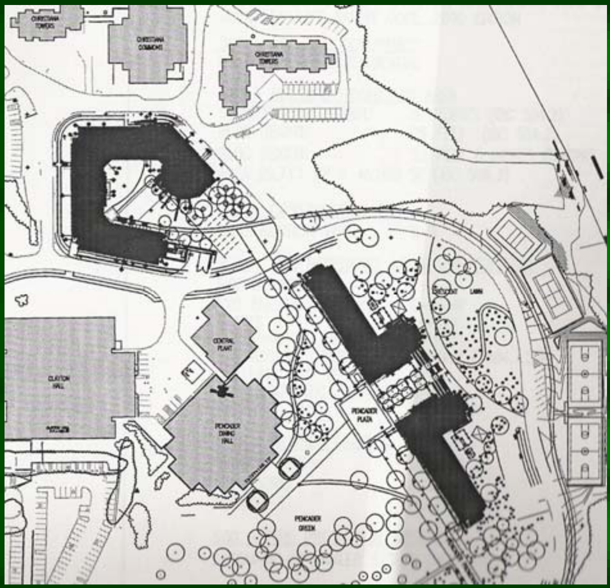
GEORGE READ HALL
UNIVERSITY OF DELAWARE
NEWARK, DELAWARE

Eric Alwine – Structural Option

Senior Thesis – April 2006
Department of Architectural Engineering
The Pennsylvania State University

George Read Hall
University of Delaware
Newark, DE

PROJECT OVERVIEW:
- 5 stories
- approx. 129,000 square feet
- Residential housing for students
- Construction: May 2004-August 2005
- $27 million



PRIMARY PROJECT TEAM:
Owner: The University of Delaware
Architect: Ayers/Saint/Gross
Construction Manager: Whiting-Turner
Structural Engineer: Skarda & Associates
Civil Engineer: Tetra Tech, Inc.
Code Consultant: Koffel & Associates
M.E.P. and Fire Protection: Sebesta blomberg & Associates

ARCHITECTURAL:
- Georgian style
- Largest of three buildings in new residential complex
- "U-Shaped"
- Pedestrian bridge to main campus

STRUCTURAL:
Foundation: Continuous & Spread footings
Superstructure: Cold formed metal studs
Floor: Hambro composite floor system
Lateral System: x-braced shear walls
Envelope: Brick or simulated stone façade

MECHANICAL:
- 750 ton chiller
- Variable Air Volume Units
- 50-5,000 CFM

LIGHTING/ELECTRICAL:
- 480/277V & 208/120V
- 3-phase, 4-wire
- 350 kW Diesel generator back-up
- Fluorescent/HID lighting

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Executive Summary:

George Read Hall is a new five story dormitory on the campus of The University of Delaware. The building is approximately 129,000 square feet. Its architecture is accentuated by its "U-shape" and Georgian style. George Read Hall is the largest of three new buildings being built to replace the existing Pencader residential complex.

The existing structural system of George Read Hall is composed of a Hambro composite floor system with light gauge metal stud bearing walls. The lateral force resisting system is X-braced shear walls composed of light gauge metal straps. The roof structure is prefabricated light gauge metal trusses.

This thesis project is an in depth study of an alternate structural system. The goal of the alternate system is to find a more suitable lateral force resisting system as well as a more economical overall system. This study investigated pre-cast hollow core planks, masonry bearing walls, and reinforced masonry shear walls. The results show lightweight 8" deep hollow core planks with a 2" concrete topping. Supporting these planks is 12" hollow blocks. The new lateral force resisting system is comprised of grouted 8" blocks with #8 reinforcing bars. Prefabricated light gauge roof trusses are still the roof framing system.

In addition to the depth study, two breadth studies were also performed. A cost analysis of the new system showed a savings of $23,643 over the existing system. The new system can be constructed in about the same amount of time. Hence, it can be concluded that this system is a very viable alternative. A study into LEED certification was also performed, and it can be concluded that a level of certification could have been achieved if properly incorporated into the original design.

Introduction:
Building Introduction
Building Systems

Introduction:

George Read Hall is one of three new state of the art buildings being built on the University of Delaware's campus in Newark, Delaware. The new buildings are being constructed to replace the existing Pencader complex. George Read Hall consists of five floors of dormitory style housing with double rooms sharing a bathroom as well as single room suites. The building also features laundry rooms, fully furnished lounge spaces, kitchen space, study rooms, and building support spaces. Housing approximately 500 beds, George Read Hall is the largest of the three new residential buildings. Also included in the building are apartments for the hall directors and complex coordinator. The exterior of the "U-shaped" building combines architectural features from the surrounding buildings into a Georgian styled look. The removal of the existing buildings will make room for playing fields, tennis, basketball, and volleyball courts, and a pedestrian bridge from main campus to the new residential complex.

Building Entrance

Interior Lounge Space

Building Envelope:

The exterior walls are brick façade with many symmetrically placed windows so that each room has a window. Each window is finished with a brick jack arch above and a simulated stone sill below. The fifth floor

is finished with dormers on each side of the building protruding from the gambrel style roof. The dormers are covered with metal paneling on the front and shingles on the top, matching the rest of the roof. On the north end of the building, a two story curved wall complex lounge adds an additional space for students to relax or study. The exterior of the lounge is finished with simulated stone and several rows of bricks at each floor level. Topping off the look of this complex lounge is a large storefront system with one inch insulating glass units. Exterior bearing walls of the building consist of metal stud framing with 2" rigid insulation and an air space. The inside of the wall is finished will $^{5}/_{8}$" gypsum board, while the outside is finished with brick or simulated stone. The roof of George Read Hall is finished with asphalt shingles.

Construction:

Building Site

The project began in May 2004 and took less than 15 months to complete. The building was completed on time, and students were able to move in for the fall semester 2005. George Read Hall cost approximately $27 million. The construction process was to occur while interrupting as little as possible, including activities on the campus and surrounding vegetation that was to remain. In addition, the process was designed so that none of the beds in the existing Pencader complex were removed until the new beds were ready for use. Deliveries to the site were scheduled so as to minimize the amount of space used for storage

as well as the amount of time required for storage of materials and equipment.

<u>Electrical</u>:

George Read Hall is powered by both 480/277V and 208/120V. The larger power supply is needed to run appliances such as dryers and stoves, while the smaller source is needed to for outlets using smaller appliances. The elevators are powered by 480 volt, 60 Hz, 3 phase, wye delta starting. Backup power is supplied by a diesel engine generator that will run at 350 kW for 17 hours.

<u>Lighting</u>:

As with a typical dormitory style building, various types of lights and light fixtures were used. Typical sizes include 2'-0" x 2'-0", 1'-0" x 4'-0", 2'-0" x 4'-0" as well as wall mounted lights providing direct lighting and direct/indirect lighting. Indirect lighting is also used. Many different varieties of bulbs are used as well, including fluorescent and high intensity discharge. T4 and T8 fluorescent bulbs are used in double and triple tube fixtures. High-pressure sodium and metal-halide lamps are used in the HID fixtures. Emergency lighting is installed with batteries and chargers.

<u>Mechanical</u>:

The construction of the three new buildings in the residential complex required the installation of a new 750 ton chiller to accommodate the building's cooling needs. The air is supplied

throughout the building by variable air volume units. The air is monitored by humidity sensors, air velocity sensors, and differential pressure sensors. Airflow ranges from 50-5,000 FPM in most situations. The temperature can be controlled by electric thermostats that are located in each residential unit.

Fire Protection:

The building is equipped with automatic sprinkler systems. The sprinkler systems are both wet-pipe and dry-pipe type systems. Spray on fireproofing is used on the beams and columns. Exit enclosures provide a two hour fire rating, and occupancy separations provide a one hour fire rating. The fire protection system includes manual pull stations, audio devices, smoke detectors, heat detectors, and duct detectors.

Telecommunications:

A security management system manages several of the building systems. The system integrates access control, alarm monitoring, and database management. Each room unit is equipped with phone and Ethernet jacks as well as cable hookups for television. Two telecommunications rooms are located on each floor and a main telecommunications room is located in the basement. As with most universities, access to the building will require authentication through card readers at the doors. Also, card access is required to operate the elevator.

Vertical Transportation:

George Read Hall is equipped with two hydraulic passenger elevators running from the basement to the fifth floor. The elevators are rated at 3500 pounds and travel at 125 feet per minute. They are equipped with battery powered lowering. If the power fails, cars at a floor level will open their doors and shut down. Cars between floors will be lowered to a pre-selected floor, open their doors, and shut down.

Existing Structural System

Existing Structural System:

The existing floor system in George Read Hall is composed of a Hambro composite floor system. This system uses 14” deep 50 ksi steel joists working compositely with a $2^{3}/_{4}$” concrete slab. The joists are spaced at 4’-$1^{1}/_{4}$” on center and typically span 23’-6” with an interior span of 6’-0” for the corridor. Typical bays are shown on pages 13 and 14.

Bearing walls are 16 gauge, 50 ksi cold formed metal studs. The first floor is supported with 3-6” studs @ 16” on center. A typical bay is 26’-8” x 23’-6”. Interior first floor framing consists of wide flange beams of various sizes. The first floor interior framing differs from the upper floors due to the need for more open space as required by the lounges. The second floor metal stud framing consists primarily of 3-6” studs @ 16” on center. Framing under the second floor hallway is wide flange beams, with the typical size being a W14x53. These interior hallway beams are located on each side of the 6’-0” wide hallway. The interior beams are replaced by metal stud bearing walls under the hallway in the third though fifth floor framing. The third through fifth floor framing is very similar. The third floor bearing walls consist mainly of 2-6” studs @ 16” on center. The fourth and fifth floor bearing walls are built with 1-6” stud @ 16” on center. Roof framing on George Read Hall consists of prefabricated light gauge metal trusses at a maximum of 4’-0” on center with $1^{1}/_{2}$” 22 gauge galvanized metal deck. The roof trusses span 54’-0” with two intermediate supports located 23’-6” from each exterior wall.

