_x^k = 111251$
$C_{vx} = 0.26643$ $F_x = 189.1^k$

$W_{x_3} = 4837^k$, $h_x = 33'$ $W_x h_x^k = 159621$
$C_{vx} = 0.38227$ $F_x = 241.3^k$

$W_{xR} = 2174^k$, $h_x = 43'$ $W_x h_x^k = 93482$
$C_{vx} = 0.22588$ $F_x = 158.9^k$

$\Sigma W_x = 16{,}685^k$
$\Sigma(h_x^k W_x) = 417561$
$\Sigma F_x = 710^k$

OVERTURNING MOMENT $= (158.9 \cdot 43) + (241.3 \cdot 33) + (189.1 \cdot 23) + (90.4 \cdot 11) = 21{,}129$'k

FLOOR SYSTEM
SPOT CHECK

SP 2
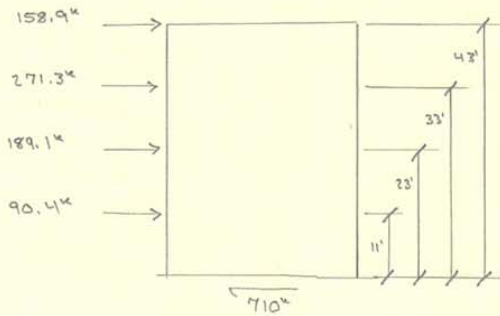
FLOOR LOADS ON PLANK $\begin{cases} SDL - 25\,psf \\ LL - 125\,psf \end{cases} = 150\,psf$

(THIS IS IN A STORAGE AREA)

SPAN IS 26'-0" (ROUNDED UP)

TOPPING TO BE ADDED (2" THICK) — CONEWAGO PRECAST

ACCORDING TO A FABRICATOR, THERE
IS AN 8" HOLLOW CORE PLANK W/
2" OF TOPPING THAT WILL SUPPORT
$125\,psf$ OF LL AND $25\,psf$ OF SUPERIMPOSED
DL.

THE ACTUAL DESIGN OF THIS MEMBER WOULD
BE LEFT UP TO THE FABRICATOR.

PLANK TO BE CHECKED

EAST WING

LATERAL SYSTEM
SPOT CHECK
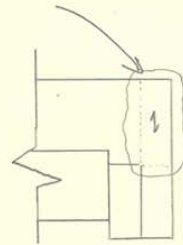
TOTAL LENGTH OF SHEAR WALLS IN

    X-DIRECTION - 1314'

    Y-DIRECTION - 945'

TRIB WIDTH OF FLOOR LOAD IS
13'
LOADING IS   FLOOR/ROOF

    LL - 40 psf / 30 PSF
    DL - 88 PSF / 63 PSF
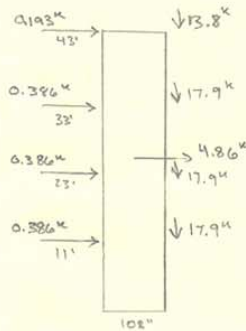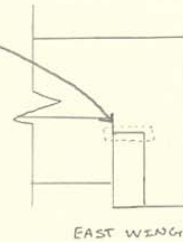   SDL - 25 PSF / 25 PSF
    $\overline{153 PSF / 118 PSF}$

WALL CHOOSEN TO SPOT CHECK

    9' IN X-DIRECTION

    $\frac{9'}{1314'} = 0.685\%$ OF TOTAL SHEAR

WALL IS 8" CMU w/ #4's @ 48" O.C.

43' TALL → SELF WEIGHT IS $45 PSF \cdot 43' \cdot 9' = 17,415$ lbs
    TOTAL LOAD ON WALL $= 67.5^k$

EAST WING

f'm = 1500 psi
GR 60 REIN.

$S = \frac{1}{6} \cdot 7.625 \text{ in} \cdot (108")^2 = 14823 \text{ in}^3$

$A = 7.625 \text{ in} \cdot 108 \text{ in} = 823.5 \text{ in}^2$

$f_t = M/S - P/A = [(0.193 \cdot 43 \cdot 12 + 0.386 \cdot 33 \cdot 12 + 0.386 \cdot 23 \cdot 12 +$
$0.386 \cdot 11 \cdot 12 + 4.86 \cdot \frac{43}{2} \cdot 12)/14823 \text{ in}^3 - 84.9/823.5] \cdot 1000$

$f_t = -75.4 \text{ psi} < 68 \text{ psi}$ OR IN TEN.

$F_g = 0.25 f'm R = 0.25 (1500 psi)(0.843) = 316 psi$ ✓

THIS WALL IS OK IN SHEAR DURING FULL LOADING