Depth Study: Alternate Structural System Design

Pre-cast Hollow Core Planks:

The floor system of the building must be designed to resist the gravity loads applied from both the dead load and live load. The dead load consists of the total weight of the materials as well as a superimposed load. Hollow core planks were selected because of their many advantages. The construction process for these planks is fairly simple, allowing it to be done quickly. They are very durable and fire resistant. Also, they are manufactured with high strength concrete, giving them excellent loading capacity. The disadvantage of hollow core planks in comparison to a Hambro composite system is that it makes the total building weight higher. This added weight increases the seismic forces on the building.

The design floor load calculation is shown below. The superimposed load accounts for furniture and other permanent fixtures. The live load accounts for the load from the occupants.

Superimposed Dead Load = 25 psf

Live Load = 40 psf

Total Load = 1.2(25) + 1.6(40) = 94 psf

Using the PCI Design Handbook's provided load tables for hollow core planks, it was determined that 4'-0" wide x 8" deep lightweight planks with a 2" normal weight concrete topping are sufficient. The reinforcing for these planks is 6-$^{3}/_{8}$" straight pre-stressing strands located $1^{1}/_{2}$" up from the bottom of the planks. The typical plank cross section is shown on the following page along with the corresponding load tables. Lightweight concrete was used for this design because it decreases the

total weight of the building. Lightweight 8" planks are actually lighter than 6" normal weight planks. In addition to less weight, less reinforcing is needed because of the added depth.

Strand Pattern Designation

76–S
- S = straight
- Diameter of strand in 16ths
- No. of strand (7)

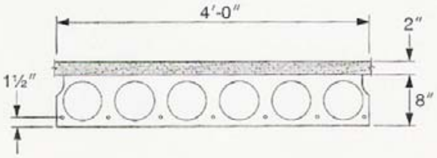
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load. Check availability of lightweight sections.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
- 346 — Safe superimposed service load, psf
- 0.3 — Estimated camber at erection, in.
- 0.4 — Estimated long-time camber, in.

HOLLOW-CORE

4′-0″ x 8″
Lightweight Concrete



f'_c = 5,000 psi
f'_{ci} = 3,500 psi

Section Properties

		Untopped		Topped	
A	=	215	in^2	—	
I	=	1,666	in^4	3,529	in^4
y_b	=	4.00	in.	5.70	in.
y_t	=	4.00	in.	4.30	in.
S_b	=	416	in^3	619	in^3
S_t	=	416	in^3	821	in^3
b_w	=	12.00	in.	12.00	in.
wt	=	184	plf	272	plf
		46	psf	68	psf
V/S	=	1.92	in.		

4LHC8+2

Table of safe superimposed service load (psf) and cambers (in.) **2″ Normal Weight Topping**

Strand Designation Code	Span, ft 16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38
66-S	320	277	242	211	186	163	144	127	113	100	88	78	69	60	53	45							
	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2							
	0.4	0.5	0.5	0.5	0.5	0.4	0.4	0.3	0.3	0.2	0.0	−0.1	−0.3	−0.5	−0.7	−1.0							
76-S		327	286	251	222	196	174	155	138	123	109	98	87	77	69	61	52	43					
		0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3					
		0.6	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.1	−0.1	−0.3	−0.6	−0.9	−1.2					
58-S				327	290	258	231	206	185	167	150	135	122	110	99	90	81	72	62	53	45		
				0.8	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.0	1.0	1.0	0.9	0.8	0.7		
				0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	−0.2	−0.5	−0.9	−1.3		
68-S					323	304	278	250	225	204	184	167	151	138	125	114	103	93	83	73	64	56	48
					1.1	1.1	1.2	1.3	1.3	1.4	1.5	1.5	1.5	1.6	1.6	1.6	1.6	1.6	1.5	1.5	1.4	1.3	1.2
					1.2	1.3	1.3	1.4	1.4	1.4	1.4	1.3	1.3	1.2	1.1	0.9	0.8	0.6	0.3	0.0	−0.3	−0.7	−1.2
78-S					332	313	297	279	263	238	216	197	179	163	149	136	125	113	102	91	81	72	64
					1.3	1.4	1.5	1.6	1.7	1.7	1.8	1.9	2.0	2.0	2.1	2.1	2.1	2.2	2.2	2.2	2.1	2.1	2.0
					1.5	1.6	1.7	1.7	1.8	1.8	1.8	1.8	1.8	1.8	1.7	1.6	1.5	1.3	1.1	0.9	0.6	0.2	−0.1