The foundation is comprised of a combination of continuous and spread footings. The continuous footings range from 3’-0” wide to 7’-0” wide and are 1’-0” deep reinforced with continuous #5 bars. Fifteen

different sizes of spread footings are used ranging from 3’-0” wide x 3’-0” wide x 1’-0” deep to 10’-0” wide x 10’-0” wide x 2’-3” deep. These spread footings carry the concentrated loads from the interior columns. Reinforcing for the spread footings are either #5 bars or #6 bars. The footings were designed using a soil bearing capacity of 4000 pounds per square foot (psf). Basement walls are 1’-4” thick with #4@12 both ways in both faces. The basement floor of George Read Hall is a 5” thick slab on grade with 6 x 6 - W1.4 x W1.4 welded wire mesh. Slab control joints are located so that there is a maximum of 40 feet in length along any one side with a maximum uninterrupted concrete area of 1200 square feet.

The lateral force resisting system of George Read Hall is X-braced shear walls. The shear walls are located along typical bay lines. First floor shear walls consist of X-bracing using 2-4$^{1}/_{2}$” metal straps. The second and third floor shear walls are X-braced walls of 2-4” metal straps. Fourth and fifth floor shear walls are 2-3” X-braced metal straps. The building footprint is shown on the following page, with the typical bay area represented by the hatching. The shaded area on the typical bay diagrams shows where the live load is 100 psf as required by code. The complete floor plans and building section can be seen in the appendix.

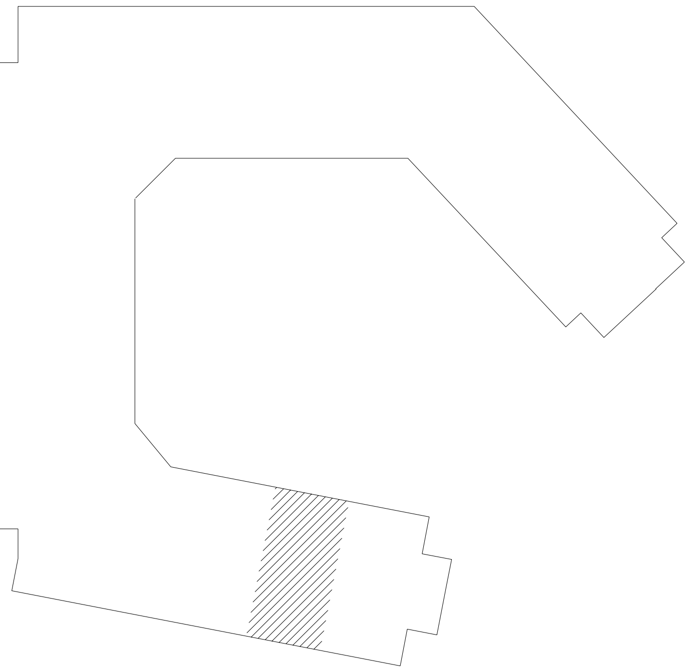


Figure 1: Building Footprint

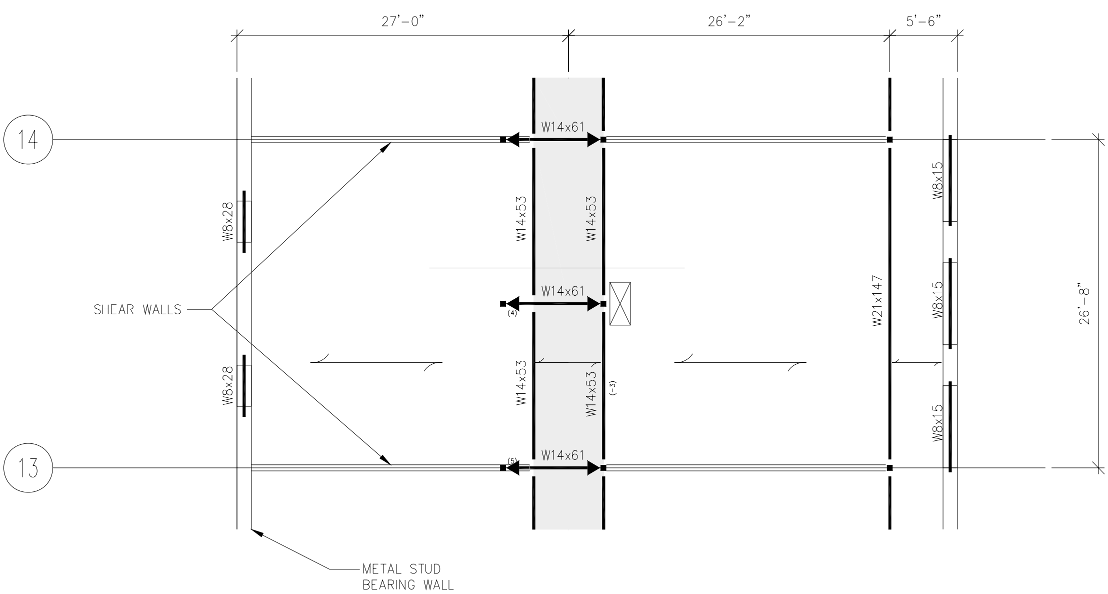


Figure 2: Typical Bay with Interior Beams

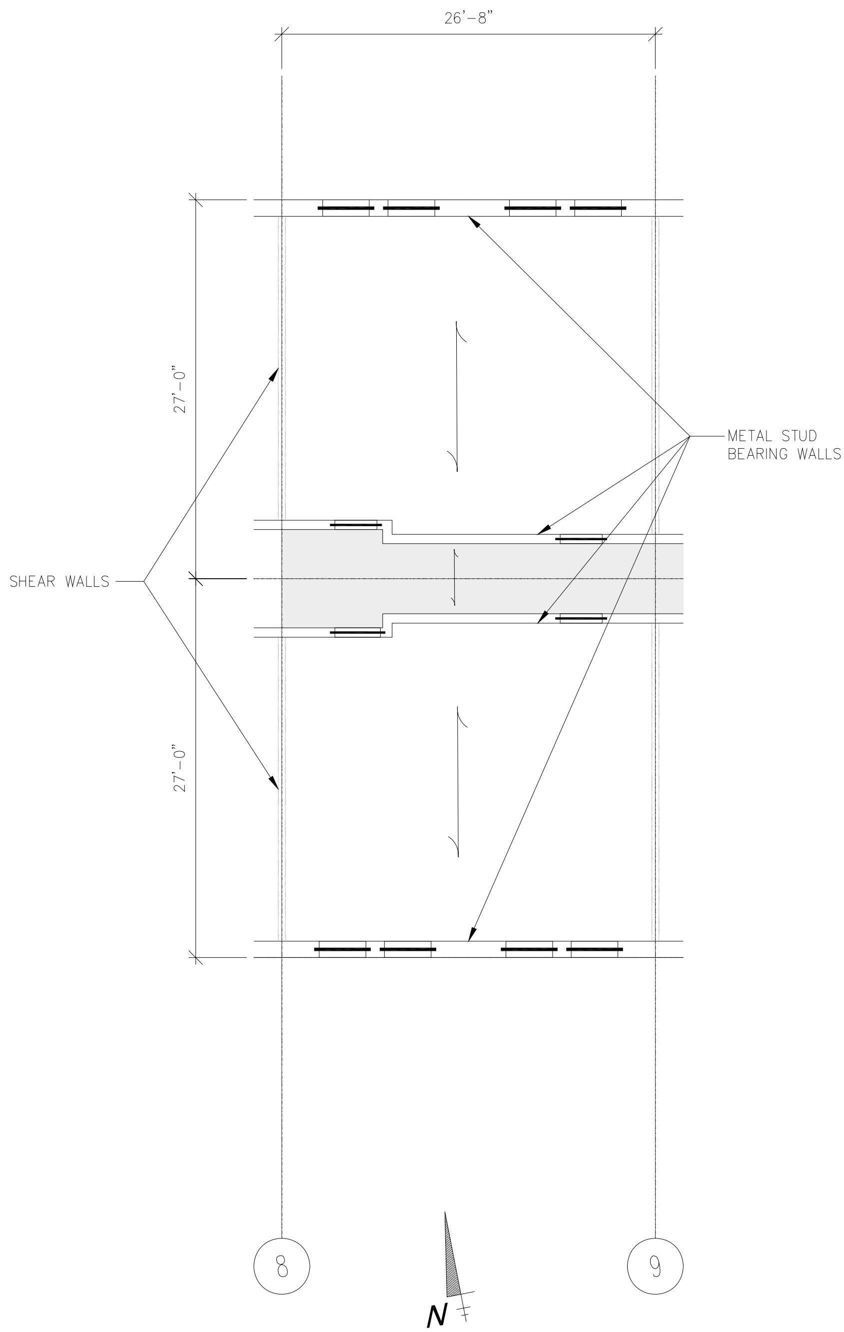


Figure 3: Typical Bay with Interior Bearing Walls

Problem Statement & Problem Solution

Problem Statement:

A building must be designed to resist all applied forces in accordance with the International Building Code (IBC). This includes gravity loads and lateral loads. The gravity loads are determined from the dead loads of the building and the live loads established in Table 1607.1 of IBC. The lateral forces take into account the effects of wind and seismic. These forces are also calculated in accordance with IBC with references to ASCE 7. Because of load combinations set forth in the IBC, the building does not have to resist both wind and seismic concurrently.

After a review of Technical Assignment #2 it was prevalent that several alternate floor systems were worth further investigation. The most viable alternative floor system is pre-cast hollow core planks. This was determined because it has the most advantages. It was concluded in Technical Assignment #3 that the seismic forces control the design of the system. This differs from the original design in which the wind forces were determined to control the design. Because of this, it was also determined that the existing lateral system is not appropriately designed to resist these higher seismic forces. Therefore an alternate lateral force resisting system will be designed.

Although the system of X-braced shear walls is fairly simple in the scheme of lateral resisting systems, the irregular shape of the building requires an analysis beyond the scope of my educational experiences thus far. The determination of the direct and torsional shear forces is more complex than in a rectangular or more regularly shaped building. Therefore, the design of an alternate lateral force resisting system will expand my experiences in structural engineering. Additionally,

introducing a new lateral system would not be appropriate with the existing gravity load resisting system. As a result, a redesign of the bearing walls system will also be done.

One of the most important things to consider when designing buildings is to make it as economical as possible. Because of this, it is very critical to investigate different systems.

Problem Solution:

The solution to this is to design an alternate system and compare it to the original design. The alternative system being considered in this proposal is a load bearing masonry system as well as masonry shear walls. Also, a new floor system of pre-cast hollow core planks will be studied. The alternate systems will then be compared to the original design to determine whether it is a considerable alternative.

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)

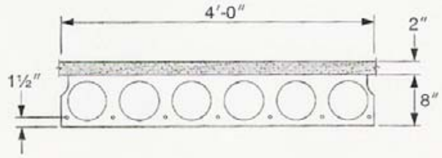
Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load. Check availability of lightweight sections.

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	in²	—	
I	=	1,666	in⁴	3,529	in⁴
y_b	=	4.00	in.	5.70	in.
y_t	=	4.00	in.	4.30	in.
S_b	=	416	in³	619	in³
S_t	=	416	in³	821	in³
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.

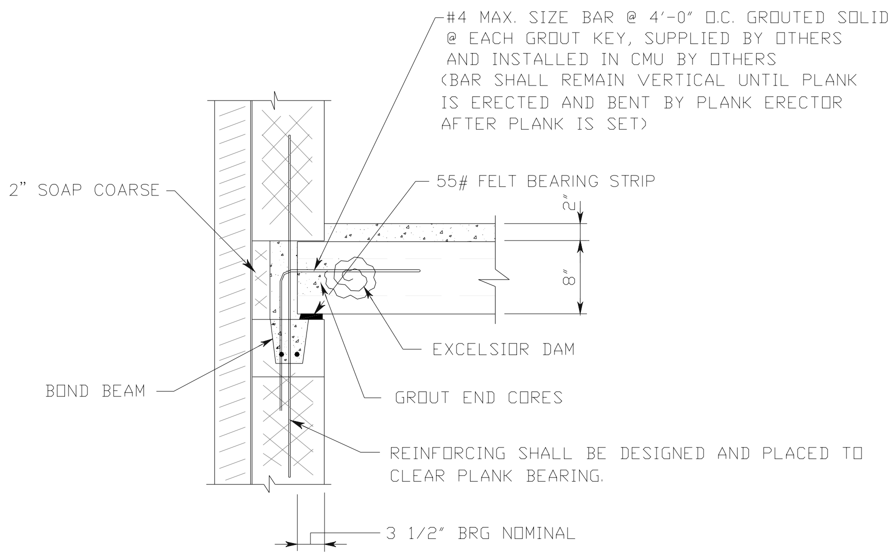


Figure 5: Exterior Bearing Wall Detail

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.

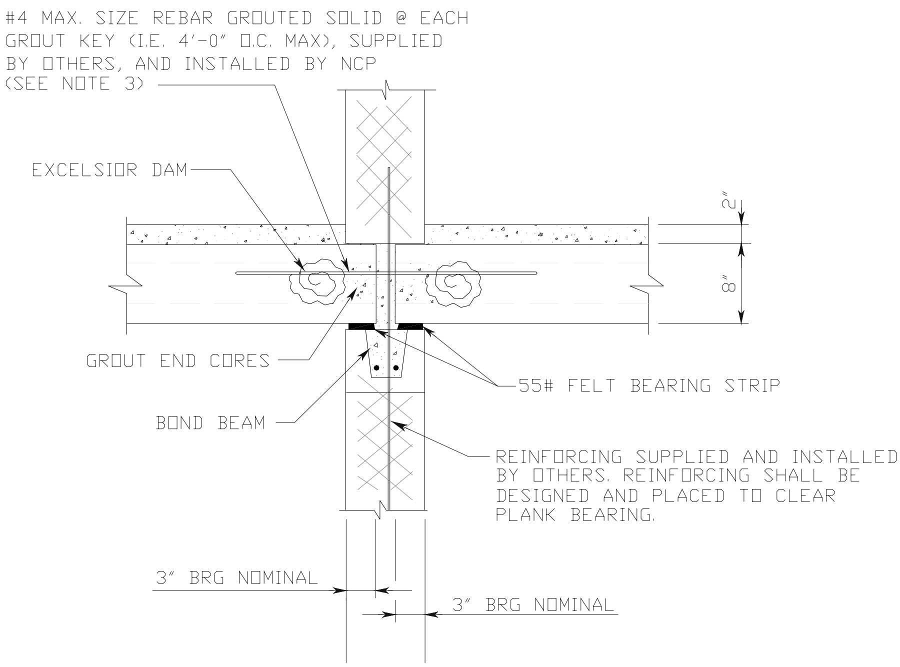


Figure 6: Interior Bearing on Masonry

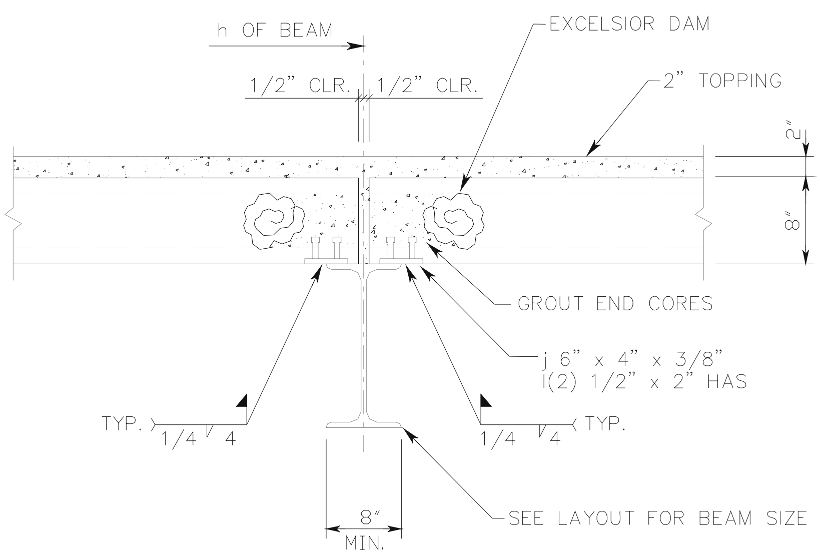


Figure 7: Interior Bearing on Wide Flange Beam

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.

	Allowable compressive stresses based on gross cross-sectional area, psi (MPa)[a]	
Gross area compressive strength of unit, psi (MPa)	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.

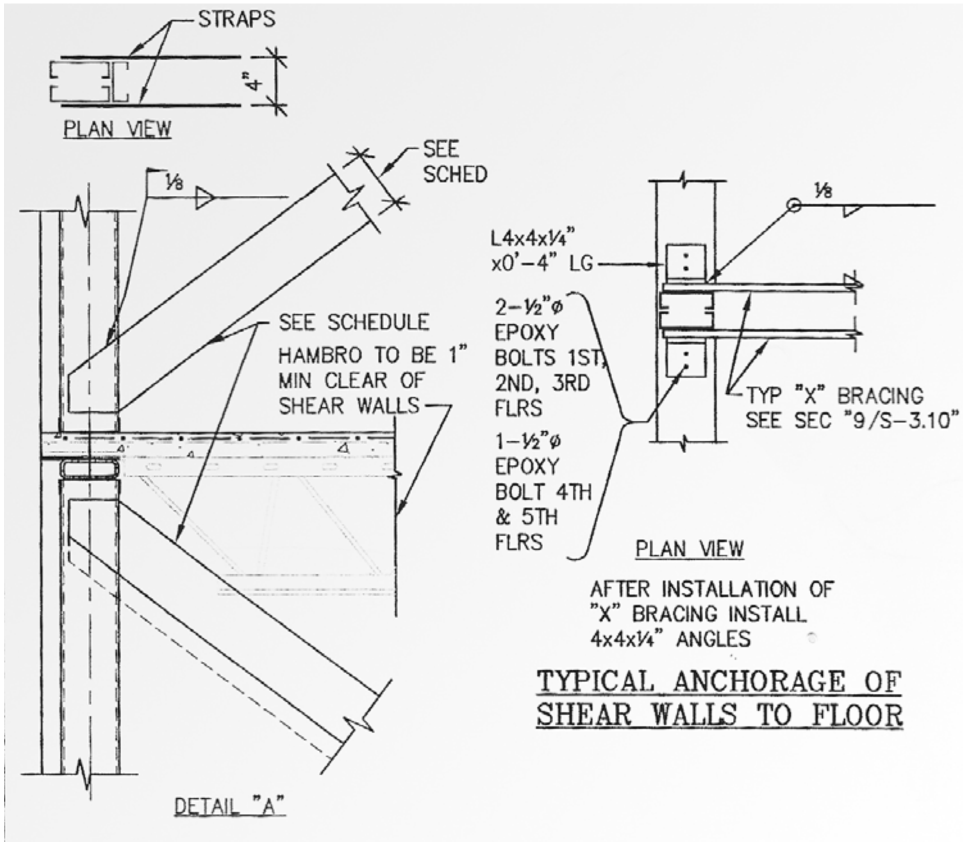


Figure 9: Existing Shear Wall Detail

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.