Figure 4: Hollow Core Plank Design Table

The typical exterior bearing wall detail is shown below.

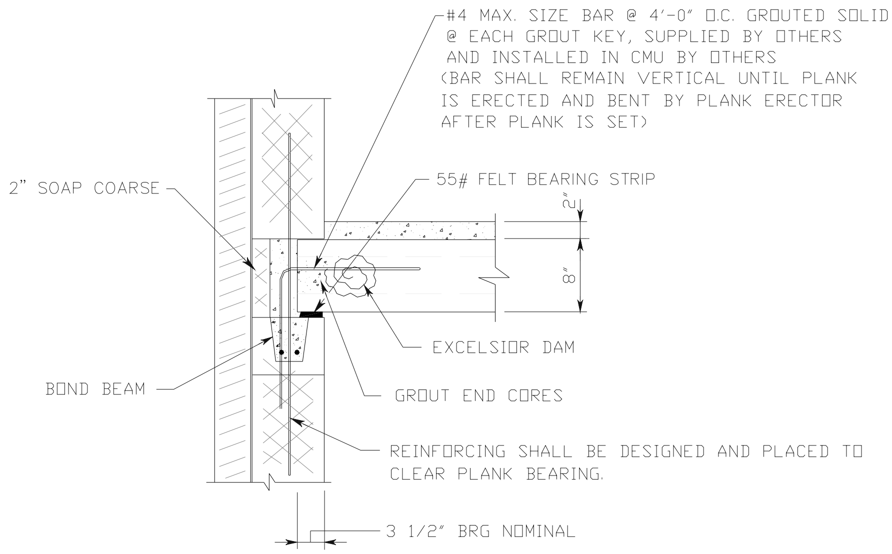


Figure 5: Exterior Bearing Wall Detail

The interior plank bearing detail is different on parts of the second floor than it is on the upper floors. Some of the second floor planks bear on wide flange steel beams along the corridor. This is because more open space is required on the floor below. The rest of the planks are supported by interior load bearing masonry walls. The two typical interior bearing wall details are shown on the next page.