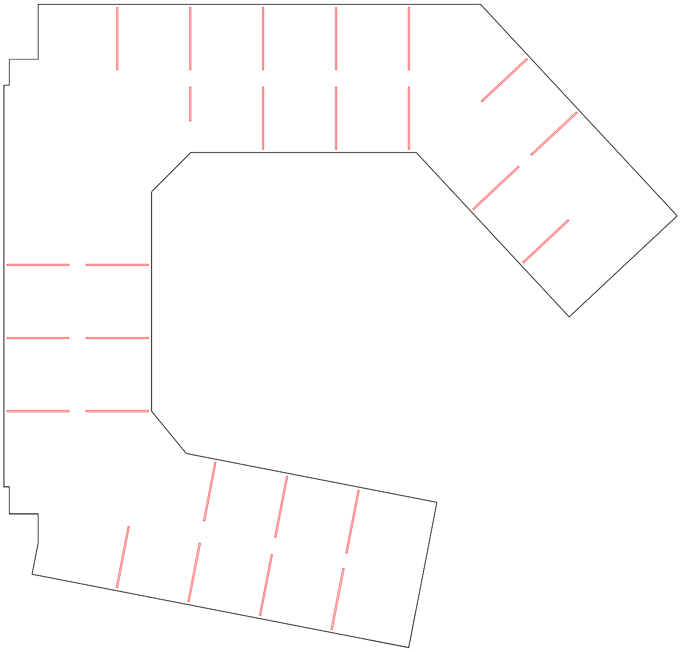


Figure 10: Existing Shear Wall Layout

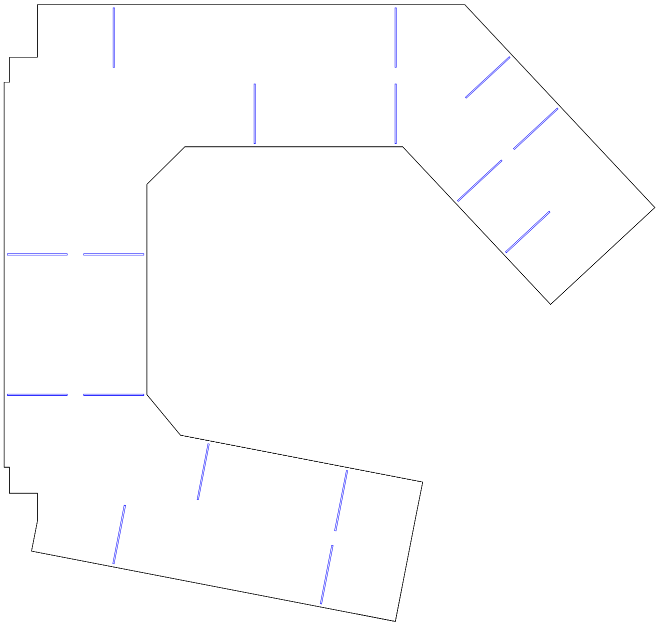


Figure 11: New Shear Wall Layout

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.

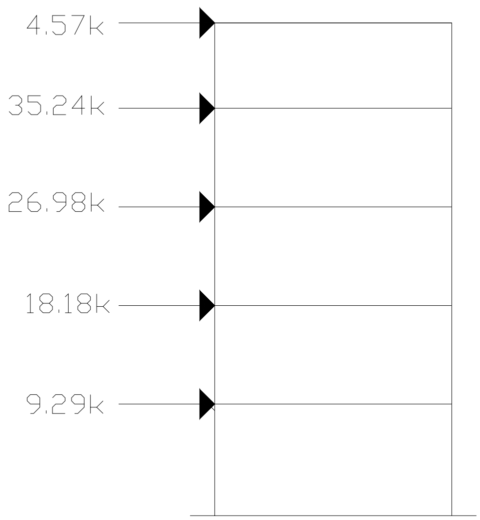


Figure 12: Design Shear Wall Loading

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.

Breadth Studies:
Construction Management
LEED Certification

Breadth Studies:

Two breadth topics were studied in addition to the structural depth work. In order to effectively compare the existing structural system with the new system, the cost and construction time must be compared. The second breadth topic will be an investigation into what could have been incorporated into the design and construction process in order to receive a LEED certification.

Construction Management Study:

The first breadth study involves construction management issues. A cost analysis was performed on the new system using the 2006 version of RS Means Building Construction Costs. In order to effectively compare the cost of the new system to the actual cost of the existing system, some items were added to the cost analysis that was not affected by this thesis project. For example, prefabricated light gauge metal trusses are still being used as the roof framing material. As a result, an estimate for these trusses was added to the cost. The actual cost of the structural package was $3.2 million. The total cost of the estimate for the new structural package is $3,176,357. This results in a savings of $23,643. I was not able to compare individual systems because the actual cost breakdown was not permitted to be released for this project. However, since the total system results in a savings of almost $24,000, it is definitely a considerable alternative to the existing system. The complete estimate calculations can be seen in the appendix. Cost is only one factor in the decision between different systems. Another major factor is the duration of the construction process.

A construction schedule was also done using Primavera Project Manager. For the purposes of this study, the building was broken down into four sections for the construction process. By breaking the building down into different sections, several activities can be done at the same time. The duration of the activities was determined based on the daily output of a typical crew found in RS Means. The schedule can be adjusted based on the crew size for each activity. The larger the crew, the faster the activity can be completed. The schedule is shown below. The red bars indicate the critical path of the work. The green bars are activities that can be done simultaneously with another activity, and the start of another activity is not directly related to their completion.

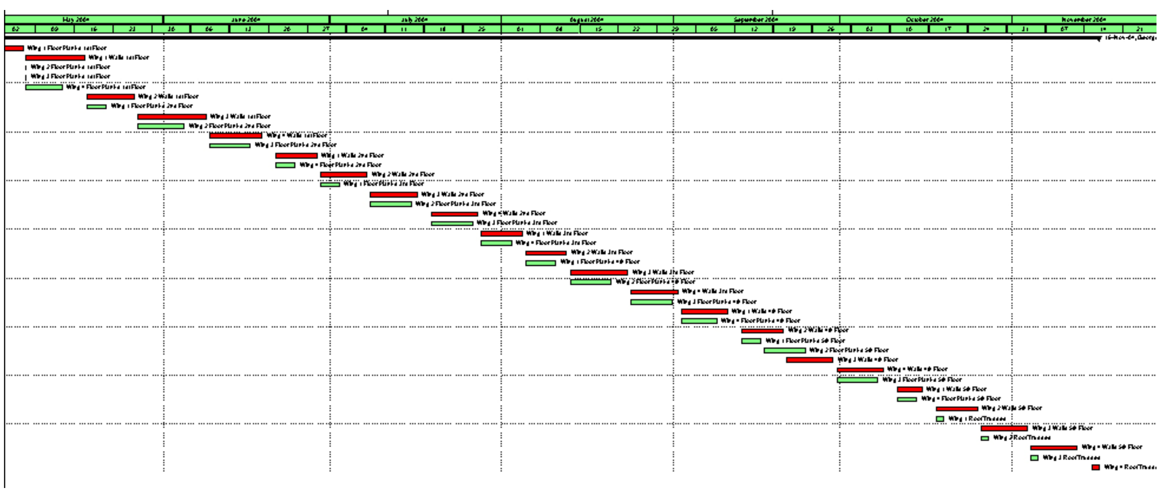


Figure 13: Construction Timeline

The actual duration of the structural aspect of the project was six months. The project began in May 2004. For this study, I assumed that the structural work began on May 3, 2004. The end of the structural construction as determined from the schedule above is November 16,

2004. The total time is approximately two weeks more than six months. This is virtually the same amount of time as the actual construction. Therefore, the new system of masonry walls and hollow core planks is a viable solution in terms of construction time. A larger version of the schedule can be seen in the appendix.

LEED Certification Study:

George Read Hall was not originally designed as a green building. This study was performed to see what changes could have been implemented to achieve certification on the LEED checklist. This was done using the US Green Building Council's Green Building Rating System for New Construction and Major Renovations, Version 2.2. In order to reach certification, twenty six points must be achieved on this checklist. Time did not allow for a complete study of all sixty nine possible points, so the sustainable sites category was chosen for a more in depth study. Fourteen possible points can be achieved in this category, as well as one required point.

Prerequisite 1: Construction Activity Pollution Prevention

In order to meet the prerequisite, construction activity pollution must be reduced. This is accomplished by creating an erosion and sedimentation control plan for the project. The goal of this credit is to prevent loss of topsoil during construction, to prevent sedimentation of storm sewer or receiving streams, and to prevent polluting the air with dust. To prevent the loss of topsoil, it will be stockpiled in the location shown on the plan on the next page. In addition, the stockpile will be temporarily seeded to prevent erosion until it is needed. The silt fence

will help to prevent run off from reaching the sewer or any surrounding streams. More care will be taken during the construction process to help reduce polluting the air with dust and other particles.

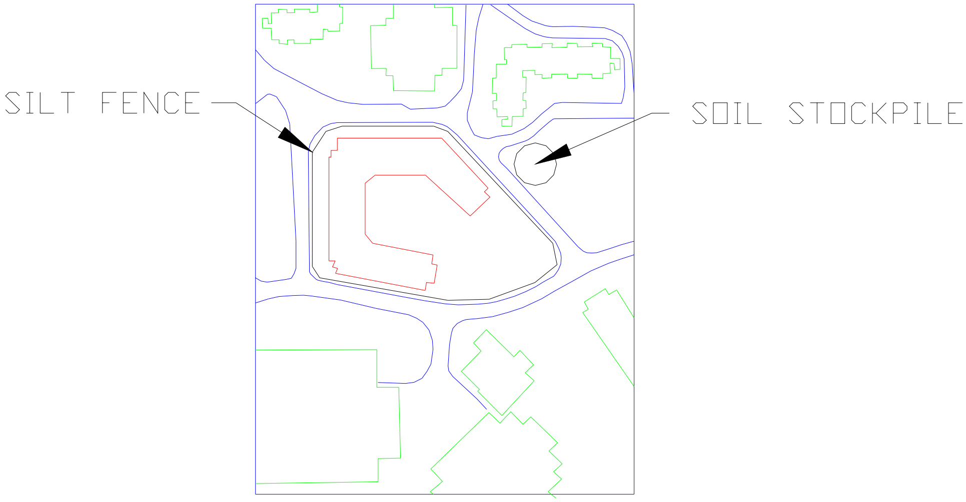


Figure 14: Erosion and Sedimentation Control Plan

<u>Credit 1: Site Selection</u>

The building must be constructed in a manner that results in the least environmental impact. The building cannot be constructed on a site that meets any of the following conditions:

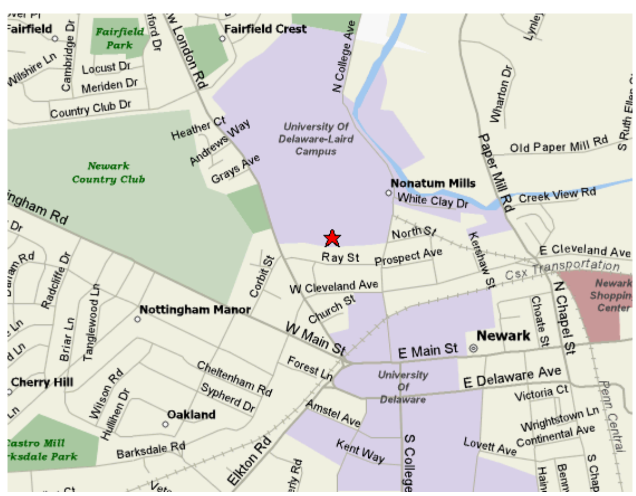
- Prime farmland as defined by the United States Department of Agriculture in the United States Code of Federal Regulations, Title 7, Volume 6, Parts 400 to 699, Section 657.5 (citation 7CFR657.5)
- Previously undeveloped land whose elevation is lower than 5 feet above the elevation of the 100-year flood as defined by FEMA (Federal Emergency Management Agency)
- Land that is specifically identified as habitat for any species on Federal or State threatened or endangered lists

- Within 100 feet of any wetlands as defined by United States Code of Federal Regulations 40 CFR, Parts 230-233 and Part 22, and isolated wetlands or areas of special concern identified by state or local rule, OR within setback distances from wetlands prescribed in state or local regulations, as defined by local or state rule or law, whichever is more stringent
- Previously undeveloped land that is within 50 feet of a water body, defined as seas, lakes, rivers, streams and tributaries which support or could support fish, recreation or industrial use, consistent with the terminology of the Clean Water Act
- Land which prior to acquisition for the project was public parkland, unless land of equal or greater value as parkland is accepted in trade by the public landowner (Park Authority projects are exempt)

The site does not meet any of these criteria; therefore, this point can be attained.

<u>Credit 2: Development Density and Community Connectivity</u>

This credit is achieved by constructing on a previously developed site and within ½ mile of a residential zone with an average density of ten units per acre net as well as within ½ mile of at least ten basic services. A few examples of basic services include banks,



Project Location

places of worship, fire stations, beauty salons, libraries, restaurants, and schools. Using Mapquest, it was determined that at least ten basic services are located within ½ mile of the building location. However, the intention of this point is to be located within ½ mile of a residential zone so that people can walk to the building for work. Because George Read Hall is a dormitory, it is unlikely that it will qualify for this point.

Credit 3: Brownfield Redevelopment

The purpose of this point is to rehabilitate a contaminated site. George Read Hall was not constructed on a contaminated site; therefore, this point cannot be earned.

Credit 4.1: Alternative Transportation: Public Transportation Access

This credit is intended to reduce pollution from automobiles. It can be achieved by locating the project within ¼ mile of one or more stops for at least two campus bus lines. Several different campus bus routes make stops right outside the building. Two of these routes are shown on the following page. The arrows along the route represent the bus stops. The arrows with an asterisk represent stops by request. George Read Hall is labeled near the top of the maps. It is evident that several bus stops are located within ¼ mile of the building. Thus, the building already qualifies for this point.

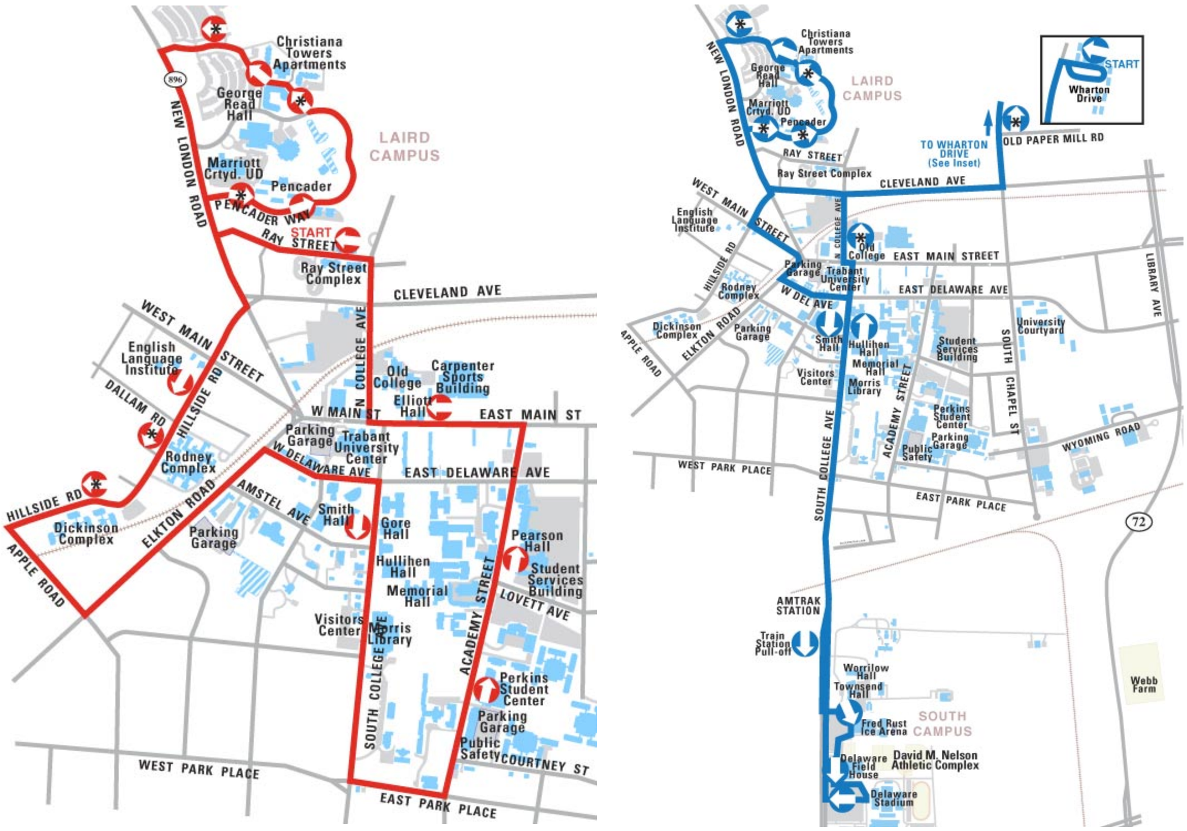


Figure 15: Campus Bus Routes

<u>Credit 4.2: Alternative Transportation: Bicycle Storage</u>

For residential buildings, this credit can be earned by providing covered storage facilities for bicycles for 15% of the building occupants. The building houses five hundred people. This results in the need for seventy five storage facilities. The easiest way to provide covered storage is the use of bike lockers. These lockers can be placed in the open space adjacent to the building. This credit can be easily attained.

Credit 4.3: Alt. Transportation: Low Emitting & Fuel Efficient Vehicles

There are three options that meet this credit. The first option is to provide low emitting, fuel efficient vehicles for 3% of the building occupants as well as providing preferred parking for these vehicles. The second option is to provide preferred parking for low emitting, fuel efficient vehicles for 5% of the building occupants. The third option is to install alternative fuel refueling stations for 3% of the total vehicle parking capacity of the building. Because of the site restraints due to existing roads, there is no space for new parking. Therefore, options one and two cannot be obtained. The third option is possible, but could be quite expensive. Space for such a refueling station is also fairly limited. Because of these reasons, this credit cannot practically be achieved.

Credit 4.4: Alternative Transportation: Parking Capacity

For residential buildings, two options exist to meet this credit. The first is to provide parking capacity no greater than the minimum local zoning requirements and implement programs that promote shared vehicle usage. The second option is not to supply any new parking. As mentioned in Credit 4.3, no new parking is provided due to lack of space. Therefore, this credit is already attained.

Credit 5.1: Site Development: Protect or Restore Habitat

The purpose of this credit is to conserve existing natural areas and also restore damaged areas. This is done by restoring or protecting 50% of the site area with native vegetation. This area does not include the building footprint. After construction was completed, the area around the building was seeded and trees were planted. The trees are represented by the circles on the following site plan.