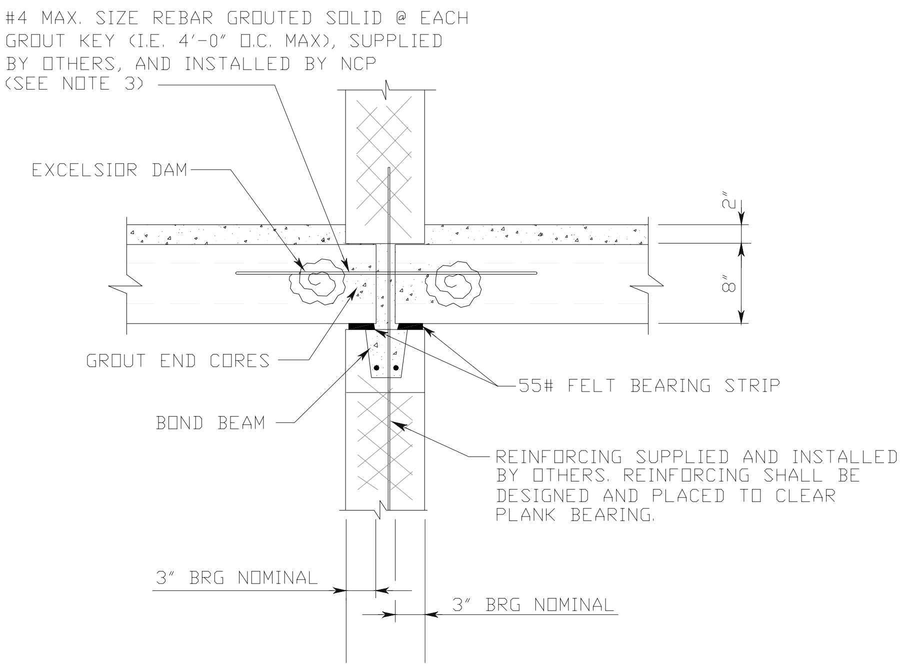


Figure 6: Interior Bearing on Masonry

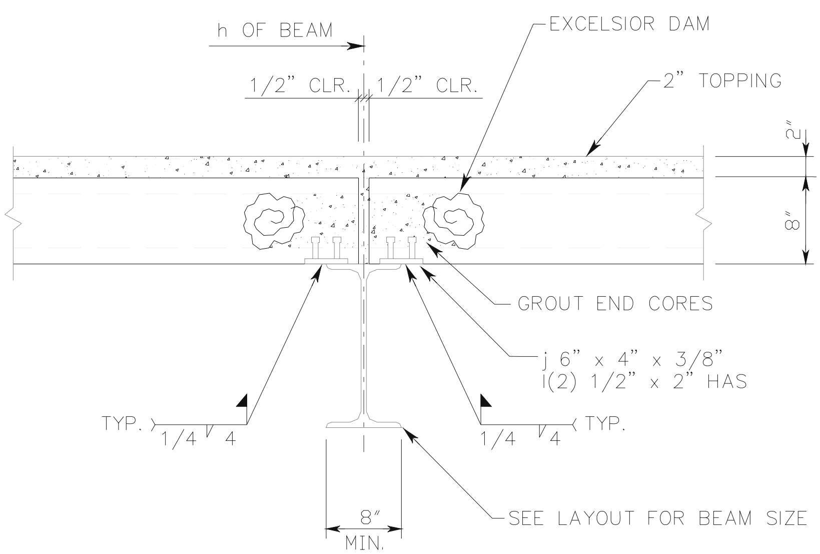


Figure 7: Interior Bearing on Wide Flange Beam

Masonry Bearing Wall System:

The bearing walls for George Read Hall are located around the exterior of the building as well as along the interior corridor in order to support the hollow core planks. The walls were designed using the empirical design method. In order to design using this method, several criteria must be met:

- length/width ≥ 4:1
- Design wind speed ≤ 110 mph
- Seismic Design Category A, B, C
- height/thickness ≤ 18

The exterior walls have a tributary width of 11'-4". The interior walls have the same 11'-4" tributary width as well as a 3'-0" width from the corridor. The final summarized design calculations are shown below. The final wall stresses are in pounds per square inch.

Exterior Wall:

Floor No.	Plank Size	Self-weight	Total DL	Live Load	Load from wall above	Load from supported floor	Estimated wall weight	Wall load	Wall Stress
5	8" + 2	68	93	40	-	1529.5	555	2084.5	14.5
4	8" + 2	68	93	40	2084.5	1529.5	555	4169	29.0
3	8" + 2	68	93	40	4169	1529.5	555	6253.5	43.4
2	8" + 2	68	93	40	6253.5	1529.5	555	8338	57.9

Interior Wall:

Floor No.	Plank Size	Self-weight	Total DL	Live Load	Corridor Live Load	Load from wall above	Load from supported floor	Estimated wall weight	Wall load	Wall Stress
5	8" + 2	68	93	40	100	-	1829.5	555	2384.5	16.6
4	8" + 2	68	93	40	100	2384.5	1829.5	555	4769	33.1
3	8" + 2	68	93	40	100	4769	1829.5	555	7153.5	49.7
2	8" + 2	68	93	40	100	7153.5	1829.5	555	9538	66.2

These calculated wall stresses were then compared to the empirical design method allowable compressive stresses in the National Concrete Masonry Association TEK-Notes. The tables shown are for 12” hollow blocks. Calculations were also done for hollow 8” blocks as well as grouted 8” blocks. It was determined that the compressive stresses on the hollow 8” blocks exceeded the allowable values. Grouted 8” blocks presented a possible solution from a strength perspective. However, grouting all of the cells creates sufficiently more labor costs. Because of this, hollow 12” blocks were chosen as a more economical solution. The allowable stress values are shown in the figure below.

Gross area compressive strength of unit, psi (MPa)	Allowable compressive stresses based on gross cross-sectional area, psi (MPa)[a] Type M or S mortar	Type N mortar
Solid concrete brick:		
8000 (55) or greater	350 (2.41)	300 (2.07)
4500 (31)	225 (1.55)	200 (1.38)
2500 (17)	160 (1.10)	140 (0.97)
1500 (10)	115 (0.79)	100 (0.69)
Grouted concrete masonry:		
4500 (31) or greater	225 (1.55)	200 (1.38)
2500 (17)	160 (1.10)	140 (0.97)
1500 (10)	115 (0.79)	100 (0.69)
Solid concrete masonry units:		
3000 (21) or greater	225 (1.55)	200 (1.38)
2000 (14)	160 (1.10)	140 (0.97)
1200 (8.3)	115 (0.79)	100 (0.69)
Hollow concrete masonry units:		
2000 (14) or greater	140 (0.97)	120 (0.83)
1500 (10)	115 (0.79)	100 (0.69)
1000 (6.9)	75 (0.52)	70 (0.48)
700 (4.8)	60 (0.41)	55 (0.38)
Hollow walls (noncomposite masonry bonded[b])		
solid units:		
2500 (17) or greater	160 (1.10)	140 (0.97)
1500 (10)	115 (0.79)	100 (0.69)
hollow units	75 (0.52)	70 (0.48)

Figure 8: Allowable Compressive Stresses

As seen in the figure on the previous page, the allowable compressive stress for masonry units with a 1500 psi unit strength is 100 psi for Type N mortar and 115 psi for Type M or S mortar. Both of these values are higher than the maximum calculated stress; therefore, Type N mortar should be used because it is the cheapest. To help control shrinkage and other movements, hot-dipped, galvanized truss type wire reinforcement will be provided in every other course. Additionally, as shown in the hollow core plank details above, a course of bond beam blocks is required for bearing of the planks.

As mentioned above, parts of the second floor framing consist of wide flange beams due to open space on the first floor. Because of the added weight of the upper floor masonry bearing walls, these beams needed to be resized to accommodate the new loads. As shown in Figure 2, a W14x53 spans 13’-3” and frames into a W14x61 that transfers the load into the supporting columns. After applying the new loads, the W14x53 needs to be increased to a W14x61. The W14x61 support beam does not need to be increased in size.

Reinforced Masonry Shear Walls:

The existing lateral resisting system consists of X-braced shear walls. The walls are cold formed metal studs with 16 gauge, 50 ksi metal straps. Shear walls are located on each side of the double loaded corridor. The typical distance between walls is 26’-8”. At the fifth and fourth floors, the shear walls are constructed with 2-3” straps. 2-4” straps are used on the third and second floor, and 2-$4^{1}/_{2}$” straps are used on the first floor. The typical shear wall details are shown on the next page.

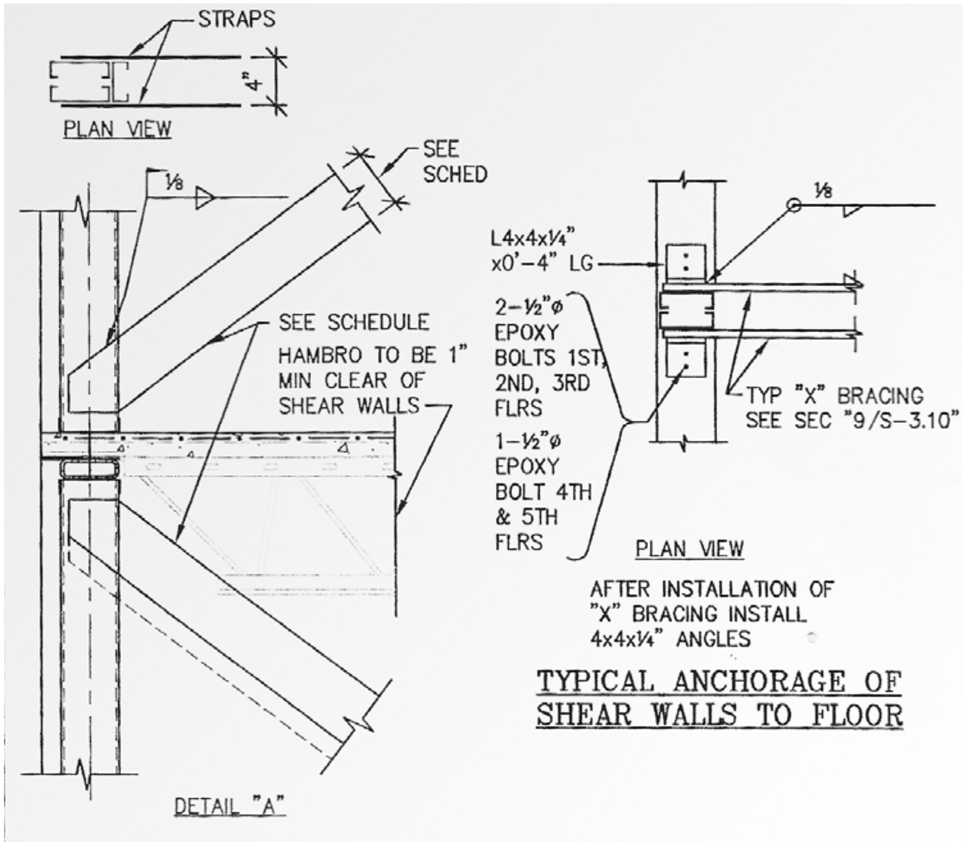


Figure 9: Existing Shear Wall Detail

As seen on the details, the vertical edge members of the shear walls are metal studs. The straps are welded to the vertical studs with a $^{1}/_{8}$" thick fillet weld. This shear wall system acts virtually as a vertical cantilevered truss.

It was previously determined that the seismic forces control the lateral force resisting system design. This becomes even more evident in the new system because of the added weight of the masonry system. The seismic story forces are calculated in the following table. More detailed seismic calculations can be seen in the appendix.

Level	w_x	h_x	$w_x h_x^{1.0}$	C_{vx}	F_x	Shear
Roof	411.3	50	20565	0.048654	23.43	-
5	3871.6	41.333	160024.8	0.378594	182.33	23.43
4	4075	31.333	127682	0.302076	145.48	182.33
3	4075	21.333	86931.98	0.205668	99.05	327.81
2	4239.2	11.333	48042.85	0.113662	54.74	426.86
Base	-	-	-	-	481.6	481.6
			422681.6	1		

In addition to changing the shear walls from X-braced metal straps to reinforced masonry shear walls, the number of shear walls was reduced. Decreasing the number of shear walls on each floor helps to lower the cost because grouting and reinforcing is not required in walls not designed to resist shear.

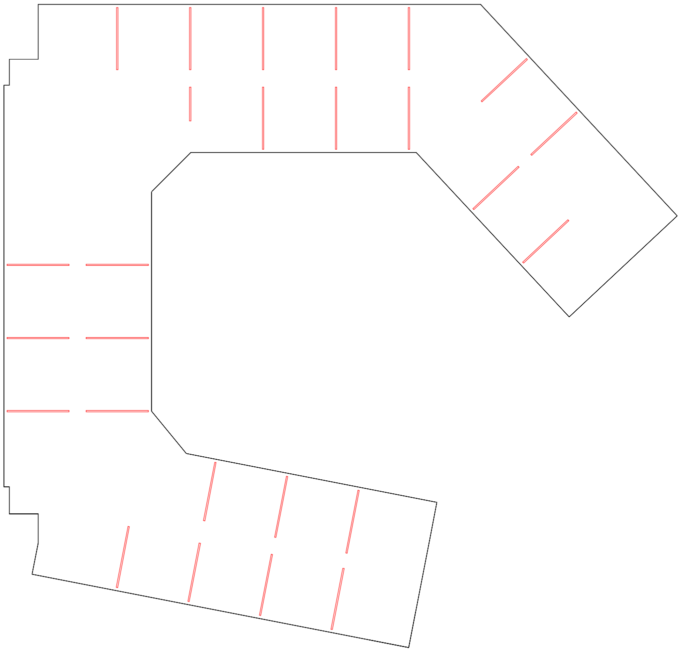


Figure 10: Existing Shear Wall Layout

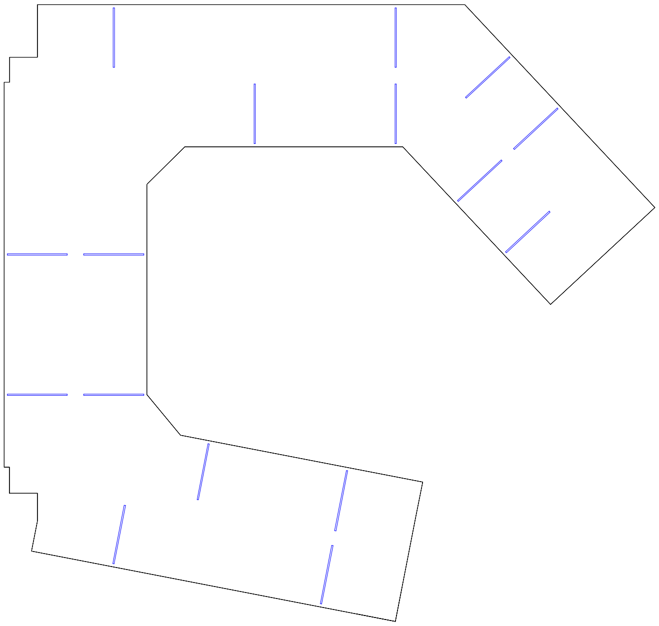


Figure 11: New Shear Wall Layout

The use of four shear walls in each wing helps to maintain that the building acts rigidly under lateral loads. Also, the placement of the walls was chosen to give the greatest resistance to torsional shear.

The story shears are distributed to the shear walls according to the rigidities of the walls. The rigidities were calculated according to the following equation:

$R = (Et)/(4(h/l)^3 + 2.78(h/l))$, where

- -E = modulus of elasticity
- -t = thickness of the wall
- -h = height of the wall
- -l = length of the wall

After distributing the story forces to the shear walls, the critical shear wall loading was determined. The design loading is shown below.

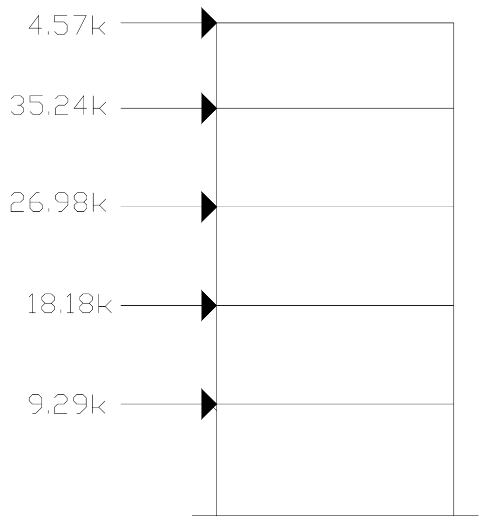


Figure 12: Design Shear Wall Loading

These shear walls must be designed to resist the direct shear forces as well as the moment created by the shear forces. The greatest shear force and resulting moment is created at the base of the building.

$$V = 94.3 \text{ kips}$$

$$M = 3023 \text{ ft-kips}$$

The design process began by assuming 8" grouted concrete masonry units. The shear stress in the masonry is determined from ACI 530-02 Structural Design Provisions section 2.3.5.2.1:

$f_v = V/(bd)$

$f_v = (94{,}300 \text{ lb.})/((7.625 \text{ in.})(22.67 \text{ ft.})(12 \text{ in/ft})) = 45.5/1.33 = 34.1 \text{ psi}$

The 1.33 factor takes into account an allowable stress increase from the code. The allowable shear stress is determined from section 2.3.5.2.2(b) of ACI 530-02. This allowable stress is based upon the ratio of M/(Vd). Where M/(Vd) =1, $F_v = \sqrt{f'_m}$ but not to exceed 35 psi. This allowable shear stress is greater than the calculated stress in the masonry. Therefore, no shear reinforcement is needed, and 8" grouted blocks can be used.

As previously mentioned, the walls must also be designed to resist the moment created by the shear forces. This is accomplished by providing vertical flexural reinforcement. Thus, the shear walls act as rectangular beams in order to resist the moments. A sample calculation is shown below.

$$f_s = M/(A_s jd);\ \text{Assume, } j=0.85,\ d=0.8l$$

Solving for A_s gives a trial steel area of 6.13 in^2. Thus, try 8-#8 bars.

$d = 240$ in.

$\rho = 0.0035$

$n = 21.5$

$k = 0.32$

$j = 0.893$

$f_s = 3023(12000)/(6.32(0.893)(240)) = 26{,}782$ psi

$f_m = (2(3023)(12000))/(7.625(0.893)(0.32)(240)^2) = 578$ psi

$F_s = 24{,}000(1.33) = 32{,}000$ psi

$F_m = {}^1/_3 f'_m = 500(1.33) = 666$ psi

Both of the allowable values are greater than the actual stresses. Therefore, 8-#8 bars can be used at the base of the building. These reinforcing bars will be placed at each end of the shear walls to account for loading in the opposite direction of the analysis. One bar will be placed in each core of the blocks until the required reinforcing is met. Similar calculations were performed for the remaining floors. The results are summed in the table below. Detailed calculations can be seen in the appendix.

Floor	**Reinforcing**
5	1-#8 Bar
4	1-#8 Bar
3	3-#8 Bars
2	5-#8 Bars
Base	8-#8 Bars

The interstory drift and total building drift were also calculated. The interstory drift was then compared to the allowable values set forth in ASCE 7-98 Table 9.5.2.8. All of the calculated story drifts were less than the allowable values. The total building drift was compared to the industry standard of L/400. The calculated total building drift was well below this allowable limit. Drift calculations are available in the appendix.

Summary:

In summary, the required floor system to resist the applied loads is pre-cast 8” deep lightweight hollow core planks with a 2” normal weight concrete topping. The required bearing wall system to support these planks is 12” thick hollow concrete masonry units. In some areas, W14x61 wide flange beams are required to support the planks. In addition to the hollow 12” blocks, reinforced 8” grouted masonry shear walls are required to resist the seismic forces. The reinforcing for these shear walls consists of #8 bars.