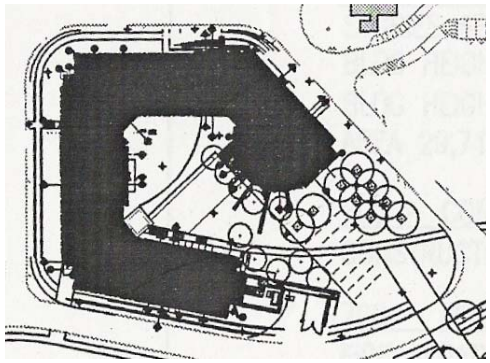


Figure 16: Site Plan

The majority of the area adjacent to the building is vegetated. The only area that is not vegetated is a limited area of sidewalks. Thus, this point is already attained as well.

<u>Credit 5.2: Site Development: Maximize Open Space</u>

This credit is similar to credit 5.1. The amount of vegetated open space must be equal to the building footprint. The building footprint is approximately 27,500 square feet. The area of the site is approximately 73,000 square feet. The majority of the area adjacent to the building is vegetated. Therefore, this point can be achieved.

<u>Credit 6.1: Stormwater Design: Quantity Control</u>

The requirement for this credit is to implement a storm water management plan that prevents the discharge rate of storm water after construction from being higher than the discharge rate before construction for the one and two year twenty four hour storm. The two year twenty four hour design storm for New Castle County, Delaware is 3.2 inches/24 hour period. The easiest way to keep the storm water from running off the site is to promote infiltration. This can be done by

allowing the storm water to discharge onto vegetated areas instead of impervious areas. Once discharged onto vegetated areas, the water can slowly perk into the ground. This prevents the storm water from leaving the site. Therefore, this credit can be earned.

Credit 6.2: Stormwater Design: Quality Control

The intention of this point is to limit the disruption and pollution of natural water flows by managing the storm water runoff. This is accomplished by treating the storm water runoff. The purpose of treating the water is to remove the suspended solids. Certain types of vegetation are able to treat the runoff. However, this requires in field monitoring to determine if the treatment level is sufficient to meet the requirements of this credit. Because of this, earning this credit requires careful monitoring, making it impractical.

Credit 7.1: Heat Island Effect: Non-roof

In order to meet the requirements of credit 7.1, 50% of the site hardscape must be a combination of shaded, paving materials with a solar reflectance index of 29, or an open grid pavement system. The use of Portland cement concrete meets the required solar reflectance value. Thus, to meet this requirement, all sidewalks should be constructed of this type of concrete. Additionally, the trees planted on the site will provide shade within five years. Therefore, with the use of Portland cement concrete, this credit can be earned.

Credit 7.2: Heat Island Effect: Roof

The intent of this credit is to reduce heat islands on the roof surface. This can be done by using roofing materials having a Solar

Reflectance Index value of 29 or higher, installing a green roof, or a combination of both for a minimum amount of the roof area. The Solar Reflectance Index is a measure of the surface's ability to reflect solar heat. Because of the slope of the roof, installing a green roof might be quite difficult. The existing roofing material is black asphalt shingles. Black shingles do not have a Solar Reflectance Index high enough to meet these standards. As a result, the roofing material must be changed in order to meet the requirements of this credit. A solution to this is to change the roofing material from asphalt shingles to a metal roof. To achieve the desired Solar Reflectance Index a coating can be applied to the metal roof. This coating is available in a variety of colors that still meet the requirements. This allows for a level of architectural freedom to give the building the desired appearance. With a change in the roofing materials, this credit can be earned.

<u>Credit 8: Light Pollution Reduction</u>

The requirements for light pollution reduction include interior and exterior lighting. For interior lighting, the angle of maximum candela must intersect opaque building surfaces instead of exiting out through windows. For exterior lighting, site and building mounted luminaries cannot produce an initial illuminance value higher than 0.20 horizontal and vertical footcandles at the site boundary and 0.01 horizontal footcandles fifteen feet beyond the site boundary. A lot of the luminaries in the building are indirect type light fixtures. This meets the requirements for interior lighting. However, in areas with different types of lights, as well as exterior lights, this credit can be earned by simply selecting appropriate fixtures and laying them out so that light does not escape through the windows.

LEED Summary

In summary, ten out of fourteen of the credits in the sustainable sites category can feasibly be incorporated into the design of George Read Hall. This is a very good indication that at least twenty six credits can be earned, and the building can be LEED certified. An important aspect to consider is the cost of this process. A cost analysis was not performed on this study, but it is evident that additional costs may be incurred from the use of different materials. Additional costs may also come from items such as bike lockers. However, these initial additional costs would be offset by the increased efficiency of the green systems. Overall, if properly incorporated into the design, this process could be done and would allow the building to make less of an environmental impact.

Conclusions & Recommendations

Conclusions & Recommendations:

The goal of this thesis project was to find a more effective lateral force resisting system. Furthermore, alternate floor and bearing wall systems were studied to determine if a more economical structural system could be found.

In conclusion, the new system of hollow core planks, masonry bearing walls, and reinforced masonry shear walls presents a very reasonable alternative to the existing system. The existing lateral force resisting system of X-braced shear walls was not able to sufficiently resist the seismic forces calculated in this project. The new system of reinforced masonry shear walls is able to resist these forces using fewer walls. The hollow core planks and masonry bearing walls are also able to appropriately resist the gravity loads as calculated by code.

An in depth cost analysis of the new system showed a savings of $23,643 over the existing system. In addition to cost analysis, a construction schedule showed that the new system can be constructed in approximately the same amount of time as the existing system. This study shows that the new system is a feasible alternative.

It can also be concluded that a level of LEED certification could have been achieved if appropriately incorporated into the original design. This would allow the building to make as little environmental impact as possible. Any additional initial costs of this aspect of the project would be offset by the long term cost savings of the green systems.

The final recommendation made from this thesis is to incorporate these results into future projects. Masonry construction should be given more consideration for projects of this size. LEED certification should also be given more consideration to help protect the environment.

Acknowledgements:

I would like to thank The University of Delaware for allowing me to use George Read Hall in this thesis project. I would also like to thank the following people at The University of Delaware for providing me with valuable information:

Mr. Joe Filippone – Plans Room Technician

Ms. Penny Person – Senior Project Manager

I would like to thank the companies involved in the project team for their assistance in various aspects of this project:

Ayers/Saint/Gross Architects & Planners

Sebesta Blomberg & Associates

Skarda & Associates

Tetra Tech, Inc.

I would like to thank my thesis advisor, Dr. Boothby, as well as Dr. Riley, Professor Parfitt, Dr. Lepage, and Kelly Salyards for answering all of my thesis questions.

I would especially like to thank my parents for their continued support throughout my education. I would also like to thank Maria Smith for helping to motivate me and for helping to edit this report.

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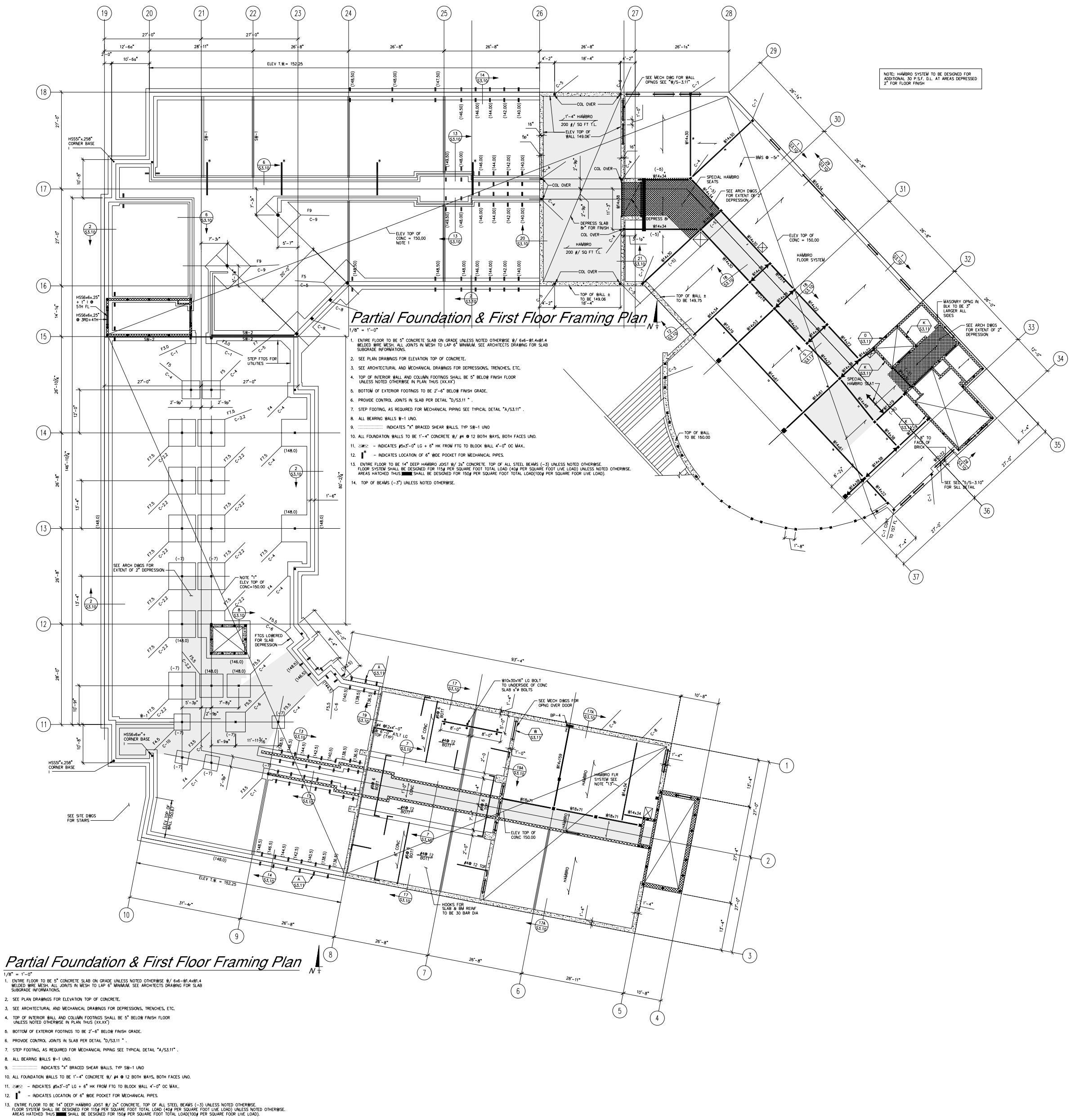
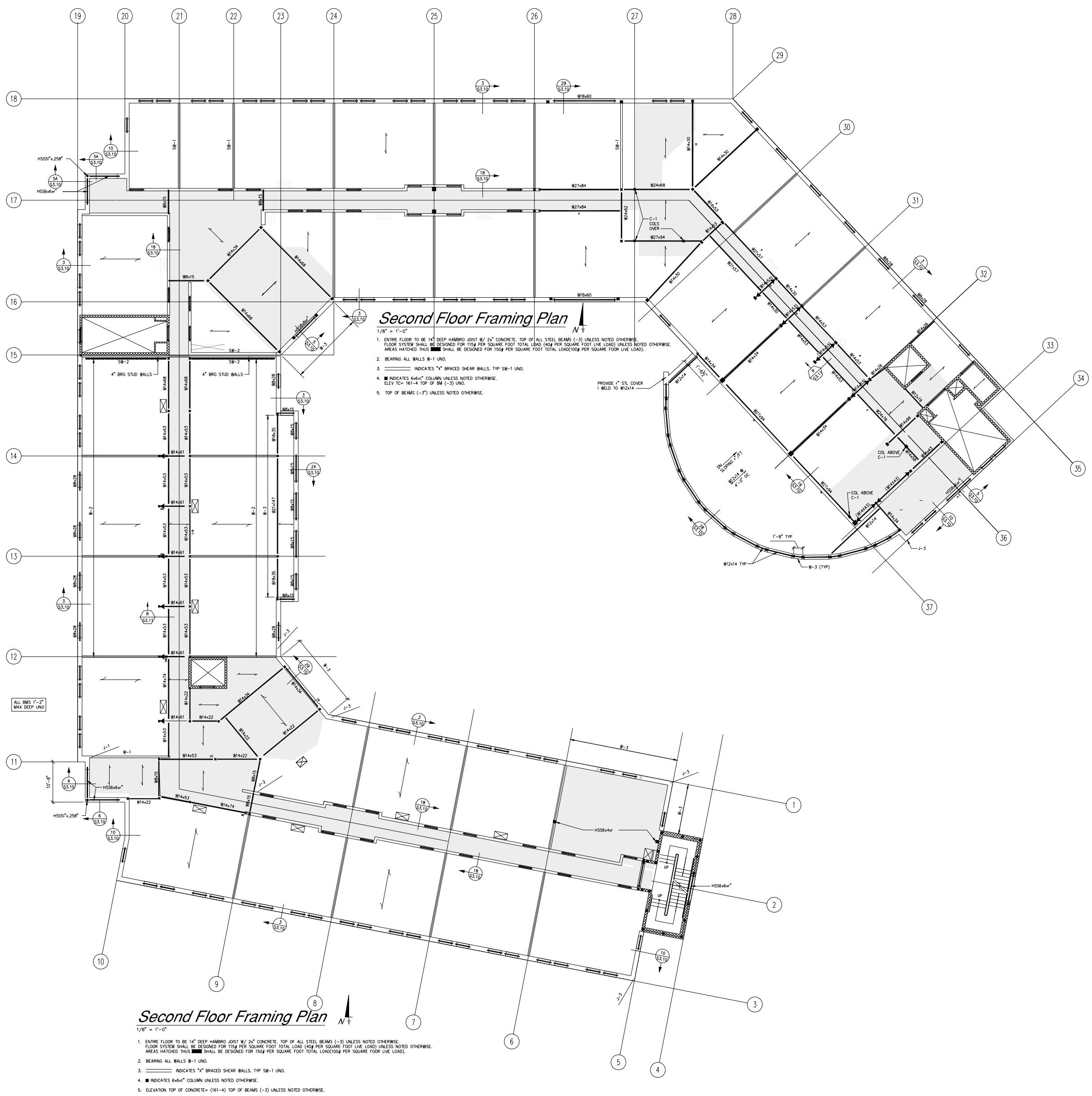
Appendix

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

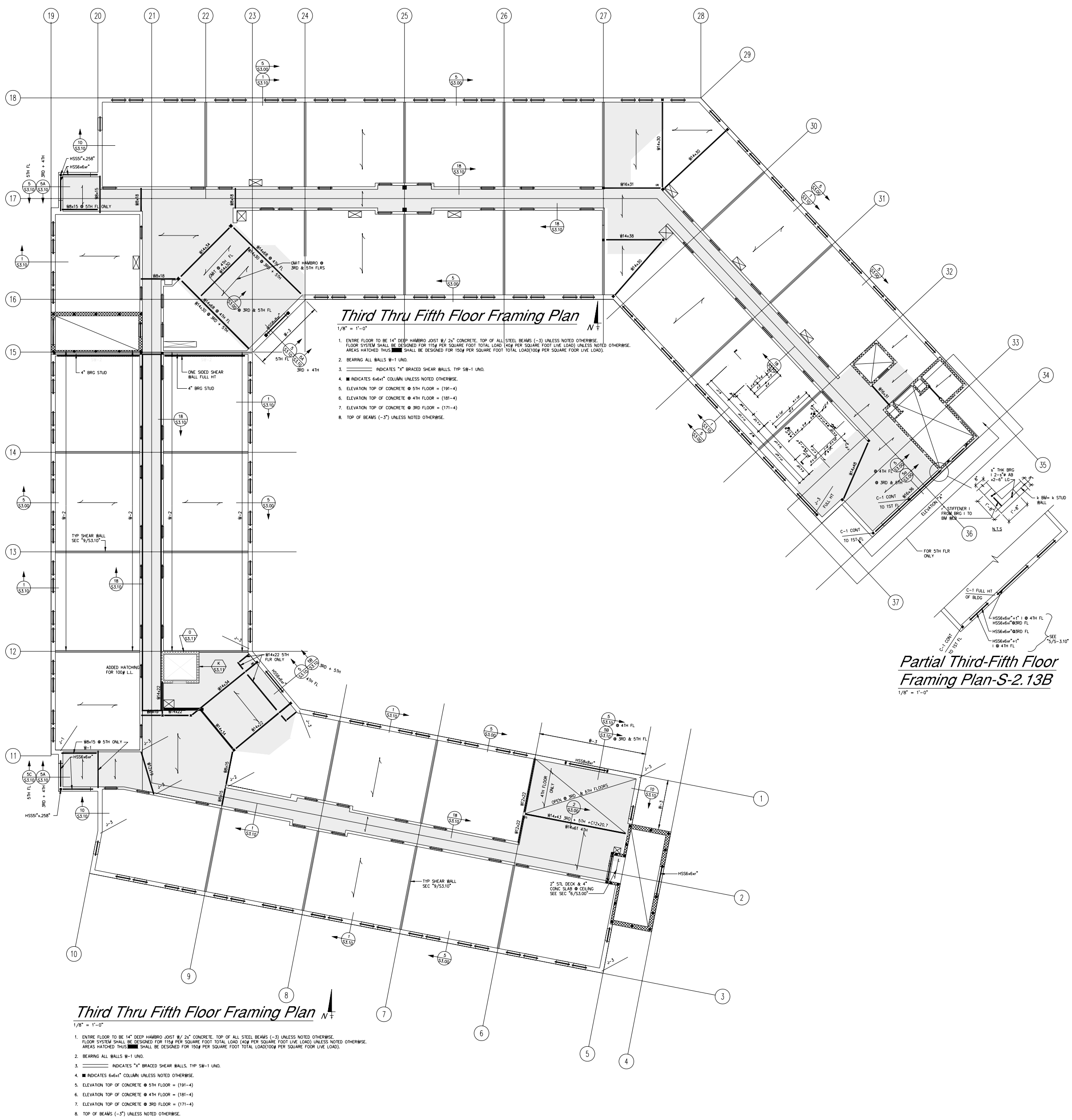
First Floor Plan:



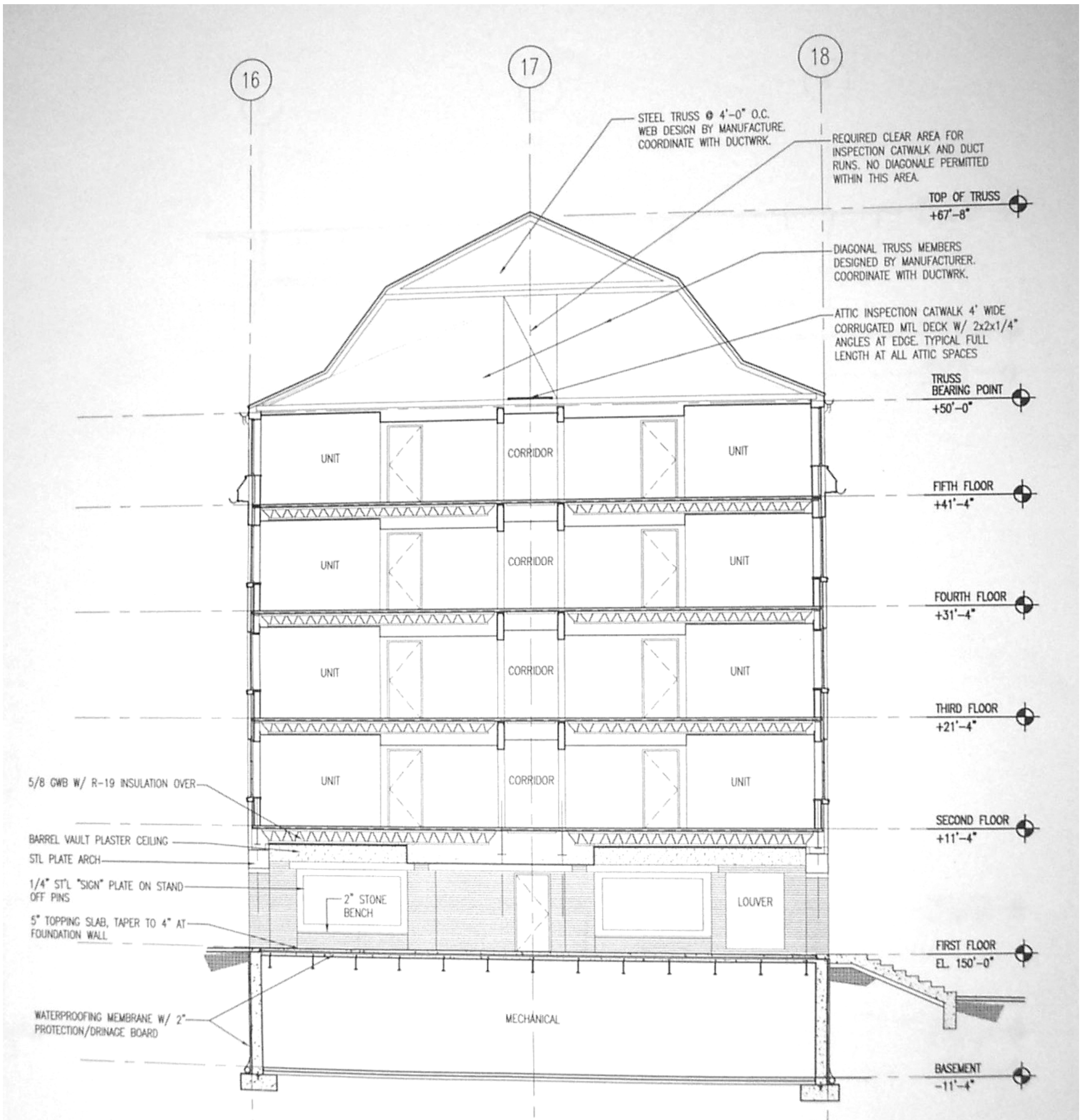
Second Floor Plan:



Third through Fifth Floor Plan:



Building Section:



Seismic Calculations:

Seismic Analysis for New Structure

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

$S_s = 0.225$
$R = 3.5 \rightarrow$ intermediate reinforced masonry shear walls
$I = 1.0$
$S_1 = 0.07$
$\left.\begin{array}{l} F_a = 1.2 \\ F_v = 1.7 \end{array}\right\}$ 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.

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

- 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 = 5 walls in each direction

Check Period

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

- $\mu = 0.057 \ \text{k-sec}^2/\text{in}^2$
- $H = 68$ 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

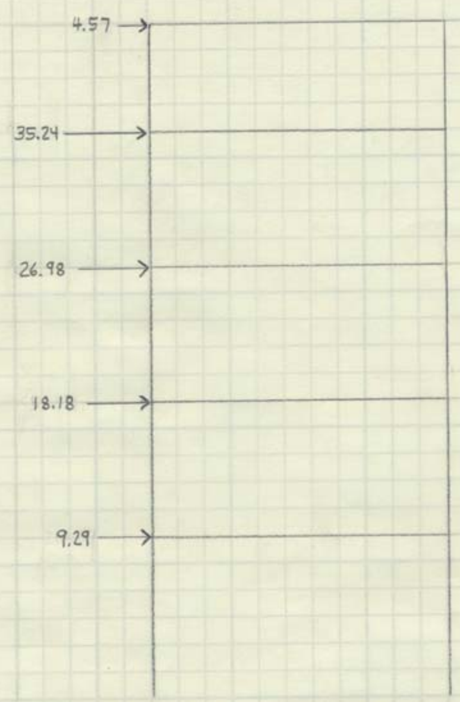
$$C_{s,max} = \frac{0.079}{0.81(3.5)} = 0.028$$

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:



5th Floor
Direct Shear + Torsional Shear = 4.12 + 0.45 = 4.57 k

4th Floor
31.79 + 3.45 = 35.24 k

3rd Floor
24.98 + 2.0 = 26.98 k

2nd Floor
16.46 + 1.72 = 18.18 k

1st Floor
8.45 + 0.84 = 9.29 k

V = 94.3 k
M = 3023 ft·k

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 → 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 → 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·k

$$32{,}000 = \frac{1105.6(12000)}{A_s(0.85)(0.8)(22.67)(12)}$$

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

$$\rho = \frac{2.37}{7.625(260)} = 0.0012$$
$k = 0.202$
$j = 0.932$

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

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

∴ USE 3 - #8 BARS @ 3rd FLOOR

4th Story

$V = 39.81^k$
$M = 437.7$ ft·k

$$32{,}000 = \frac{437.7(12000)}{A_s(0.85)(0.8)(22.67)(12)}$$

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

$$\rho = \frac{0.79}{7.625(268)} = 0.00039$$
$k = 0.121$
$j = 0.96$

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

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

∴ USE 1 - #8 BAR @ 4th FLOOR

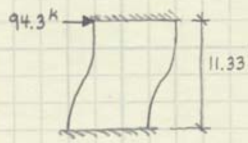
5th Story

$V = 4.57^k$
$M = 39.6$ ft·k

∴ USE 1 - #8 BAR @ 5th FLOOR

Drift Calculations:

Story Drift



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

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

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

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

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

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

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

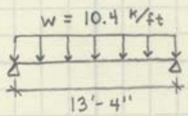
$$\left.\begin{array}{l} \Delta_{1\,allow} = 0.007h_x = 0.079 \text{ in.} \\ \Delta_{2\text{-}4\,allow} = 0.007h_x = 0.07 \text{ in.} \\ \Delta_{5\,allow} = 0.01h_x = 0.0867 \text{ in.} \end{array}\right\} \therefore \text{OK}$$

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

$\therefore$ DRIFT IS OK

Interior Beam Calculations:

Beam @ 2nd Floor Corridor



$$M_{max} = \frac{w\ell^2}{8} = \frac{10.4(13.33)^2}{8} = 231\ k\cdot ft$$

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

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

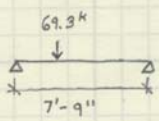
$$0.44 = \frac{5(10.4)(13.33)^4(1728)}{384(29000)\ I_{req}}$$

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

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

While the W21 × 48 is the lightest and probably most economical, it is a fairly deep member. Therefore, the W14×61 is a better choice.

2nd Floor Support Beam



$$M = \frac{Pab}{\ell} = \frac{69.3(2.25)(5.5)}{7.75} = 110.7\ k\cdot ft$$

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

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

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

∴ USE W14×61

Cost Analysis:

Cost Estimate

Bearing Walls:

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

$(10.65\ \$/ft^2)(87,019.5\ ft^2) = \boxed{\$926,758}$

Mortar for Bearing Walls:

7.5 bags / 300 blocks

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

$(2900\ bags)(7.10\ \$/bag) = \boxed{\$20,590}$

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

$(387\ tons)(19\ \$/ton) = \boxed{\$7353}$

Shear / Interior Walls:

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

$(25,202\ ft^2)(7.15\ \$/ft^2) = \boxed{\$180,196}$

Mortar for Shear / Interior Walls:

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

$(708\ bags)(7.10\ \$/bag) = \boxed{\$5027}$

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

$(95\ tons)(19\ \$/ton) = \boxed{\$1805}$

Grout for Shear Walls:

– 8" Thk. pumped

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

$(18,136\ ft^2)(3.53\ \$/ft^2) = \boxed{\$64,020}$

Scaffolding:

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

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.) = $\boxed{\$18,891}$

Precast Planks:

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

(121,432 ft^2)(8.40 $/$ft^2$) = $\boxed{\$1,020,029}$

Concrete Topping:

(121,432 ft^2)(2.11 $/$ft^2$) = $\boxed{\$256,222}$

Bond Beams:

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

(8189 ft)(14.90 $/ft) = $\boxed{\$122,016}$

Waste:

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

(20,590 + 5027 + 7353 + 1805)(0.25) = $\boxed{\$8694}$

Wide Flange Beams:

W14x22 – 115 ft. (35.50 $/ft) = $\boxed{\$4082}$

W14x30 – 81 ft (40.50 $/ft) = $\boxed{\$3281}$

W14x34 – 170 ft. (45.50 $/ft) = $\boxed{\$7735}$

W14x61 – 264 ft. (75.67 $/ft) = $\boxed{\$19,976}$

W14x68 – 81 ft (83.83 $/ft) = $\boxed{\$6791}$

Light Gauge Metal Trusses:

(277 Trusses)(400 \$/Truss) = $\boxed{\$110{,}800}$

Pre-cast Lintels:

(856 lintels)(369 \$/lintel) = $\boxed{\$315{,}864}$

Total Cost = $\boxed{\$3{,}176{,}357}$

Construction Schedule: