Senior Thesis

Final Report

AC Hotel Philadelphia

Philadelphia, Pennsylvania

Submittal in partial fulfillment of the requirements for a baccalaureate degree in Architectural Engineering at The Pennsylvania State University.

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4/8/2016

AC Hotel Philadelphia

Baywood Hotels | 230 North 13th Street, Philadelphia, Pa

Project Information

- Occupancy: Residential transient hotel
- Stories: parking garage + 14 levels above grade + Mech. Penthouse & Rooftop Terrace
  - 192ft. Above sidewalk grade
- Overall project cost: $35,000,000
- Size: 107,680 sq.ft.
- Construction Dates: Fall 2015 – Summer 2017
- Project delivery method: Design-Bid-Build

Courtesy of Holbert Apple Associates

Project Team

Owner: Kurt Blorstad
General Contractor: Clemens Construction
Architect: Spg3
Structural Engineer: Holbert Apple Associates
MEP: McHugh Engineering

Features:

- 150 luxury units
- Underground, valet parking via car elevator
- Exclusive restaurant for guests
- Fitness center & indoor pool
- Green Roofs
  - Extensive (2nd & 3rd Levels)
  - Intensive (Rooftop Terrace)

Structure:

- Foundation
  - Mat-slab
  - Underpinning of adjacent structures during construction
- Framing
  - Structural steel framing
  - Composite deck (normal-weight concrete)
  - 8″ thick precast hollow-core plank (@4′ O.C.) girder slab system
- Lateral System
  - Concrete shear walls (lower levels)
  - Concentric braced frames (upper levels)

MEP:

- Mechanical
  - (4) three-ton air handling units
  - Water-source heat pump
  - Energy recovery wheel on the roof used to mix outside air with return air
  - Plethora of fans used to exhaust class 3&4 air
- Electrical
  - 600KW Emergency generator on roof
  - 2500A Main Circuit Breaker

JESSE BORDEAU ~ Structural Option
http://jbordeau18.wix.com/thesis

Areas of Discussion

List of Figures

Executive Summary

AC Hotel Philadelphia is a 15-story residential transient hotel (including penthouse) located in the heart of downtown Philadelphia. This new hotel, owned by Baywood Hotels, will be built on top of the previous NFL Films and Warner Bros distribution center, a historic two-story building located at the corner of Florist and North 13th Street in Philadelphia.

The original two-story, 31'-0" tall building is a load bearing masonry structure. In order to properly satisfy the proposed addition, a mat foundation of varying thickness will be installed and the building will be gutted and restructured. The new construction will consist of composite steel at the bottom two levels, supporting a 12-story steel-frame structure atop, capped with a penthouse. The typical floor to floor height measures 10'6". Concentric braced frames support the building against lateral loads.

The building was redesigned with a one-way concrete slab with concrete beams with varying spacing from 4'6" to 5'8" based on bay size. Concrete girders transfer loads from the slab and beams to concrete columns which disperse building loads into the mat slab foundation. The existing lateral system was also switched from concentric braced frames to concrete shear walls and moment frames. Four shear walls resist lateral building movement in the N-S direction and moment frames run in the E-W direction. The column layout was slightly modified to create a more evenly-spaced grid. Even with the change in structure, the overall building mass decreased, and wind still controlled for lateral load conditions. Both structural systems were designed by hand. The gravity system was verified by the use of StructurePoint programs and the lateral design by ETABS (2015). Assumptions for calculations can be located at the beginning of each section.

Acknowledgments

I would like to express my sincere gratitude to the following people for assisting me throughout the year on my thesis project:

- The engineers at Holbert Apple Associates (especially Scott Molongoski & David Smith) for allowing me access and use of the AC Hotel Philadelphia building project.
- The AE faculty, especially my thesis advisors (Dr. Hanagan & Prof. Sustersic) for their advice and assistance.
- My fellow AE friends, classmates and colleagues.
- My parents for continuously supporting me through my higher-education endeavors.

Existing Conditions

Site Location

230 North 13th St. is located in Philadelphia, Pennsylvania in proximity to the Liberty Bell, The Franklin Institute and the Eastern State Penitentiary. The site is positioned ever so slightly off axis and lies northwest of center city, offering dwellers a beautiful view of the Philadelphia skyline. Figure 1 clarifies the exact location below.

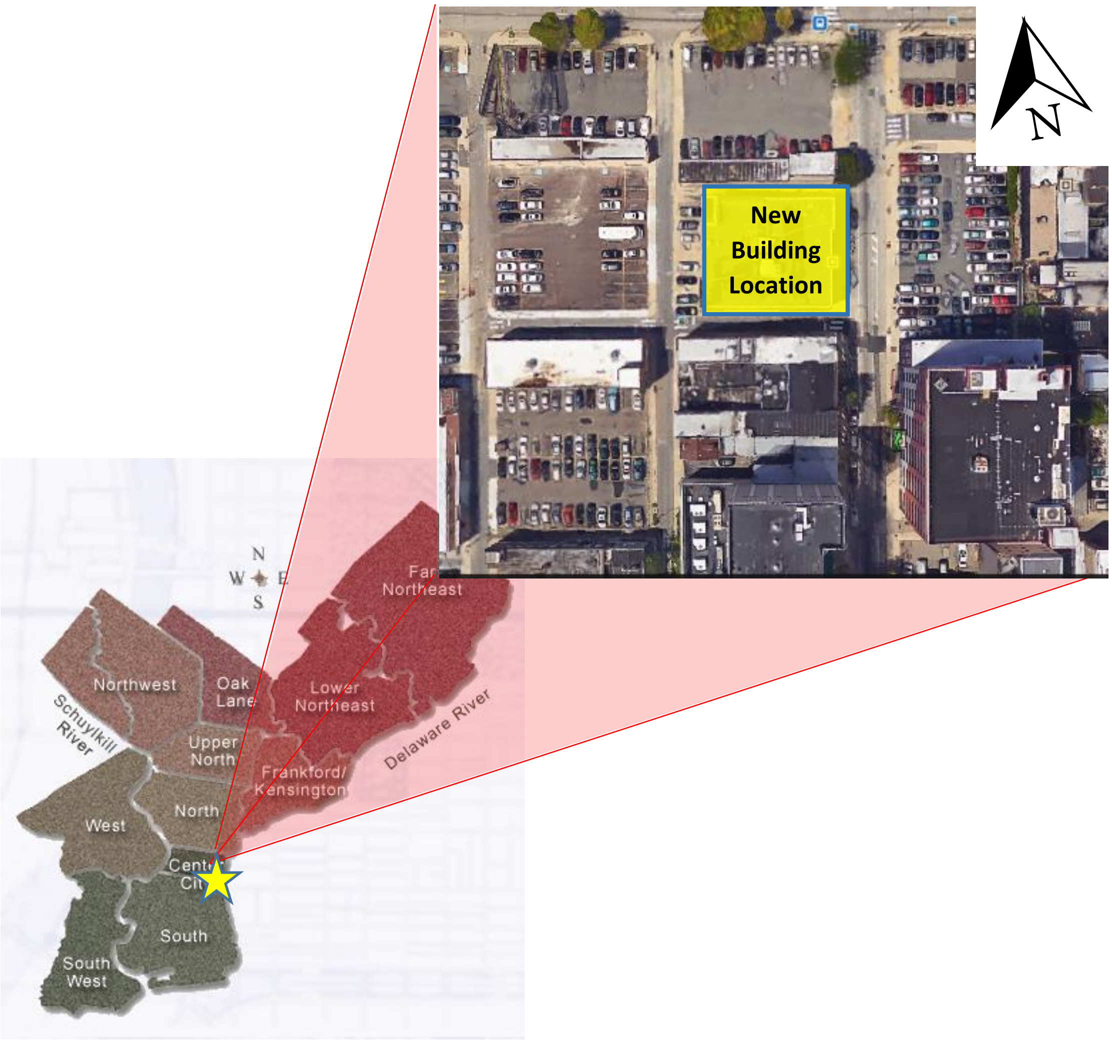


Figure 1: Site location of 230 North 13th Street in Philadelphia, Pa. (Courtesy of Google Maps)

Building Description

230 North 13th Street is a residential transient hotel located in downtown Philadelphia, Pa. This modernized hotel will provide 150 luxurious guest rooms, a private dining area solely for guests, and underground valet parking accessible only by car elevator. There is also a rooftop penthouse which includes an intensive green roof. There are also several extensive green roofs on the low roof areas at the second and third levels. The original two-story structure will be partially demolished and remodified in order to support the 192' superstructure. It is important to note that the existing structure will not support the new building, new steel and concrete columns will be installed to compensate for the added building mass. The design team and the Philadelphia Historical Commission came to an agreement that in order to historically preserve the existing facades, the building must step back 18ft on the southern and eastern sides.

AC Philadelphia occupies 107,680 SF, with the typical floor occupying near 6,000 SF. The lower three levels have a slightly larger footprint than the typical level (levels 3-13). The main means of vertical circulation are through two stairwells located at the northern corners of the building and two elevators (side-by-side) at the center of the floor plan, helping to keep the center of rigidity and center of mass towards the middle of the structure, reducing overall building eccentricity. The bottom floor (at grade level) features a lobby, café, lounge and a kitchen. The second floor is occupied by a small indoor pool, meeting rooms, and several guest rooms. Above

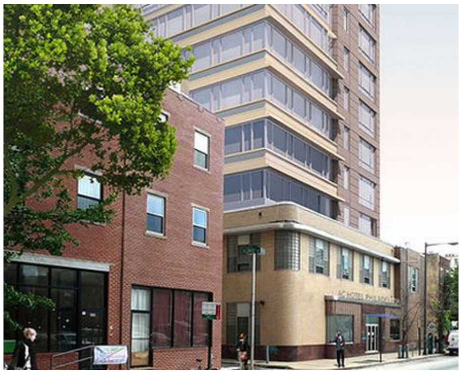


Figure 2: Rendering revealing new hotel atop the existing two-story building. (Courtesy Google Maps)

this, the typical floor contains only guest rooms, and the penthouse at level 14 includes a fitness room, a green roof terrace and some of the mechanical equipment. The majority of the mechanical equipment is contained on the mechanical penthouse (level 15).

Design Codes & Standards

Relevant codes and standards used while designing AC Hotel Philadelphia are listed below:

- International Code Council
    - International Building Code 2009
    - Chapter 11 (IBC 2012) Accessibility Requirements
- American Society of Civil Engineers
    - ASCE 7-05
- AC Hotels By Marriott Design Standards 2014 edition
- AC Hotels By Marriott Module 14 FLS Design Standards January 2015 edition
- City of Philadelphia Building Code with Current Amendments
- AISC Steel Manual (14th Edition)
- ACI 318-11 Concrete Code

Existing Structure Overview

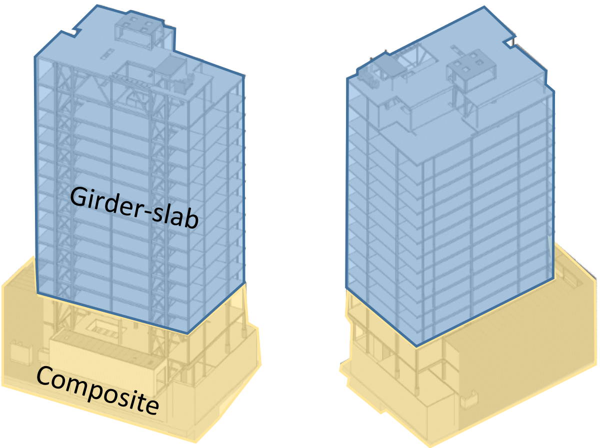


Figure 3: Existing structural Skeleton of AC Hotel Philadelphia.

Columns/Foundation

Figure 4: Existing exterior walls to remain after demolition. (Courtesy Google Maps)

The existing building is comprised of two main floor systems (figure 3) and is supported primarily by steel (wide-flange) columns. At the base of the structure, the columns are supported by the mat slab foundation. Partial demolition will take place to allow for the construction of AC Hotel Philadelphia. Remaining foundation walls can be seen in Figure 4. Underpinning will be needed for the one-story garage to the North and for a portion of the three-story building to the North. The AC Hotel Philadelphia building will be supported on a varying 30”-42” mat foundation, and micropiles will support existing structures on the northern side. Extra steel columns will complement the concrete columns at the basement level to support the entire building load. At the basement level, a mix of concrete (30”x30”, typ.) and steel columns (W10x54, W12x136 and W14x211 typ.) are used. Beginning at the first floor, steel columns (W10x54 and W14x211, typ.) are used. At the top level, W10x33 and W14x120 columns are used. As elevation increases, column weight per foot decreases; however, steel column depths remain the same full height to minimize splice connection detailing.

Lateral System

Laterally, multiple 14” concrete shear walls are utilized up to grade, with braced frames (HSS8x8 and HSS6x6) on all floors above grade (Figures 5 and 6). Braced frame beam sizes are W14x26 typ. Braced frames are utilized around the stair towers located on the northern façade and at the centralized elevator shaft.

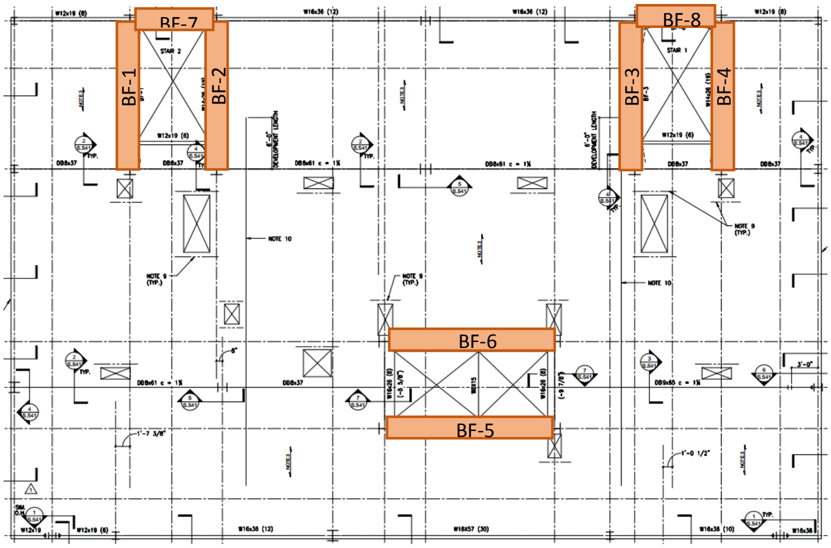


Figure 5: Typ. floor plan showing Lateral Force Resisting Elements (LFRE).

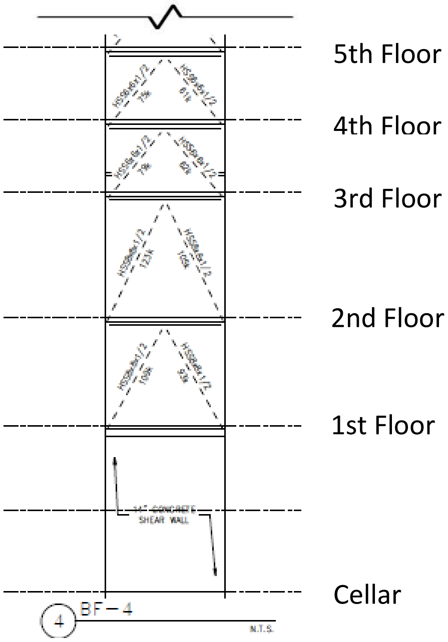


Figure 5: Elevation of LFRE's. The configuration is comprised of chevron-shaped braces.

Typical Floor Bay

Despite a rectangular structural grid, the bay sizes of AC Philadelphia are quite irregular. Bay sizes range anywhere from 14’-25’ in width by 17’-30’ in length. Highlighted in Figure 7 is the average bay size chosen for the AC Philadelphia building. Due to the bay irregularity, the loads on each girder vary, hence why girder (d-beam) sizes range from DB8x37 to DB 9x65. One can see the architects’ intent to open up the floor plan in the building by creating larger interior bays than exterior bays.

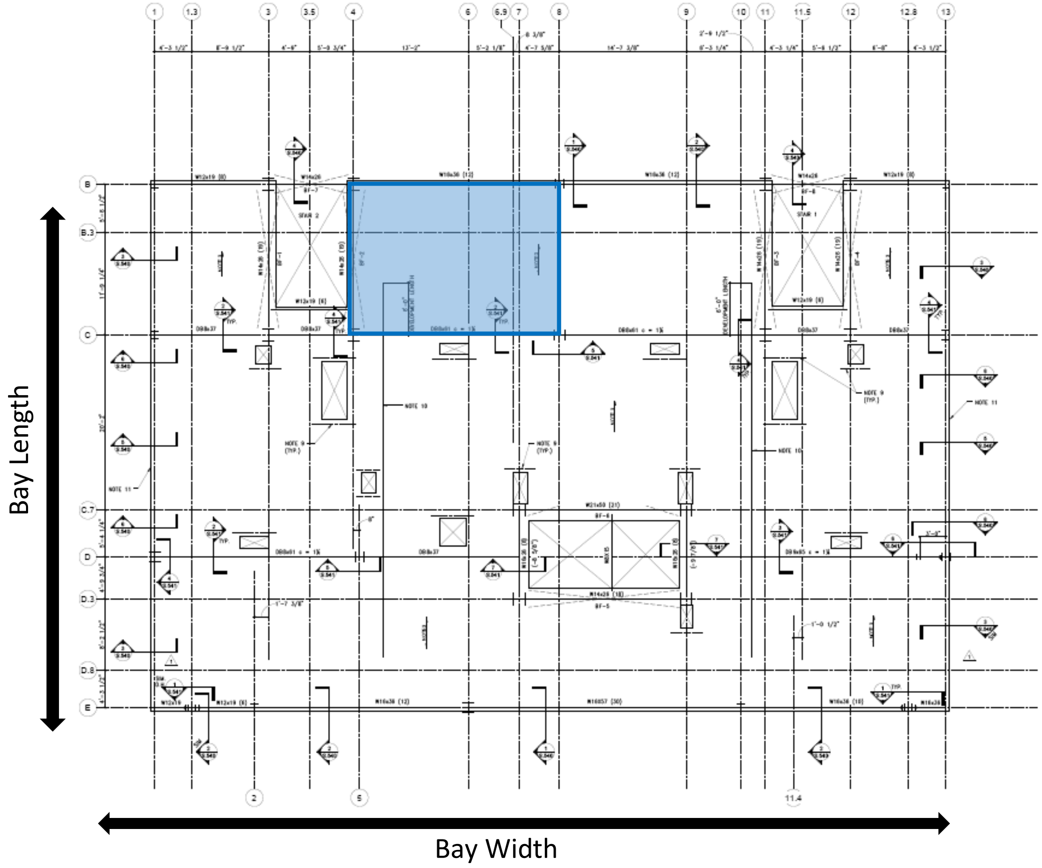


Figure 6: Typical Framing plan showing irregular structural grid. The average bay size (24’x17.5’) is highlighted above.

Shown below are section cuts through the two main floor structures: girder-slab assembly (Figure 8) and a composite floor system (Figure 9). The girder-slab configuration is comprised of 8" thick, precast hollow core planks that sit directly on the bottom flange of a structural steel beam, and protrude past the top flange, concealing the top of the beam. The bottom of the beam is exposed; however this issue can be solved by adding a drop-panel ceiling. Proper construction for inspection requires 2'0" width openings (minimum) at 24" O.C. in order to place #4 transverse rebar. Once all rebar is placed, the openings are backfilled with grout. The grouted transverse rebar helps transfer load between the concrete and steel, therefore, this floor system is assumed to be composite. Since concrete planks are being utilized, infill beams are not needed for the system, and as noted in the previous section, the typical dissymmetrical (D-beam) used for the project ranges from DB8x37 to DB 9x65. Since the floor is a girder-slab, shear studs are not needed. However, in other areas of the building, ¾" diameter, 5" long shear studs are used for composite sections. The D-beams are commonly cambered 1 ¼" to ensure allowable beam deflections. The three lower floors are comprised of: 3 ¼" lightweight concrete over a 3" deep (18 gage) composite metal deck (6 ¼" total floor depth) with 6"x6" welded wire fabric mesh.

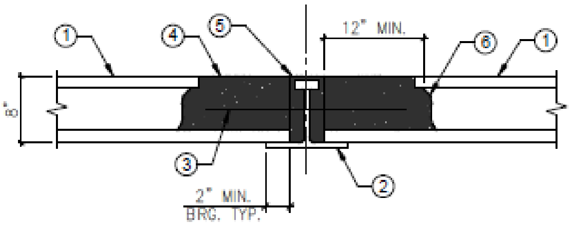


Figure 8: The typical floor plan is 8" hollow-core concrete planks (4'0" wide typ.) that sit on dissymmetrical beams. This system makes up what is known as a Girder-Slab.

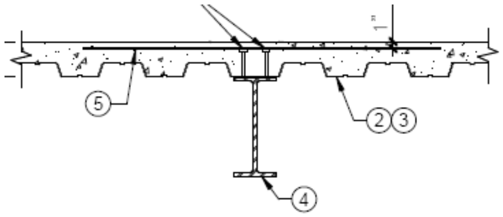


Figure 7: The three lower floors of AC Philadelphia are 3 ¼" LWC over 3", 18 gage composite metal decking.

Load Paths

Gravity

Starting from the rooftop penthouse, loads are applied from the penthouse green roof and transmit through the floor decking onto the girder slab floor system, and then into the columns (Figure 10). The building façade is primarily an ALPHATON Terracotta Panel Rainscreen system. The façade load is transferred into the aluminum substructure, through the panel clips and into the girder slab floor system. Loads are applied on girders and brought down through the columns (W10x33& W14x120) and dispersed onto the mat foundation which will evenly spread the full load into the soil beneath. Loads from the lower floors will follow the same path except that loads will transfer from the composite floor into the girders and down through the columns.

Lateral

Lateral loads are absorbed by the diaphragm and transferred into the column lines where the concentric braced frames will resist the force. This bracing transfers the load down through the cross members and is collected at the base where the foundation walls distribute the load into the surrounding soils.

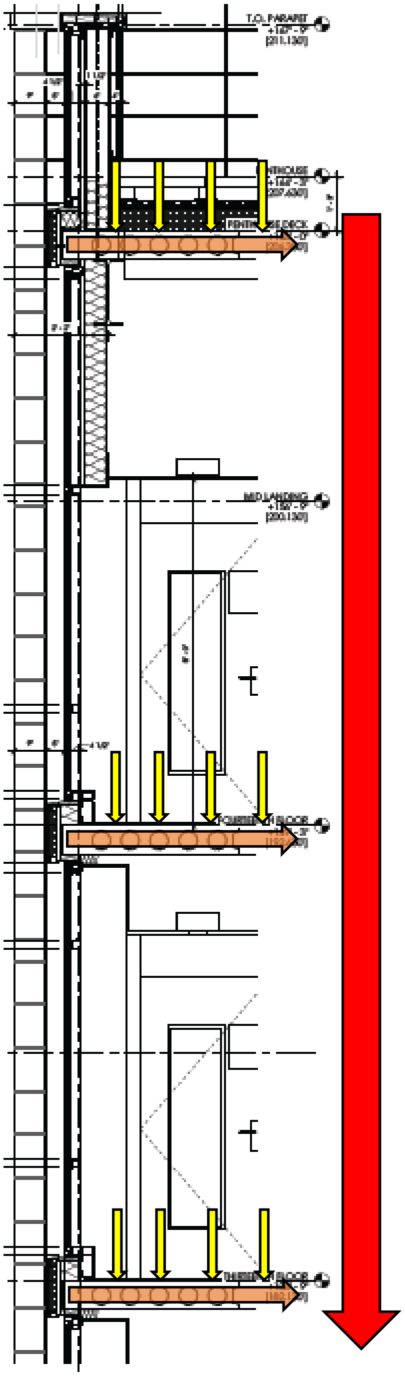


Figure 9: Gravity Load Path

Other Elements

Project designers of AC Philadelphia incorporated multiple green roofs (both intensive and extensive) in their design (Figure 11-13). On the second and third levels, smaller, extensive green roofs are utilized. On the upper penthouse level, a larger, intensive green roof was installed. Since intensive green roofs are designed to support dynamic activity, higher design loads must be accounted for.

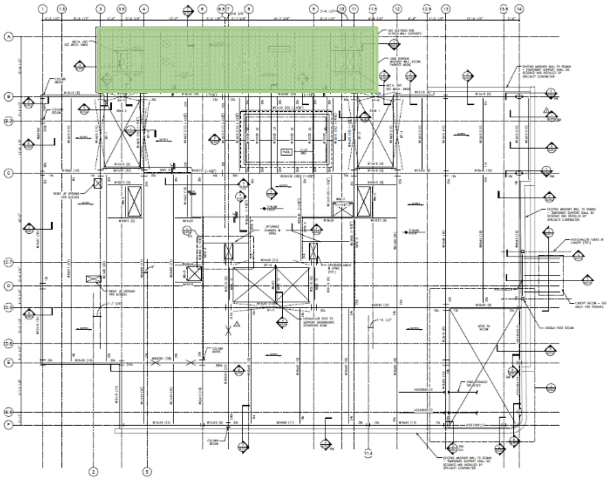


Figure 10: 2nd Floor Green Roof

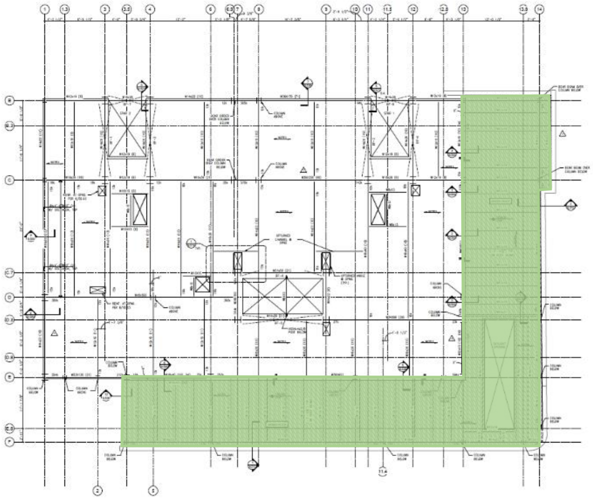
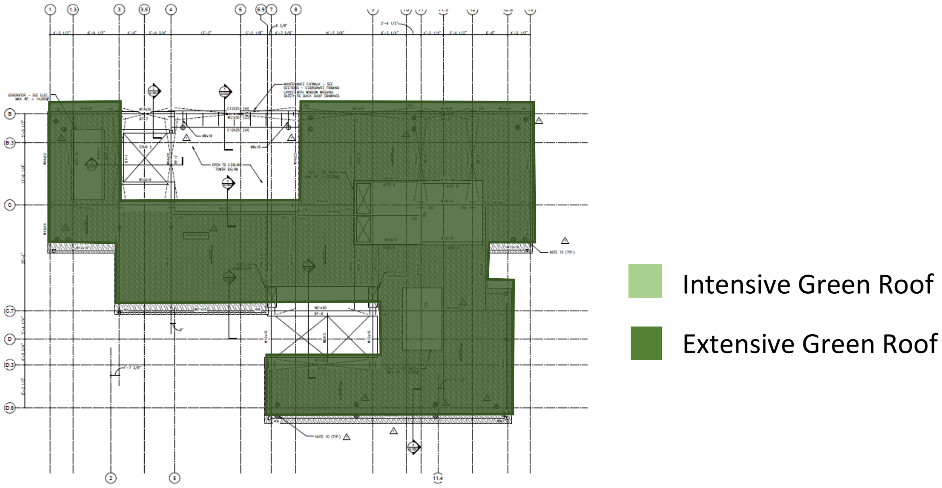


Figure 11: 3rd Floor Green Roof



🟩 Intensive Green Roof

🟩 Extensive Green Roof

Figure 12: Penthouse Green Roof

Depth Study

Problem Statement

The current gravity and lateral framing systems of 221 N. 13th Street have been determined to be satisfactory for strength and serviceability requirements based on the findings in Notebook Submission A, B & C. Although the design is sufficient, the owner and architect have decided to alter the structure, utilizing structural concrete instead of structural steel. A one-way slab system with beams will be implemented as the new floor system and will be supported by concrete columns. The main reason for this is to simplify floor plan layout and determine if it is feasible to implement a cast-in-place slab instead of a precast system in order to maintain the AC Marriott requirement of a floor-to-ceiling height of 9’0”. The existing building design also incorporates a 100% LED lighting scheme. With the efficiency and effectiveness of LED’s on the rise, it was of interest to see if it is cost-efficient to have an LED system. A combination of fluorescent and compact fluorescent (CFL) luminaires will be implemented in place of the LED’s and the initial luminaire cost, along with annual power costs, will be compared to see which scheme is more appropriate.

The Solution

The new design will incorporate a new gravity system, which acts as a one-way concrete slab with beams, and a lateral system comprised of shear walls resisting forces in the N-S direction and concrete moment frames resisting forces in the E-W direction.

Simplified Column Layout

In order to perform a structural redesign, it was determined that the most cost-efficient scheme would evolve from simplifying the existing column geometry. As seen in figures 14 and 15, several grid lines were removed to simplify load paths, and span lengths were established in order to create a more balanced grid system. It was decided to maintain the same floor openings to keep the designs comparable. Although the overall floor dimensions were modified, the changes were not deemed critical enough to have a big impact on the structural design.

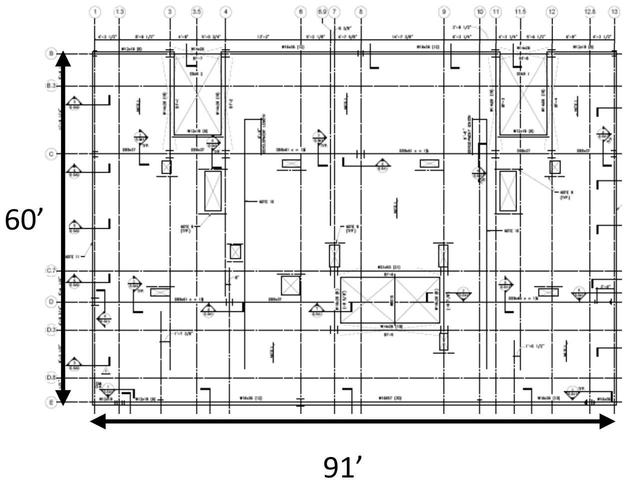


Figure 13: Typical floor plan of the existing girder-slab design.

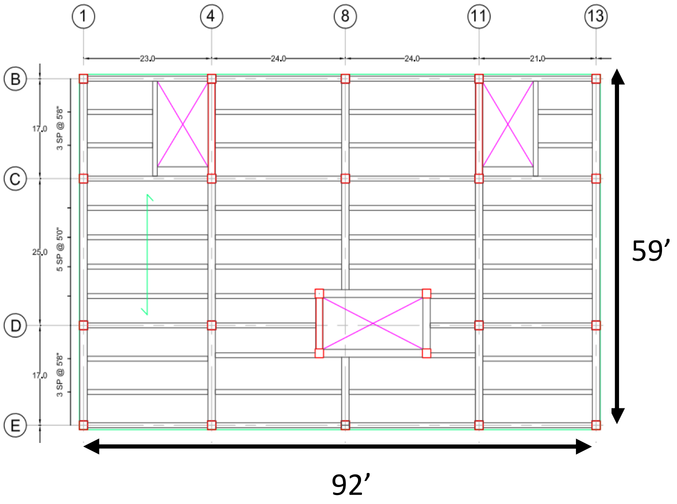


Figure 14: Typ. Floor plan of the proposed one way slab with beams. Multiple grid lines were altered/removed as necessary to create a systematic grid.

Floor System

Assumptions

- Beam sizes
  - Beams-> $\frac{d}{b} < 1.5$
  - Girders-> $\frac{d}{b} < 2.0$

  $\}$ Where "d" is the distance from the top of the beam to the middle of the reinforcement and "b" is the width of the concrete section.

- Rebar
  - The use of rebar larger than #10 is not recommended due to issues with constructability of mechanically splicing and bending thick bars
  - Min clear spacing for parallel bars=diameter of the bar (db), but >1"

- Live Loads
  - Roof-unreducible (30psf)
  - 1st & 2nd-reducible (100psf) public rooms & corridors serving them
  - Typ. Floor- reducible (40psf+10psf) private rooms & corridors serving them
  - LL Reduction (KLL)
    - Int Col-4
    - Ext Col w/ cant slab-3
    - Corner Col w/ cant slab-2

Slab

Design alternatives were investigated in the Fall semester to use as a redesign concept for the Spring semester. Through the systems that were researched, it was determined that none of the three were the ideal fit for application. Through further research it was decided that a one way slab would be best to implement. Minimum slab thickness was established in accordance with Table 9.5(a) in ACI 318-11 as seen in the equation 1 below. Slab thickness was calculated based on the assumption that both ends are continuous. It was also determined that multiple beams beneath the slab would be required for slab deflection control because they act as intermediate supports. The use of beams also helps to decrease overall slab thickness from 8" to 5", which in turn, will maintain the required floor-to-ceiling height. The alternative option would be to remove the beams, increasing the required slab thickness to 11" which would decrease the ceiling-to-floor height beneath the allowable specifications for AC standards. Therefore, the design would require increasing overall building height, which would drive the cost of the proposed system significantly up and the design would be less feasible. This was the driving reason to utilize beams. As seen in equation 1, the minimum required thickness is less than 3". However, this value was increased to 5" for the typical floor and 6" for the lower three floors to allow for adequate reinforcement cover and placement. Additional calculations (found in Appendix B) were executed for the worst case scenario (simply supported) and the slab thicknesses selected remained adequate. Maximum deflections were not necessary to compute because of the conservative slab depth selected.

Equation 1: Minimum slab thickness for a typical floor. The allowable thickness is so thin due to the use of interior beams act as supports for the slab. This value was increased to account for MEP equipment.

$$\min thickness\ (h) = \frac{l}{28} = \frac{5.67*12}{28} = 2.43”$$

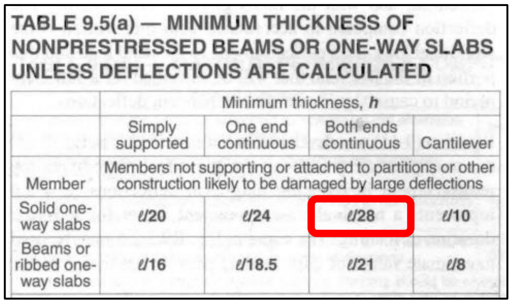


TABLE 9.5(a) — MINIMUM THICKNESS OF NONPRESTRESSED BEAMS OR ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED

	Minimum thickness, *h*			
	Simply supported	One end continuous	Both ends continuous	Cantilever
Member	Members not supporting or attached to partitions or other construction likely to be damaged by large deflections			
Solid one-way slabs	$\ell/20$	$\ell/24$	$\ell/28$	$\ell/10$
Beams or ribbed one-way slabs	$\ell/16$	$\ell/18.5$	$\ell/21$	$\ell/8$

Figure 15: Minimum allowable thickness for non-prestressed beams and slabs without needing to calculate deflections. (Courtesy ACI 318-11)

Reinforcement

Reinforcement for the one-way slab will run in the N-S direction, allowing the slab weight to distribute evenly to the beams beneath. Slab reinforcement was calculated using equations (10-3) and (10-4) from ACI for minimum bar area required and maximum bar spacing respectively. It was found that #4@12” O.C. is adequate for reinforcement. In figure 17 below, the floor load path for a typical floor is displayed. Once the slab distributes itself to the beams, the load is transferred to girders (N-S) where the load is deposited down into the columns.

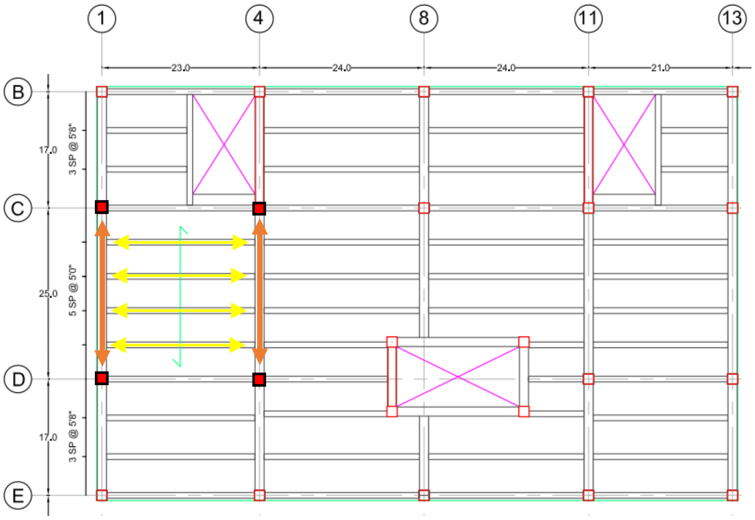
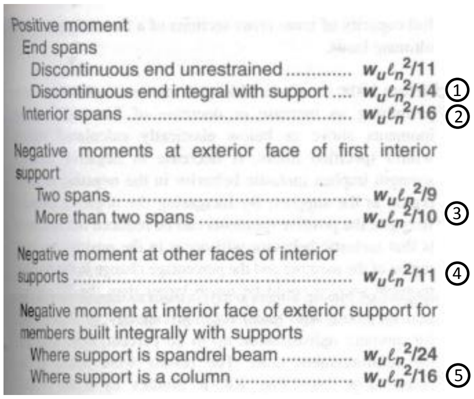


Figure 16: Load path for a typical floor. The one way slab system evenly distributes load to the beams (running E-W). Beams transfer the load into the girders (running N-S) which carry the load into the columns that frame the particular bay.

Beam Design

Beams and girders were also designed in accordance with Table 9.5(a) assuming both ends are continuous. Design moments from ACI (section 8.3.3) can be found in figure 18 below. The longest span (24') was selected, granting the rest of the spans a conservative design approach.



Positive moment		
End spans		
Discontinuous end unrestrained	$w_u\ell_n^2/11$	
Discontinuous end integral with support	$w_u\ell_n^2/14$	①
Interior spans	$w_u\ell_n^2/16$	②
Negative moments at exterior face of first interior support		
Two spans	$w_u\ell_n^2/9$	
More than two spans	$w_u\ell_n^2/10$	③
Negative moment at other faces of interior supports	$w_u\ell_n^2/11$	④
Negative moment at interior face of exterior support for members built integrally with supports		
Where support is spandrel beam	$w_u\ell_n^2/24$	
Where support is a column	$w_u\ell_n^2/16$	⑤

Figure 17: Approximate moments for various locations along the beam span. (Courtesy ACI 318-11, section 8.3.3.)

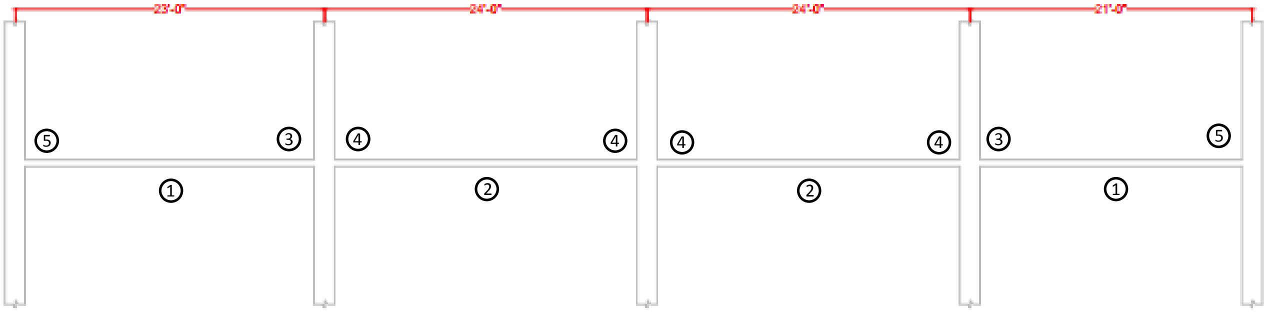


Figure 18: Location of approximate moments along a full beam span for a typical floor.

Beam sizes were approximated based on the following equation:

Equation 2: Derivation of the flexural capacity equation for concrete. If the equation below holds true, a simplified equation for the required area of steel can be used.

$$20Mu < bd^2$$

As stated in the assumptions at the beginning of the section, beams and girders were designed to certain proportions to reduce issues with shear and deflection. Beams were designed that d < 1.5b and girders, d < 2.0b. Using equation 2 above enables the designer to calculate the required area of steel using equation 3 below:

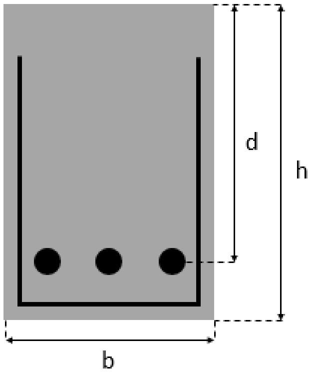


Figure 19: Concrete beam section dimensions labeled.

Equation 3: Area of reinforcement required to satisfy strain requirements.

$$As_{reqd} = \frac{Mu}{4d}$$

Equation 4: Standard procedure to determine the area of reinforcement required for a flexural member.

$$As_{reqd} = \frac{Mu}{0.9fyjd} \rightarrow a = \frac{Asfy}{0.85f'cb} \rightarrow c = \frac{a}{\beta} \rightarrow \varepsilon_s = 0.005 < \varepsilon_c(\frac{d-c}{c}) \rightarrow if\ \varepsilon_c > \varepsilon_s \therefore \phi = 0.9$$

After sufficient rebar sizes were selected, beams were also checked for shear capacity. It was established that all beams and girders need shear rebar (stirrups). Figures 21 and 22 below show an example of a detailed section of a designed beam and the rebar layout for a typical floor.

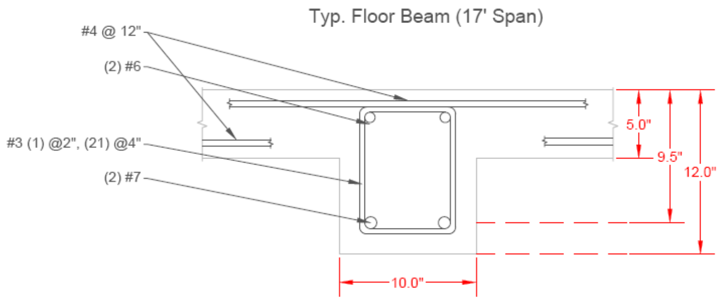


Figure 21: Detailed beam section for a typical floor (17' span).

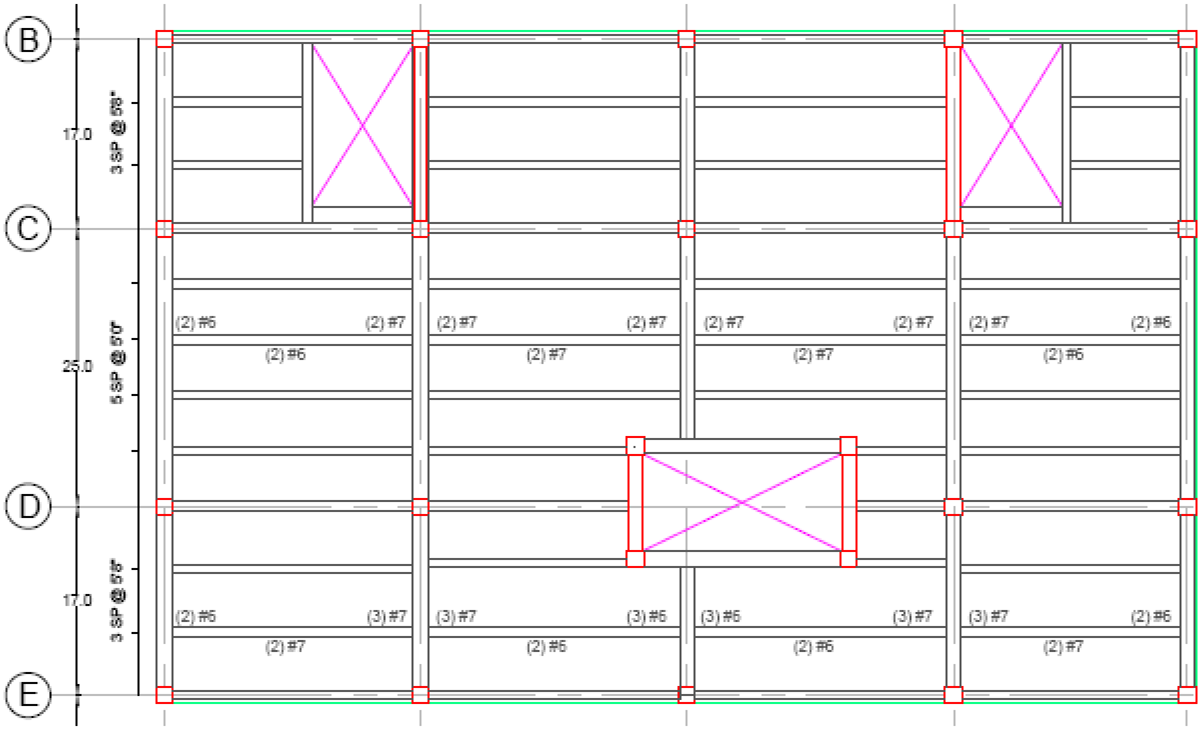


Figure 20: Reinforcement layout for a typical floor.

Certain areas within the building require particular attention to detailing because of special loading conditions. In particular, the green roofs and pool area were designed differently. A full set of section details and floor plans can be found in Appendix B.

Column Design

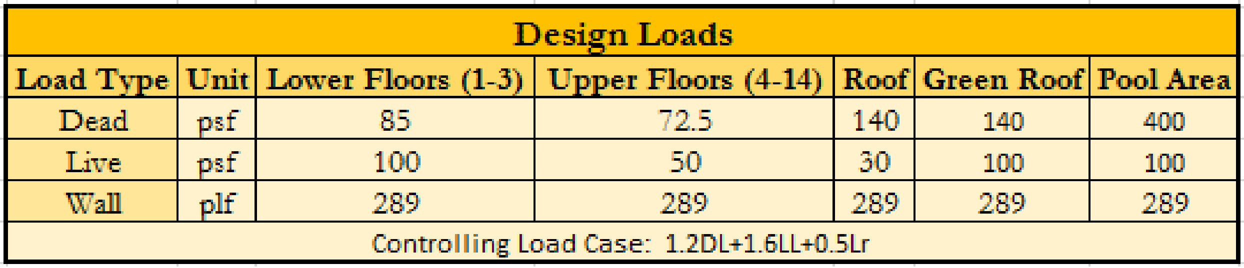
Assumptions

- Concrete strength-4ksi
- Exterior columns-add wall weight when considering loads
- Rebar for compression members: $0.01Ag < Ast < 0.08Ag$
- LL Reduction
  - Typ. Floor - reducible
  - Roof - not reducible
- Columns under consideration ($K_{LL} = 4$ for both columns)
  - Exterior: C1 (Mu = 29kft)
  - Interior: C4 (Mu ~ 0kft)

Process

Sizes were determined for a typical interior and exterior column with the use of the CRSI Design Handbook (2008 edition). Before columns could be selected from the manual, design loads were established as the following:

Table 1: Design loads used for various areas within the building.



Design Loads						
Load Type	**Unit**	**Lower Floors (1-3)**	**Upper Floors (4-14)**	**Roof**	**Green Roof**	**Pool Area**
Dead	psf	85	72.5	140	140	400
Live	psf	100	50	30	100	100
Wall	plf	289	289	289	289	289
Controlling Load Case: 1.2DL+1.6LL+0.5Lr						

Dead loads above include an additional 10psf for superimposed loads. Live load for the upper floors include 10psf for partitions. Once design loads were determined, column loads were calculated. Square columns were selected from Table 3-12 in the CRSI handbook for architectural reasons with column sizes ranging from 18"x18" to 24"x24". Selected columns and their capacities are displayed in table 2. Column sizes are displayed in table 3 which summarizes loads and sizes for a typical interior and exterior column. Extra capacity was reserved for the columns so they can withstand lateral effects from wind forces.

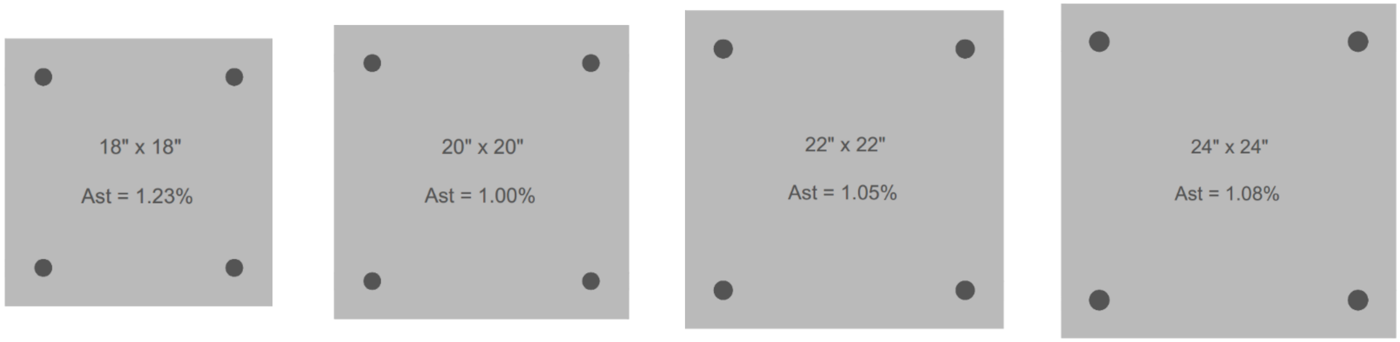
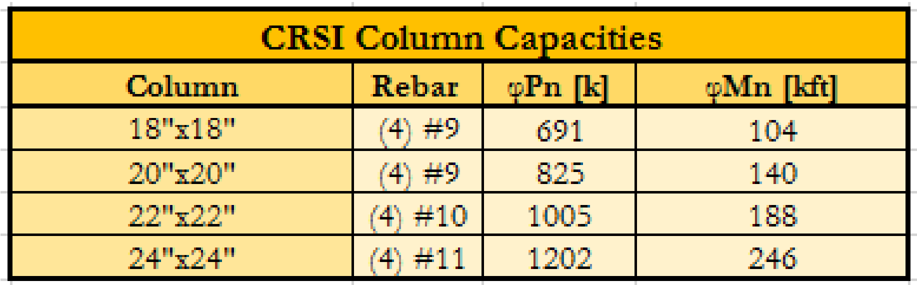


Figure 22: Various column sizes specifying their respective areas of steel.

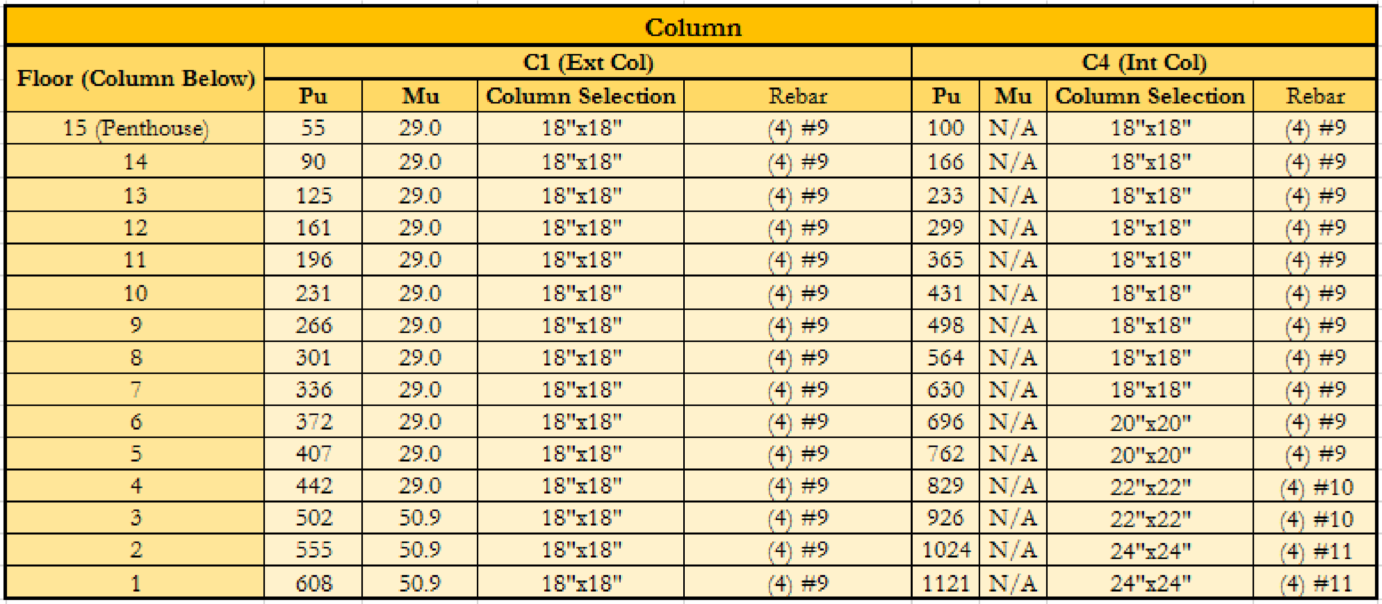
Table 2: Column capacities from the CRSI Manual.



CRSI Column Capacities			
Column	Rebar	φPn [k]	φMn [kft]
18"x18"	(4) #9	691	104
20"x20"	(4) #9	825	140
22"x22"	(4) #10	1005	188
24"x24"	(4) #11	1202	246

It should be noted that larger reinforcement sizes require additional on-site labor to mechanically bend and weld the steel. To avoid these issues, selected rebar sizes are #11 or smaller. Slenderness effects were also taken into account as per ACI section 10.10 (Eqn. 10-6) and it was found that slenderness effects were permitted to be neglected.

Table 3: Column size and reinforcement summary. Calculated design moments and axial loads are displayed as well.



Column								
Floor (Column Below)	C1 (Ext Col)				C4 (Int Col)			
	Pu	Mu	Column Selection	Rebar	Pu	Mu	Column Selection	Rebar
15 (Penthouse)	55	29.0	18"x18"	(4) #9	100	N/A	18"x18"	(4) #9
14	90	29.0	18"x18"	(4) #9	166	N/A	18"x18"	(4) #9
13	125	29.0	18"x18"	(4) #9	233	N/A	18"x18"	(4) #9
12	161	29.0	18"x18"	(4) #9	299	N/A	18"x18"	(4) #9
11	196	29.0	18"x18"	(4) #9	365	N/A	18"x18"	(4) #9
10	231	29.0	18"x18"	(4) #9	431	N/A	18"x18"	(4) #9
9	266	29.0	18"x18"	(4) #9	498	N/A	18"x18"	(4) #9
8	301	29.0	18"x18"	(4) #9	564	N/A	18"x18"	(4) #9
7	336	29.0	18"x18"	(4) #9	630	N/A	18"x18"	(4) #9
6	372	29.0	18"x18"	(4) #9	696	N/A	20"x20"	(4) #9
5	407	29.0	18"x18"	(4) #9	762	N/A	20"x20"	(4) #9
4	442	29.0	18"x18"	(4) #9	829	N/A	22"x22"	(4) #10
3	502	50.9	18"x18"	(4) #9	926	N/A	22"x22"	(4) #10
2	555	50.9	18"x18"	(4) #9	1024	N/A	24"x24"	(4) #11
1	608	50.9	18"x18"	(4) #9	1121	N/A	24"x24"	(4) #11

Lateral Design

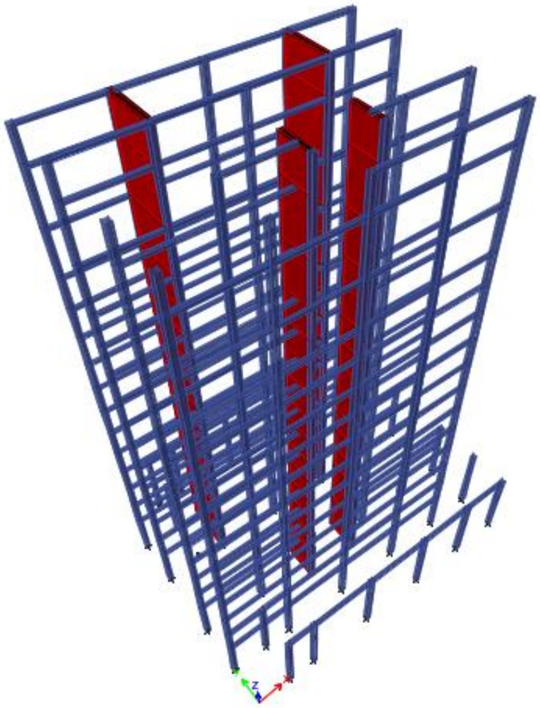


Figure 24: 3D model of AC Hotel Philadelphia created in ETABS. Certain elements of the building that do not have an impact on the proposed investigation were not modeled.

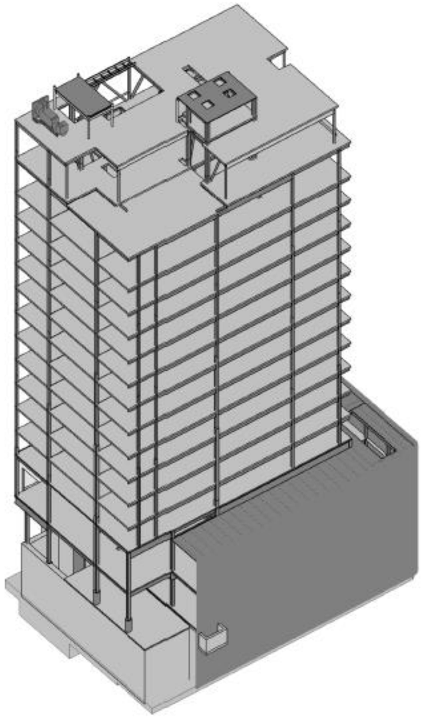


Figure 23: 3D representation of AC Hotel Philadelphia. (Courtesy Holbert Apple Associates)

Process

Due to the adjustment in floor systems, it was determined to modify the LFRE's of the building to better compliment the gravity system. In the existing design, multiple concentric braced frames (HSS 6x6x1/2 typ.) were used in both directions. Concrete moment frames will now resist lateral loads in the E-W direction and concrete shear walls (both full building height) will resist forces in the N-S direction as seen in FIG. Shear walls are much more rigid than moment frames, making them a more suitable fit for the N-S direction since greater wind loads are experienced. In order to gain a better understanding of the new system being implemented, ETABS (2015 edition) was utilized. Wind and seismic loads were recalculated to account for the change in structure material and stiffness. In order to acquire accurate computer-generated results, careful measures were taken while using ETABS. The elements modeled were:

- Slabs
- Slab openings
- Columns
- Column piers
- Shear walls
- Beams that compose the moment frames

All other elements were not modeled because they do not resist any lateral forces and therefore do not impact the sizing and design. Moment connections were specified for the frames and the walls were set to mesh every four feet so that the elements in a particular direction were properly analyzed when lateral loads are applied. It was also extremely important to specify that the diaphragm is semi-rigid (flexible) to allow proper joint-fixity movement. The last step was to apply wind loads (user defined) in both directions. Full wind load calculations can be found in Appendix B. It was determined through ASCE 7-05, Equivalent Lateral Force Procedure (section 12.8) that wind still controlled the design in both orthogonal directions. This makes sense due to the geographical site location and realizing that wind would most likely control on the east coast. Through the investigation, it was found that the overall wind forces declined and base shears decreased nearly 33% even with the switch to concrete. Beam sizes originally sized for gravity loads were adequate for lateral forces as well, therefore, they do not need to be upsized. ETABS was also utilized for its detailing capabilities. The shear walls defined in the program were detailed and it was determined that boundary elements would be needed for extra capacity against lateral forces.

Pictured below in figures 26 and 27 are the LRFE's on a typical floor and lower floor for both directions.

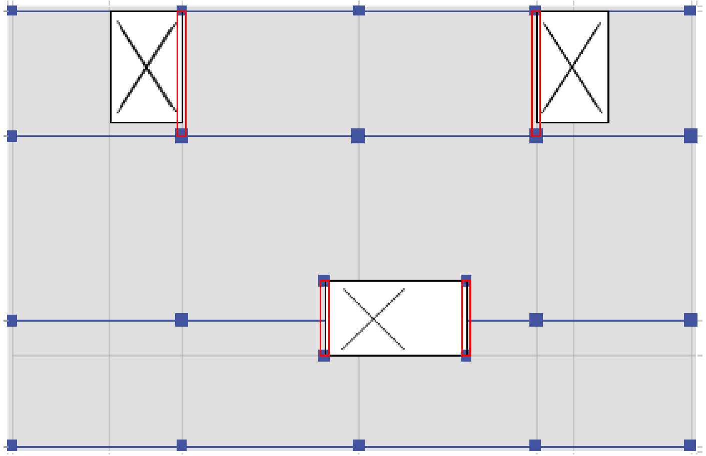


Figure 25: LFRE's displayed for both orthogonal directions for a typical floor. (4) Shear walls, (2) 17'-long and (2) 10'-long resist forces in the N-S direction. (4) Concrete moment frames resist forces in the E-W direction.

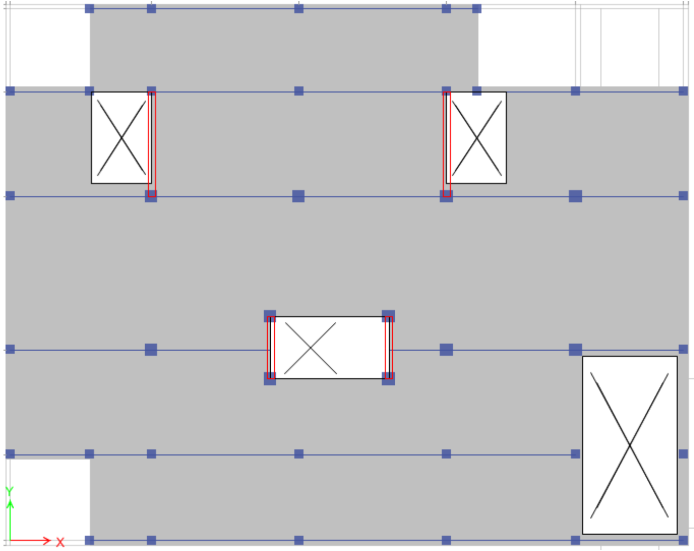


Figure 26: LFRE's displayed for both orthogonal directions for a lower floor. Two additional concrete moment frames were placed to control inter-story drifts at the lower levels.

Figures 28 and 29 below shows typical detailing for a shear wall and full results can be found in Appendix B.

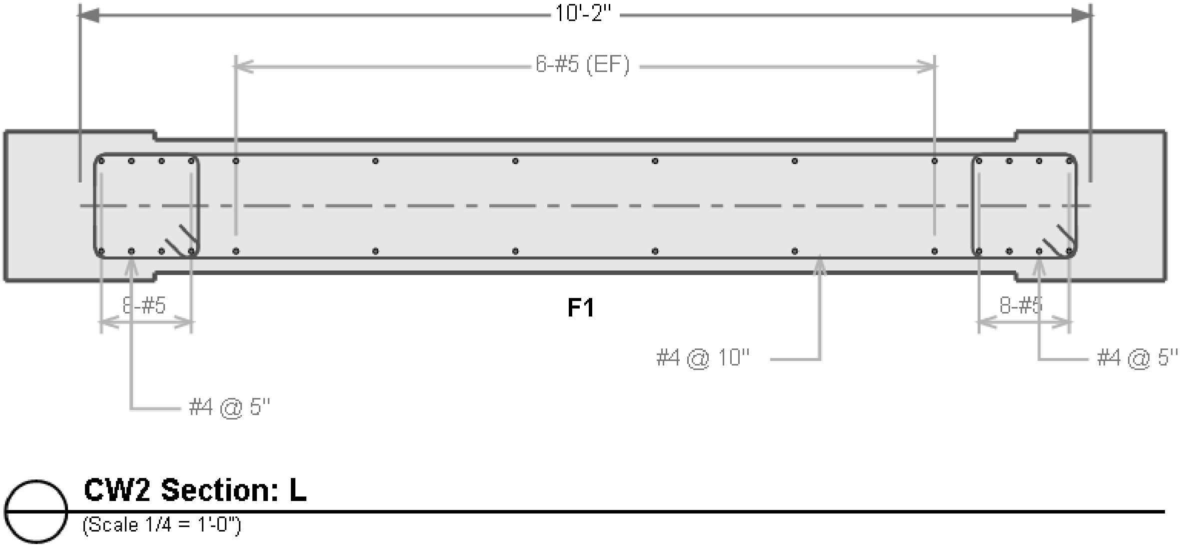


Figure 27: Typical detail for 10' long shear walls (SW-1 and SW-4). Boundary elements were needed for extra resistance against the overturning moment of the building. (Courtesy of ETABS 2015)

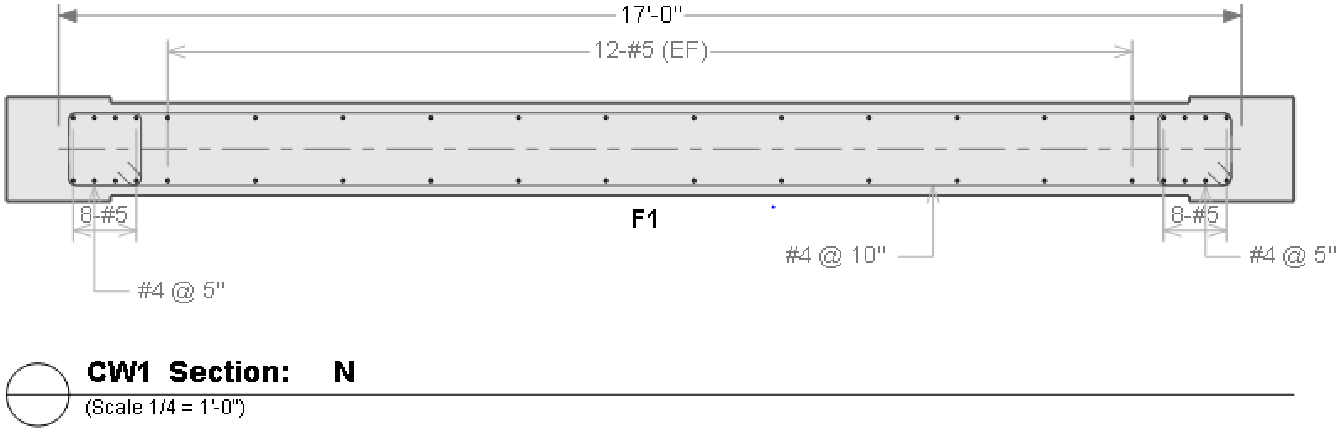


Figure 28: Typical detail for 17' long shear walls (SW-2 and SW-3). Boundary elements were also required for OTM. (Courtesy of ETABS 2015)

Results

Results in figure 30 and table 3 below reveal that each orthogonal direction achieved allowable drift values for all elevations. After a preliminary analysis was run, it was determined that the lower floors required additional moment frames to keep inter-story drifts within allowable limits. Hand calculations and other results located in Appendix B verify that the LFRE's are adequate for lateral forces applied.

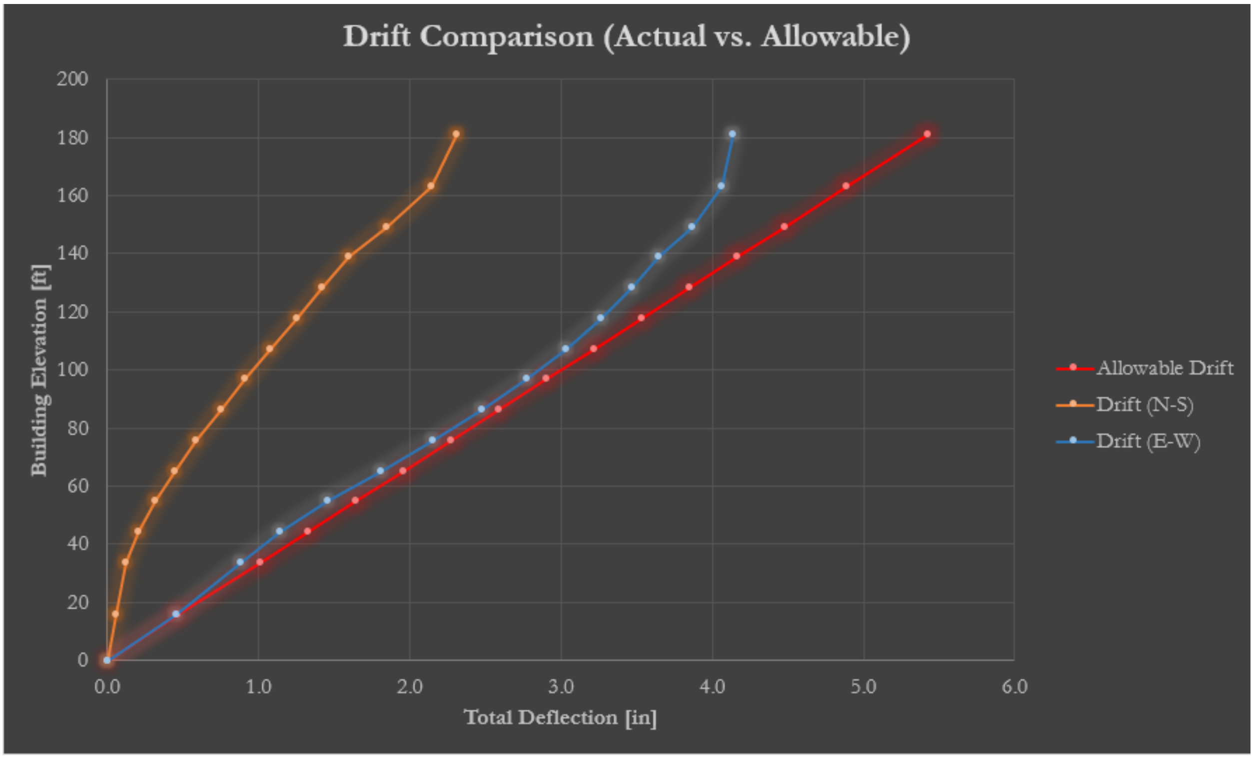
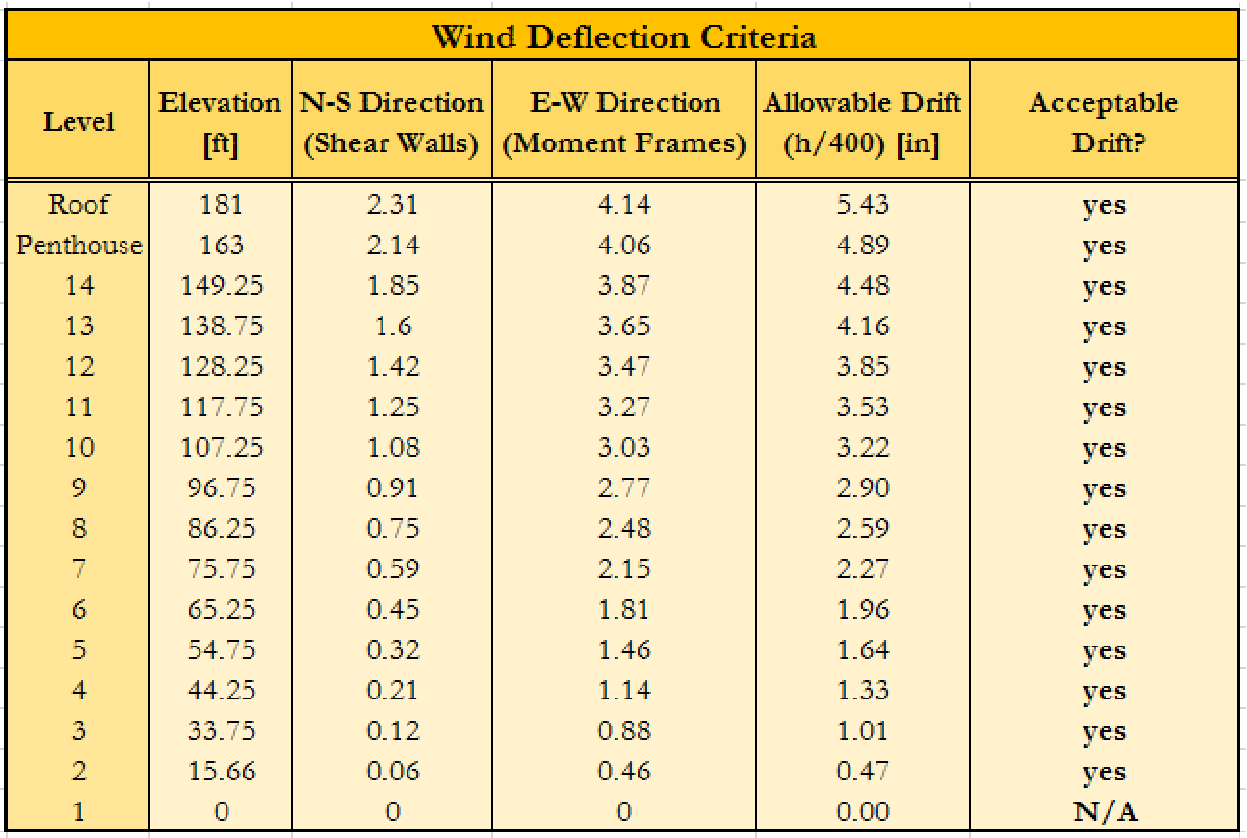


Figure 29: Building drifts displayed for both orthogonal directions. Drift values were measured at each story height.

Table 2: Wind drift values displayed for all story elevations. These values were compared with the allowable drift of (h/400), where h is the elevation of the story being analyzed.



Wind Deflection Criteria

Level	Elevation [ft]	N-S Direction (Shear Walls)	E-W Direction (Moment Frames)	Allowable Drift (h/400) [in]	Acceptable Drift?
Roof	181	2.31	4.14	5.43	yes
Penthouse	163	2.14	4.06	4.89	yes
14	149.25	1.85	3.87	4.48	yes
13	138.75	1.6	3.65	4.16	yes
12	128.25	1.42	3.47	3.85	yes
11	117.75	1.25	3.27	3.53	yes
10	107.25	1.08	3.03	3.22	yes
9	96.75	0.91	2.77	2.90	yes
8	86.25	0.75	2.48	2.59	yes
7	75.75	0.59	2.15	2.27	yes
6	65.25	0.45	1.81	1.96	yes
5	54.75	0.32	1.46	1.64	yes
4	44.25	0.21	1.14	1.33	yes
3	33.75	0.12	0.88	1.01	yes
2	15.66	0.06	0.46	0.47	yes
1	0	0	0	0.00	N/A

Since the LFRE's passed for both gravity and lateral requirements, the proposed lateral system is considered adequate for analysis purposes.

Breadth Studies

Breadth #1: Construction Management Cost Analysis

Introduction

The construction management breadth entails a comparison of a detailed cost estimate between the original structure (girder slab system) and the proposed design (one-way slab). The cost analysis encompasses all necessary materials to form/erect the structure. Material, labor and equipment values were extracted from RS Means: Facilities Construction Cost Data (2014 edition). The total estimate does not include overhead and profit due to the fact that values can vary depending on the contractor. Time and location factors were also considered. Philadelphia is a densely populated area, which is why the location factor (1.139) is higher than the national average (1.0). Although the project has not gotten underway, the time factor was determined based off the midpoint (08/2016) of the initial project timeline (10/2015-06/2017) and assumed 3% annual inflation lending to a factor of 1.08.

Process

Starting with the existing steel design, values for the slab (hollow-core planks), structural steel (W-shapes) and shear studs were accounted for. Bare cost values also encompass structural bolts, delivery, installation and erection. The majority of takeoffs were in linear feet [LF] with the exception of the precast planks, measured by the required square footage [SF]. To account for all steel members, assumptions were made for beams sizes due to only certain members receiving values in the manual. There were also associated crane costs because erecting the steel structure will require the use of two cranes due to site logistics. Each crane was assumed to be needed for six months, with a cost of $143k per month. A sample line was extracted from the estimate and can be seen below in table 5. A full detailed estimate (separated by category) is available in Appendix C for all assumptions and details.

Table 3: Sample line taken from the existing structure estimate. A full estimate can be found in Appendix C.

Category	Line Number	Description	Crew	Daily Output	Labor Hours	Unit	Material
Slab	03 41 13.50.0100	Precast Structural Concrete, Slab, Hollow-Core Planks, 8" Thick	C-11	3200	0.023	SF	7.10

For the proposed design, formwork, rebar, concrete and concrete placement were all accounted for. Bracing and shoring costs are pre-included in unit prices. Formwork is measured by total contact area between the formwork and concrete [SFCA], reinforcement by the total rebar [LB] and concrete by the cubic yard [CY]. In an effort to reduce total cost it was assumed that the forms will be used four times before they are considered waste. Careful measures will be taken by construction workers when handling the forms in the field so that the forms will be in workable condition for multiple uses. Concrete will be placed by pump to avoid needing a hoist/crane until the upper floors.

Estimate totals yielded within 1% of each other (approx. $3.2 million), giving reason to believe that the proposed design is feasible and can be verified below in table 4. However, the lack of qualified concrete subcontractors in the region with the capabilities of such a large project

Table 4: Total estimated costs of the existing and proposed structure are compared.

System	Structure Cost	Difference
Steel	$3,164,409	0.9%
Concrete	$3,193,185	

most likely explains why the steel structure was selected. Figure 31 below shows a component breakdown by percentage of the total cost.

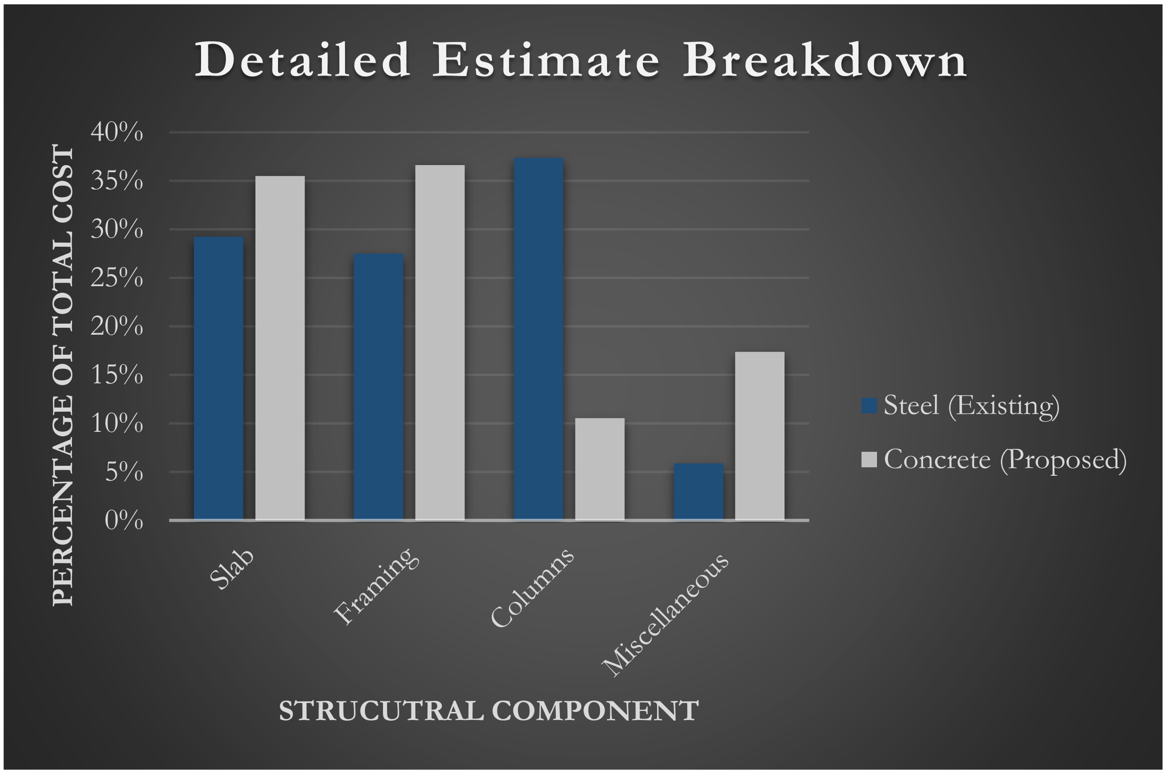


Figure 30: Cost breakdown by structural component comparing the existing and proposed structural systems.

Summary

In summary, the proposed one-way slab with beams structure only costs 1% more than the existing structure and is a viable design due to its simplicity and redundancy. However, it would decrease the floor-to-floor height which would be a major concern and it may be difficult to locate a concrete subcontractor that could supply the entire building. One large schedule impact would be the curing time needed for all of the concrete forms, compared to the ease of construction with the precast girder-slab system. Overall, the proposed design is feasible so long as a reliable concrete source could supply the project at a reasonable cost; however, it resulted in a longer construction schedule overall.

Breadth #2: Alternative Lighting Analysis

Introduction

AC Philadelphia utilizes an LED lighting scheme throughout the entire building. Since LEDs are already implemented, there was a study performed to see if there were more appropriate solutions for the reception/lobby space. This involved replacing LED fixtures with a combination of linear and compact fluorescent (CFL) luminaires. It is known that on average, LEDs consume less power than fluorescents, however fluorescents tend to be cheaper to purchase up front. All of the LED luminaires will be directly switched out with fluorescent fixtures that have similar distributions and lumen outputs; the lighting layout however may be slightly altered to better suit illuminance requirements for the spaces. With these thoughts in mind, this lighting breadth will investigate the two systems, and determine which is more cost efficient initially, as well as over a 20-year period.

Process

In order to correctly model the space, AGi32-16.7 was utilized for light calculations and renderings. It was determined that the most appropriate way to compare the two designs was to model the original design first, so that illuminance values (vertical and horizontal) of the original system could be compared with those found in Table 28.2 of the IES Lighting Handbook (10th edition) for Hospitality and Entertainment Facilities: Lobbies and Lounge (table 5). A reference model of the part of the lobby in interest was created in AutoCAD and exported into AGi32. Part of the lobby was not included after it was determined to be out of the scope of this breadth. Once in AGi32, the model was modified to replicate the space as closely as possible, including finishes and relevant furniture. IES files for the existing luminaires were located and used to create the most accurate model. In figure 32 below, a rendering shows a view of how the existing space would appear, and a Pseudo-color shows the light levels in the space. It was also essential to achieve uniformity within the space so the contrast of light does not distract occupants while still attaining target illuminance values which depends on the task being performed. To compare uniformities, calculation grids were placed in the models. The "Lobby" grid measures illuminance values for the entire space shown below. The "Uniformity" grid (highlighted in pseudo-rendering) was strategically placed at the center of the room so that corners of the room (outliers) did not skew results. All relevant values are in figure 33 below.

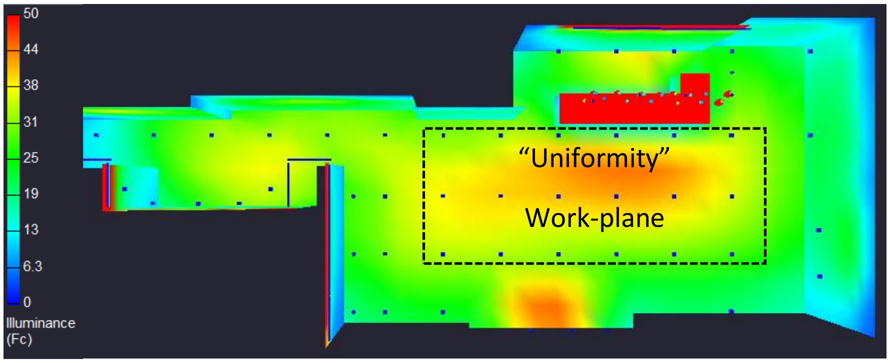
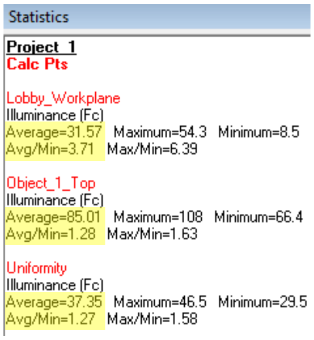


Figure 31: A pseudo-color rendering displaying the lighting layout for the lobby and reception area. Results reveal higher light levels near prominent surfaces and work planes (reception desk and seating area). (Courtesy AGi32-16.7)



Project 1
Calc Pts

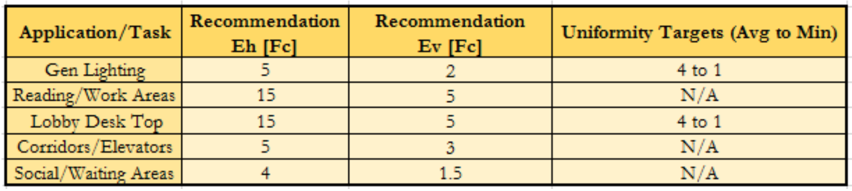
Lobby_Workplane
Illuminance (Fc)
Average=31.57 Maximum=54.3 Minimum=8.5
Avg/Min=3.71 Max/Min=6.39

Object_1_Top
Illuminance (Fc)
Average=85.01 Maximum=108 Minimum=66.4
Avg/Min=1.28 Max/Min=1.63

Uniformity
Illuminance (Fc)
Average=37.35 Maximum=46.5 Minimum=29.5
Avg/Min=1.27 Max/Min=1.58

Figure 32: Displays illuminance results from AGi32 for three work planes defined within the program. (Courtesy AGi32-16.7)

Table 5: Recommended illuminance targets for various tasks are displayed. Values extracted from the IES handbook are given in lux. These values were converted to foot-candles so light levels in AGi32 could be compared directly.



Application/Task	Recommendation Eh [Fc]	Recommendation Ev [Fc]	Uniformity Targets (Avg to Min)
Gen Lighting	5	2	4 to 1
Reading/Work Areas	15	5	N/A
Lobby Desk Top	15	5	4 to 1
Corridors/Elevators	5	3	N/A
Social/Waiting Areas	4	1.5	N/A

As one can see, the values attained by the original design exceed the recommended values by over 500%, which gives reason to believe it was an intentional attempt by the architect to create a bright, welcoming area. The same approach was repeated for the proposed system; the only difference being new fluorescent luminaires were found and substituted for the original LED fixtures. These luminaires were selected based on criteria that was determined after examining the space, attempting to incorporate: low wattage and similar lumen output in order to keep the comparison consistent. Before creating the layout, IES files were examined to determine the distribution of light from each luminaire. The LED lights tend to have a higher percentage of direct downlight, giving reason to why an abundance of luminaires were needed to light the space, and why light levels on surfaces surpass their targets by so much. Fluorescents on the other hand, have a wider light distribution, allowing for better overall uniformity. Because of this, it was possible to remove luminaires from the scheme, bringing the initial cost down, while the design still exceeded illuminance targets, however they were closer compared to what the LEDs provided. The results of the proposed scheme are found below.

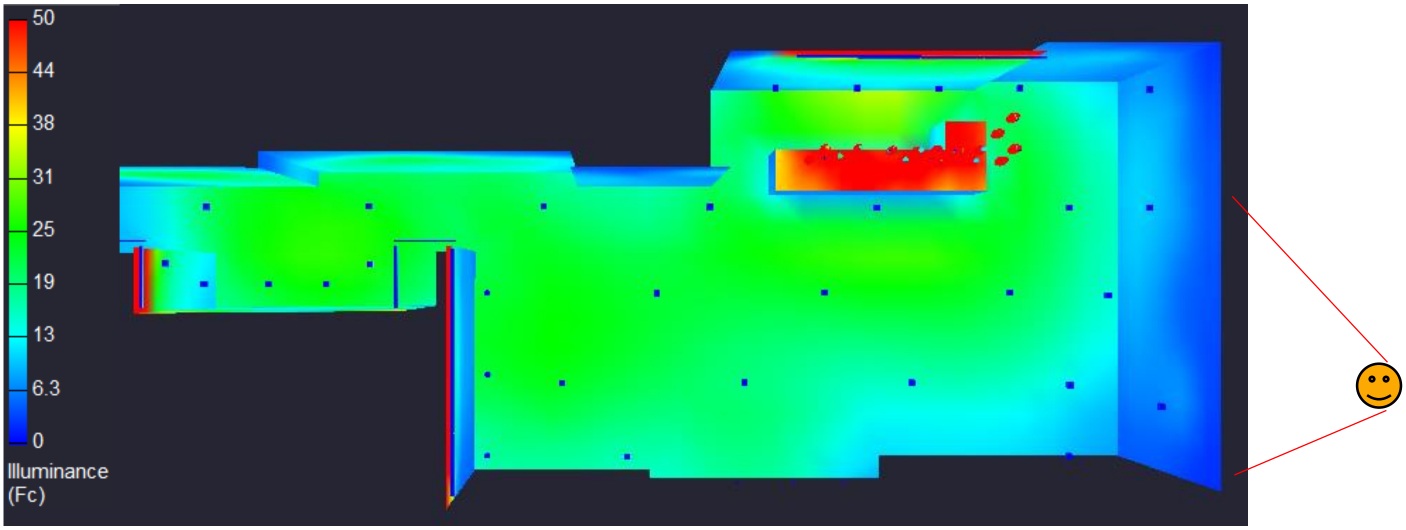
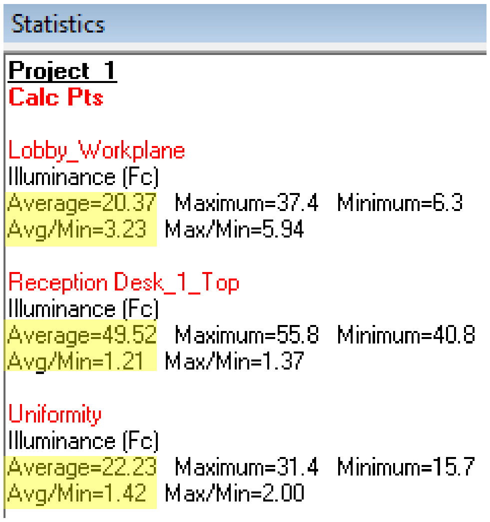


Figure 33: Proposed lighting scheme is revealed. Multiple fixtures were removed and illuminance targets were still attained, therefore the initial cost decreased. Better uniformity was achieved from altering the layout and the fact that the CFL selected has a wider light distribution. (Courtesy AGi32-16.7)



Statistics

Project 1
Calc Pts

Lobby_Workplane
Illuminance (Fc)
Average=20.37 Maximum=37.4 Minimum=6.3
Avg/Min=3.23 Max/Min=5.94

Reception Desk_1_Top
Illuminance (Fc)
Average=49.52 Maximum=55.8 Minimum=40.8
Avg/Min=1.21 Max/Min=1.37

Uniformity
Illuminance (Fc)
Average=22.23 Maximum=31.4 Minimum=15.7
Avg/Min=1.42 Max/Min=2.00

Figure 34: Displays illuminance results from AGi32 for three work planes defined within the program. Light levels are closer to the target values, while still achieving overall uniformity. (Courtesy AGi32-16.7)

Figure 35: Rendering of the lobby and reception areas is displayed. A combination of fluorescent and CFL downlights, wall washers and pendants combine to produce an acceptable lighting scheme for the LED to fluorescent redesign. (Courtesy AGi32-16.7)

Once the models were replicated, a 20-year cost analysis was performed. As seen in figure 37 and table 6 below, the fluorescent lighting scheme is initially about half the cost compared to LEDs. However, after approximately 13yrs, the LED scheme lends itself as the less expensive option because the system only requires 3/4 of the necessary power needed for the fluorescent option.

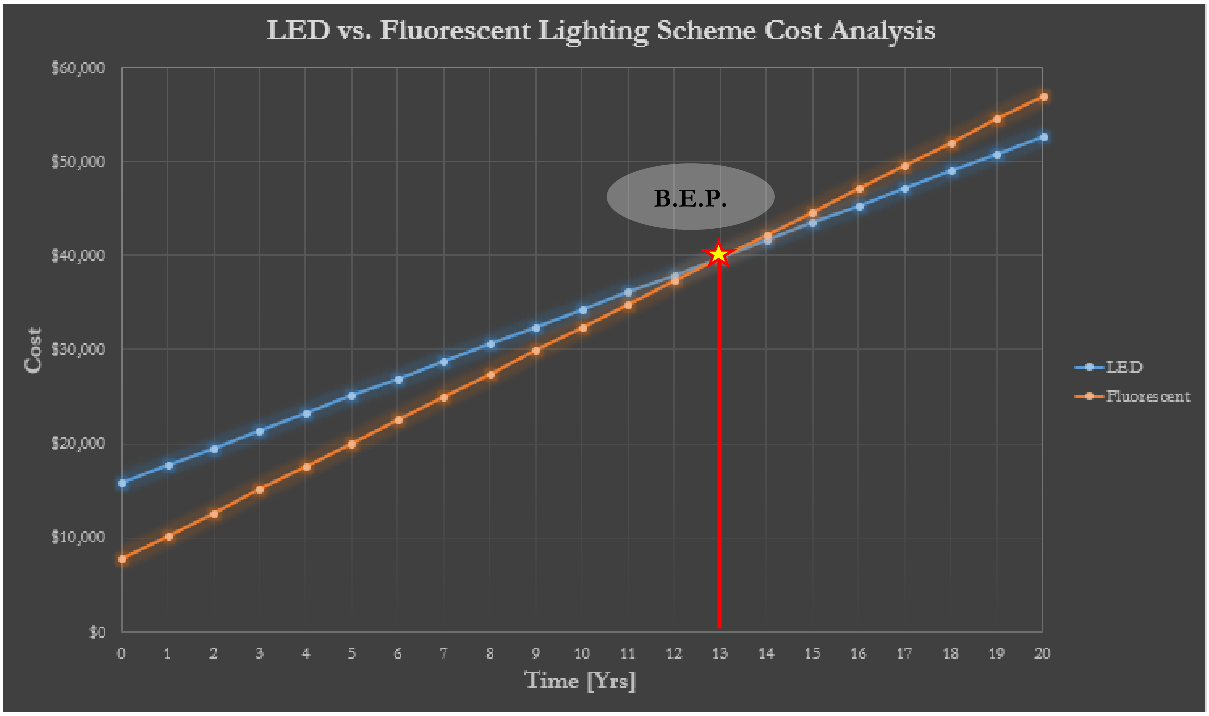
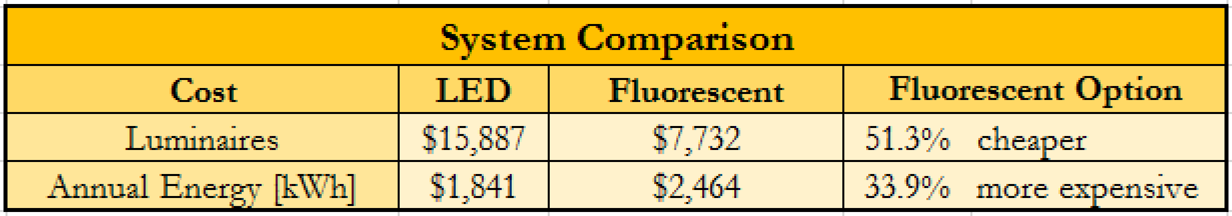


Figure 36: Existing and proposed lighting schemes are compared over a 20-year span to determine which design is feasible in the long run. The break-even point is also marked to show the point that the LED scheme would become cheaper due to lower annual power consumption.

Table 6: Lighting schemes are compared based on their initial cost for fixtures and their annual cost in energy.



System Comparison			
Cost	LED	Fluorescent	Fluorescent Option
Luminaires	$15,887	$7,732	51.3% cheaper
Annual Energy [kWh]	$1,841	$2,464	33.9% more expensive

Summary

After performing this interesting investigation, it was recognized that employing linear and CFLs luminaires is indeed cheaper in the short run. However, it is now evident why the design professionals decided to select an LED lighting scheme. Not only do LED fixtures have a lifespan of over twice that of CFLs, but it is also statistically proven that the cost of LED luminaires are decreasing too, which essentially explains the decision for the hotel to pursue LED's with optimism that AC Philadelphia will stay in operation for longer than the break-even point of approximately 13 years.

Conclusion

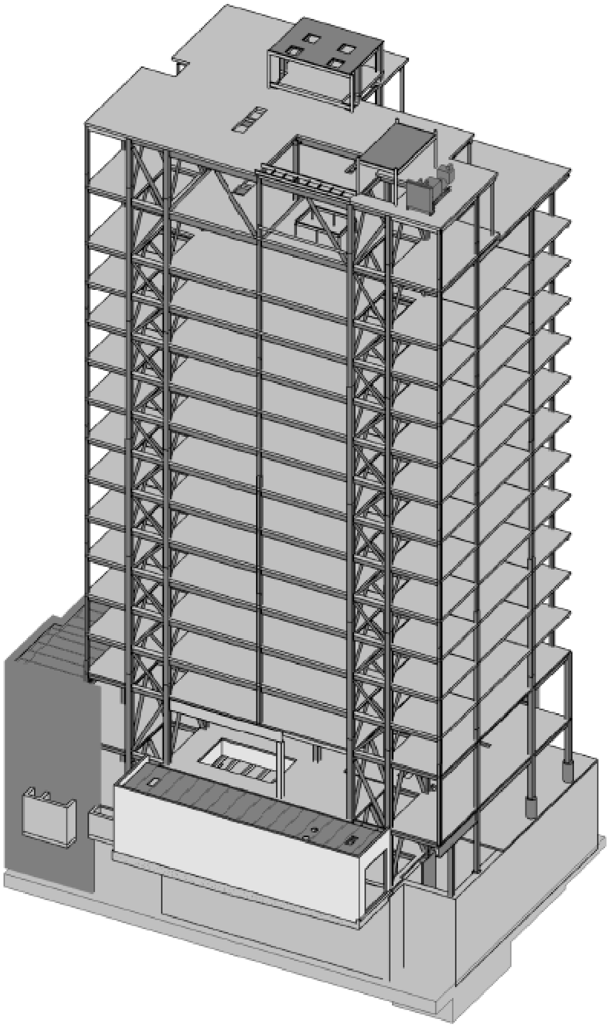
AC Hotel Philadelphia is a 15 story (including penthouse) transient hotel occupying a little over 107,000SF. This report explored the proposed design of replacing the existing steel structure with a concrete structure. The proposed gravity system entails a one-way concrete slab with intermediate beams. For lateral support, concrete moment frames will be employed in the E-W direction and (4) 14” shear walls will resist loads in the N-S direction. A cost analysis was performed to determine the feasibility of the proposed design. After analyzing the data, it was determined that the suggested design is within 1% of the cost of the existing structure, making it feasible. With this said, it may be difficult to locate a qualified concrete supplier, driving the final price to the point that it may not be feasible. Due to the fact that all of the concrete is cast-in-place, the overall schedule of the project would be longer.

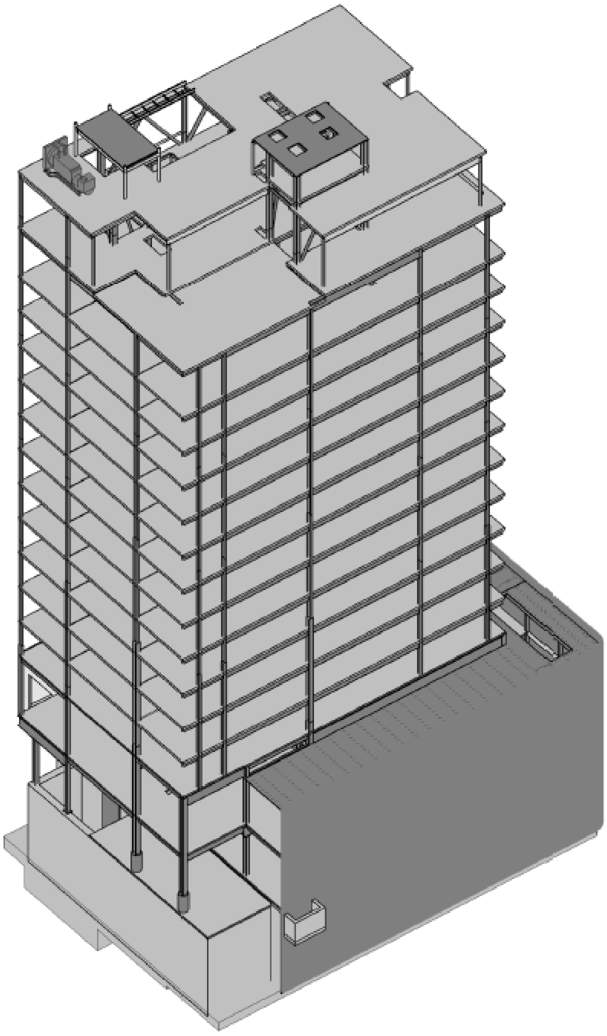
The lighting layout for the lobby/reception area was also analyzed to determine whether or not another scheme would be appropriate. Fluorescents and CFLs were implemented in place of the LED’s and it was determined that the fluorescent option would be less expensive in the short run. However, after approximately 13 years, the LED scheme lends itself as the less expensive option because of lower power consumption cumulatively.

Overall, the proposed changes to AC Hotel Marriott are feasible within reason. Given that the hotel is built to last longer than the B.E.P. of 13 years for LEDs, it is evident that the existing scheme is appropriate. If the project is constructed at the proper time, the proposed structural design would also be feasible. MEP equipment would fit within the depths of the beams, allowing my goal of maintaining the required floor-to-ceiling height of 9’0” to be achieved.

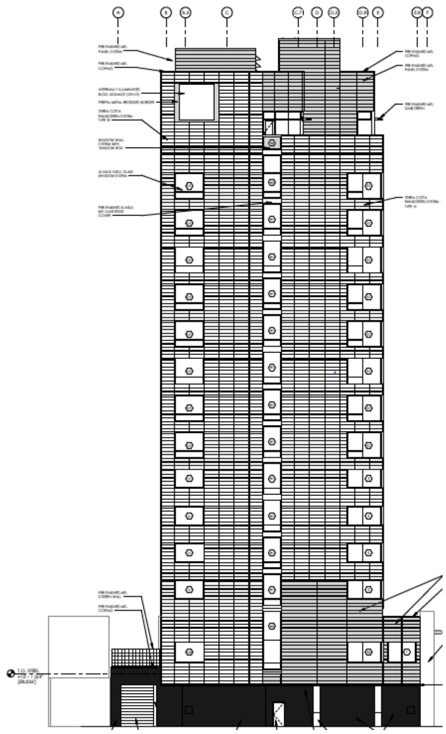
Appendix A (Existing Structure Supplementary Info)

Isometric Views



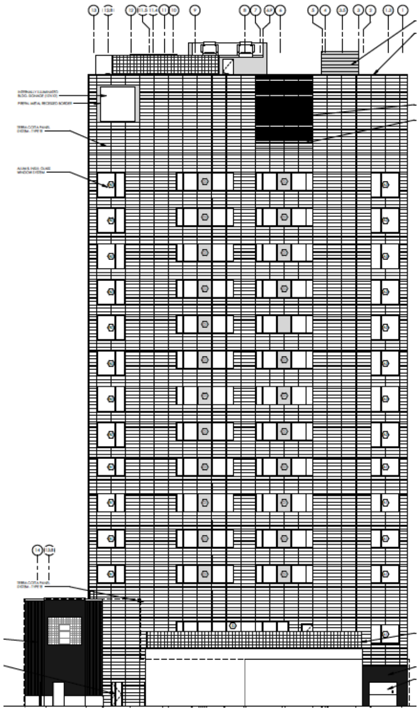


Building Elevations

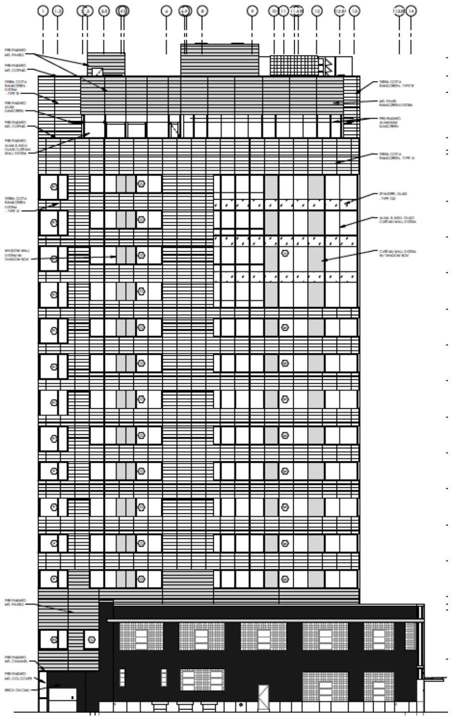


West Elevation

East Elevation



North Elevation

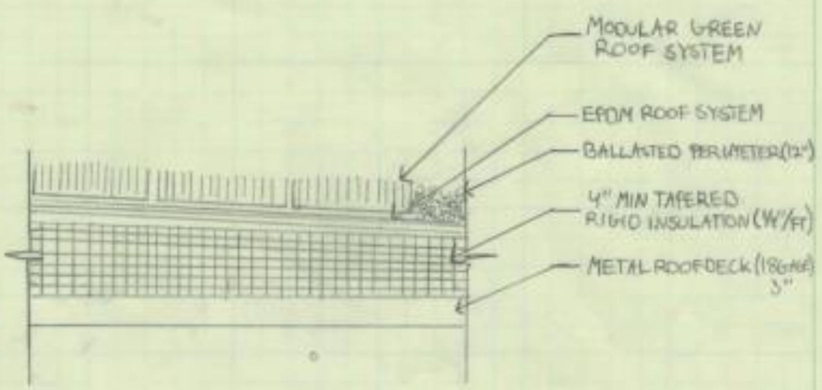


South Elevation

Gravity Loads

NOTEBOOK SUBMISSION A | GRAVITY LOADS | JESSE BORDEAU | 1

Roof Dead & Live Loads (2nd, 3rd Story, Roof & Penthouse)



Design Loads:

- Live Load (Min): 100psf → roof garden [IBC 2009, Section 1607.1, Table 1607.1]

- Dead Load:

Modular Green Roof System = 50psf

Ballasted EPDM Roof System = 3 PSF b/c only @ perimeter

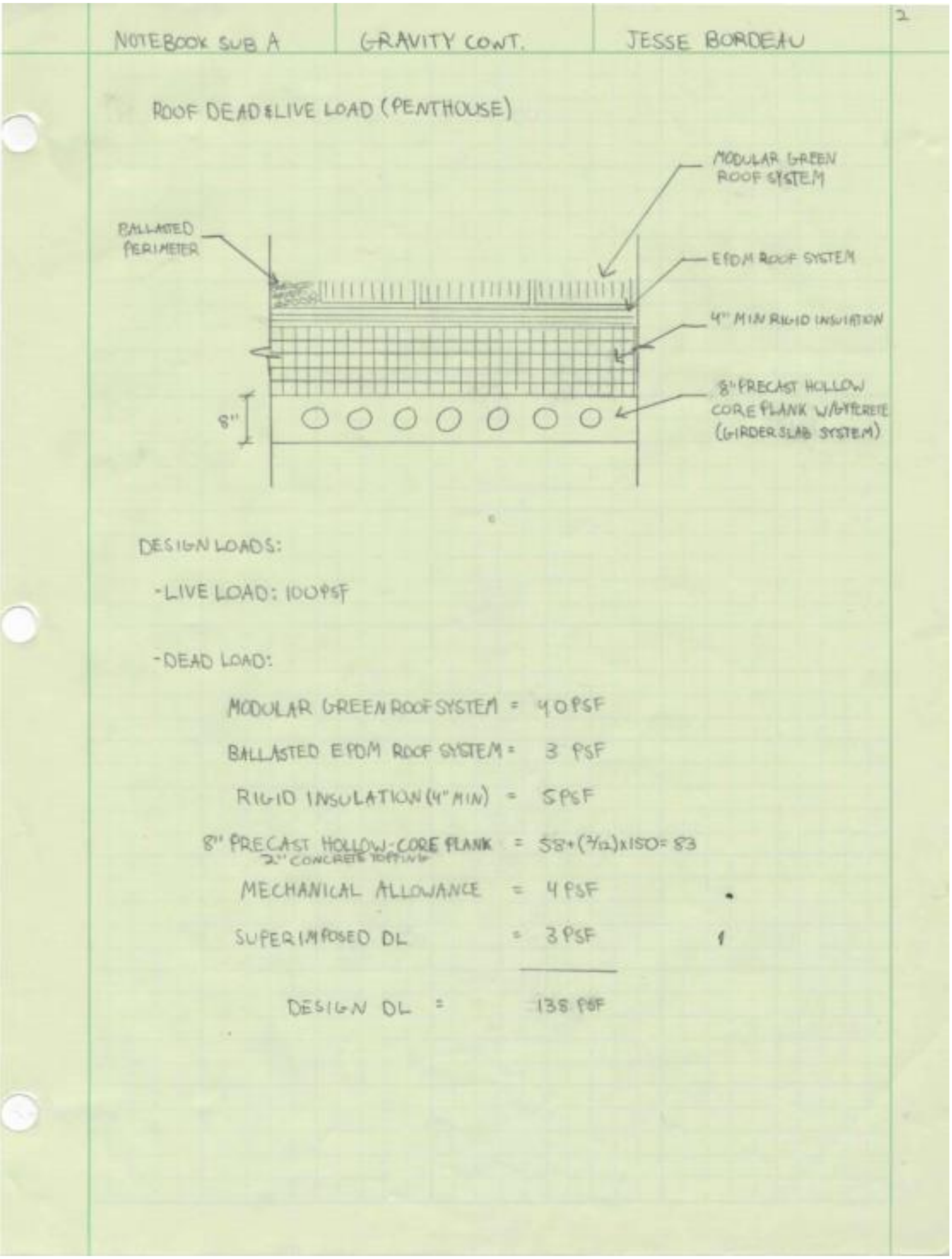
Tapered Rigid Insulation = (5")(1 psf/in) = 5psf

Metal Roof Deck (18gage) = 3psf

Mechanical Allowance = 4psf

Superimposed Dead Load = + 3 psf

Design DL = 68psf compared to 60psf for extensive green roof



NOTEBOOK SUB A | GRAVITY CONT. | JESSE BORDEAU | 2

ROOF DEAD & LIVE LOAD (PENTHOUSE)

DESIGN LOADS:

-LIVE LOAD: 100PSF

-DEAD LOAD:

MODULAR GREEN ROOF SYSTEM = 40 PSF

BALLASTED EPDM ROOF SYSTEM = 3 PSF

RIGID INSULATION (4" MIN) = 5PSF

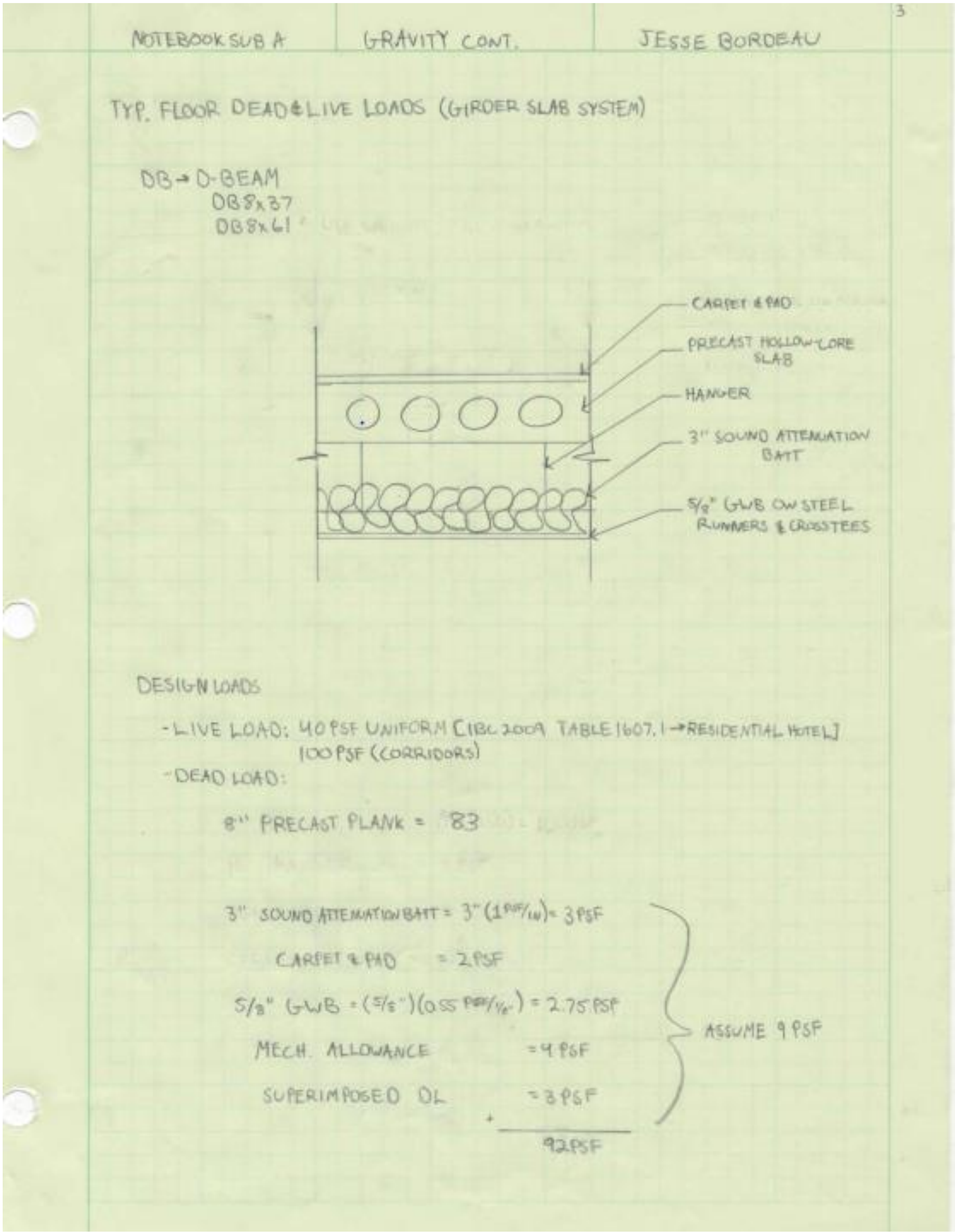
8" PRECAST HOLLOW-CORE PLANK = 58+(2/12)x150= 83

2" CONCRETE TOPPING

MECHANICAL ALLOWANCE = 4 PSF

SUPERIMPOSED DL = 3 PSF

DESIGN DL = 138 PSF



NOTEBOOK SUB A | GRAVITY CONT. | JESSE BORDEAU | 3

TYP. FLOOR DEAD & LIVE LOADS (GIRDER SLAB SYSTEM)

DB → D-BEAM
DB8x37
DB8x61

DESIGN LOADS

- LIVE LOAD: 40 PSF UNIFORM [IBC 2009 TABLE 1607.1 → RESIDENTIAL HOTEL]
  100 PSF (CORRIDORS)
- DEAD LOAD:

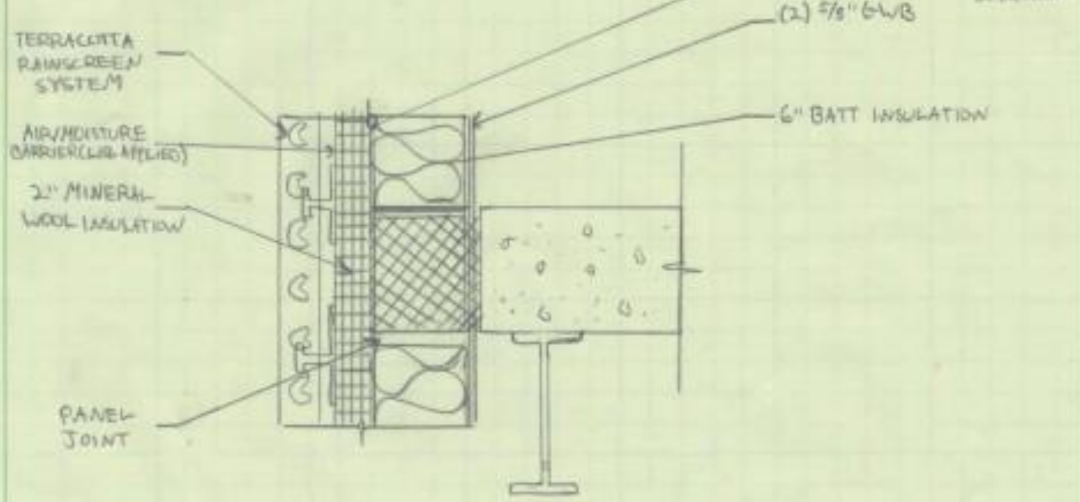
8" PRECAST PLANK = 83

3" SOUND ATTENUATION BATT = 3" (1 PSF/IN) = 3 PSF
CARPET & PAD = 2 PSF
5/8" GWB = (5/8")(0.55 PSF/1/8") = 2.75 PSF
MECH. ALLOWANCE = 4 PSF
SUPERIMPOSED DL = 3 PSF
ASSUME 9 PSF
+ 92 PSF

NOTEBOOK SUB A | GRAVITY CONT | JESSE BORDEAU | 4

TYP. WALL SECTION DEAD & LIVE LOADS

[A.320]



DESIGN LOADS:

- DEAD LOAD

TERRACOTTA RAIN SCREEN SYSTEM = 10 PSF

MINERAL WOOL INSULATION (2") = (2")(1 PSF/IN) = 2 PSF

BATT INSULATION (6") = (6")(1.5 PSF/IN) = 9 PSF

GYPSUM WALLBOARD (2) 5/8" = (2)(5)(0.55 PSF/1/8") = 5.5 PSF

AIR/MOISTURE BARRIER LIQUID APPLIED = 1 PSF

+ ______

FLOOR-TO-FLOOR HT

DESIGN DL = 27.5 PSF x 10.5 FT = 289 PLF

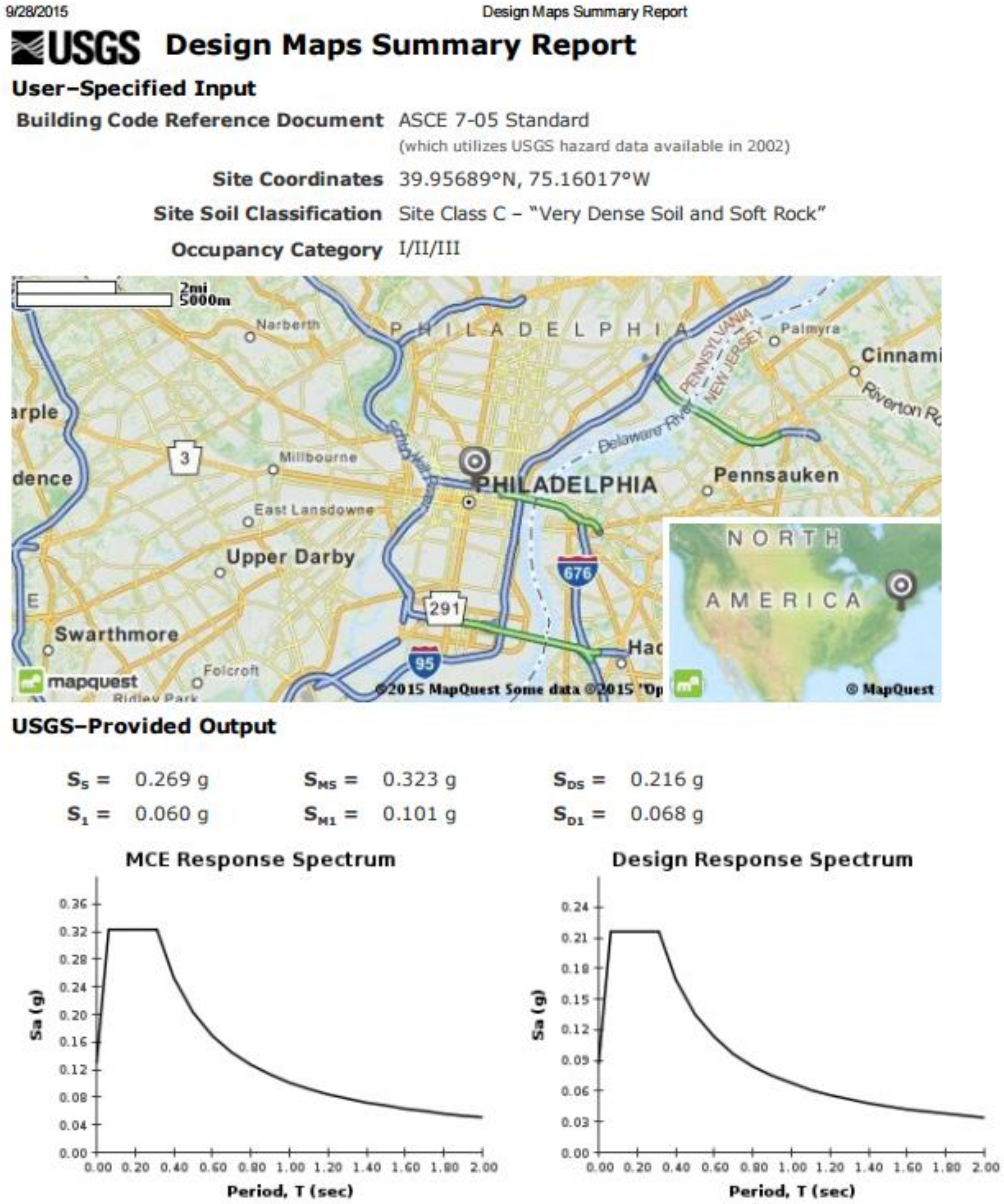
LOAD PATH: TYP EXTERIOR WALLS DO NOT BEAR ANY LOADS. THE LOAD IS TRANSFERRED FROM THE PANELS THROUGH THE CLIPS AND INTO THE GIRDER SLAB SYSTEM. FROM HERE, THE LOAD IS ABLE TO TRANSFER INTO THE BRACE FRAMES AND DOWN INTO THE FOUNDATION.

Live Loads & Wind Information

Permissible Live Loads		
Area	Loading (PSF)	Live Load Reduction Permitted
First Floor	100	Yes
Second Floor	100	Yes
Typical Floor	40+10 partitions	Yes
Loading Dock	250	No
Roof Live Load	30	No

Wind Criteria	**Value**
Basic Wind Speed (3 sec gust)	90 mph
Occupancy Category	II
Site Exposure Category	B
Wind Importance Factor (I_w)	1.0
Internal Pressure Coefficient (GC_{pi})	+0.18, -0.18
External Pressure Coefficient (GC_p)	+0.88(windward), -0.50(leeward)

Seismic Information



9/28/2015 Design Maps Summary Report

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document	ASCE 7-05 Standard (which utilizes USGS hazard data available in 2002)
Site Coordinates	39.95689°N, 75.16017°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Occupancy Category	I/II/III

USGS–Provided Output

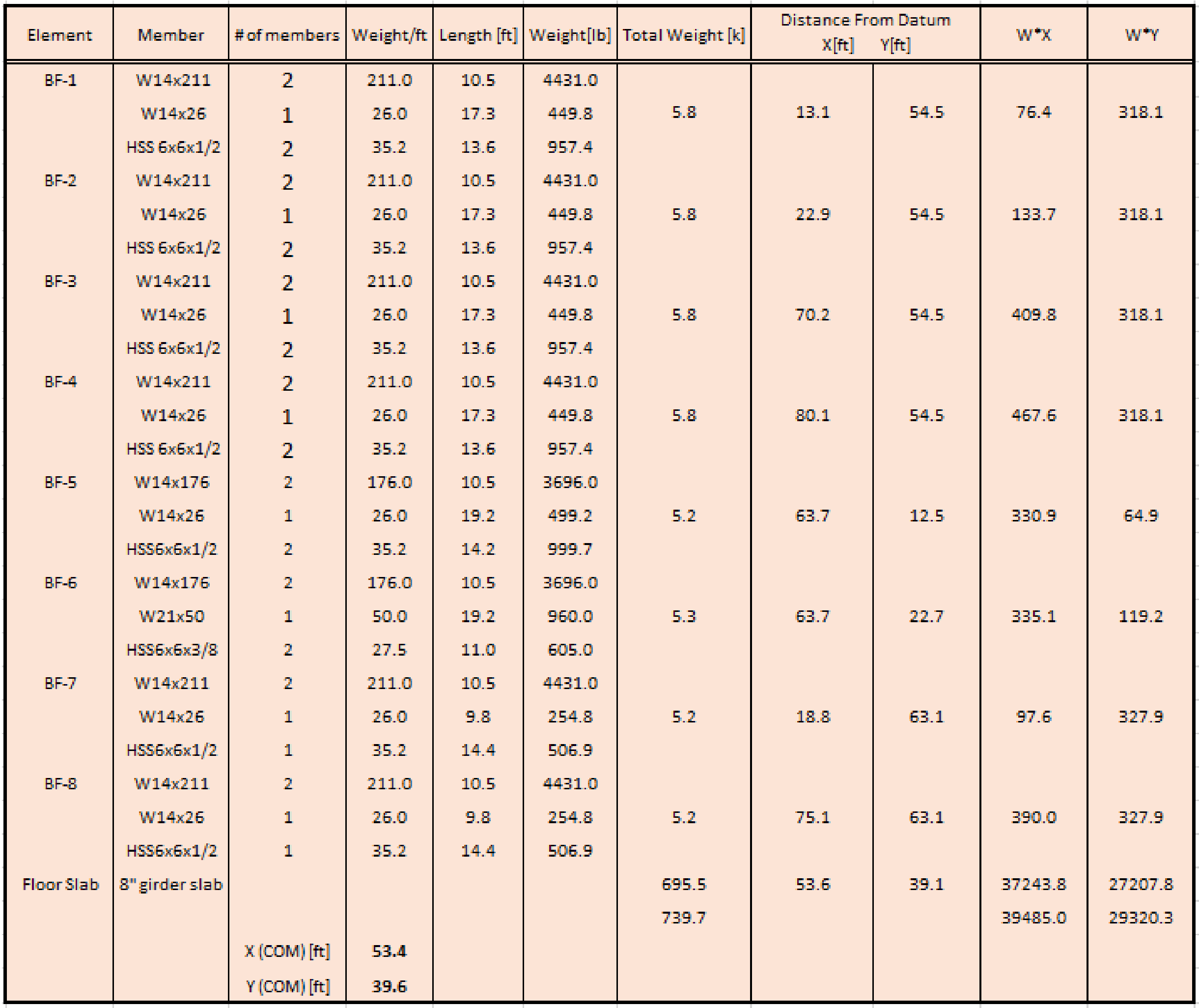
$S_S =$ 0.269 g $S_{MS} =$ 0.323 g $S_{DS} =$ 0.216 g

$S_1 =$ 0.060 g $S_{M1} =$ 0.101 g $S_{D1} =$ 0.068 g

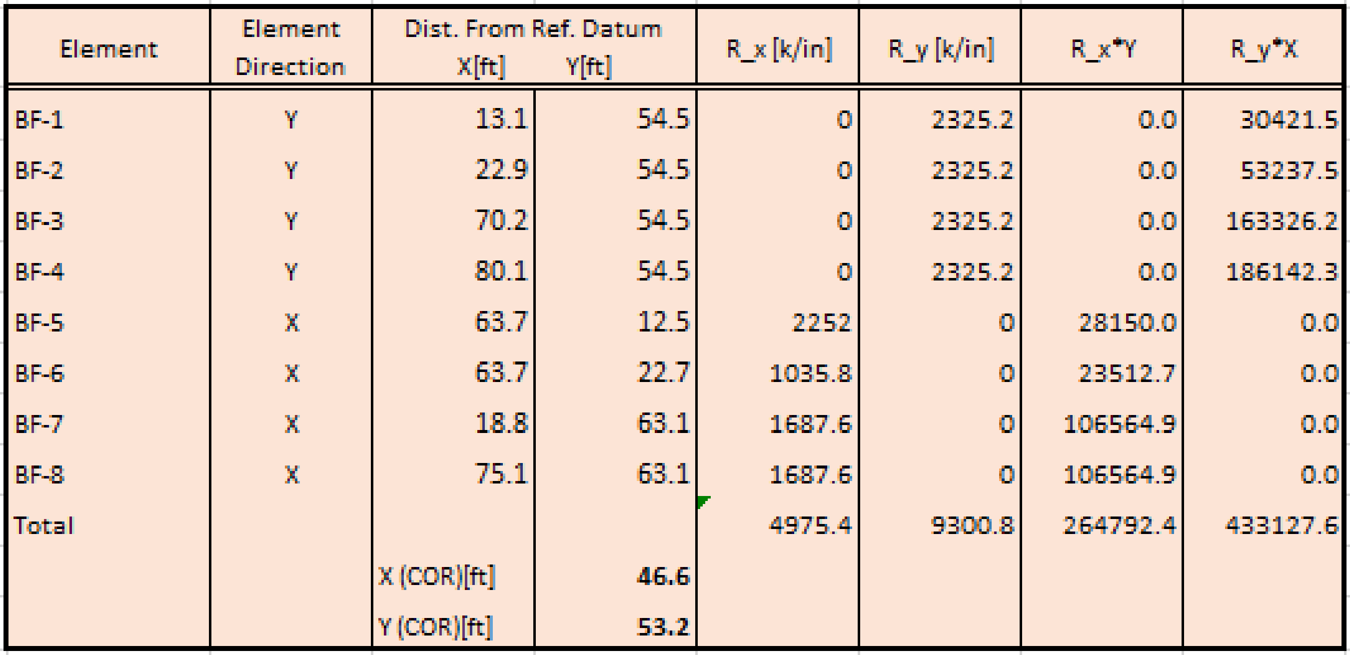
MCE Response Spectrum

Design Response Spectrum

Center of Mass (COM) & Center of Rigidity (COR)

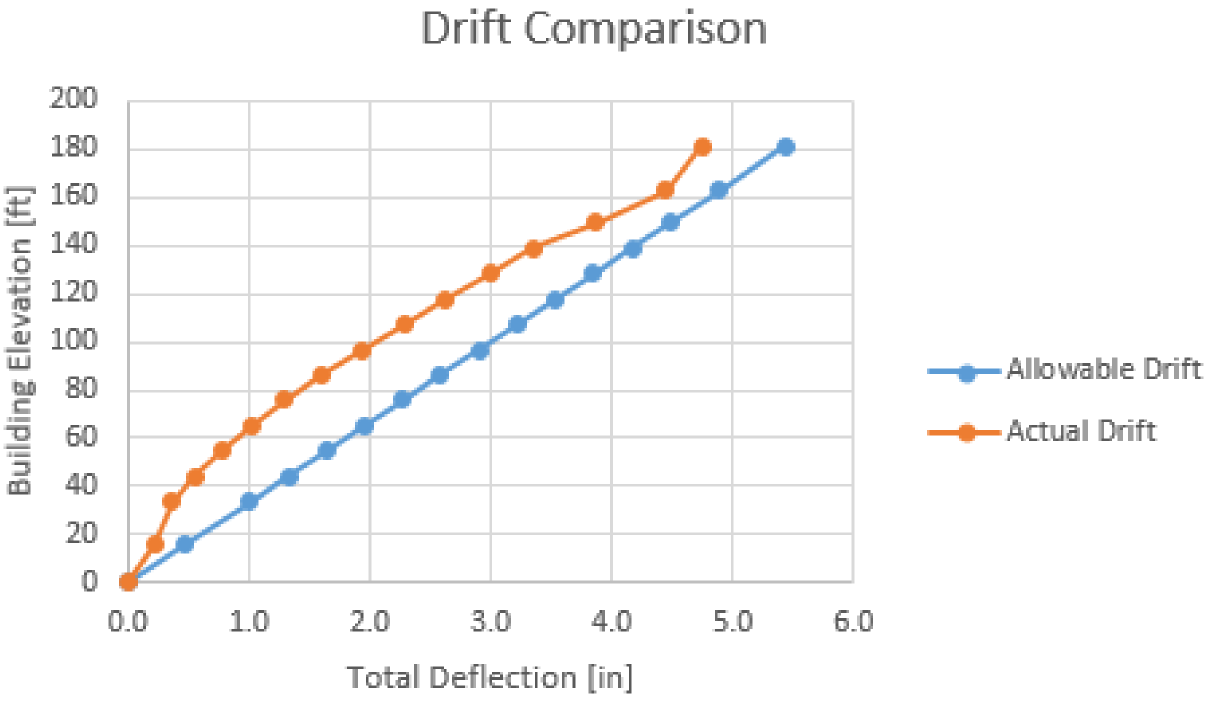


Element	Member	# of members	Weight/ft	Length [ft]	Weight[lb]	Total Weight [k]	Distance From Datum		W*X	W*Y
							X[ft]	Y[ft]		
BF-1	W14x211	2	211.0	10.5	4431.0					
	W14x26	1	26.0	17.3	449.8	5.8	13.1	54.5	76.4	318.1
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-2	W14x211	2	211.0	10.5	4431.0					
	W14x26	1	26.0	17.3	449.8	5.8	22.9	54.5	133.7	318.1
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-3	W14x211	2	211.0	10.5	4431.0					
	W14x26	1	26.0	17.3	449.8	5.8	70.2	54.5	409.8	318.1
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-4	W14x211	2	211.0	10.5	4431.0					
	W14x26	1	26.0	17.3	449.8	5.8	80.1	54.5	467.6	318.1
	HSS 6x6x1/2	2	35.2	13.6	957.4					
BF-5	W14x176	2	176.0	10.5	3696.0					
	W14x26	1	26.0	19.2	499.2	5.2	63.7	12.5	330.9	64.9
	HSS6x6x1/2	2	35.2	14.2	999.7					
BF-6	W14x176	2	176.0	10.5	3696.0					
	W21x50	1	50.0	19.2	960.0	5.3	63.7	22.7	335.1	119.2
	HSS6x6x3/8	2	27.5	11.0	605.0					
BF-7	W14x211	2	211.0	10.5	4431.0					
	W14x26	1	26.0	9.8	254.8	5.2	18.8	63.1	97.6	327.9
	HSS6x6x1/2	1	35.2	14.4	506.9					
BF-8	W14x211	2	211.0	10.5	4431.0					
	W14x26	1	26.0	9.8	254.8	5.2	75.1	63.1	390.0	327.9
	HSS6x6x1/2	1	35.2	14.4	506.9					
Floor Slab	8" girder slab					695.5	53.6	39.1	37243.8	27207.8
						739.7			39485.0	29320.3
		X (COM) [ft]	53.4							
		Y (COM) [ft]	39.6							



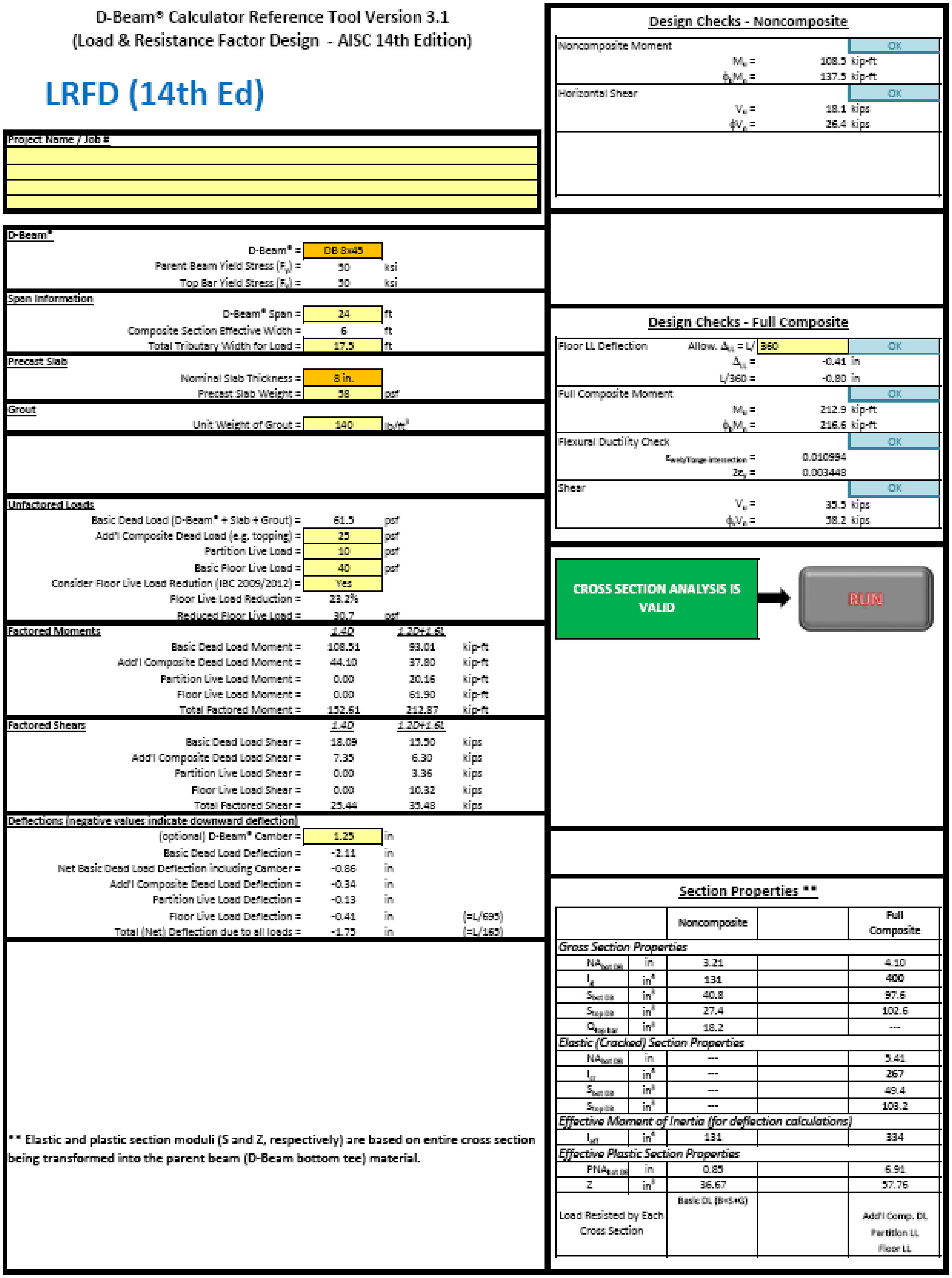
Element	Element Direction	Dist. From Ref. Datum		R_x [k/in]	R_y [k/in]	R_x*Y	R_y*X
		X[ft]	Y[ft]				
BF-1	Y	13.1	54.5	0	2325.2	0.0	30421.5
BF-2	Y	22.9	54.5	0	2325.2	0.0	53237.5
BF-3	Y	70.2	54.5	0	2325.2	0.0	163326.2
BF-4	Y	80.1	54.5	0	2325.2	0.0	186142.3
BF-5	X	63.7	12.5	2252	0	28150.0	0.0
BF-6	X	63.7	22.7	1035.8	0	23512.7	0.0
BF-7	X	18.8	63.1	1687.6	0	106564.9	0.0
BF-8	X	75.1	63.1	1687.6	0	106564.9	0.0
Total				4975.4	9300.8	264792.4	433127.6
		X (COR)[ft]	46.6				
		Y (COR)[ft]	53.2				

Controlling Lateral Case (Wind) Building Drifts



Level	Elevation [ft]	Total Drift @ Particular Level [in]	Allowable Drift (h/400) [in]	Acceptable Drift?
Roof	181	4.74	5.43	yes
Penthouse	163	4.44	4.89	yes
14	149.25	3.87	4.48	yes
13	138.75	3.35	4.16	yes
12	128.25	2.99	3.85	yes
11	117.75	2.63	3.53	yes
10	107.25	2.28	3.22	yes
9	96.75	1.94	2.90	yes
8	86.25	1.61	2.59	yes
7	75.75	1.3	2.27	yes
6	65.25	1.03	1.96	yes
5	54.75	0.77	1.64	yes
4	44.25	0.55	1.33	yes
3	33.75	0.36	1.01	yes
2	15.66	0.22	0.47	yes
1	0	0	0.00	N/A

D-Beam Design Aid



D-Beam® Calculator Reference Tool Version 3.1
(Load & Resistance Factor Design - AISC 14th Edition)

LRFD (14th Ed)

Project Name / Job #

D-Beam®
D-Beam® = DB 8x45
Parent Beam Yield Stress (F_y) = 50 ksi
Top Bar Yield Stress (F_y) = 50 ksi

Span Information
D-Beam® Span = 24 ft
Composite Section Effective Width = 6 ft
Total Tributary Width for Load = 17.5 ft

Precast Slab
Nominal Slab Thickness = 8 in.
Precast Slab Weight = 58 psf

Grout
Unit Weight of Grout = 140 lb/ft^3

Unfactored Loads
Basic Dead Load (D-Beam® + Slab + Grout) = 61.5 psf
Add'l Composite Dead Load (e.g. topping) = 25 psf
Partition Live Load = 10 psf
Basic Floor Live Load = 40 psf
Consider Floor Live Load Redution (IBC 2009/2012) = Yes
Floor Live Load Reduction = 23.2%
Reduced Floor Live Load = 30.7 psf

Factored Moments

	1.4D	1.2D+1.6L	
Basic Dead Load Moment =	108.51	93.01	kip-ft
Add'l Composite Dead Load Moment =	44.10	37.80	kip-ft
Partition Live Load Moment =	0.00	20.16	kip-ft
Floor Live Load Moment =	0.00	61.90	kip-ft
Total Factored Moment =	152.61	212.87	kip-ft

Factored Shears

	1.4D	1.2D+1.6L	
Basic Dead Load Shear =	18.09	15.50	kips
Add'l Composite Dead Load Shear =	7.35	6.30	kips
Partition Live Load Shear =	0.00	3.36	kips
Floor Live Load Shear =	0.00	10.32	kips
Total Factored Shear =	25.44	35.48	kips

Deflections (negative values indicate downward deflection)
(optional) D-Beam® Camber = 1.25 in
Basic Dead Load Deflection = -2.11 in
Net Basic Dead Load Deflection including Camber = -0.86 in
Add'l Composite Dead Load Deflection = -0.34 in
Partition Live Load Deflection = -0.13 in
Floor Live Load Deflection = -0.41 in (=L/695)
Total (Net) Deflection due to all loads = -1.75 in (=L/165)

** Elastic and plastic section moduli (S and Z, respectively) are based on entire cross section being transformed into the parent beam (D-Beam bottom tee) material.

Design Checks - Noncomposite

Noncomposite Moment OK
M_u = 108.5 kip-ft
$\phi_b M_n$ = 137.5 kip-ft

Horizontal Shear OK
V_u = 18.1 kips
ϕV_n = 26.4 kips

Design Checks - Full Composite

Floor LL Deflection Allow. Δ_{LL} = L/ 360 OK
Δ_{LL} = -0.41 in
L/360 = -0.80 in

Full Composite Moment OK
M_u = 212.9 kip-ft
$\phi_b M_n$ = 216.6 kip-ft

Flexural Ductility Check OK
$\varepsilon_{web/flange\ intersection}$ = 0.010994
$2\varepsilon_y$ = 0.003448

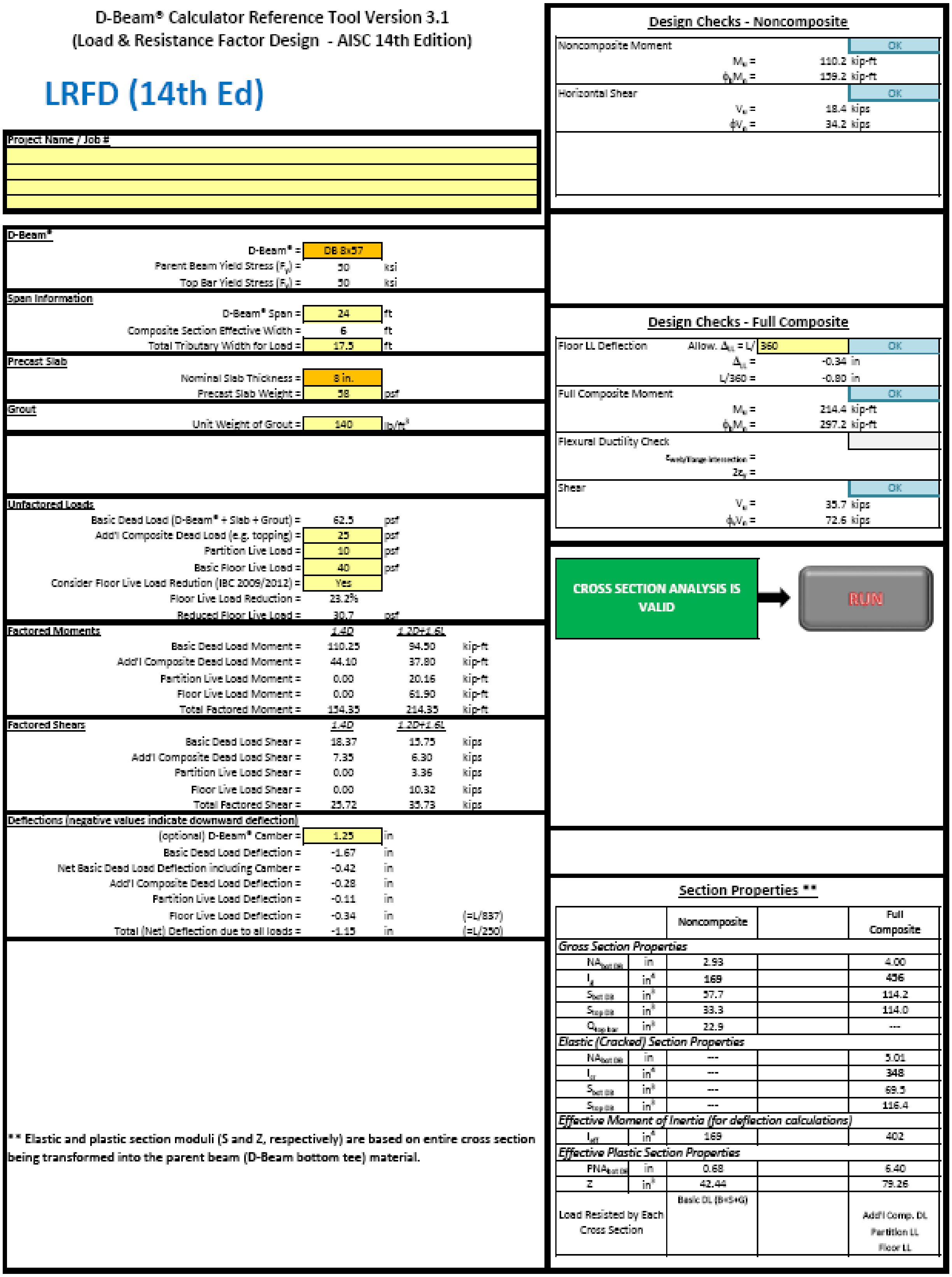
Shear OK
V_u = 35.5 kips
$\phi_v V_n$ = 58.2 kips

CROSS SECTION ANALYSIS IS VALID

RUN

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	3.21		4.10
I_g	in^4	131		400
$S_{bot\ DB}$	in^3	40.8		97.6
$S_{top\ DB}$	in^3	27.4		102.6
$Q_{top\ bar}$	in^3	18.2		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.41
I_{cr}	in^4	---		267
$S_{bot\ DB}$	in^3	---		49.4
$S_{top\ DB}$	in^3	---		103.2
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in^4	131		334
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.85		6.91
Z	in^3	36.67		57.76
Load Resisted by Each Cross Section		Basic DL (B+S+G)		Add'l Comp. DL Partition LL Floor LL



D-Beam® Calculator Reference Tool Version 3.1
(Load & Resistance Factor Design - AISC 14th Edition)

LRFD (14th Ed)

Project Name / Job #

D-Beam®

D-Beam® = DB 8x57
Parent Beam Yield Stress (F_y) = 50 ksi
Top Bar Yield Stress (F_y) = 50 ksi

Span Information

D-Beam® Span = 24 ft
Composite Section Effective Width = 6 ft
Total Tributary Width for Load = 17.5 ft

Precast Slab

Nominal Slab Thickness = 8 in.
Precast Slab Weight = 58 psf

Grout

Unit Weight of Grout = 140 lb/ft^3

Unfactored Loads

Basic Dead Load (D-Beam® + Slab + Grout) = 62.5 psf
Add'l Composite Dead Load (e.g. topping) = 25 psf
Partition Live Load = 10 psf
Basic Floor Live Load = 40 psf
Consider Floor Live Load Redution (IBC 2009/2012) = Yes
Floor Live Load Reduction = 23.2%
Reduced Floor Live Load = 30.7 psf

Factored Moments	1.4D	1.2D+1.6L	
Basic Dead Load Moment =	110.25	94.50	kip-ft
Add'l Composite Dead Load Moment =	44.10	37.80	kip-ft
Partition Live Load Moment =	0.00	20.16	kip-ft
Floor Live Load Moment =	0.00	61.90	kip-ft
Total Factored Moment =	154.35	214.35	kip-ft

Factored Shears	1.4D	1.2D+1.6L	
Basic Dead Load Shear =	18.37	15.75	kips
Add'l Composite Dead Load Shear =	7.35	6.30	kips
Partition Live Load Shear =	0.00	3.36	kips
Floor Live Load Shear =	0.00	10.32	kips
Total Factored Shear =	25.72	35.73	kips

Deflections (negative values indicate downward deflection)

(optional) D-Beam® Camber = 1.25 in
Basic Dead Load Deflection = -1.67 in
Net Basic Dead Load Deflection including Camber = -0.42 in
Add'l Composite Dead Load Deflection = -0.28 in
Partition Live Load Deflection = -0.11 in
Floor Live Load Deflection = -0.34 in (=L/837)
Total (Net) Deflection due to all loads = -1.15 in (=L/250)

** Elastic and plastic section moduli (S and Z, respectively) are based on entire cross section being transformed into the parent beam (D-Beam bottom tee) material.

Design Checks - Noncomposite

Noncomposite Moment OK
M_u = 110.2 kip-ft
$\phi_b M_n$ = 159.2 kip-ft

Horizontal Shear OK
V_u = 18.4 kips
ϕV_n = 34.2 kips

Design Checks - Full Composite

Floor LL Deflection Allow. Δ_{LL} = L/ 360 OK
Δ_{LL} = -0.34 in
L/360 = -0.80 in

Full Composite Moment OK
M_u = 214.4 kip-ft
$\phi_b M_n$ = 297.2 kip-ft

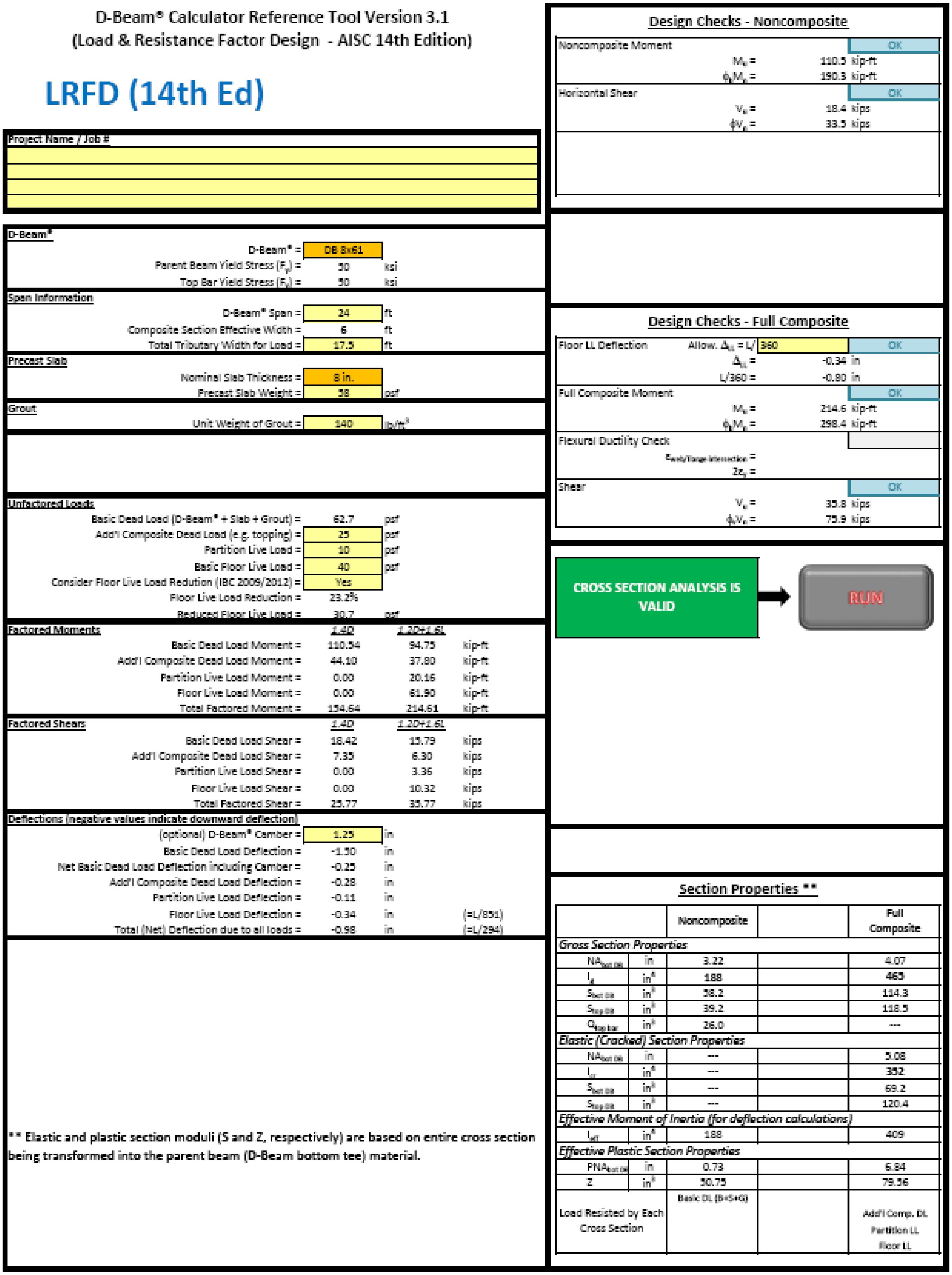
Flexural Ductility Check
$\varepsilon_{web/flange\ intersection}$ =
$2\varepsilon_y$ =

Shear OK
V_u = 35.7 kips
$\phi_v V_n$ = 72.6 kips

CROSS SECTION ANALYSIS IS VALID → RUN

Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	2.93		4.00
I_g	in^4	169		456
$S_{bot\ DB}$	in^3	57.7		114.2
$S_{top\ DB}$	in^3	33.3		114.0
$Q_{top\ bar}$	in^3	22.9		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.01
I_{cr}	in^4	---		348
$S_{bot\ DB}$	in^3	---		69.5
$S_{top\ DB}$	in^3	---		116.4
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in^4	169		402
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.68		6.40
Z	in^3	42.44		79.26
Load Resisted by Each Cross Section		Basic DL (B+S+G)		Add'l Comp. DL Partition LL Floor LL



D-Beam® Calculator Reference Tool Version 3.1
(Load & Resistance Factor Design - AISC 14th Edition)

LRFD (14th Ed)

Project Name / Job #

D-Beam®

D-Beam® = DB 8x61
Parent Beam Yield Stress (F_y) = 50 ksi
Top Bar Yield Stress (F_y) = 50 ksi

Span Information

D-Beam® Span = 24 ft
Composite Section Effective Width = 6 ft
Total Tributary Width for Load = 17.5 ft

Precast Slab

Nominal Slab Thickness = 8 in.
Precast Slab Weight = 58 psf

Grout

Unit Weight of Grout = 140 lb/ft^3

Unfactored Loads

Basic Dead Load (D-Beam® + Slab + Grout) = 62.7 psf
Add'l Composite Dead Load (e.g. topping) = 25 psf
Partition Live Load = 10 psf
Basic Floor Live Load = 40 psf
Consider Floor Live Load Redution (IBC 2009/2012) = Yes
Floor Live Load Reduction = 23.2%
Reduced Floor Live Load = 30.7 psf

Factored Moments	1.4D	1.2D+1.6L	
Basic Dead Load Moment =	110.54	94.75	kip-ft
Add'l Composite Dead Load Moment =	44.10	37.80	kip-ft
Partition Live Load Moment =	0.00	20.16	kip-ft
Floor Live Load Moment =	0.00	61.90	kip-ft
Total Factored Moment =	154.64	214.61	kip-ft

Factored Shears	1.4D	1.2D+1.6L	
Basic Dead Load Shear =	18.42	15.79	kips
Add'l Composite Dead Load Shear =	7.35	6.30	kips
Partition Live Load Shear =	0.00	3.36	kips
Floor Live Load Shear =	0.00	10.32	kips
Total Factored Shear =	25.77	35.77	kips

Deflections (negative values indicate downward deflection)

(optional) D-Beam® Camber = 1.25 in
Basic Dead Load Deflection = -1.50 in
Net Basic Dead Load Deflection including Camber = -0.25 in
Add'l Composite Dead Load Deflection = -0.28 in
Partition Live Load Deflection = -0.11 in
Floor Live Load Deflection = -0.34 in (=L/851)
Total (Net) Deflection due to all loads = -0.98 in (=L/294)

** Elastic and plastic section moduli (S and Z, respectively) are based on entire cross section being transformed into the parent beam (D-Beam bottom tee) material.

Design Checks - Noncomposite

Noncomposite Moment OK
M_u = 110.5 kip-ft
$\phi_b M_n$ = 190.3 kip-ft

Horizontal Shear OK
V_u = 18.4 kips
ϕV_n = 33.5 kips

Design Checks - Full Composite

Floor LL Deflection Allow. Δ_{LL} = L/ 360 OK
Δ_{LL} = -0.34 in
L/360 = -0.80 in

Full Composite Moment OK
M_u = 214.6 kip-ft
$\phi_b M_n$ = 298.4 kip-ft

Flexural Ductility Check
$\varepsilon_{web/flange\ intersection}$ =
$2\varepsilon_y$ =

Shear OK
V_u = 35.8 kips
$\phi_v V_n$ = 75.9 kips

CROSS SECTION ANALYSIS IS VALID

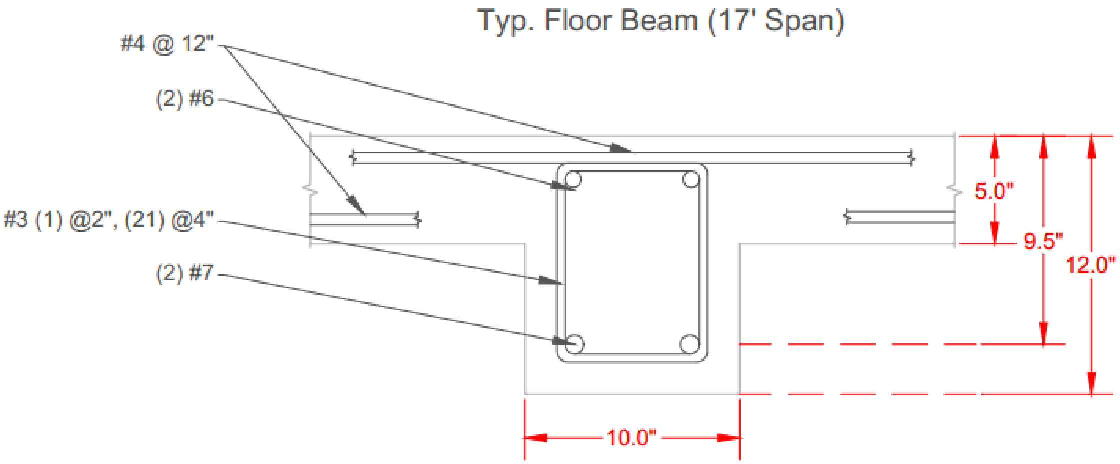
RUN

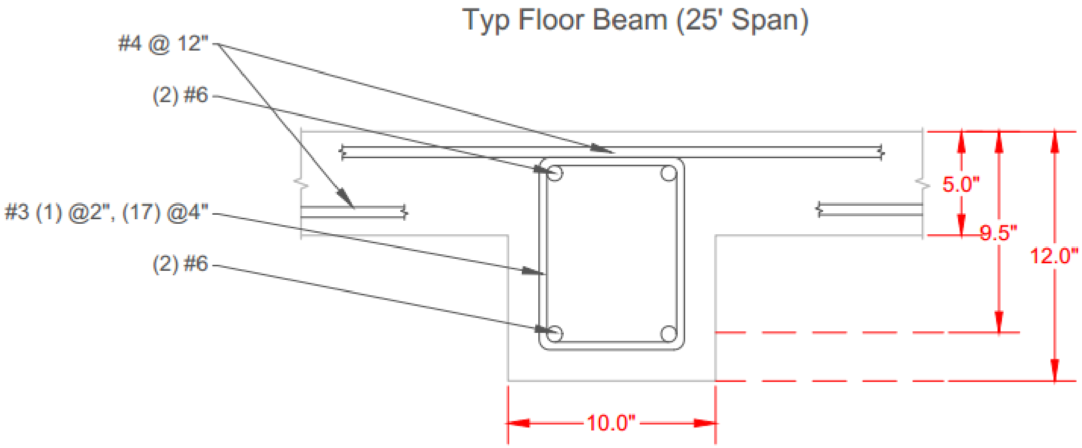
Section Properties **

		Noncomposite		Full Composite
Gross Section Properties				
$NA_{bot\ DB}$	in	3.22		4.07
I_g	in^4	188		465
$S_{bot\ DB}$	in^3	58.2		114.3
$S_{top\ DB}$	in^3	39.2		118.5
$Q_{top\ bar}$	in^3	26.0		---
Elastic (Cracked) Section Properties				
$NA_{bot\ DB}$	in	---		5.08
I_{cr}	in^4	---		352
$S_{bot\ DB}$	in^3	---		69.2
$S_{top\ DB}$	in^3	---		120.4
Effective Moment of Inertia (for deflection calculations)				
I_{eff}	in^4	188		409
Effective Plastic Section Properties				
$PNA_{bot\ DB}$	in	0.73		6.84
Z	in^3	50.75		79.56
Load Resisted by Each Cross Section		Basic DL (B+S+G)		Add'l Comp. DL Partition LL Floor LL

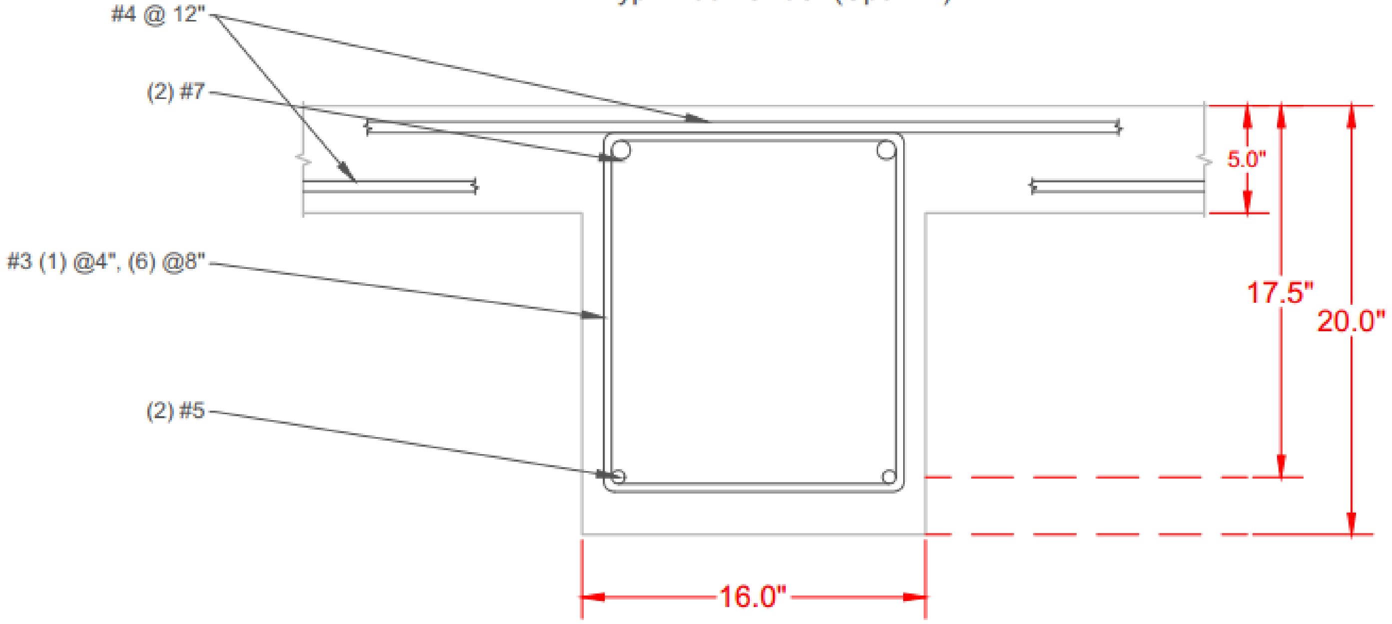
Appendix B (Proposed Structure Supplementary Info)

Typ. Floor Beam & Girder Sections

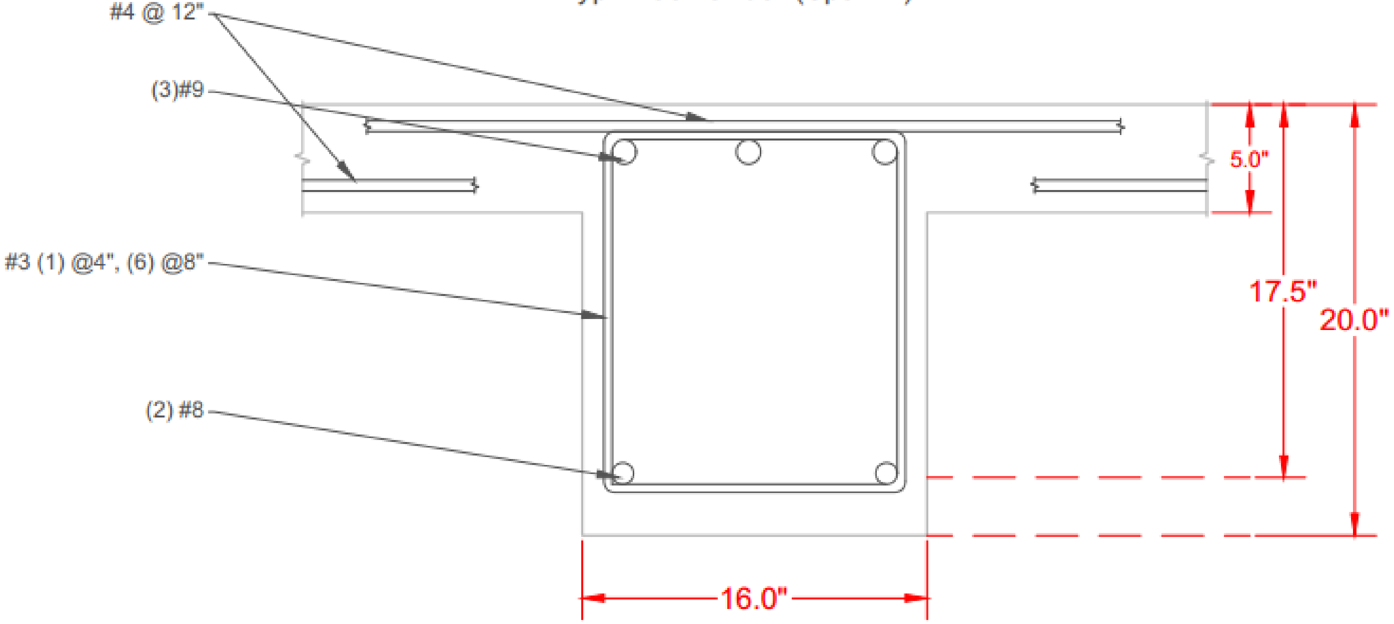




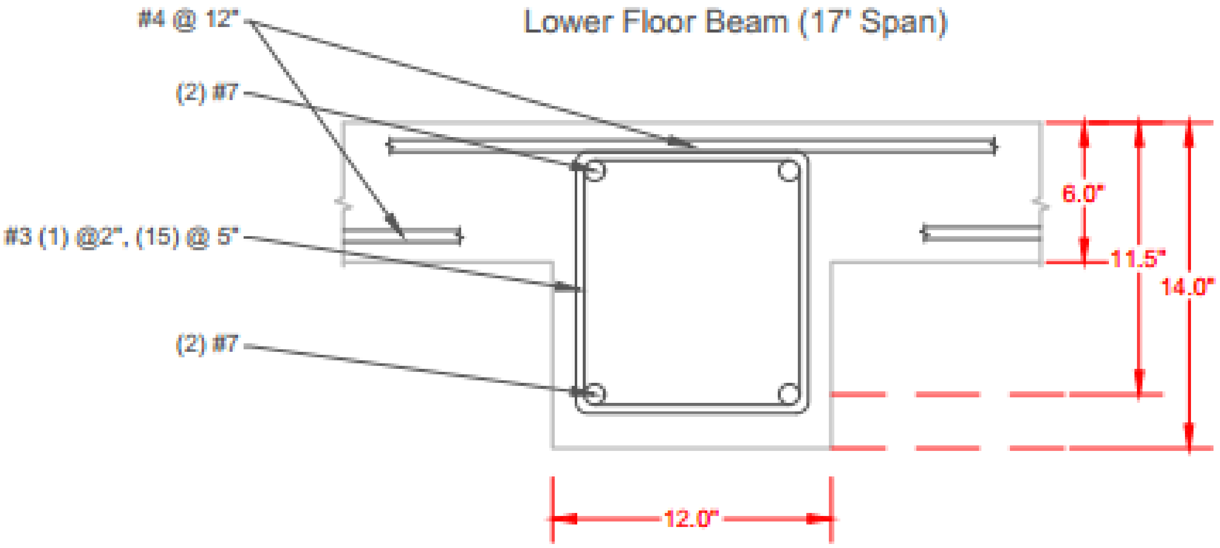
Typ. Floor Girder (Span A)

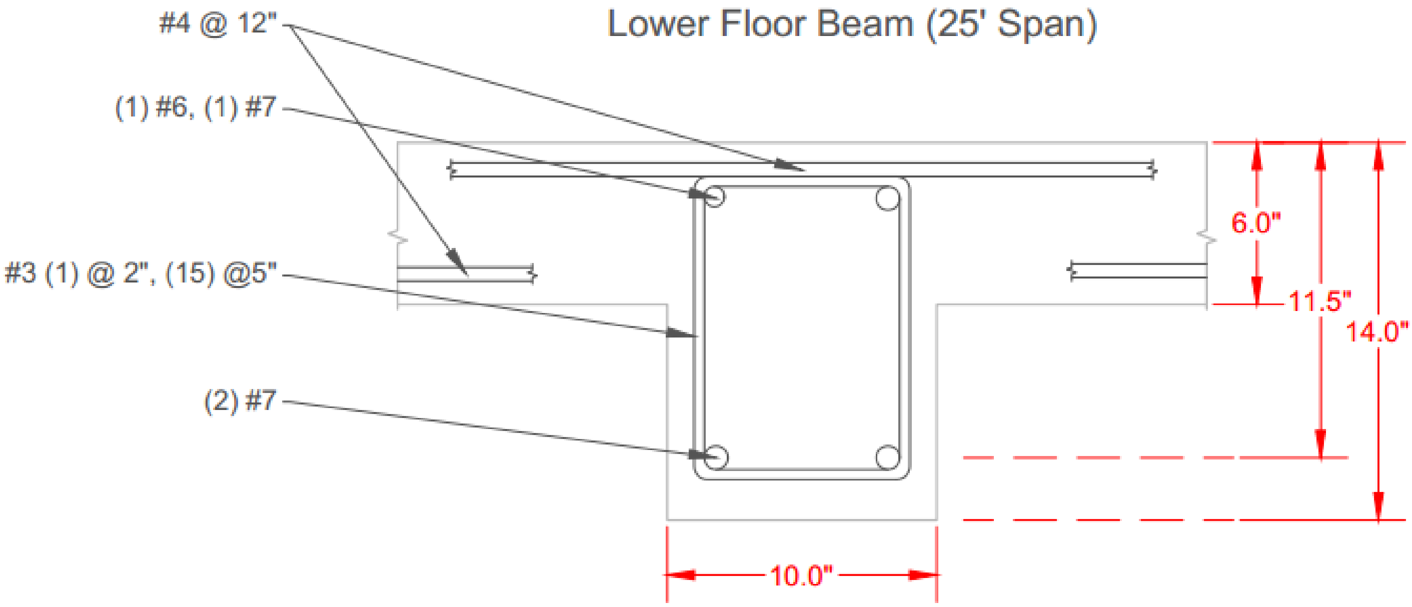


Typ. Floor Girder (Span B)

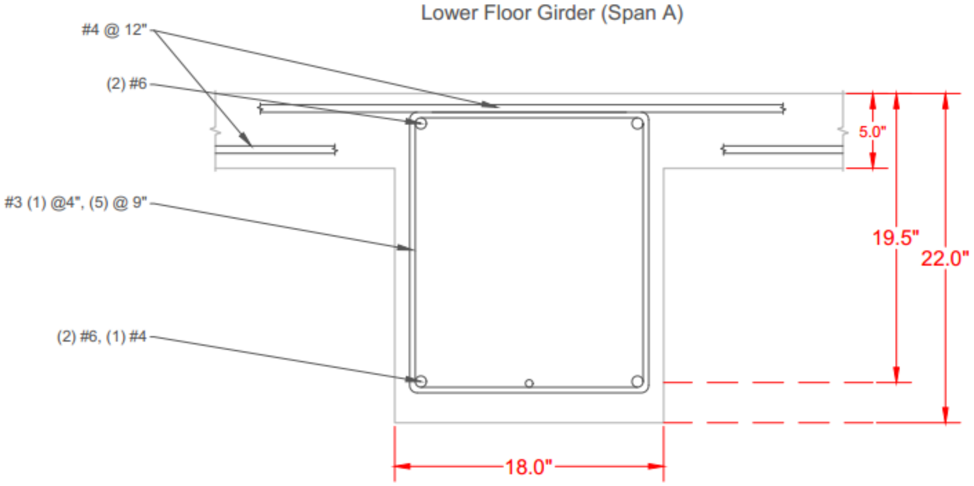


Lower Floor Beam & Girder Sections

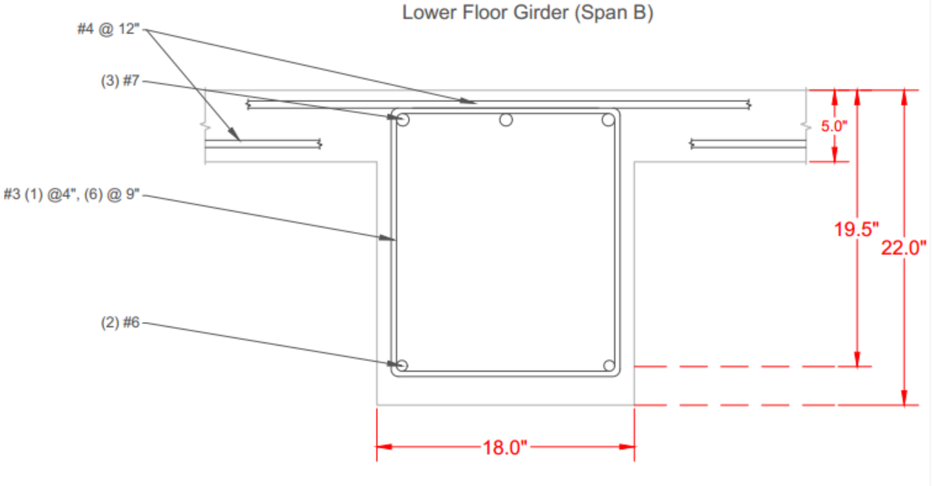




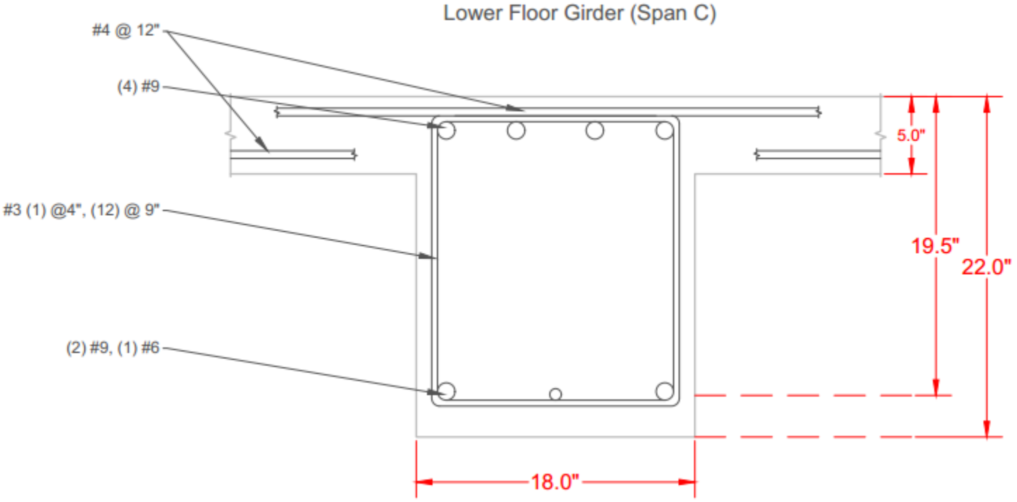
Lower Floor Girder (Span A)



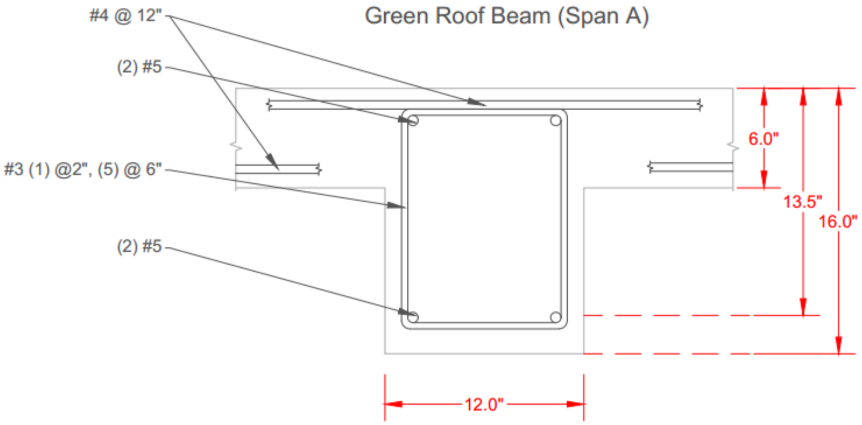
Lower Floor Girder (Span B)

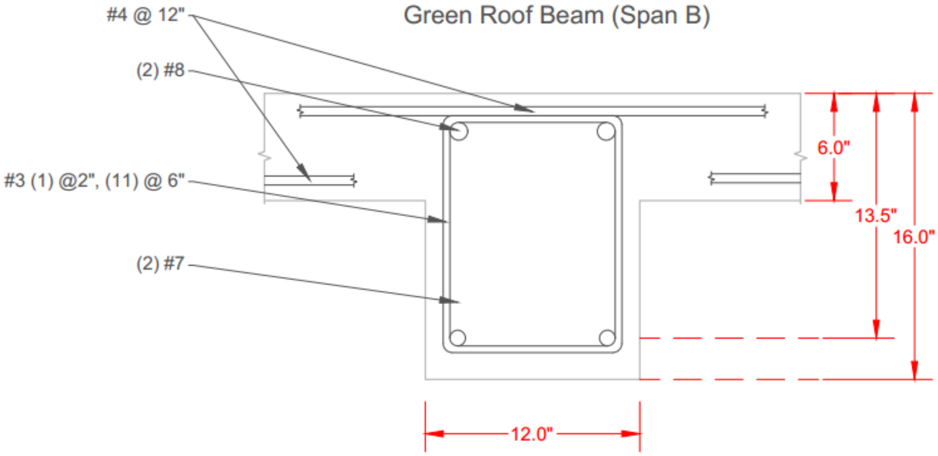


Lower Floor Girder (Span C)

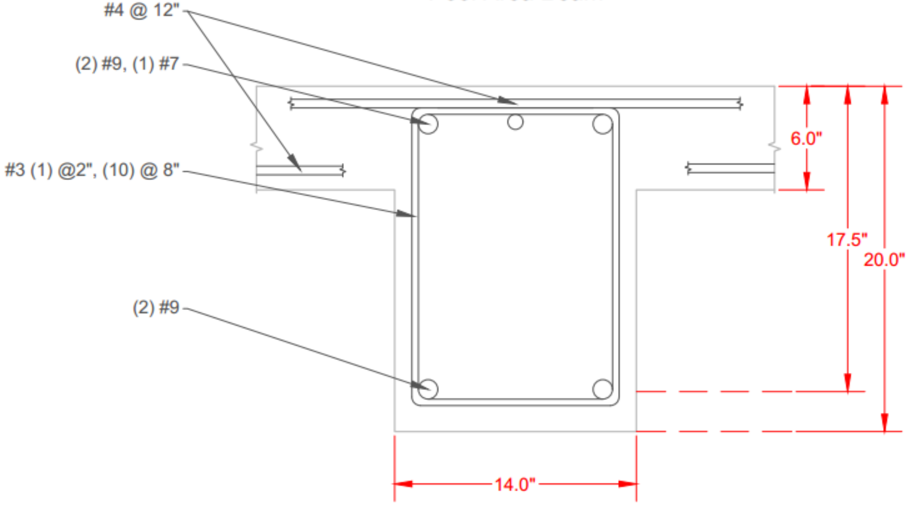


Specialized Elements' Sections

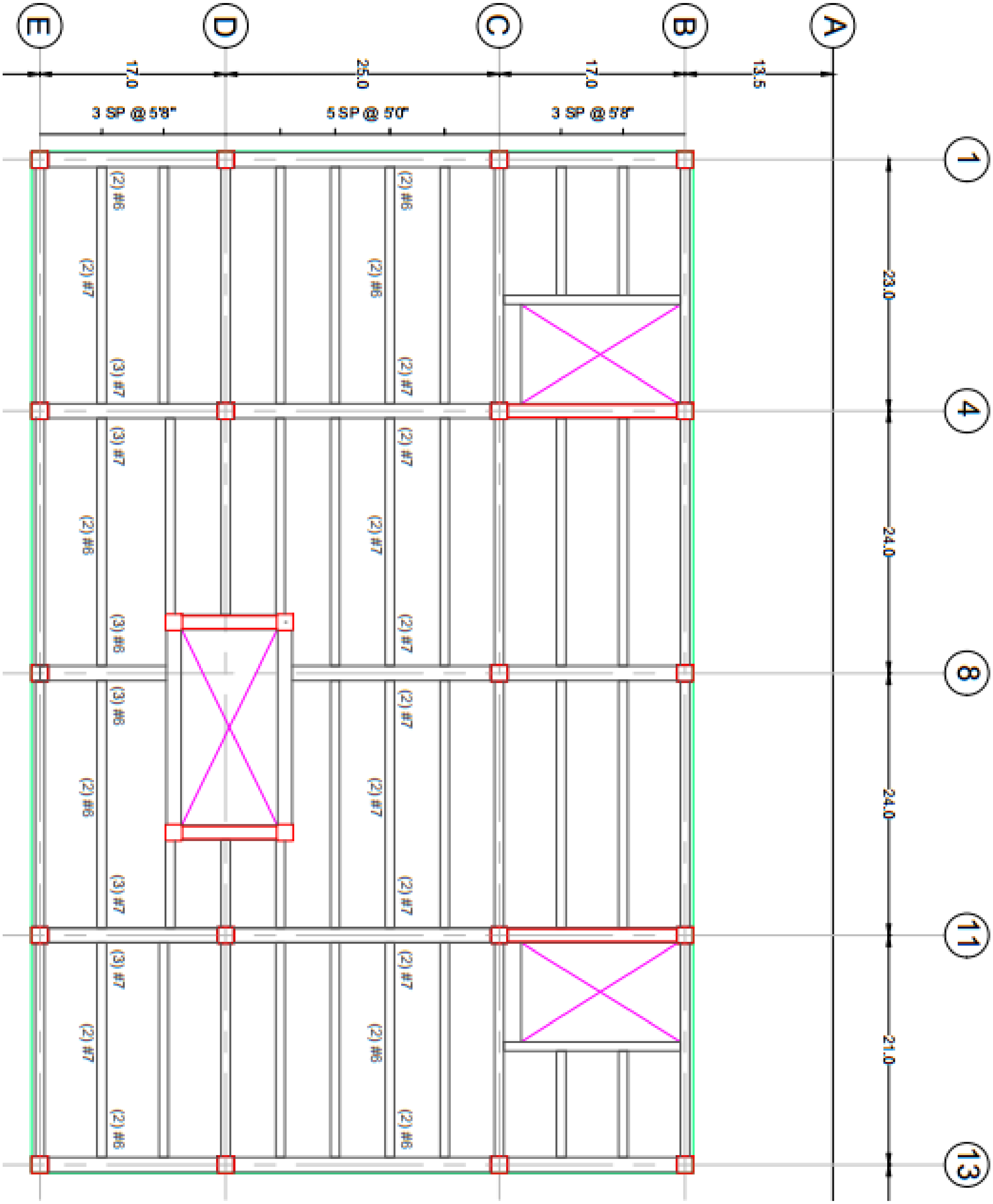

Green Roof Beam (Span A)


Green Roof Beam (Span B)

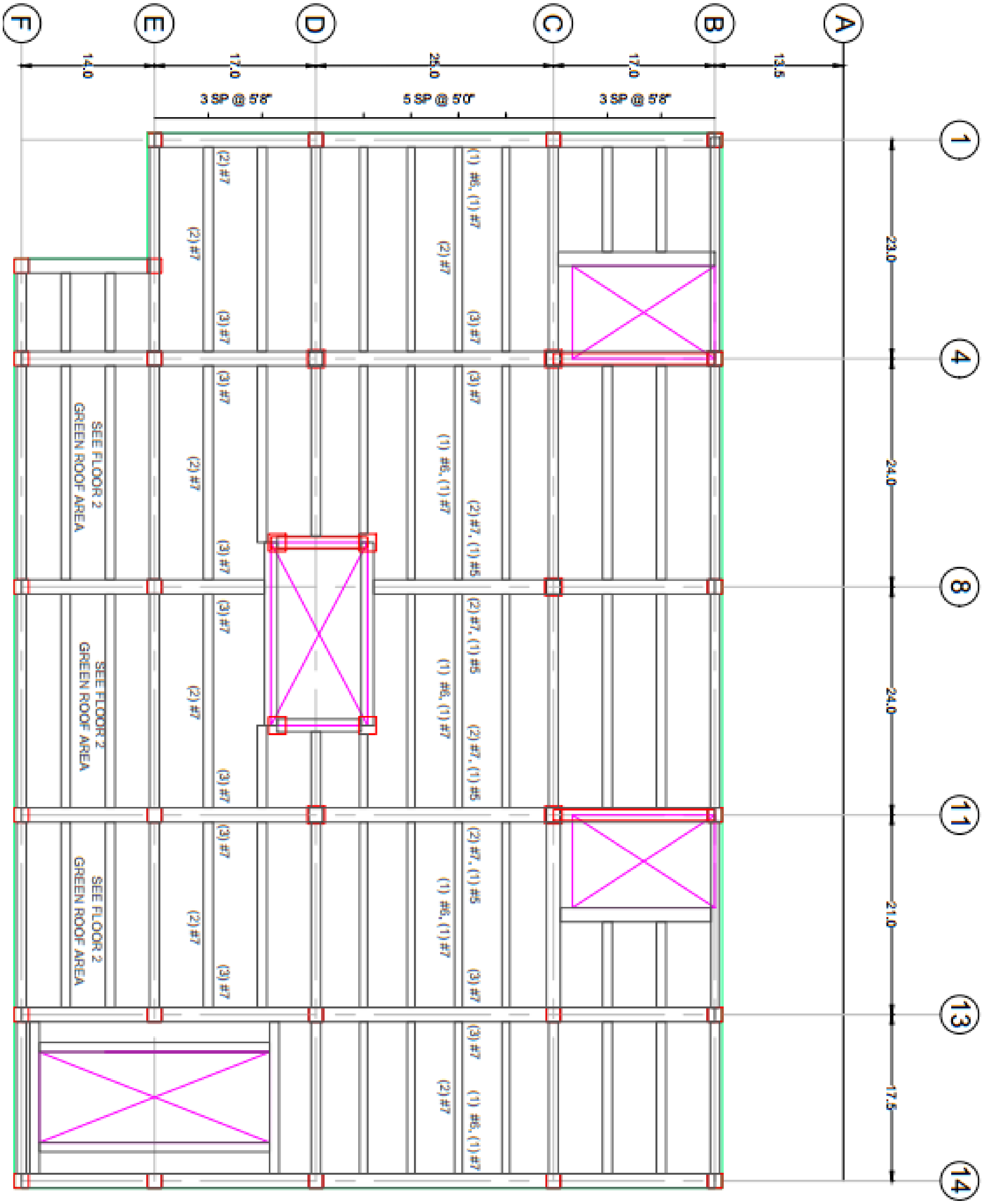
Pool Area Beam



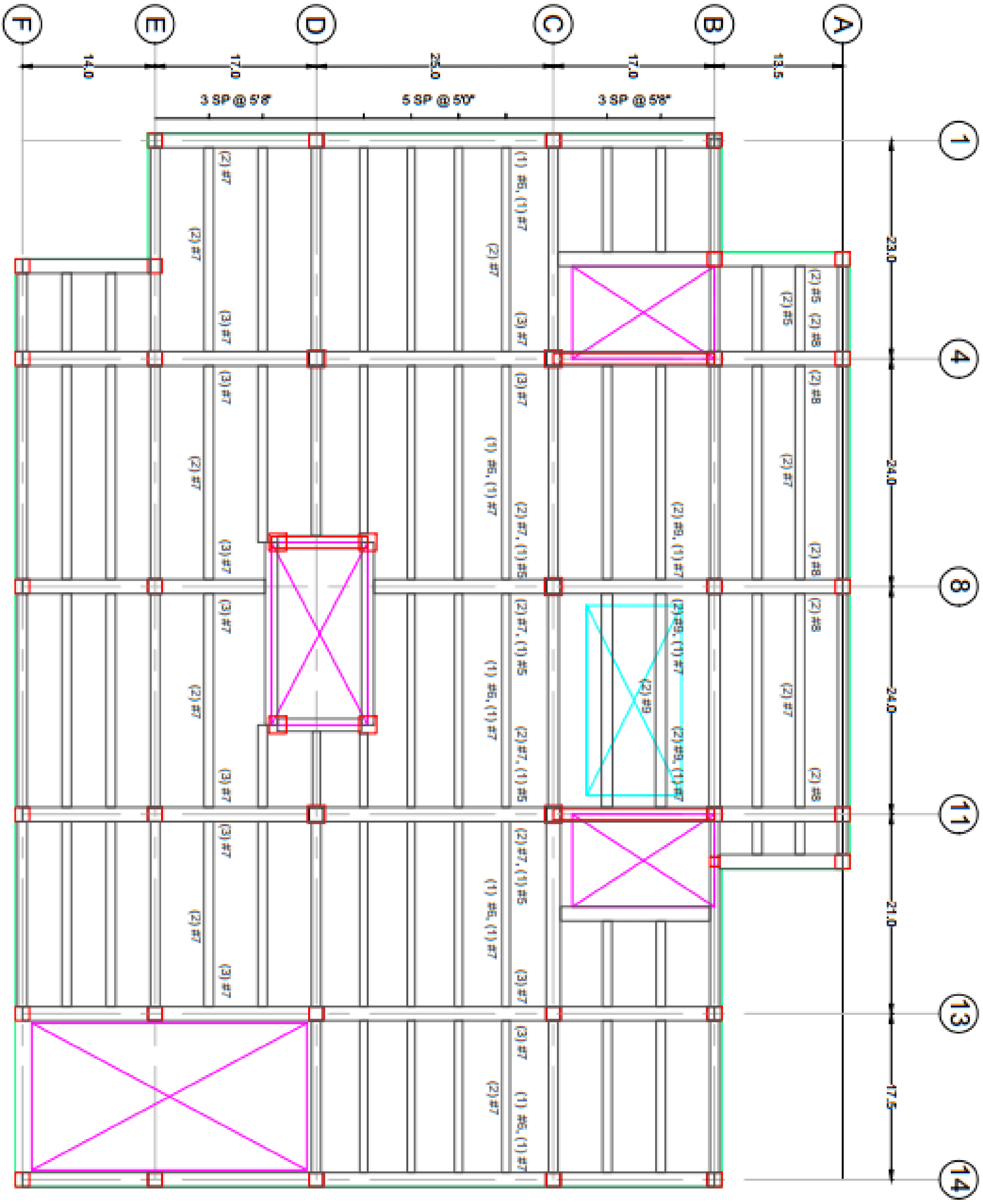
Typical Floor Rebar Layout



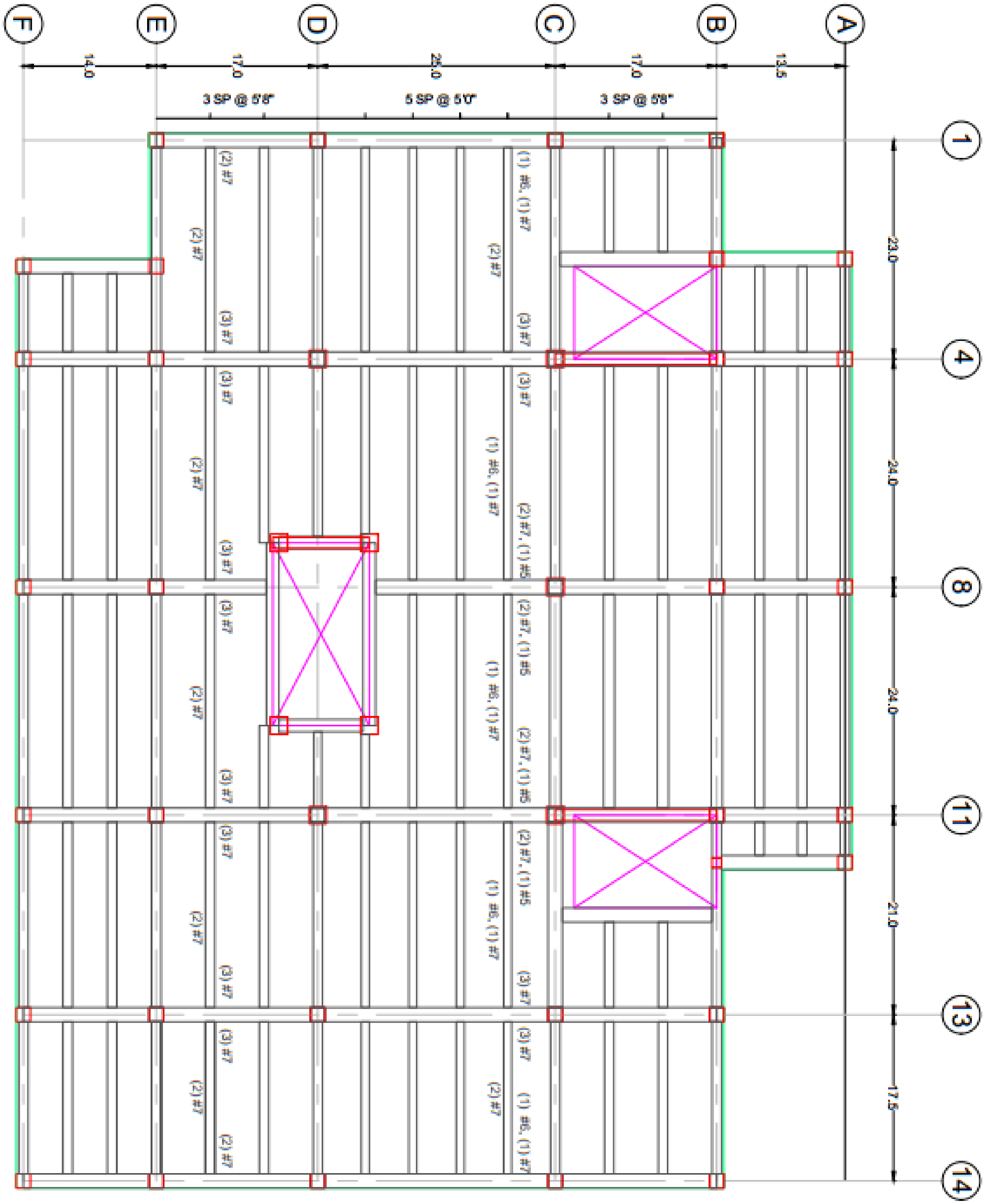
3rd Floor Rebar Layout



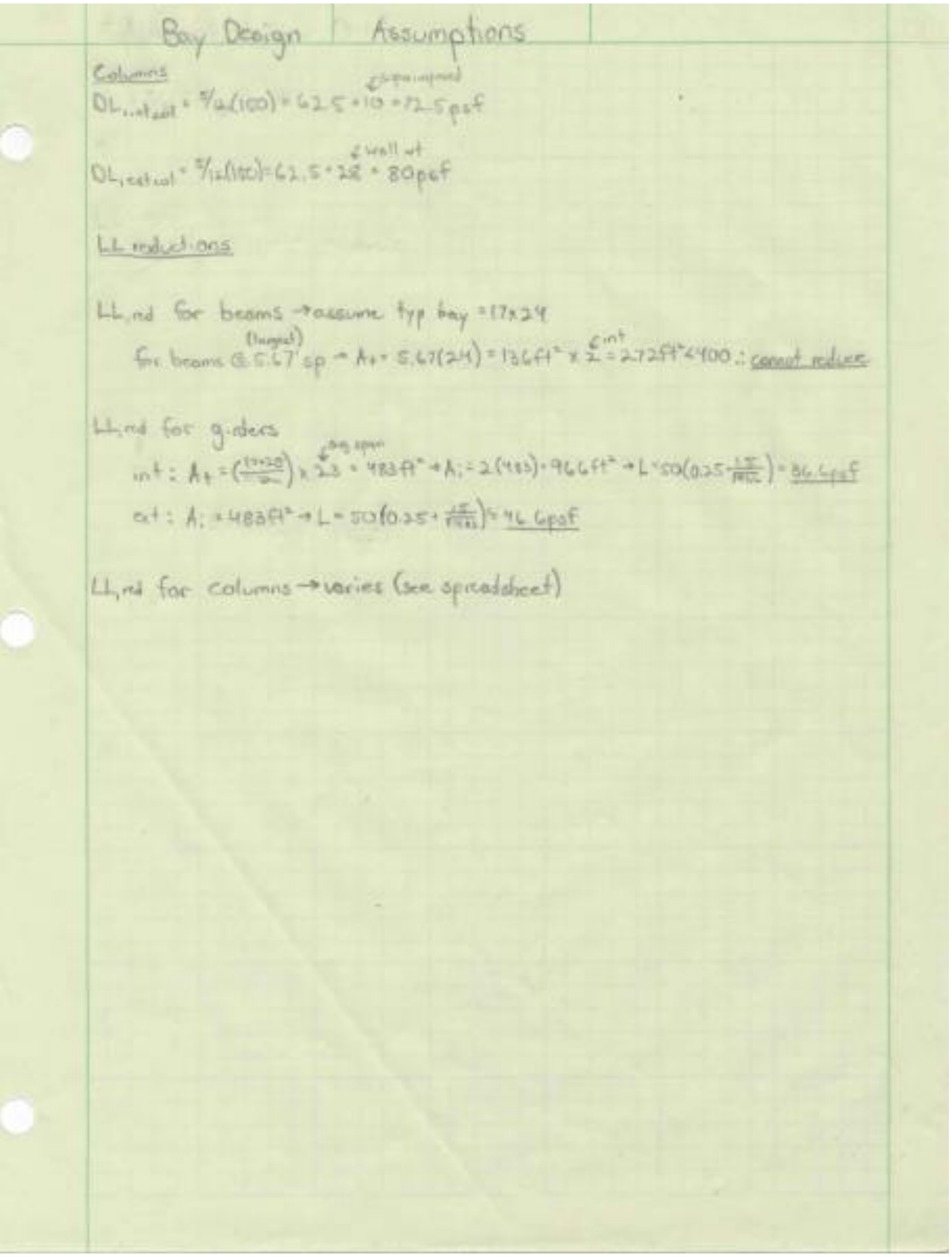
2nd Floor Rebar Layout



1st Floor Rebar Layout



Assumptions



Bay Design | Assumptions

Columns

$DL_{i,int\,col} = \frac{5}{12}(150) = 62.5 + \overset{\swarrow \text{superimposed}}{10} = 72.5\text{psf}$

$DL_{i,ext\,col} = \frac{5}{12}(150) = 62.5 + \overset{\swarrow \text{wall wt}}{18} = 80\text{psf}$

LL reductions

LL,red for beams → assume typ bay = 17x24

for beams @ $\overset{\text{(longest)}}{5.67'}$ sp → $A_t = 5.67(24) = 136\text{ft}^2 \times \overset{\swarrow int}{2} = 272\text{ft}^2 < 400$ ∴ cannot reduce

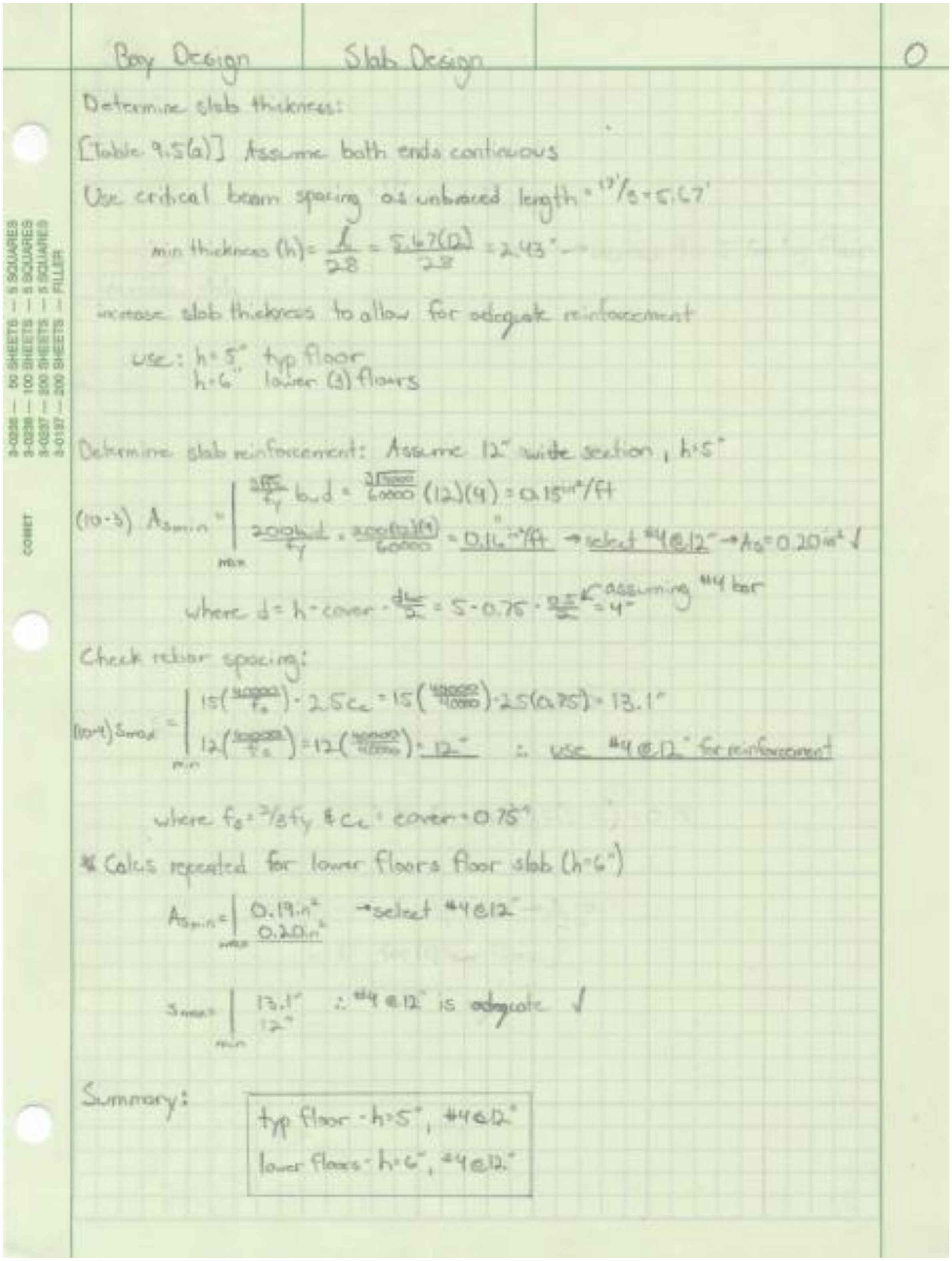
LL,red for girders

int: $A_t = \left(\frac{17+25}{2}\right) \times \overset{\swarrow \text{avg span}}{23} = 483\text{ft}^2 \rightarrow A_i = 2(483) = 966\text{ft}^2 \rightarrow L = 50\left(0.25 + \frac{15}{\sqrt{966}}\right) = \underline{36.6\text{psf}}$

ext: $A_i = 483\text{ft}^2 \rightarrow L = 50\left(0.25 + \frac{15}{\sqrt{483}}\right) = \underline{46.6\text{psf}}$

LL,red for columns → varies (see spreadsheet)

Slab Design



Bay Design | Slab Design | 0

Determine slab thickness:

[Table 9.5(a)] Assume both ends continuous

Use critical beam spacing as unbraced length = 17/3 = 5.67'

$$\text{min thickness } (h) = \frac{l}{28} = \frac{5.67(12)}{28} = 2.43" \rightarrow$$

increase slab thickness to allow for adequate reinforcement

use: h = 5" typ floor
h = 6" lower (3) floors

Determine slab reinforcement: Assume 12" wide section, h = 5"

$$(10\text{-}3)\ A_{s,min} = \max\begin{cases} \frac{3\sqrt{f'_c}}{f_y} b_w d = \frac{3\sqrt{5000}}{60000}(12)(4) = 0.15\ \text{in}^2/\text{ft} \\ \underline{\frac{200 b_w d}{f_y} = \frac{200(12)(4)}{60000} = 0.16\ \text{in}^2/\text{ft}} \rightarrow \underline{\text{select \#4@12"}} \rightarrow A_s = 0.20\ \text{in}^2\ \checkmark \end{cases}$$

$$\text{where } d = h - \text{cover} - \frac{d_b}{2} = 5 - 0.75 - \frac{0.5}{2} = 4" \leftarrow \text{assuming \#4 bar}$$

Check rebar spacing:

$$(10\text{-}4)\ s_{max} = \min\begin{cases} 15\left(\frac{40000}{f_s}\right) - 2.5c_c = 15\left(\frac{40000}{40000}\right) - 2.5(0.75) = 13.1" \\ 12\left(\frac{40000}{f_s}\right) = 12\left(\frac{40000}{40000}\right) = \underline{12"} \end{cases} \therefore \underline{\text{use \#4@12" for reinforcement}}$$

where $f_s = 2/3 f_y$ & c_c = cover + 0.75"

* Calcs repeated for lower floors floor slab (h = 6")

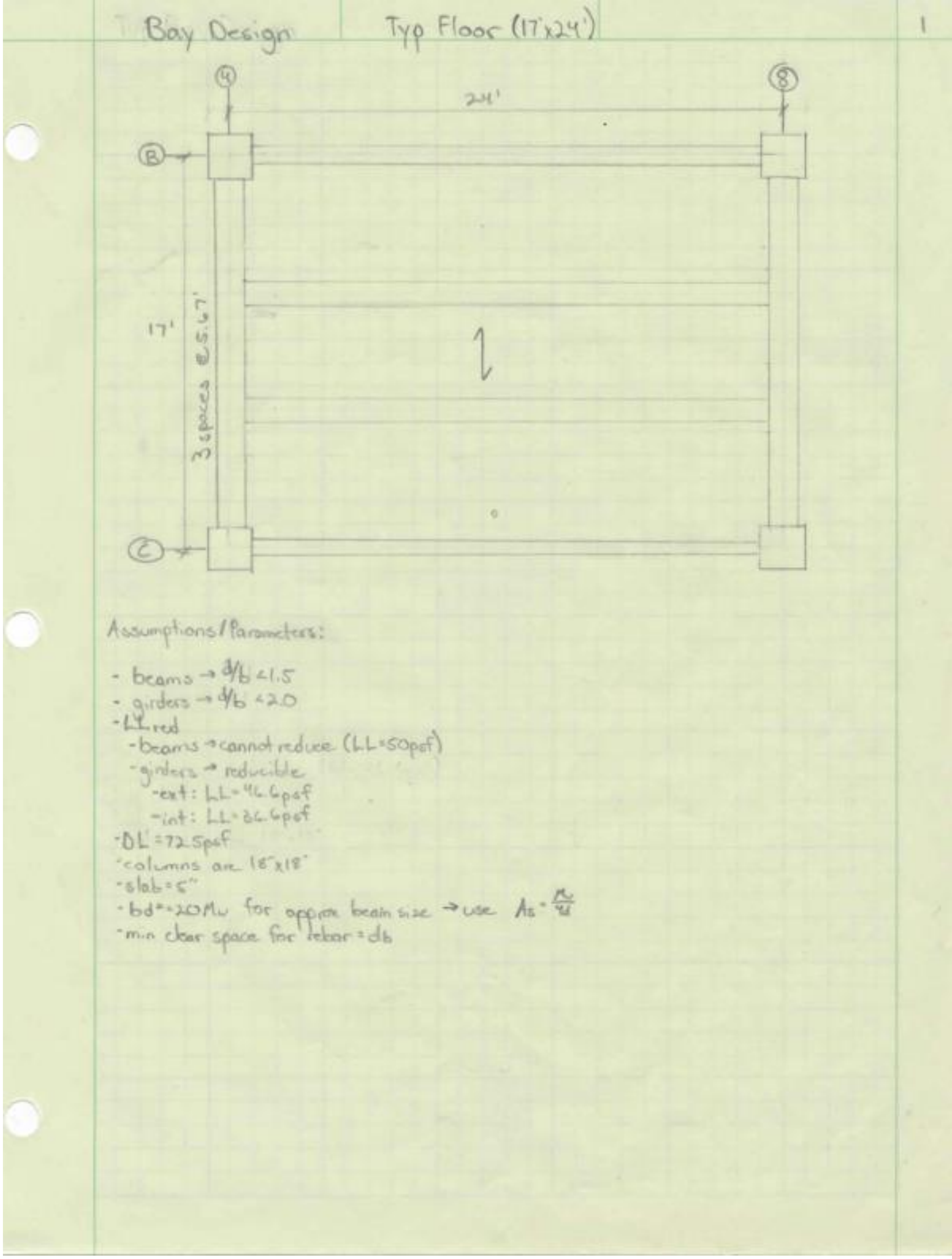
$$A_{s,min} = \max\begin{cases} 0.19\ \text{in}^2 \\ \underline{0.20\ \text{in}^2} \end{cases} \rightarrow \text{select \#4@12"}$$

$$s_{max} = \min\begin{cases} 13.1" \\ 12" \end{cases} \therefore \text{\#4@12" is adequate}\ \checkmark$$

Summary:

$$\boxed{\begin{array}{l} \text{typ floor - h = 5", \#4@12"} \\ \text{lower floors - h = 6", \#4@12"} \end{array}}$$

Beam Design



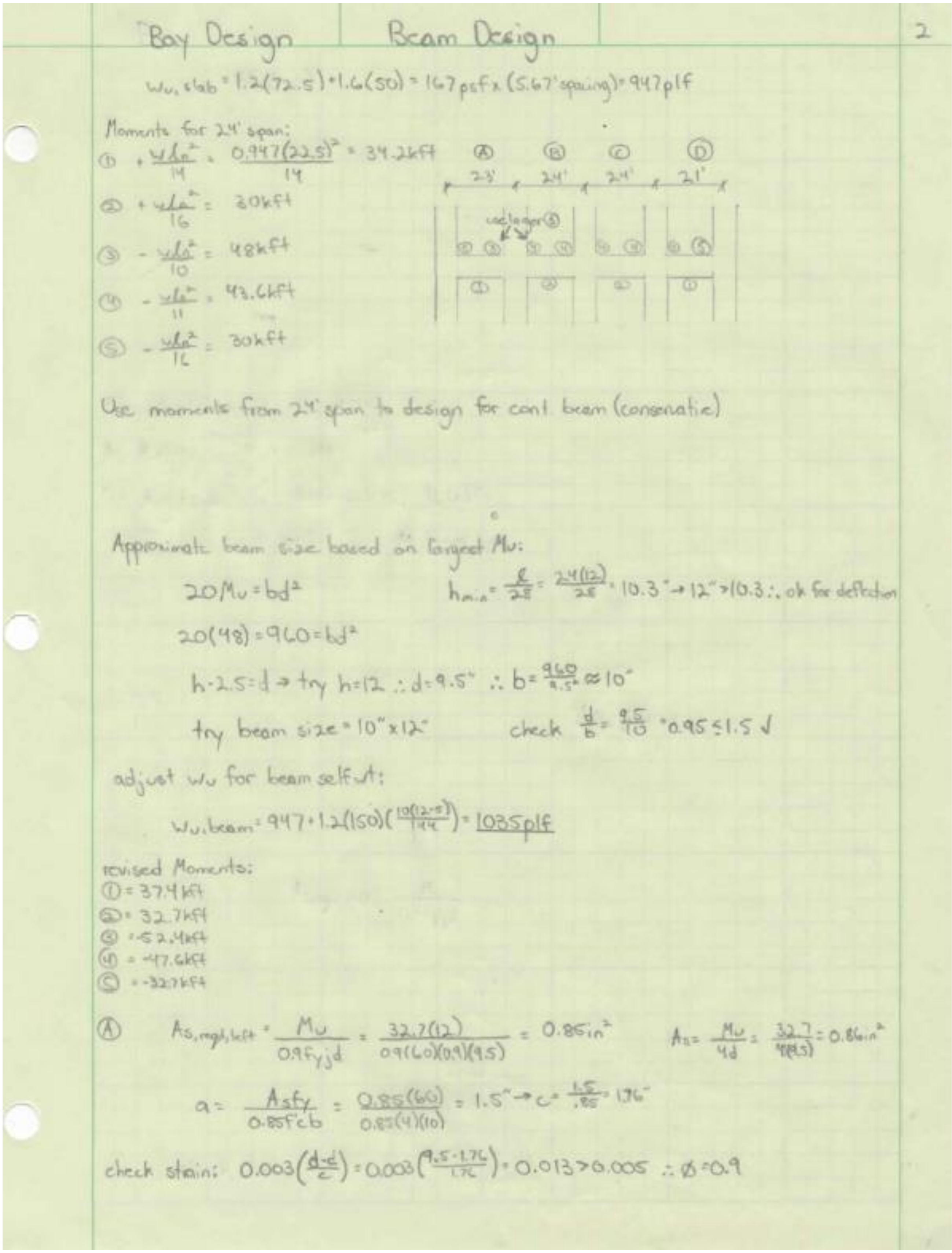
Bay Design

Typ Floor (17'x24')

1

Assumptions/Parameters:

- beams → d/b < 1.5
- girders → d/b < 2.0
- LL red
  - beams → cannot reduce (LL = 50psf)
  - girders → reducible
    - ext: LL = 46.6psf
    - int: LL = 36.6psf
- DL = 72.5psf
- columns are 18"x18"
- slab = 5"
- $bd^2 = 20M_u$ for approx beam size → use $A_s = \frac{M_u}{4d}$
- min clear space for rebar = d_b



Bay Design | Beam Design | 2

$w_{u,slab} = 1.2(72.5)+1.6(50) = 167\,psf \times (5.67'\,spacing) = 947\,plf$

Moments for 24' span:

① $+\frac{wl_n^2}{14} = \frac{0.947(22.5)^2}{14} = 34.2kft$

② $+\frac{wl_n^2}{16} = 30kft$

③ $-\frac{wl_n^2}{10} = 48kft$

④ $-\frac{wl_n^2}{11} = 43.6kft$

⑤ $-\frac{wl_n^2}{16} = 30kft$

Use moments from 24' span to design for cont beam (conservative)

Approximate beam size based on largest Mu:

$20M_u = bd^2$ $\quad h_{min} = \frac{\ell}{28} = \frac{24(12)}{28} = 10.3" \rightarrow 12" > 10.3 \therefore$ ok for deflection

$20(48) = 960 = bd^2$

$h - 2.5 = d \rightarrow$ try $h = 12 \therefore d = 9.5" \therefore b = \frac{960}{9.5^2} \approx 10"$

try beam size = 10"x12" $\quad$ check $\frac{d}{b} = \frac{9.5}{10} = 0.95 \leq 1.5$ ✓

adjust w_u for beam selfwt:

$w_{u,beam} = 947 + 1.2(150)\left(\frac{10(12-5)}{144}\right) = \underline{1035plf}$

revised Moments:

① = 37.4kft

② = 32.7kft

③ = -52.4kft

④ = -47.6kft

⑤ = -32.7kft

Ⓐ $A_{s,reqd,left} = \frac{M_u}{0.9f_yjd} = \frac{32.7(12)}{0.9(60)(0.9)(9.5)} = 0.85in^2$ $\quad A_s = \frac{M_u}{4d} = \frac{32.7}{4(9.5)} = 0.86in^2$

$a = \frac{A_sf_y}{0.85f'_cb} = \frac{0.85(60)}{0.85(4)(10)} = 1.5" \rightarrow c = \frac{1.5}{.85} = 1.76"$

check strain: $0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{9.5-1.76}{1.76}\right) = 0.013 > 0.005 \therefore \phi = 0.9$

Bay Design Beam Design 3

$A_{s,reqd,left,actual} = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{32.7(12)}{0.9(60)(9.5-0.75)} = 0.83 in^2$

select (2)#6 - $A_s = 0.88 in^2 > 0.83$ ∴ ok

$A_{s,reqd,mid} = \frac{37.4(12)}{\underbrace{0.9(60)(0.9)(9.5)}_{461.7}} = 0.97 in^2$

$a = \frac{0.97(60)}{\underbrace{0.85(4)(10)}_{34}} = 1.71" \rightarrow c = 2.01" \rightarrow 0.003\left(\frac{9.5-2.01}{2.01}\right) = 0.011 > 0.005$ ✓

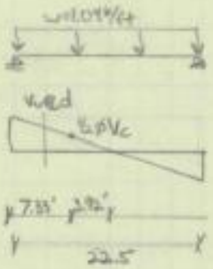
$A_{s,reqd,mid,actual} = \frac{37.4(12)}{0.9(60)(9.5 - \frac{1.71}{2})} = 1.02 in^2 \rightarrow$ select (2)#7

$A_{s,reqd,right} = \frac{52.4(12)}{461.7} = 1.36 in^2$

$a = \frac{1.36(60)}{34} = 2.4" \rightarrow c = 2.82" \rightarrow 0.003\left(\frac{9.5-2.82}{2.82}\right) = 0.0071 > 0.005$ ✓

$A_{s,reqd,right,actual} = \frac{52.4(12)}{0.9(60)(9.5 - \frac{2.4}{2})} = 1.4 in^2 \rightarrow$ select (3)#7 - $A_s = 1.8 in^2 > 1.4$ ∴ ok

check shear:



$V_u @ d$: $\frac{22.5}{2} = 11.25 - \frac{9.5}{12} = 10.46'$ = crit length

$V_u @ d = (1.04)(10.46) = 10.9k$

$\phi V_c = \phi 2 \lambda \sqrt{f'c}\, bd = 0.75(2)(1)\sqrt{4000}(10)(9.5) = 9k$

∴ $\frac{1}{2}\phi V_c = 4.5k$ → provide shear rebar until $\frac{1}{2}\phi V_c$

since $V_u @ d > \frac{1}{2}\phi V_c$ → need shear reinforcement

$S_{max} = \min \begin{cases} d/2 = 9.5/2 = 4.75" \rightarrow \text{use 4" spacing} \\ 24" \end{cases}$

$A_{v,min} = \phi \sqrt{f'c} \frac{b_w s}{f_y} = 0.75\sqrt{4000} \frac{(10)(4)}{60000} = 0.01 in^2$

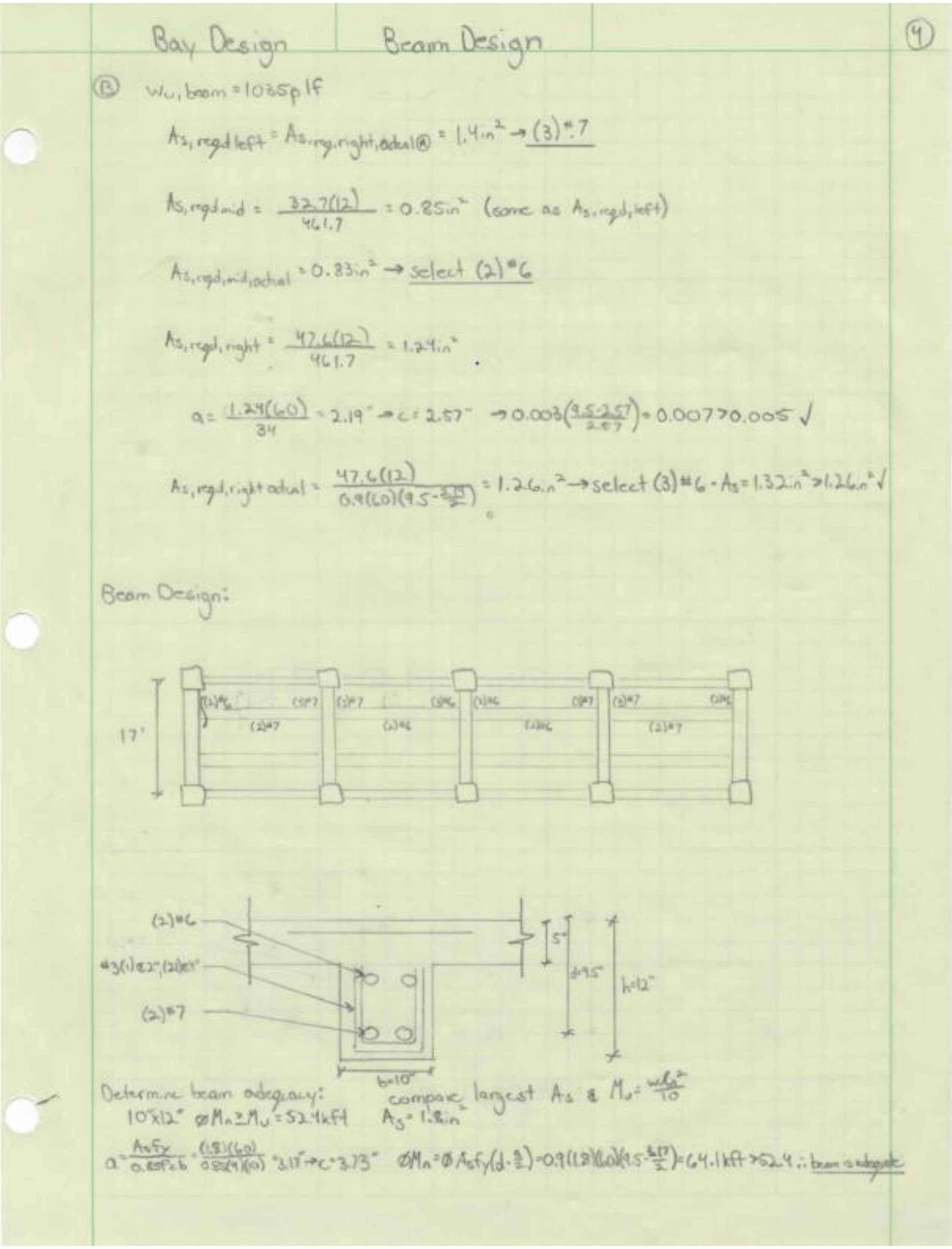
but not less than $\frac{50 b_w s}{f_y} = \frac{50(10)(4)}{60000} = \underline{0.03 in^2} \rightarrow$ select #3 @4" ($A_v = 0.11 in^2$)

$\phi V_n = \phi(V_c + V_s)$
$V_c = 9k$
$V_s = \frac{A_v f_y d}{s} = \frac{2(0.11)(60)(9.5)}{4} = 31.4k$

$\phi V_n = 0.75(9+31.4) = 30.3k > 10.9k$ ∴ ok ✓

$\frac{10.9-4.5}{1.04} = 6.2' + \left(\frac{9.5}{12}\right) = 6.9'$ provide shear rebar

$\frac{6.9(12)}{4} = 21$ ∴ provide #3 (1)@2", 21@4" each end



Bay Design | Beam Design ④

Ⓑ $w_{u,beam} = 1035plf$

$A_{s,reqd\,left} = A_{s,req,right,actual}Ⓐ = 1.4in^2 \rightarrow \underline{(3)\#7}$

$A_{s,reqd\,mid} = \frac{32.7(12)}{461.7} = 0.85in^2$ (same as $A_{s,reqd,left}$)

$A_{s,reqd,mid,actual} = 0.83in^2 \rightarrow \underline{\text{select } (2)\#6}$

$A_{s,reqd,right} = \frac{47.6(12)}{461.7} = 1.24in^2$

$a = \frac{1.24(60)}{34} = 2.19" \rightarrow c = 2.57" \rightarrow 0.003\left(\frac{9.5-2.57}{2.57}\right) = 0.007 > 0.005 ✓$

$A_{s,reqd,right\,actual} = \frac{47.6(12)}{0.9(60)(9.5-\frac{2.19}{2})} = 1.26in^2 \rightarrow$ select (3)#6 - $A_s = 1.32in^2 > 1.26in^2 ✓$

Beam Design:

Determine beam adequacy:

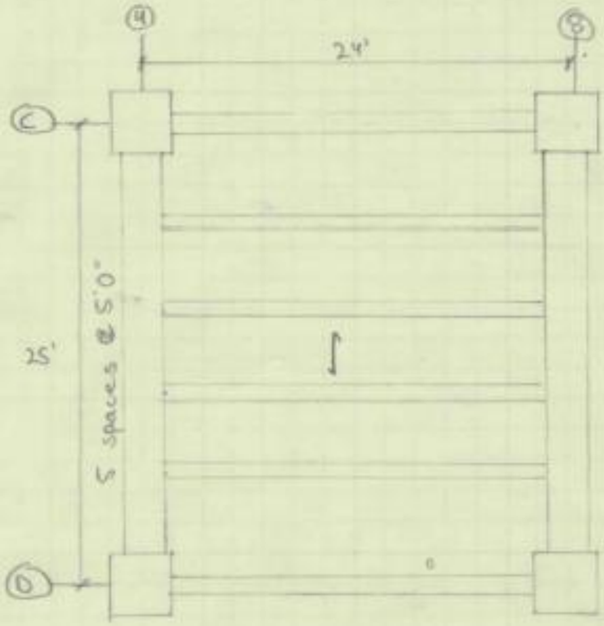
$10"\times12"$ $\phi M_n \geq M_u = 52.4kft$

compare largest A_s & $M_u = \frac{wl_n^2}{10}$

$A_s = 1.8in^2$

$a = \frac{A_s f_y}{0.85f'_c b} = \frac{(1.8)(60)}{0.85(4)(10)} = 3.17" \rightarrow c = 3.73"$ $\phi M_n = \phi A_s f_y(d-\frac{a}{2}) = 0.9(1.8)(60)(9.5-\frac{3.17}{2}) = 64.1kft > 52.4 \therefore \underline{\text{beam is adequate}}$

Bay Design | Typ Floor (24'x25') | 5



LLred:

Beams - N/A → LL=50psf

Girder - int: $A_t = \left(\frac{23+24}{2}\right) \times 25 = 588ft^2 \times 2 = \overset{A_i}{1175ft^2} \rightarrow L = 50\left(.25 + \frac{15}{\sqrt{1175}}\right) = 34.4psf$

ext: $A_i = 588ft^2 \rightarrow L = 50\left(.25 + \frac{15}{\sqrt{1176}}\right) = 43.2psf$

DL = 72.5psf

$W_{u,slab} = 1.2(72.5) + 1.6(50) = 167psf(5) = 835plf$

use $\frac{w_u l_n^2}{10}$ to approx beam size: $\frac{(.835)(22.5)^2}{10} = 42.3kft$

$20M_u = bd^2 \rightarrow 20M_u = d^3$

$20(42.3) = d^3$

$d = 9.5 \rightarrow h = 12", b = 10"$ select 10"x12" beam

$(9.5)^2(10) = 903 > 20M_u \therefore$ use $A_s = \frac{M_u}{4d}$

adjust w_u for beam SW:

$W_{u,beam} = 835 + 1.2(150)\left(\frac{10(12-5)}{144}\right) = \underline{923plf}$

Design Moments: $w_u = .923 klf$, $l_n = 22.5'$

① +33.4kft
② +29.2kft
③ +46.7kft
④ −42.5kft
⑤ −29.2kft

span Ⓐ:

$A_{s,reqd,left} = \frac{29.2}{\underset{38}{4(9.5)}} = 0.77 in^2$ → <u>select (2)#6</u> ($A_s = 0.88 in^2$)

$A_{s,reqd,mid} = \frac{33.4}{38} = 0.88 in^2$ → <u>select (2) #6</u>

$A_{s,reqd,right} = \frac{46.7}{38} = 1.23 in^2$ → <u>select (2)#7</u> ($A_s = 1.2 in^2$) b/c $bd^2 > 20M_u$, can be conservative w/ A_s

shear: V_u @ d: $\frac{22.5}{2} = 11.25' - (\frac{9.5}{12}) = 10.46'$ = crit length

$V_u @ d = (.923)(10.46) = 9.7k$

$\phi V_c = 9k \rightarrow \frac{1}{2}\phi V_c = 4.5k$

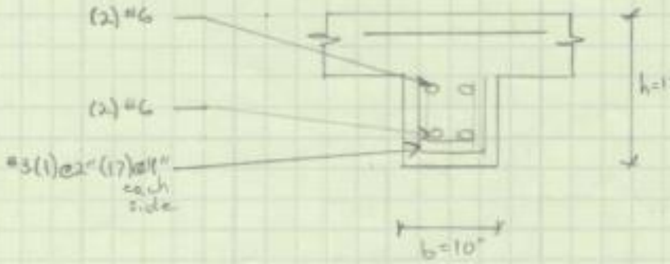
$s_{max} = 4"$ (from prev. beam since same d)

select #3 @ 4"

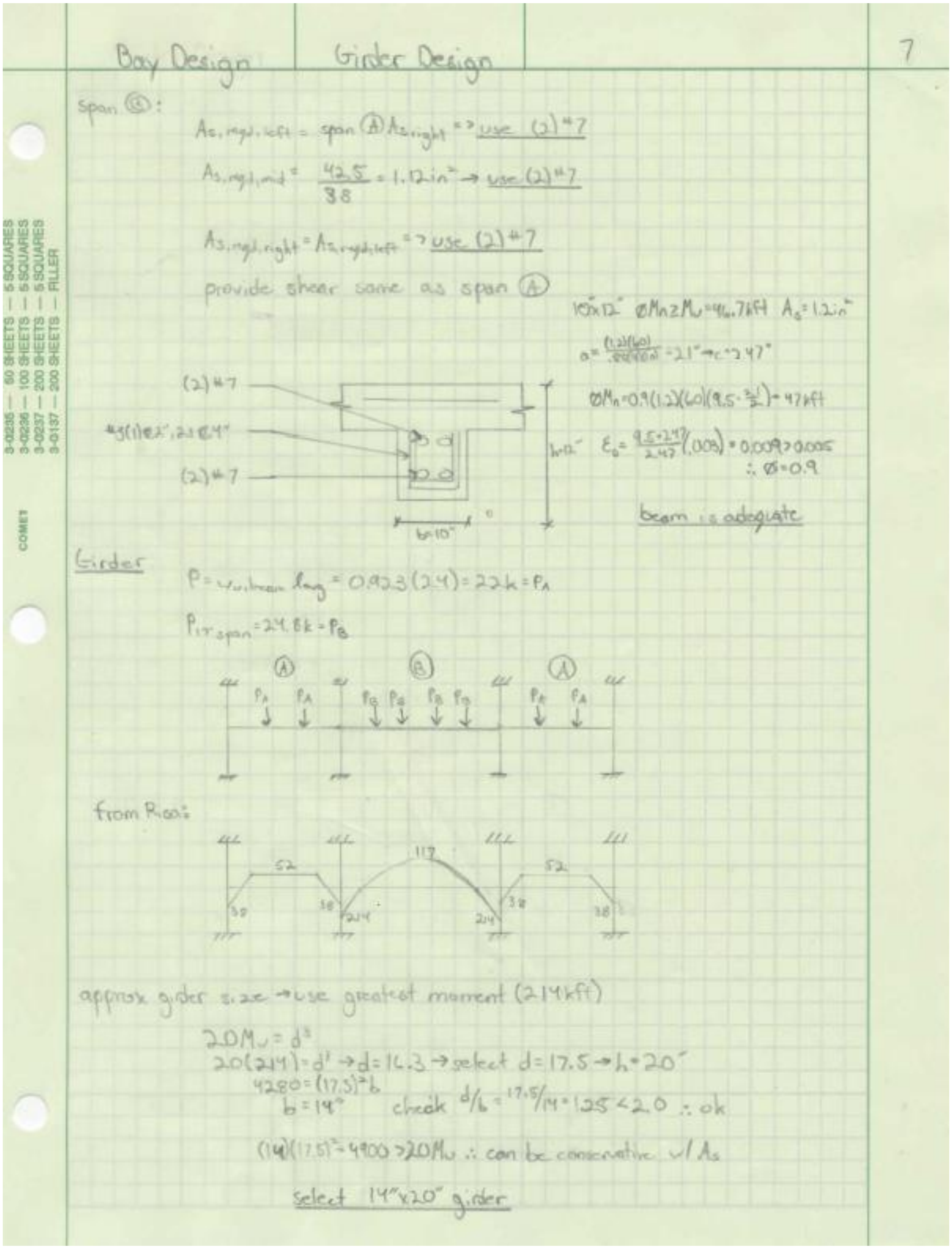
$\left.\begin{array}{l} V_c = 9k \\ V_s = 31.4k \end{array}\right\} \phi V_n = 0.75(9+31.4) = 30.3k > 9.7k$

$\frac{9.7-4.5}{.923} = 5.6'$ provide shear rebar

<u>$\frac{5.6(12)}{4} = 17$ ∴ provide #3 (1)@2", (17)@4" each side</u>



Girder Design



Bay Design | Girder Design | 7

Span Ⓒ:

$A_{s,reqd,left}$ = span Ⓐ $A_{s,right}$ => use (2)#7

$A_{s,reqd,mid} = \frac{42.5}{38} = 1.12 in^2$ → use (2)#7

$A_{s,reqd,right} = A_{s,reqd,left}$ => use (2)#7

provide shear same as span Ⓐ

10"x12" $\phi M_n \geq M_u = 46.7 kft$ $A_s = 1.2 in^2$

$a = \frac{(1.2)(60)}{.85(4)(10)} = 2.1" \rightarrow c = 2.47"$

$\phi M_n = 0.9(1.2)(60)(9.5 - \frac{2.1}{2}) = 47 kft$

$\varepsilon_t = \frac{9.5 - 2.47}{2.47}(.003) = 0.009 > 0.005$
$\therefore \phi = 0.9$

beam is adequate

Girder

$P = w_{u,beam} l_{avg} = 0.923(24) = 22k = P_A$

$P_{1r\,span} = 24.8k = P_B$

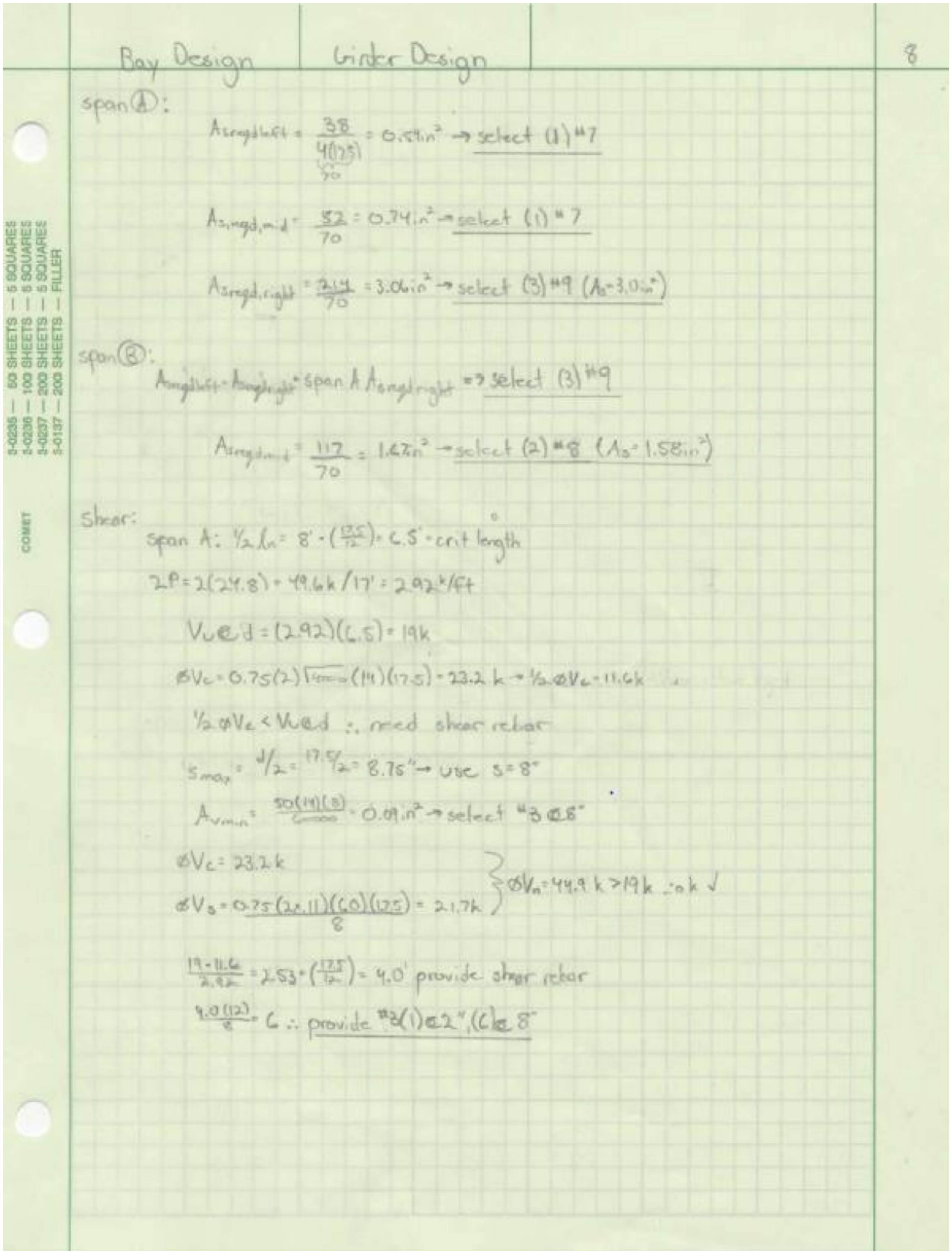
from Risa:

approx girder size → use greatest moment (214kft)

$20M_u = d^3$
$20(214) = d^3 \rightarrow d = 16.3 \rightarrow$ select $d = 17.5 \rightarrow h = 20"$
$4280 = (17.5)^2 b$
$b = 14"$ check $d/b = 17.5/14 = 1.25 < 2.0 \therefore$ ok

$(14)(17.5)^2 = 4900 > 20M_u \therefore$ can be conservative w/ A_s

select 14"x20" girder



Bay Design | Girder Design | 8

span Ⓐ:

$A_{s,reqd,left} = \frac{38}{\underset{70}{4(17.5)}} = 0.54 in^2 \rightarrow$ select (1) #7

$A_{s,reqd,mid} = \frac{52}{70} = 0.74 in^2 \rightarrow$ select (1) #7

$A_{s,reqd,right} = \frac{214}{70} = 3.06 in^2 \rightarrow$ select (3) #9 ($A_s = 3.0 in^2$)

span Ⓑ:

$A_{s,reqd,left} = A_{s,reqd,right} =$ span A $A_{s,reqd,right}$ ⇒ select (3) #9

$A_{s,reqd,mid} = \frac{117}{70} = 1.67 in^2 \rightarrow$ select (2) #8 ($A_s = 1.58 in^2$)

shear:

span A: $\frac{1}{2} l_n = 8' - \left(\frac{17.5}{12}\right) = 6.5'$ = crit length

$2P = 2(24.8) = 49.6k / 17' = 2.92^k/ft$

$V_u @ d = (2.92)(6.5) = 19k$

$\phi V_c = 0.75(2)\sqrt{4000}(14)(17.5) = 23.2 k \rightarrow \frac{1}{2}\phi V_c = 11.6k$

$\frac{1}{2}\phi V_c < V_u @ d$ ∴ need shear rebar

$s_{max} = d/2 = 17.5/2 = 8.75'' \rightarrow$ use $s = 8''$

$A_{v,min} = \frac{50(14)(8)}{60000} = 0.09 in^2 \rightarrow$ select #3 @ 8"

$$\left.\begin{array}{l}\phi V_c = 23.2k \\ \phi V_s = \frac{0.75(2 \times .11)(60)(17.5)}{8} = 21.7k\end{array}\right\} \phi V_n = 44.9 k > 19k \therefore ok ✓$$

$\frac{19 - 11.6}{2.92} = 2.53 + \left(\frac{17.5}{12}\right) = 4.0'$ provide shear rebar

$\frac{4.0(12)}{8} = 6$ ∴ provide #3 (1) @ 2", (6) @ 8" span B: $\frac{1}{2} l_n = 11.75' - \left(\frac{17.5}{12}\right) = 10.29'$

$4P = 4(22) = 88^k / 25 = 3.52^k/ft$

$V_u @ d = (3.52)(10.29) = 36.2k$

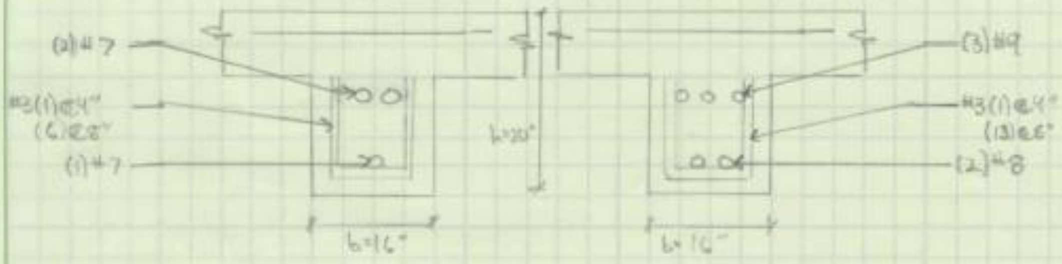
$\phi V_c = 23.2k \rightarrow \frac{1}{2}\phi V_c = 11.6k$

use $s = 8"$

$\left.\begin{array}{l} \phi V_c = 23.2k \\ \phi V_s = 21.7k \text{ (from prev calc)} \end{array}\right\} \phi V_n = 44.9k > 36.2k$

$\frac{36.2 - 11.6}{3.52} = 7' + \left(\frac{17.5}{12}\right) = 8.4'$ provide shear rebar

$\frac{8.4(12)}{8} = 13 \therefore$ <u>provide #3 (1) @ 2", (13) @ 8"</u>


Girder (Span A)

Girder (Span B)

Determine girder adequacy: $16" \times 20"$ $\phi M_n \geq M_u = 214kft$ $A_s = 3.0 in^2$

$a = \frac{(3)(60)}{.85(4)(16)} = 3.3" \rightarrow c = 3.89"$

$\phi M_n = 0.9(3)(60)\left(17.5 - \frac{3.3}{2}\right) = 214kft = 214 \therefore ok$

$\varepsilon_s = \frac{17.5 - 3.89}{3.89}(0.003) = 0.01 > 0.005 \therefore \phi = 0.9$ ∴ <u>girder is adequate</u>

Bay Design | Lower Floors (1-3) | 10

DL = 6/12(150) + 10 = 85psf
LL = 100psf → can't reduce for beams

For 17' span:

$w_{u,slab} = 1.2(85) + 1.6(100) = 262psf(5.67) = 1486plf$

approx beam size w/ $\frac{w_u l_n^2}{10} = \frac{1.49(22.5)^2}{10} = 75.4kft$

$20M_u = d^3$
$20(75.4) = 1508 = d^3 \rightarrow d = 11.5" \therefore h = 14"$ & $b = 12"$ select 12"x14" beam

$(11.5)^2(12) = 1587 > 1508 \therefore$ use $A_s = \frac{M_u}{4d}$

adjust for beam SW:

$w_{u,beam} = 1486 + 1.2(150)\left(\frac{12(14-5)}{144}\right) = \underline{1606plf}$

design moments: $w_u = 1.61klf$, $l_n = 22.5'$

① +58.2
② +50.9
③ -81.5
④ -74.1
⑤ -50.9

Span A&E:

$A_{s,reqd,left} = \frac{50.9}{\underbrace{4(11.5)}_{46}} = 1.1in^2 \rightarrow$ select (2)#7 ($A_s = 1.2in^2$)

$A_{s,reqd,mid} = \frac{58.2}{46} = 1.27in^2 \rightarrow$ select (2)#7

$A_{s,reqd,right} = \frac{81.5}{46} = 1.77in^2 \rightarrow$ select (3)#7

shear: crit length $= \frac{22.5}{2} = 11.25 - \left(\frac{11.5}{12}\right) = 10.3'$

$V_u@d = (1.61)(10.3) = 16.6k$

$\phi V_c = 0.75(2)\sqrt{4000}(12)(11.5) = 13.09k \rightarrow 1/2\phi V_c = 6.6k$

$s_{max} = 11.5/2 = 5.75" \rightarrow$ use $s = 5"$

$A_{v,min} = \frac{50(12)(5)}{60000} = 0.05in^2 \rightarrow$ select #3@5"

$\frac{16.6-6.6}{1.61} = 6.2'$ provide shear rebar

$\frac{6.2(12)}{5} = 15$

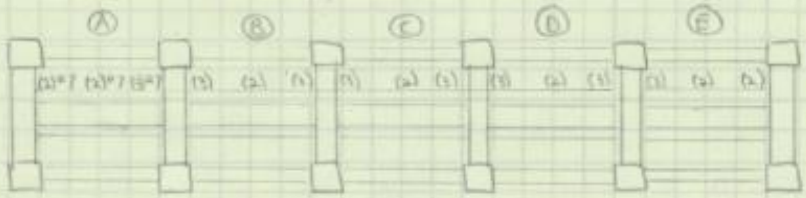
provide #3 (1)@2", (15)@5" each end

Bay Design | Beam Design | 11.

span B,C,D: w_u = 1.61klf

$A_{s\,reqd\,left} = A_{s\,reqd\,right}$ = span A $A_{s\,reqd\,right}$ = ? $\underline{\text{select (3)\#7}}$

$A_{s\,mid} = \frac{50.9}{46} = 1.1\text{in}^2 \rightarrow \underline{\text{select (2)\#7}}$



For 25' span:

$w_{u,slab} = 1.2(85)+1.6(100) = 262(5') = \underline{1310\text{plf}}$

approx beam w/ $\frac{w_u l_n^2}{10} = \frac{(1.31)(22.5)^2}{10} = 66.3\text{kft}$

$20M_u = 1326 = d^3 \rightarrow d = 11.5 \rightarrow h = 14, b = 10$ $\underline{\text{select 10"x14" beam}}$

$(11.5)^2(10) = 1323 \approx 1326$ ∴ use $A_s = \frac{M_u}{4d}$

adjust for beam SW:

$w_{u\,beam} = 1310 + 1.2(150)\left(\frac{10(14-6)}{144}\right) = 1410\text{plf}$

design moments: $w_u = 1.41\text{klf}$ $l_n = 22.5$

① +51
② +44.6
③ -71.4
④ -64.9
⑤ -44.6

span A&E:

$A_{s\,reqd\,left} = \frac{44.6}{46} = 0.97\text{in}^2 \rightarrow \underline{\text{select (1)\#6, (1)\#7 } (A_s = 1.04")}$

$A_{s\,reqd\,mid} = \frac{51}{46} = 1.11\text{in}^2 \rightarrow \underline{\text{select (2)\#7}}$

$A_{s\,reqd\,right} = \frac{71.4}{46} = 1.55\text{in}^2 \rightarrow \underline{\text{select (3)\#7,}}$

shear: crit length = 10.3' (from prev calc)

$V_u@d = (1.41)(10.3) = 14.5k$

$\phi V_c = 0.75(2)\sqrt{4000}(10)(11.5) = 10.9k \rightarrow \frac{1}{2}\phi V_c = 5.5k$

use s = 5"

$A_{vmin} = \frac{50(10)(5)}{60000} = 0.04\text{in}^2 \rightarrow$ select #3@5"

$\frac{14.5-5.5}{1.41} = 6.4'$ provide shear rebar

$\frac{(6.4)(12)}{5} = 15$

$\underline{\text{provide \#3 (1)@2", (15)@5" each end}}$

Bay Design | Beam Design | 12.

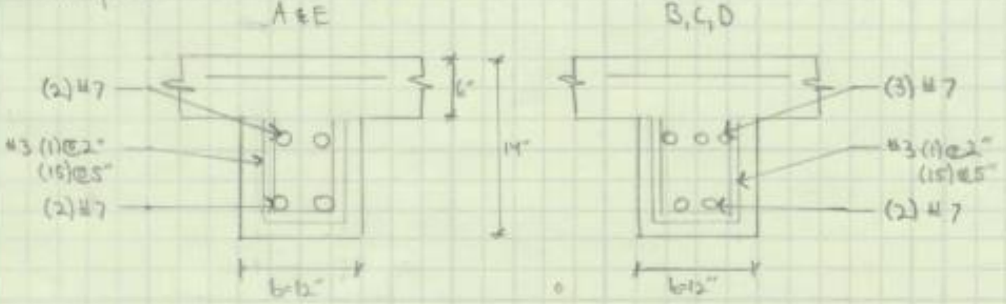
span B,C,D: w_u=1.41klf

$A_{s,reqd,left} = A_{s,reqd,right}$ = span A $A_{s,reqd,right}$ => select (2)#7, (1)#5

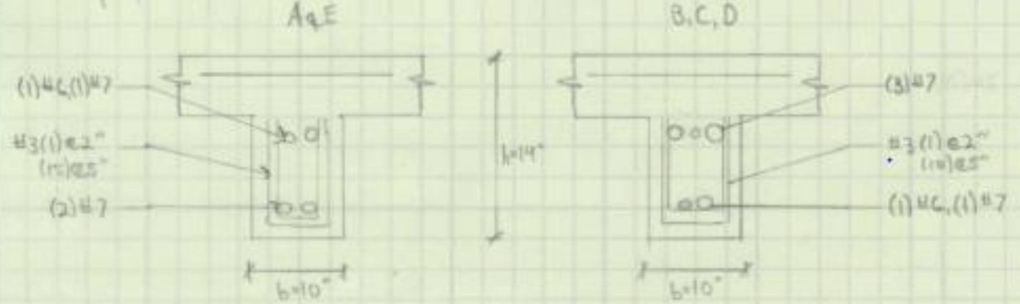
$A_{s,reqd,mid} = \frac{44.6}{46} = 0.97 in^2$ → select (1)#6, (1)#7

Beam Summary:

17' span:



25' span:



Determine if beams are adequate: compare largest A_s & $M_u = \frac{w l_n^2}{10}$

17' span: 12"x14" $\phi M_n \geq M_u$ = 81.5kft A_s = 1.8in²

$\phi M_n = \phi A_s f_y (d - \frac{a}{2})$ where $a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(1.8)(60)}{0.85(4)(12)} = 2.64" \rightarrow c = 3.06"$

$= 0.9(1.8)(60)(11.5 - \frac{2.64}{2})$ = 82.5kft > 81.5kft ✓

$\varepsilon_s = \frac{d-c}{c}(\varepsilon_c) = \frac{11.5-3.06}{3.06}(.003) = 0.008 \geq 0.005$ ∴ ϕ=0.9 → beam is adequate

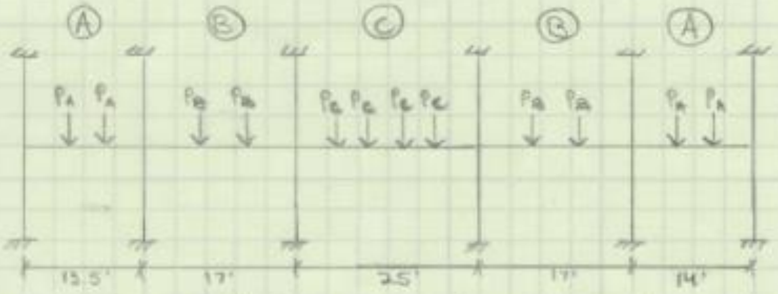
25' span: 10"x14" $\phi M_n \geq M_u = \frac{1.41(22.5)^2}{10}$ = 71.4kft A_s=1.8in²

$a = \frac{(1.8)(60)}{.85(4)(10)} = 3.17" \rightarrow c = 3.73"$

$\phi M_n = 0.9(1.8)(60)(11.5 - \frac{3.17}{2}) = 80.3 > 71.4$ ✓ $\varepsilon_s = \frac{11.5-3.73}{3.73}(.003) = .006$ ∴ beam is adequate

Bay Design | Girder Design | 13.

Girder

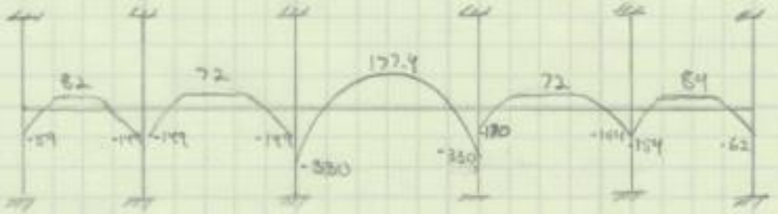


$P_A = W_{A\,beam}\, l_{avg} = (1.95)(24) = 46.8k$ $l_{avg} = 24'$ conservative

$P_B = W_{B\,beam}\, l_{avg} = (1.61)(24) = 38.6k$

$P_C = W_{C\,beam}\, l_{avg} = (1.41)(24) = 33.8k$

from Risa:



approx girder size → use greatest M_u (330kft)

$20(330) = d^3$

$6600 = d^3 \rightarrow d = 18.8$ choose $d = 19.5 \therefore h = 22$

$6600 = (19.5)^2 b$

$b = 18''$ check $d/b = 19.5/18 = 1.08 < 2.0 \therefore$ ok for dimensions

$(18)(19.5)^2 = 6845 > 20M_u \therefore$ use $A_s = \frac{M_u}{4d}$

<u>select 18"x22" girder</u>

Bay Design | Beam Design (greenroof) | 14

Green Roof:

DL = 140 psf

LL = 100 psf (unreducible b/c $K_{LL}A_T < 400 ft^2$)

span = 14' → use 4 beams, 3sp @ 4.67' (use larger of (2) green roof spans = 14')

$W_{u,beam} = (1.2(140) + 1.6(100) = 328 psf(4.67) = 1532 plf$ → 1.70 klf for beam SW

approx beam size w/ $\frac{w_u l_n^2}{10} = \frac{1.7(22.5)^2}{10} = 86.1$ kft

$20M_u = 1722 = d^3 \rightarrow d = 12.0 \rightarrow d = 13.5 \therefore h = 16"$ } select 12"x16" beam
$1722 = (13.5)^2 b \therefore b = 9.5 \rightarrow b = 10"$

$13.5^2(10) = 1732 > 20M_u \therefore$ use $A_s = \frac{M_u}{4d}$

SW check:

$W_{u beam} = 1532 + 1.2(150)\left(\frac{10(16-6)}{144}\right) = 1.66 klf < 1.70 \therefore$ assumption ok for SW ✓

① +61.4
② +53.8
③ -86
④ -78.2
⑤ -53.8

there are multiple extensive green roofs ∴ use largest spans to determine A_s (conservative)

2nd Floor (4-14, E-F)

span A: $W_u = 1.7 klf$, $l_n = 16'$

① +31.1
② +27.2
③ -43.5
④ -39.5
⑤ -27.2

$A_{s,reqd\,left} = \frac{27.2}{\underbrace{4(13.5)}_{54}} = 0.5 in^2$ → select (2) #5 ($A_s = 0.62 in^2$)

$A_{s,reqd\,mid} = \frac{31.1}{54} = 0.58 in^2$ → select (2) #5

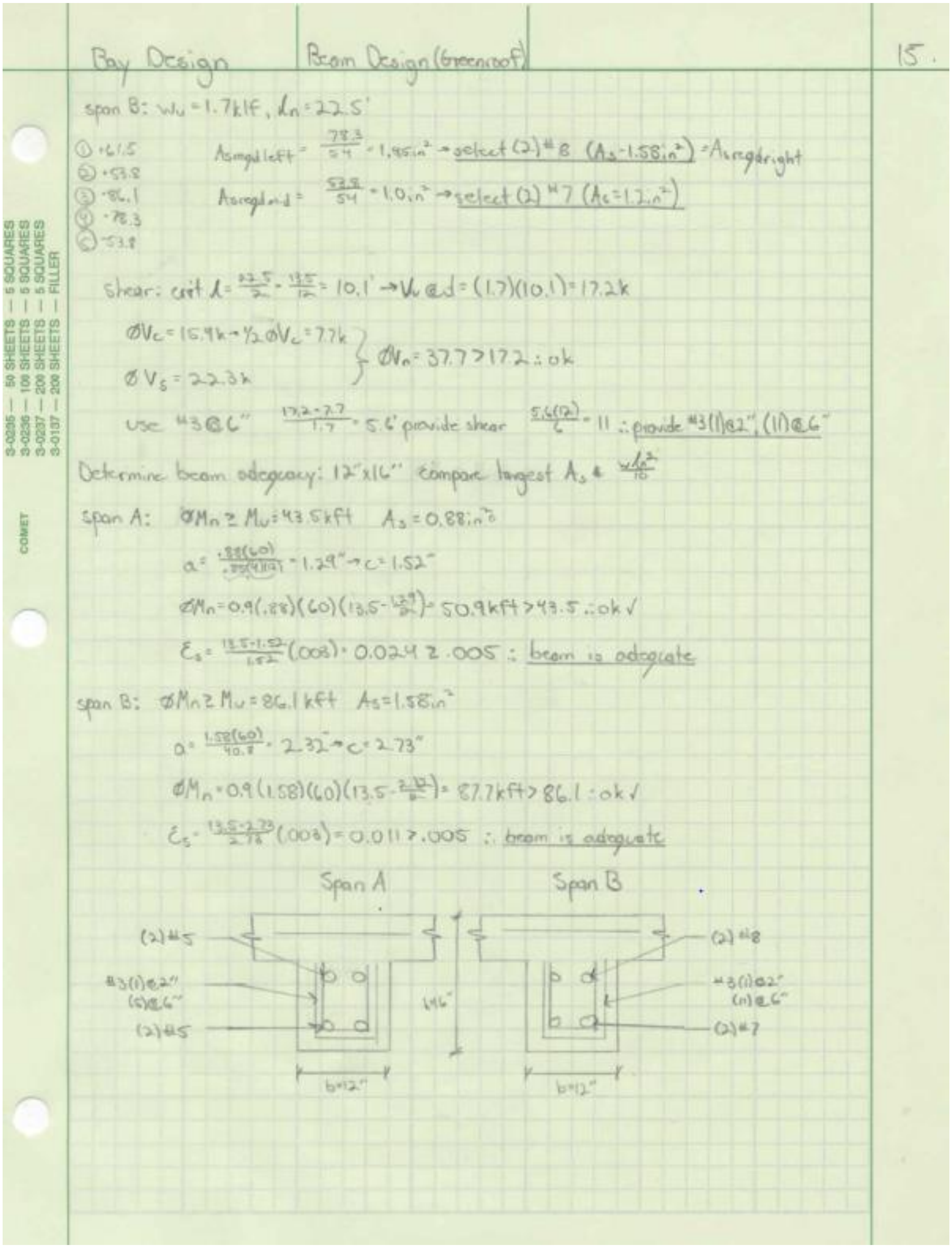
$A_{s,reqd\,right} = \frac{43.5}{54} = 0.81 in^2$ → select (2) #6 ($A_s = 0.88 in^2$)

shear: crit length = $16/2 - 13.5/12 = 6.9' \rightarrow V_u @ d = (1.7)(6.9) = 11.7k$

$\phi V_c = 0.75(2)\sqrt{4000}(12)(13.5) = 15.4k \rightarrow \frac{1}{2}\phi V_c = 7.7k$ $\phi V_s = \frac{0.75(0.22)(60)(13.5)}{6} = 22.3 \rightarrow \phi V_n = 37.7k > 11.7k \therefore$ ok ✓

$S_{max} = 13.5/2 \rightarrow$ use s = 6" $\frac{11.7 - 7.7}{1.7} = 2.35' \rightarrow \frac{2.35(12)}{6} = 5$ provide #3 (1) @ 2", (5) @ 6"

$A_{v,min} = \frac{50(12)(6)}{60000} = 0.06 in^2 \rightarrow$ use #3 @ 6"



Bay Design | Beam Design (Greenroof) | 15.

span B: $w_u = 1.7\text{klf}$, $l_n = 22.5'$

① +61.5
② +53.8
③ -86.1
④ -78.3
⑤ -53.8

$A_{s\,reqd\,left} = \frac{78.3}{54} = 1.45\text{in}^2 \rightarrow$ select (2) #8 ($A_s = 1.58\text{in}^2$) $= A_{s\,reqd\,right}$

$A_{s\,reqd\,mid} = \frac{53.8}{54} = 1.0\text{in}^2 \rightarrow$ select (2) #7 ($A_s = 1.2\text{in}^2$)

shear: crit $l = \frac{22.5}{2} - \frac{13.5}{12} = 10.1' \rightarrow V_u @ d = (1.7)(10.1) = 17.2k$

$\phi V_c = 15.4k \rightarrow \frac{1}{2}\phi V_c = 7.7k$

$\phi V_s = 22.3k$

$\left.\right\} \phi V_n = 37.7 > 17.2 \therefore$ ok

use #3 @ 6" $\frac{17.2 - 7.7}{1.7} = 5.6'$ provide shear $\frac{5.6(12)}{6} = 11 \therefore$ provide #3 (1) @ 2", (11) @ 6"

Determine beam adequacy: 12"x16" compare largest A_s & $\frac{w l_n^2}{10}$

span A: $\phi M_n \geq M_u = 43.5\text{kft}$ $A_s = 0.88\text{in}^2$

$a = \frac{.88(60)}{.85(4)(12)} = 1.29" \rightarrow c = 1.52"$

$\phi M_n = 0.9(.88)(60)(13.5 - \frac{1.29}{2}) = 50.9\text{kft} > 43.5 \therefore$ ok ✓

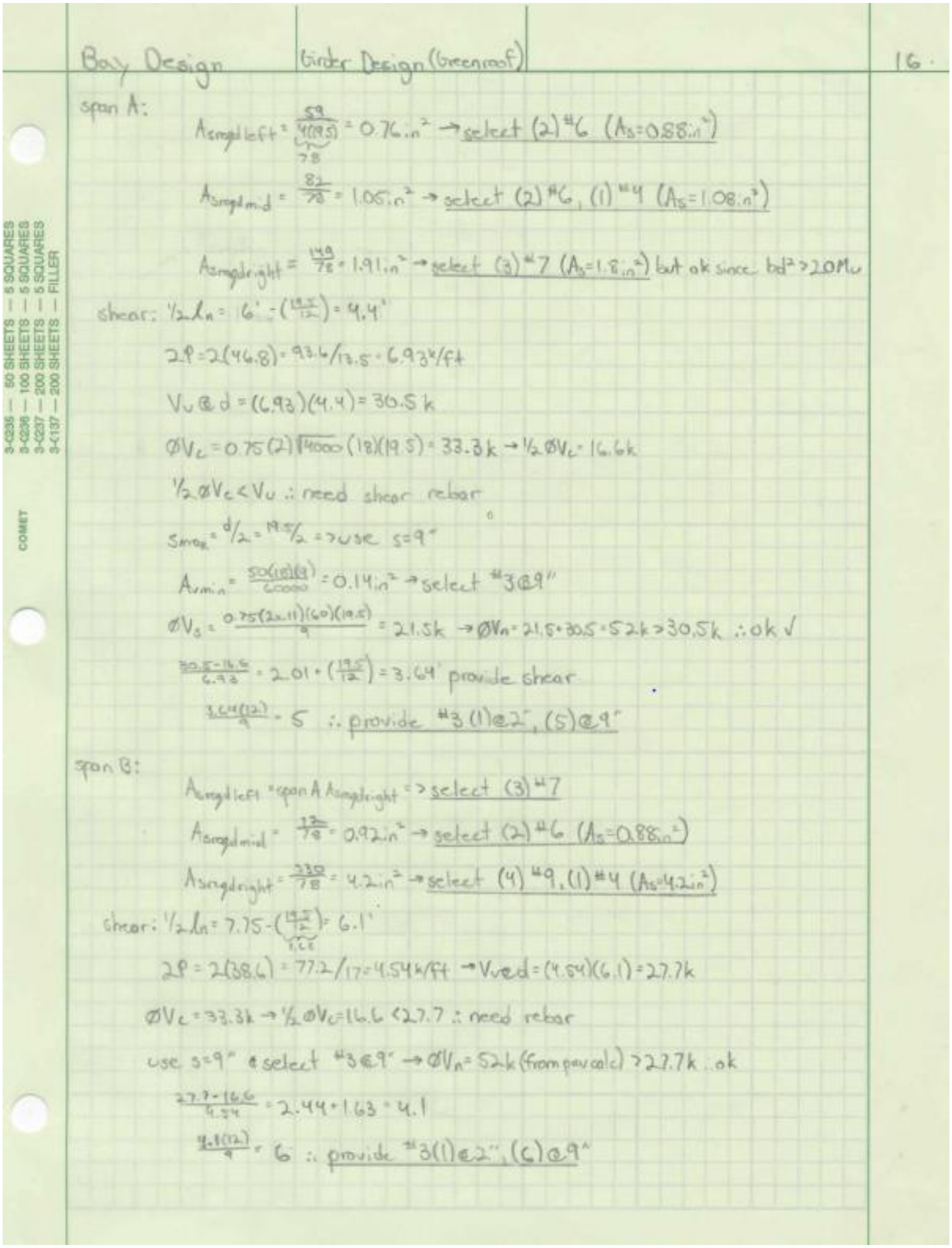
$\varepsilon_s = \frac{13.5 - 1.52}{1.52}(.003) = 0.024 \geq .005 \therefore$ beam is adequate

span B: $\phi M_n \geq M_u = 86.1\text{kft}$ $A_s = 1.58\text{in}^2$

$a = \frac{1.58(60)}{40.8} = 2.32" \rightarrow c = 2.73"$

$\phi M_n = 0.9(1.58)(60)(13.5 - \frac{2.32}{2}) = 87.7\text{kft} > 86.1 \therefore$ ok ✓

$\varepsilon_s = \frac{13.5 - 2.73}{2.73}(.003) = 0.011 > .005 \therefore$ beam is adequate



Bay Design | Girder Design (Greenroof) | 16.

span A:

$A_{s\,reqd\,left} = \frac{59}{\underbrace{4(19.5)}_{78}} = 0.76\,in^2 \rightarrow$ <u>select (2) #6 ($A_s = 0.88\,in^2$)</u>

$A_{s\,reqd\,mid} = \frac{82}{78} = 1.05\,in^2 \rightarrow$ <u>select (2) #6, (1) #4 ($A_s = 1.08\,in^2$)</u>

$A_{s\,reqd\,right} = \frac{149}{78} = 1.91\,in^2 \rightarrow$ <u>select (3) #7 ($A_s = 1.8\,in^2$)</u> but ok since $bd^2 > 20M_u$

shear: $\frac{1}{2}\ell_n = 16' - \left(\frac{19.5}{12}\right) = 4.4'$

$2P = 2(46.8) = 93.6/13.5 = 6.93^k/ft$

$V_u @ d = (6.93)(4.4) = 30.5\,k$

$\phi V_c = 0.75(2)\sqrt{4000}(18)(19.5) = 33.3k \rightarrow \frac{1}{2}\phi V_c = 16.6k$

$\frac{1}{2}\phi V_c < V_u$ ∴ need shear rebar

$s_{max} = d/2 = 19.5/2 \Rightarrow$ use $s = 9"$

$A_{v\,min} = \frac{50(18)(9)}{60000} = 0.14\,in^2 \rightarrow$ select #3@9"

$\phi V_s = \frac{0.75(2 \times .11)(60)(19.5)}{9} = 21.5k \rightarrow \phi V_n = 21.5 + 30.5 = 52k > 30.5k$ ∴ ok ✓

$\frac{30.5 - 16.6}{6.93} = 2.01 + \left(\frac{19.5}{12}\right) = 3.64'$ provide shear

$\frac{3.64(12)}{9} = 5$ ∴ <u>provide #3 (1)@2", (5)@9"</u>

span B:

$A_{s\,reqd\,left}$ = span A $A_{s\,reqd\,right} \Rightarrow$ <u>select (3) #7</u>

$A_{s\,reqd\,mid} = \frac{72}{78} = 0.92\,in^2 \rightarrow$ <u>select (2) #6 ($A_s = 0.88\,in^2$)</u>

$A_{s\,reqd\,right} = \frac{330}{78} = 4.2\,in^2 \rightarrow$ <u>select (4) #9, (1) #4 ($A_s = 4.2\,in^2$)</u>

shear: $\frac{1}{2}\ell_n = 7.75 - \underbrace{\left(\frac{19.5}{12}\right)}_{1.63} = 6.1'$

$2P = 2(38.6) = 77.2/17 = 4.54k/ft \rightarrow V_u @ d = (4.54)(6.1) = 27.7k$

$\phi V_c = 33.3k \rightarrow \frac{1}{2}\phi V_c = 16.6 < 27.7$ ∴ need rebar

use $s = 9"$ & select #3@9" $\rightarrow \phi V_n = 52k$ (from prev calc) $> 27.7k$ ∴ ok

$\frac{27.7 - 16.6}{4.54} = 2.44 + 1.63 = 4.1$

$\frac{4.1(12)}{9} = 6$ ∴ <u>provide #3 (1)@2", (6)@9"</u>

Bay Design | Girder Design (Greenroof) | 17

span C:

$A_{s reqd left} = A_{s reqd right} = $ span B $A_{s reqd right} \Rightarrow$ select (4)#9 (1)#4

$A_{s reqd mid} = \frac{177.4}{78} = 2.3in^2 \rightarrow$ select (2)#9, (1)#6 $(A_s = 2.44in^2)$

Shear: $\frac{1}{2} l_n = 11.75' - 1.63 = 10.1'$

$4P = 4(33.8) = 135.2/25 = 5.4k/ft \rightarrow V_u @ d = 5.4(10.1) = 54.6k$

$\phi V_c = 33.3k \rightarrow \frac{1}{2} \phi V_c = 16.6k$

use $s = 9" \rightarrow A_{v min} = 0.14in^2 \rightarrow$ select #4 @ 9"

$\phi V_s = \frac{0.75(2 \times .2)(60)(19.5)}{9} = 39k \rightarrow \phi V_n = 16.6 + 39 = 55.6k > 54.6k \therefore ok$ ✓

$\frac{54.6 - 16.6}{5.4} = 7.04 + \left(\frac{19.5}{12}\right) = 8.67'$ provide shear

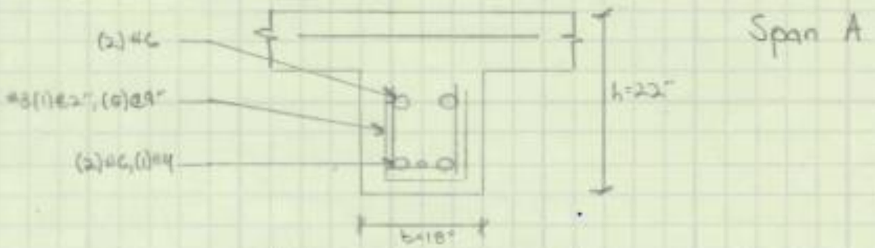
$\frac{8.67(12)}{9} = 12 \therefore$ provide #4 (1) @ 2", (12) @ 9"

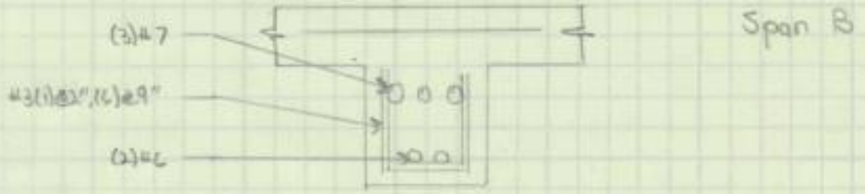
Determine girder adequacy: 18" x 22" $\phi M_n \geq M_u = 330kft$ $A_s = 4.2in^2$

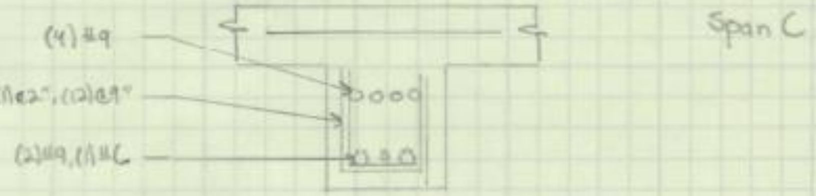
$a = \frac{(4.2)(6.0)}{.85(4)(18)} = 4.12" \rightarrow c = 4.84"$ $\phi M_n = 0.9(4.2)(60)\left(19.5 - \frac{4.12}{2}\right) = 330kft = 330 \therefore ok$

$\varepsilon_s = \frac{19.5 - 4.84}{4.84}(.003) = 0.009 > .005 \therefore \phi = 0.9$ & girder is adequate

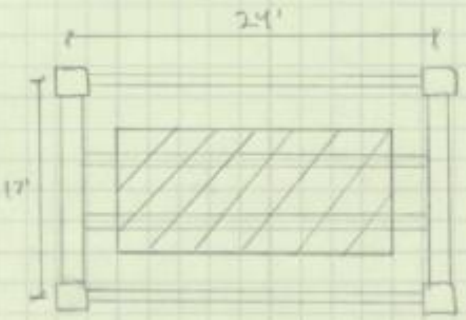
Girder Design:







Bay Design | Beam Design (Pool) | 18



Pool Area = 10'x20' = 200ft²

DL = 62.4 lb/ft³ x 5' deep
= 312psf + 85
= 397psf ≈ 400psf

DL = 400psf
LL = 100psf

$w_u = 1.2(400) + 1.6(100) = 640(5.67) = 3.63$ k/ft

$l_n = 22.5'$ $M_u = \frac{(3.63)(22.5)^2}{10} = 184$ kft

$20M_u = 3675 = d^3 \to d = 15.4 \to$ use $d = 17.5 \to h = 20''$

$3675 = (17.5)^2 b \to b = 14$ select 14"x18" beam

$17.5^2(14) = 4288 > 20M_u \to$ use $A_s = \frac{M_u}{4d}$

adjust for beam SW:

$w_u = 3630 + 1.2(150)\left(\frac{14(20-6)}{144}\right) = 3.88$ k/ft

design moments: $w_u = 3.88$ $l_n = 22.5'$

① +140.3
② +123
③ -196
④ -178
⑤ -123

$A_{s,req'd\,left} = \frac{178}{4(17.5)} = 2.54\,in^2 \to$ select (2)#9 (1)#7 = $A_{s,req'd\,right}$ ($A_s = 2.6\,in^2$)

$A_{s,req'd\,mid} = \frac{123}{70} = 1.76\,in^2 \to$ select (2)#9

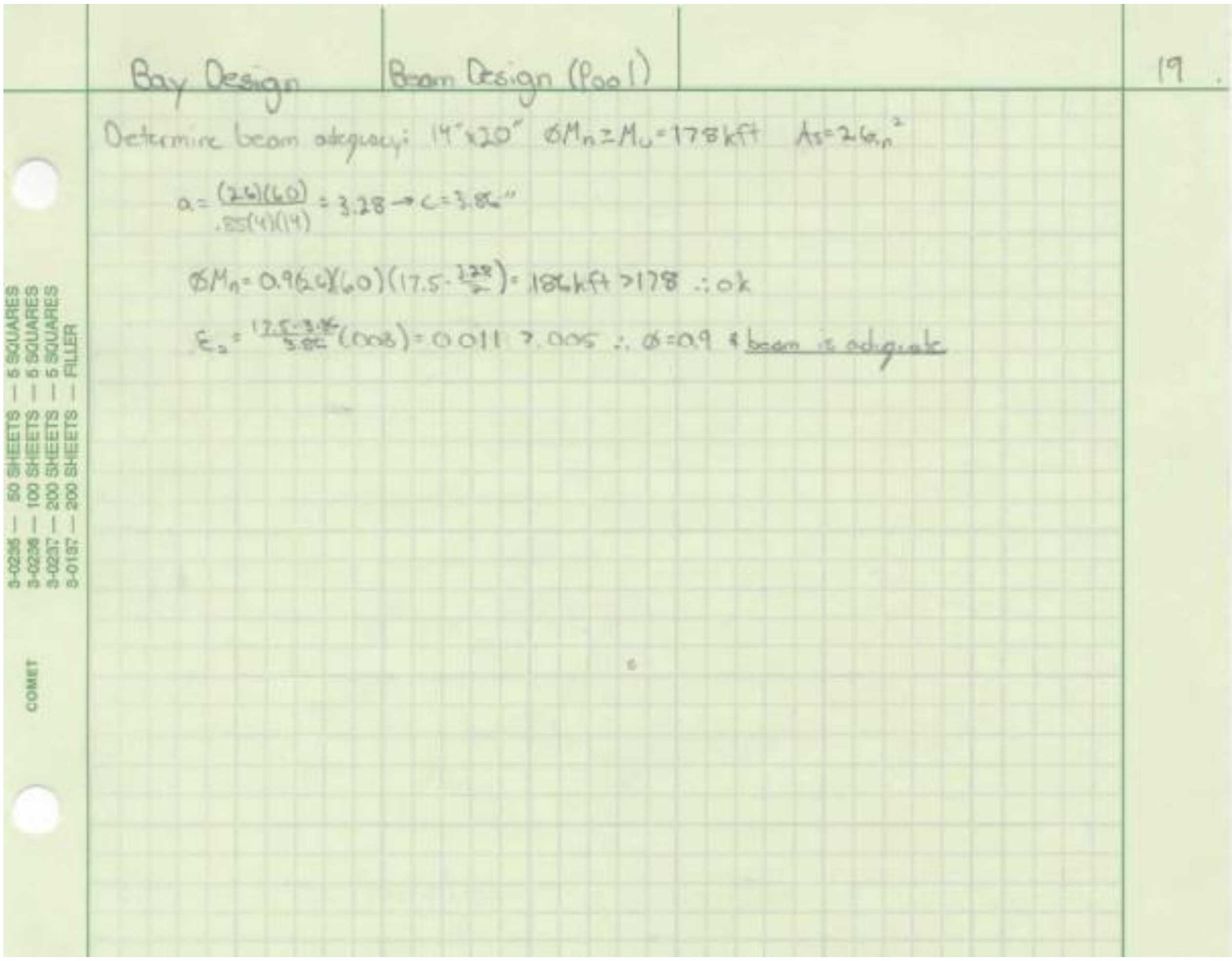
shear: crit length = $\frac{22.5}{2} - \frac{17.5}{12} = 9.8' \to V_u@d = (3.88)(9.8) = 38$ k

$\phi V_c = 0.75(2)\sqrt{4000}(14)(17.5) = 23.2k \to \frac{1}{2}\phi V_c = 11.6k$

$s_{max} = 17.5/2 \to$ use $s = 8''$ $A_{v,min} = \frac{50(14)(8)}{60000} = 0.09\,in^2 \to$ select #3@8"

$\phi V_s = \frac{0.75(.22)(60)(17.5)}{8} = 21.7k \to \phi V_n = 21.7 + 23.2 = 45k > 38$ ∴ ok ✓

$\frac{38-11.6}{3.88} = 6.8' \to \frac{6.8(12)}{8} = 10$ provide #3 (1)@2", (10)@8"



Bay Design | Beam Design (Pool) | 19

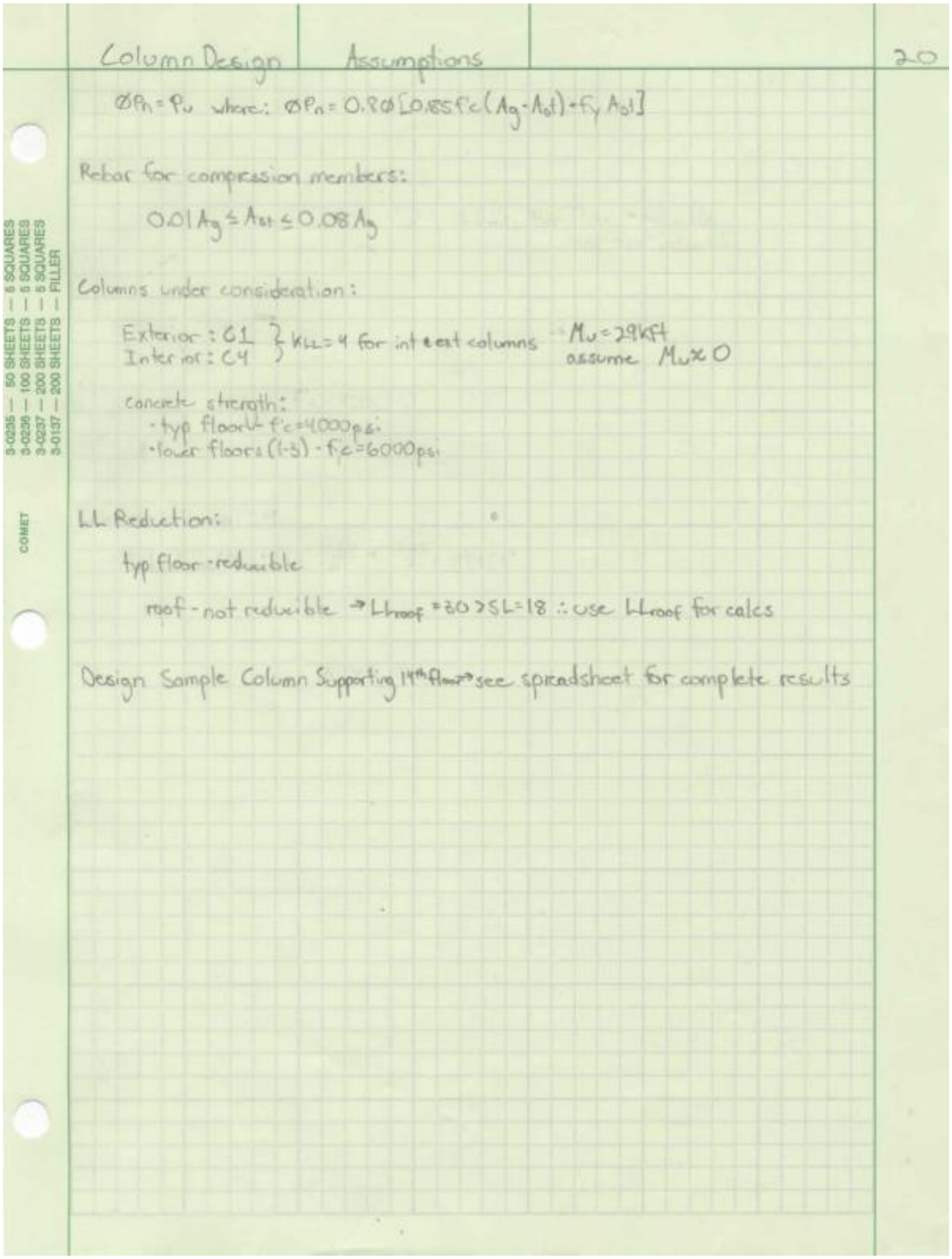
Determine beam adequacy: 14"x20" $\phi M_n \geq M_u = 178\text{kft}$ $A_s = 2.6\text{in}^2$

$$a = \frac{(2.6)(60)}{.85(4)(14)} = 3.28 \rightarrow c = 3.86''$$

$$\phi M_n = 0.9(2.6)(60)(17.5 - \tfrac{3.28}{2}) = 186\text{kft} > 178 \therefore ok$$

$$\varepsilon_s = \frac{17.5 - 3.86}{3.86}(.003) = 0.011 > .005 \therefore \phi = 0.9$$ & beam is adequate

Column Design



Column Design | Assumptions | 2.0

$\phi P_n = P_u$ where: $\phi P_n = 0.80\phi[0.85f'c(A_g - A_{st}) + f_y A_{st}]$

Rebar for compression members:

$0.01A_g \le A_{st} \le 0.08A_g$

Columns under consideration:

Exterior: C1 } $K_{LL} = 4$ for int & ext columns $M_u = 29$kft
Interior: C4 } assume $M_u \approx 0$

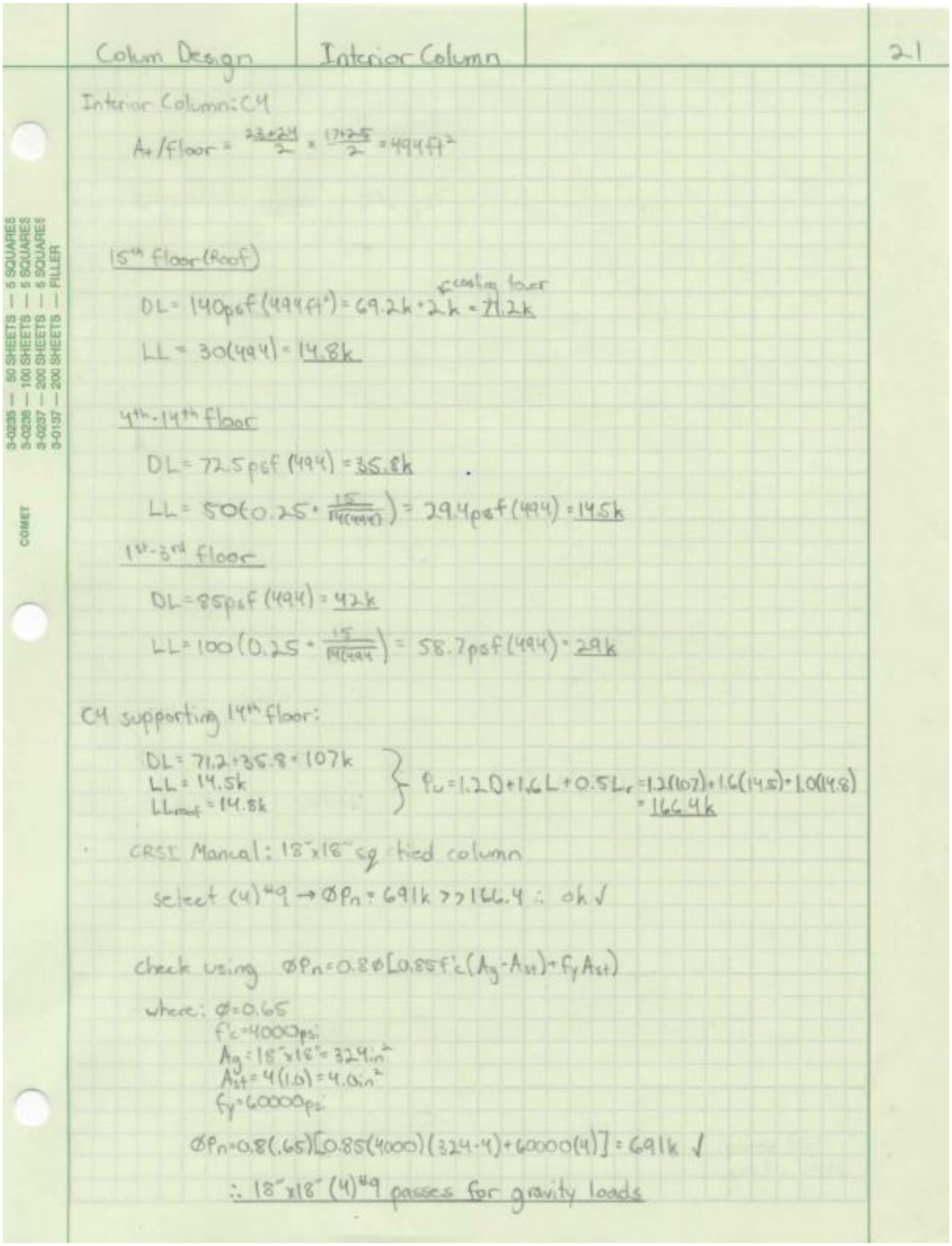
concrete strength:
- typ floor - f'c=4000psi
- lower floors (1-3) - f'c=6000psi

LL Reduction:

typ floor - reducible

roof - not reducible → $LL_{roof} = 30 > SL = 18$ ∴ use LL_{roof} for calcs

Design Sample Column Supporting 14th floor → see spreadsheet for complete results



Colum Design | Interior Column | 21

Interior Column: C4

$A_t/\text{floor} = \frac{23+24}{2} \times \frac{17+25}{2} = 494\text{ft}^2$

15th floor (Roof)

DL = 140psf (494ft²) = 69.2k + 2k = 71.2k (2k: cooling tower)

LL = 30(494) = 14.8k

4th-14th floor

DL = 72.5psf (494) = 35.8k

$LL = 50\left(0.25 + \frac{15}{\sqrt{4(494)}}\right) = 29.4\text{psf}(494) = 14.5k$

1st-3rd floor

DL = 85psf (494) = 42k

$LL = 100\left(0.25 + \frac{15}{\sqrt{4(494)}}\right) = 58.7\text{psf}(494) = 29k$

C4 supporting 14th floor:

$$\left.\begin{array}{l} DL = 71.2 + 35.8 = 107k \\ LL = 14.5k \\ LL_{roof} = 14.8k \end{array}\right\} P_u = 1.2D + 1.6L + 0.5L_r = 1.2(107) + 1.6(14.5) + 1.0(14.8) = \underline{166.4k}$$

CRSI Manual: 18"x18" sq tied column

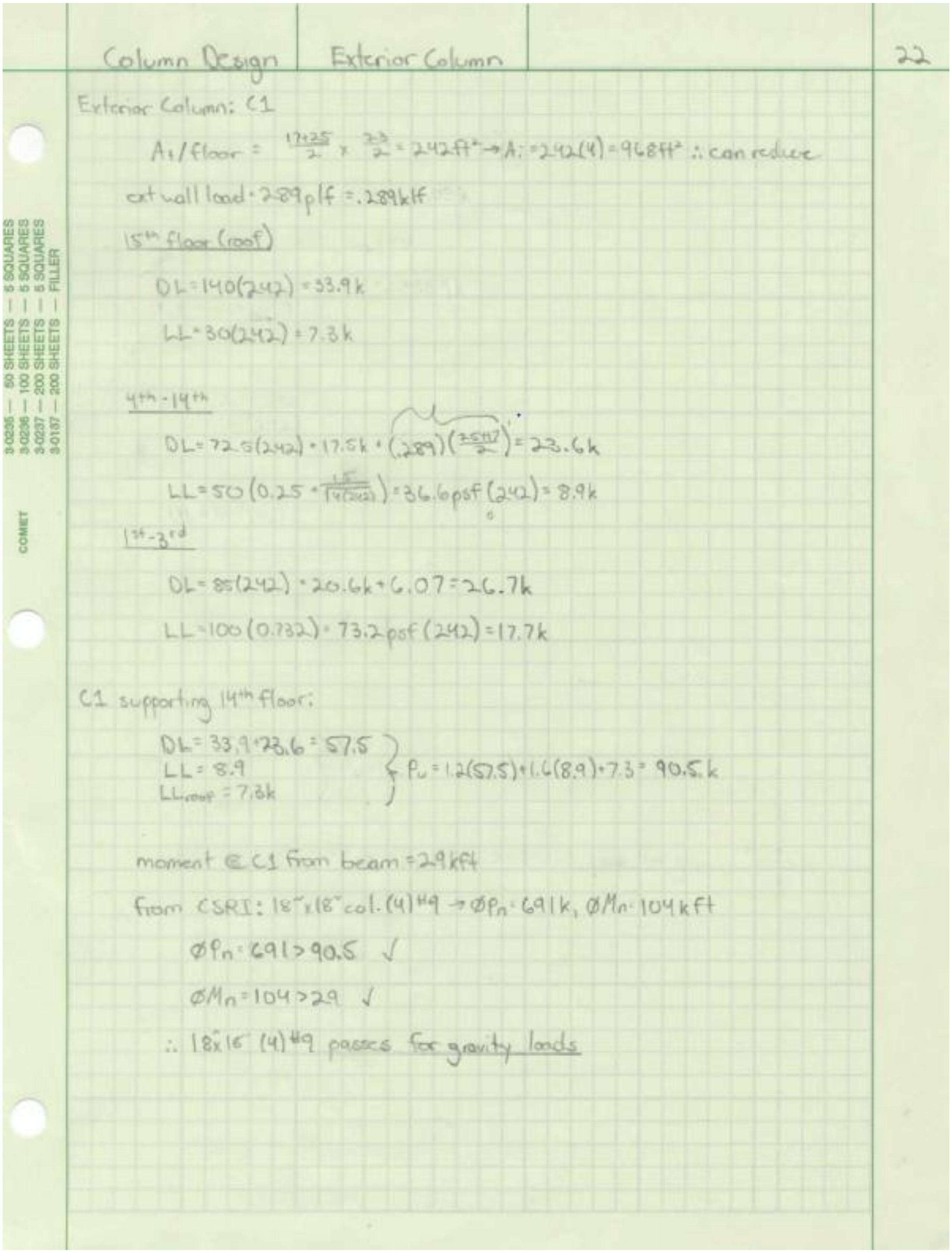
select (4)#9 → ØPn = 691k >> 166.4 ∴ ok ✓

check using $\phi P_n = 0.8\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st})$

where: $\phi = 0.65$
$f'_c = 4000\text{psi}$
$A_g = 18" \times 18" = 324\text{in}^2$
$A_{st} = 4(1.0) = 4.0\text{in}^2$
$f_y = 60000\text{psi}$

$\phi P_n = 0.8(.65)[0.85(4000)(324-4) + 60000(4)] = 691k$ ✓

∴ 18"x18" (4)#9 passes for gravity loads



Column Design | Exterior Column | 22

Exterior Column: C1

$A_t/\text{floor} = \frac{17+25}{2} \times \frac{23}{2} = 242\text{ft}^2 \rightarrow A_I = 242(4) = 968\text{ft}^2$ ∴ can reduce

ext wall load = 289plf = .289klf

<u>15th floor (roof)</u>

DL = 140(242) = 33.9k

LL = 30(242) = 7.3k

<u>4th - 14th</u>

$DL = 72.5(242) = 17.5k + \overbrace{(.289)\left(\frac{25+17}{2}\right)} = 23.6k$

$LL = 50\left(0.25 + \frac{15}{\sqrt{4(242)}}\right) = 36.6\text{psf}(242) = 8.9k$

<u>1st - 3rd</u>

DL = 85(242) = 20.6k + 6.07 = 26.7k

LL = 100(0.732) = 73.2psf(242) = 17.7k

C1 supporting 14th floor:

$$\left.\begin{array}{l} DL = 33.9 + 23.6 = 57.5 \\ LL = 8.9 \\ LL_{roof} = 7.3k \end{array}\right\} P_u = 1.2(57.5) + 1.6(8.9) + 7.3 = 90.5k$$

moment @ C1 from beam = 29kft

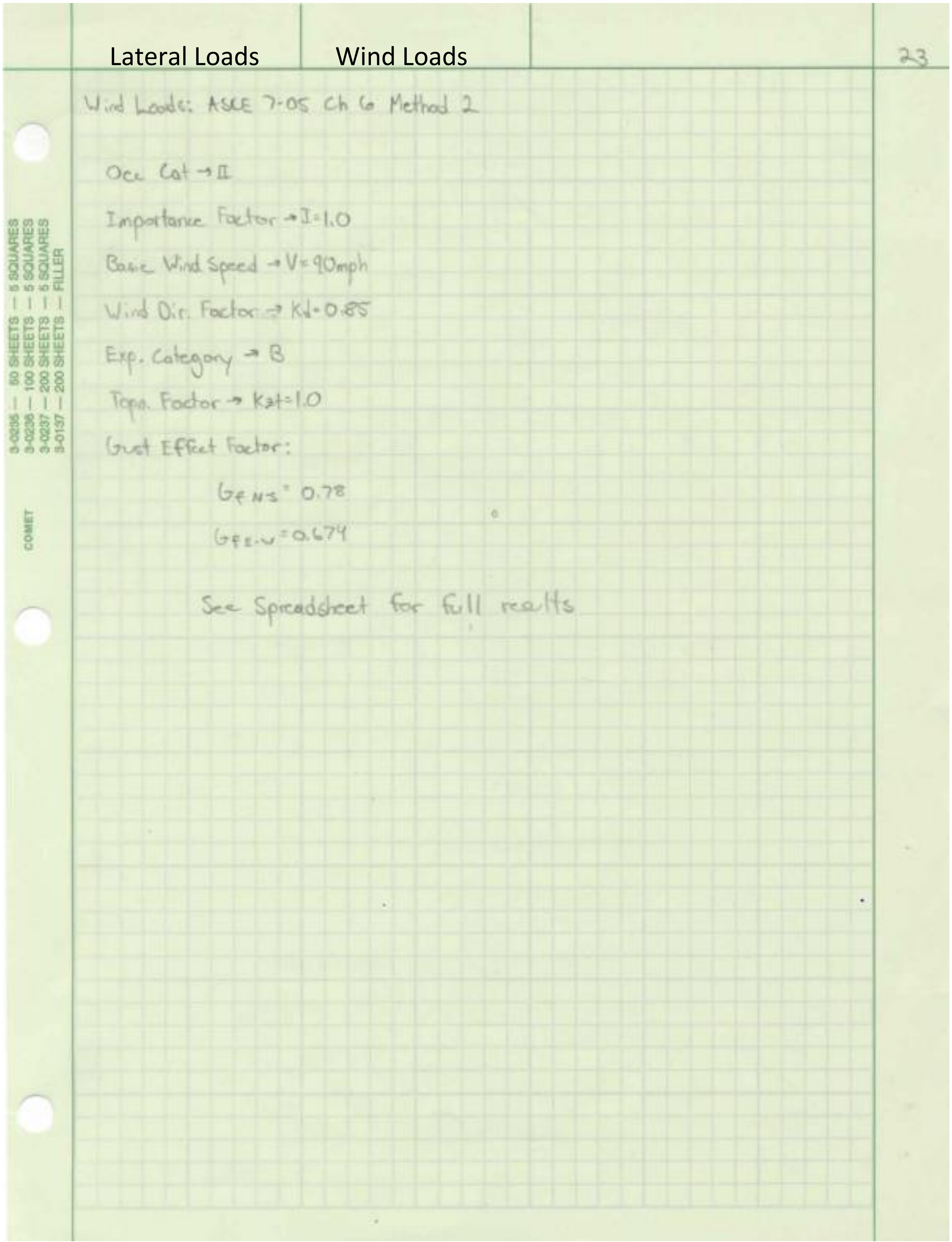
from CSRI: 18"x18" col. (4)#9 → $\phi P_n = 691k$, $\phi M_n = 104$kft

$\phi P_n = 691 > 90.5$ ✓

$\phi M_n = 104 > 29$ ✓

∴ 18"x18" (4)#9 passes <u>for gravity loads</u>

Wind Information



Lateral Loads

Wind Loads

23

Wind Loads: ASCE 7-05 Ch 6 Method 2

Occ Cat → II

Importance Factor → $I=1.0$

Basic Wind Speed → $V=90$mph

Wind Dir. Factor → $K_d=0.85$

Exp. Category → B

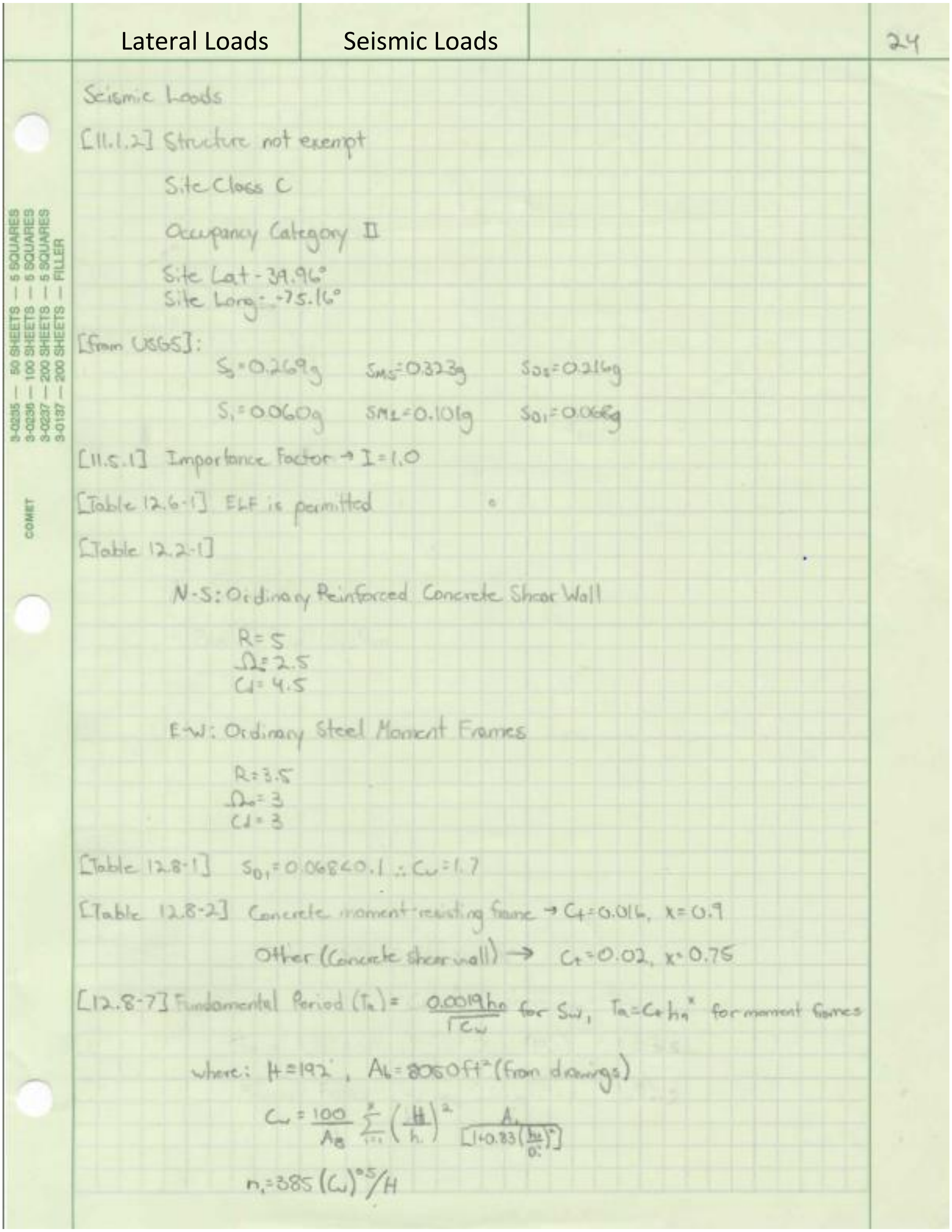
Topo. Factor → $K_{zt}=1.0$

Gust Effect Factor:

$G_{f\,N\text{-}S} = 0.78$

$G_{f\,E\text{-}W} = 0.674$

See Spreadsheet for full realts



Lateral Loads | Seismic Loads | 24

Seismic Loads

[11.1.2] Structure not exempt

Site Class C

Occupancy Category II

Site Lat - 39.96°
Site Long: -75.16°

[from USGS]:

$S_S = 0.269g$ $S_{MS} = 0.323g$ $S_{DS} = 0.216g$

$S_1 = 0.060g$ $S_{M1} = 0.101g$ $S_{D1} = 0.068g$

[11.5.1] Importance Factor → $I = 1.0$

[Table 12.6-1] ELF is permitted

[Table 12.2-1]

N-S: Ordinary Reinforced Concrete Shear Wall

$R = 5$
$\Omega = 2.5$
$C_d = 4.5$

E-W: Ordinary Steel Moment Frames

$R = 3.5$
$\Omega_o = 3$
$C_d = 3$

[Table 12.8-1] $S_{D1} = 0.068 < 0.1 \therefore C_u = 1.7$

[Table 12.8-2] Concrete moment-resisting frame → $C_t = 0.016$, $x = 0.9$

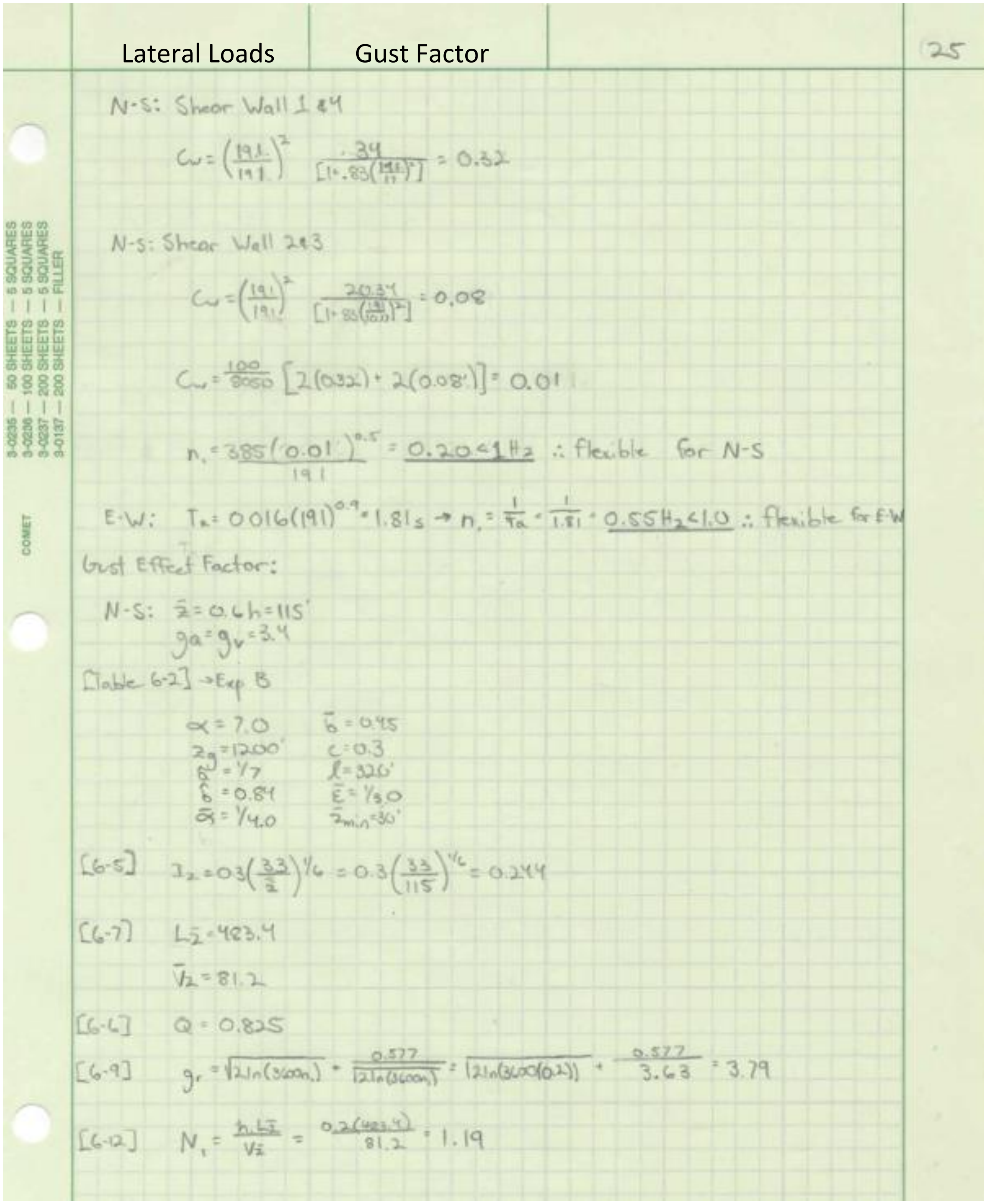
Other (Concrete shear wall) → $C_t = 0.02$, $x = 0.75$

[12.8-7] Fundamental Period $(T_a) = \frac{0.0019 h_n}{\sqrt{C_w}}$ for Sw, $T_a = C_t h_n^x$ for moment frames

where: $H = 192'$, $A_B = 8060 ft^2$ (from drawings)

$$C_w = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{H}{h_i}\right)^2 \frac{A_i}{\left[1+0.83\left(\frac{h_i}{D_i}\right)^2\right]}$$

$$n_1 = 385 (C_w)^{0.5}/H$$



Lateral Loads Gust Factor 25

N-S: Shear Wall 1 & 4

$$C_w = \left(\frac{191}{191}\right)^2 \frac{.34}{\left[1+.83\left(\frac{191}{17}\right)^2\right]} = 0.32$$

N-S: Shear Wall 2 & 3

$$C_w = \left(\frac{191}{191}\right)^2 \frac{20.34}{\left[1+.83\left(\frac{191}{100}\right)^2\right]} = 0.08$$

$$C_w = \frac{100}{8050}\left[2(0.32)+2(0.08)\right] = 0.01$$

$$n_1 = \frac{385(0.01)^{0.5}}{191} = \underline{0.20<1 Hz}$$ ∴ flexible for N-S

E-W: $T_a = 0.016(191)^{0.9} = 1.81s \rightarrow n_1 = \frac{1}{T_a} = \frac{1}{1.81} = \underline{0.55 Hz<1.0}$ ∴ flexible for E-W

Gust Effect Factor:

N-S: $\bar{z} = 0.6h = 115'$
$g_Q = g_v = 3.4$

[Table 6-2] → Exp B

$\alpha = 7.0$
$z_g = 1200'$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$

$\bar{b} = 0.45$
$c = 0.3$
$\ell = 320'$
$\bar{\epsilon} = 1/3.0$
$\bar{z}_{min} = 30'$

[6-5] $I_{\bar{z}} = 0.3\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{115}\right)^{1/6} = 0.244$

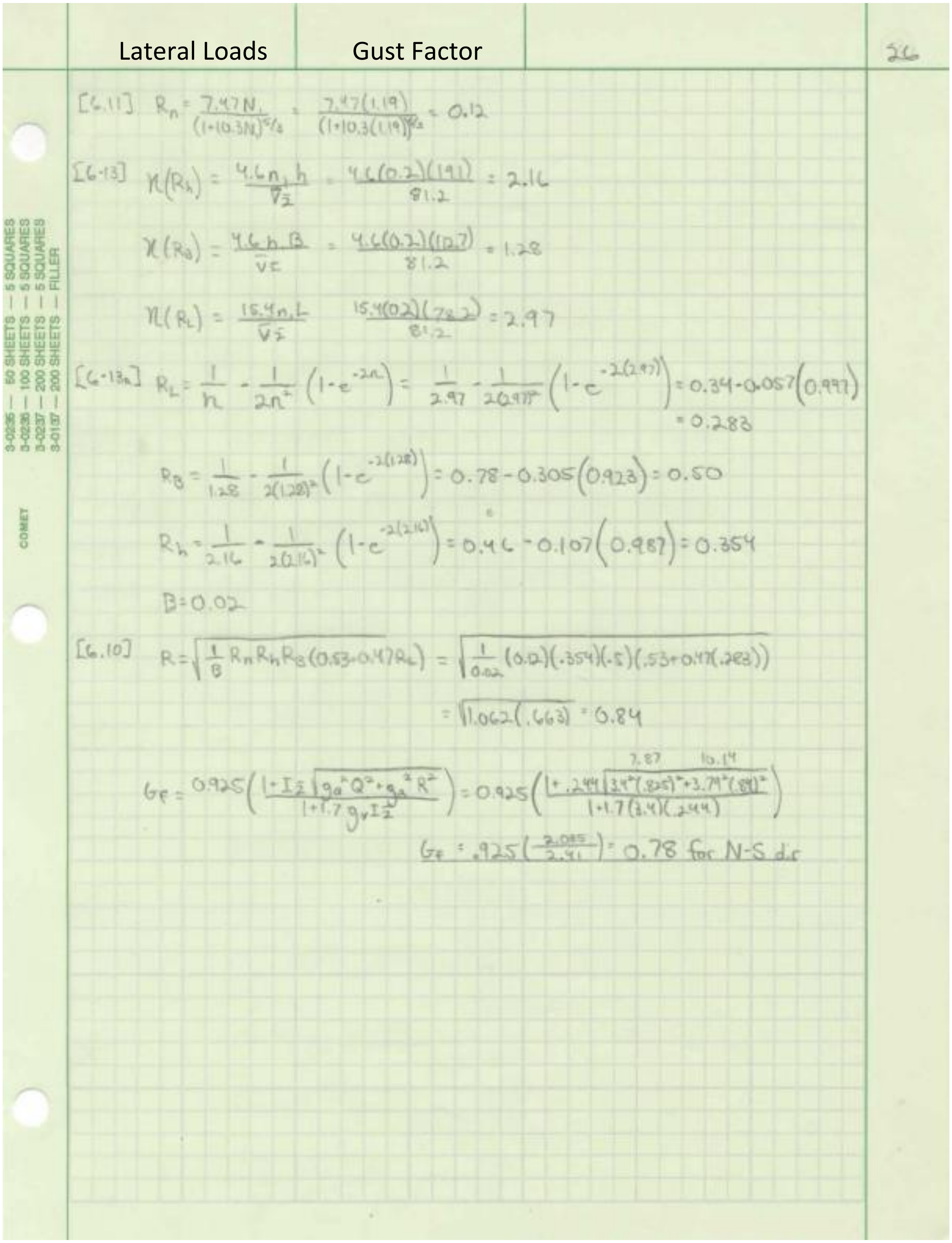
[6-7] $L_{\bar{z}} = 423.4$

$\bar{V}_{\bar{z}} = 81.2$

[6-6] $Q = 0.825$

[6-9] $g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} = \sqrt{2\ln(3600(0.2))} + \frac{0.577}{3.63} = 3.79$

[6-12] $N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.2(423.4)}{81.2} = 1.19$



Lateral Loads | Gust Factor | 26

[6.11] $R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(1.19)}{(1+10.3(1.19))^{5/3}} = 0.12$

[6-13] $\eta(R_h) = \frac{4.6n_1h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.2)(191)}{81.2} = 2.16$

$\eta(R_B) = \frac{4.6n_1B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.2)(112.7)}{81.2} = 1.28$

$\eta(R_L) = \frac{15.4n_1L}{\bar{V}_{\bar{z}}} \quad \frac{15.4(0.2)(78.2)}{81.2} = 2.97$

[6-13a] $R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) = \frac{1}{2.97} - \frac{1}{2(2.97)^2}\left(1-e^{-2(2.97)}\right) = 0.34 - 0.057(0.997)$
$= 0.283$

$R_B = \frac{1}{1.28} - \frac{1}{2(1.28)^2}\left(1-e^{-2(1.28)}\right) = 0.78 - 0.305(0.923) = 0.50$

$R_h = \frac{1}{2.16} - \frac{1}{2(2.16)^2}\left(1-e^{-2(2.16)}\right) = 0.46 - 0.107(0.987) = 0.354$

$B = 0.02$

[6.10] $R = \sqrt{\frac{1}{B}R_nR_hR_B(0.53+0.47R_L)} = \sqrt{\frac{1}{0.02}(0.12)(.354)(.5)(.53+0.47(.283))}$

$= \sqrt{1.062(.663)} = 0.84$

$G_f = 0.925\left(\frac{1+I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_vI_{\bar{z}}}\right) = 0.925\left(\frac{1+.244\sqrt{\overset{7.87}{3.4^2(.825)^2}+\overset{10.14}{3.79^2(.84)^2}}}{1+1.7(3.4)(.244)}\right)$

$\underline{G_f = .925\left(\frac{2.085}{2.41}\right) = 0.78}$ for N-S dir

Lateral Loads Gust Factor

E-W:

$Q = 0.835$

$$g_r = \sqrt{2\ln(3600(0.55))} + \frac{0.577}{\sqrt{2\ln(3600(.55))}} = 3.90 + .148 = 4.05$$

$$\eta(R_h) = \frac{4.6(.55)(191)}{81.2} = 5.95$$

$$N_1 = \frac{.55(483.4)}{81.2} = 3.27$$

$$R_n = \frac{7.47(3.27)}{(1+10.3(3.27))^{5/3}} = 0.066$$

$$\eta(R_B) = \frac{4.6(.55)(78.2)}{81.2} = 2.44$$

$$\eta(R_L) = \frac{15.4(.55)(112.7)}{81.2} = 11.76$$

$$R_L = \frac{1}{11.76} - \frac{1}{2(11.76)^2}\left(1-e^{-2(11.76)}\right) = 0.09 - 0.004\left(1\right) = 0.086$$

$$R_B = \frac{1}{2.44} - \frac{1}{2(2.44)^2}\left(1-e^{-2(2.44)}\right) = 0.41 - 0.084\left(.992\right) = 0.327$$

$$R_h = \frac{1}{5.95} - \frac{1}{2(5.95)^2}\left(1-e^{-2(5.95)}\right) = .168 - 0.014\left(.99\right) = 0.154$$

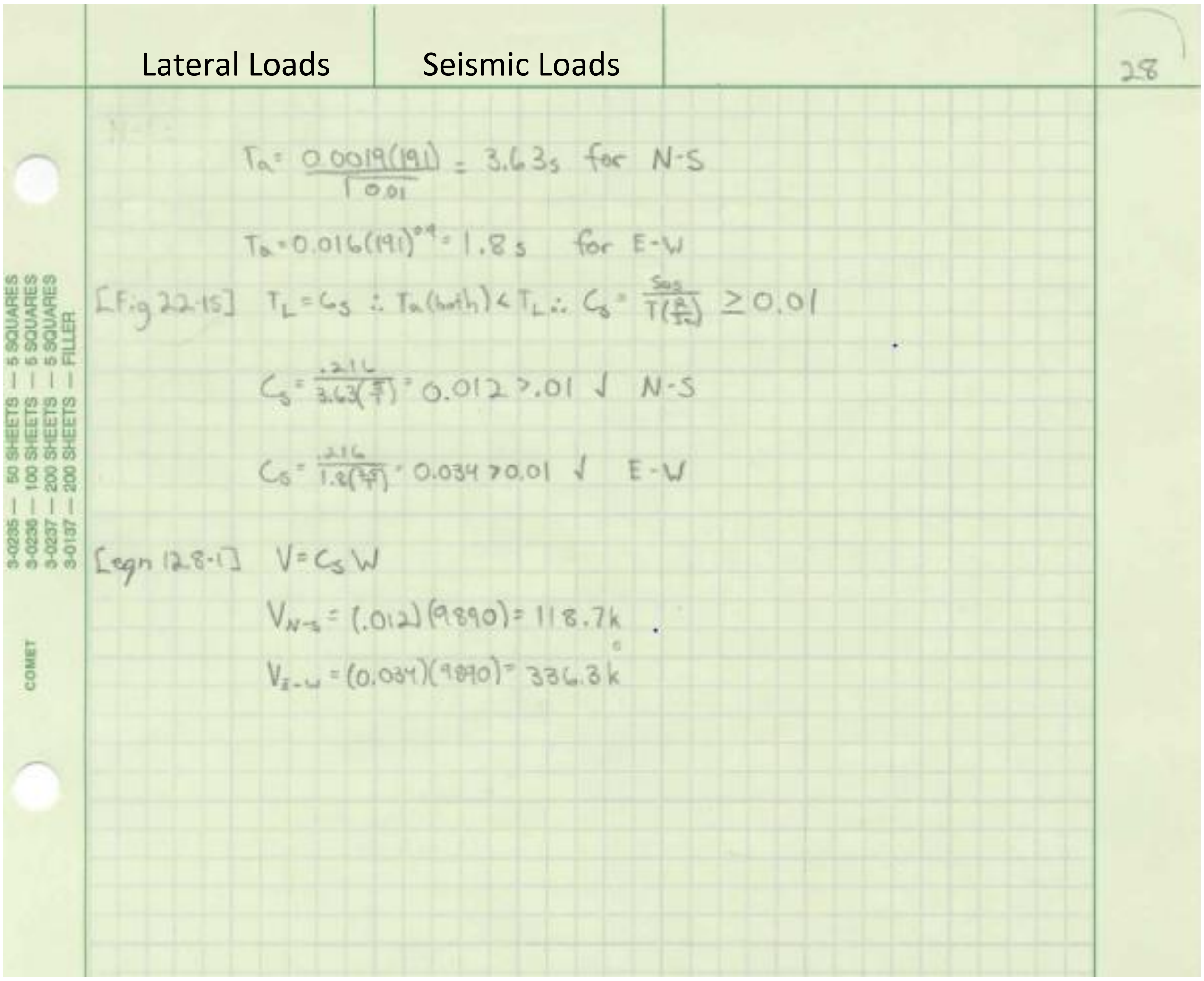
$$R = \sqrt{\frac{1}{0.02}(0.066)(.154)(.327)(.53+.47(.086))} = \sqrt{.166(.57)} = 0.308$$

$$G_f = 0.925\left(\frac{1+.244\sqrt{\overset{8.06}{3.4^2(.835)^2} + 4.05^2(.308)^2}}{1+1.7(3.4)(.244)}\right) = .925\left(\frac{1.757}{2.41}\right) = \underline{0.674}$$

$\underline{G_{f\,N\text{-}S} = 0.78}$

$\underline{G_{f\,E\text{-}W} = 0.674}$

Seismic Information



Lateral Loads Seismic Loads 28

$T_a = \frac{0.0019(191)}{\sqrt{0.01}} = 3.63s$ for N-S

$T_a = 0.016(191)^{0.9} = 1.8s$ for E-W

[Fig 22-15] $T_L = 6s \therefore T_a(\text{both}) < T_L \therefore C_s = \frac{S_{DS}}{T(\frac{R}{I_e})} \geq 0.01$

$C_s = \frac{.216}{3.63(\frac{5}{1})} = 0.012 > .01$ ✓ N-S

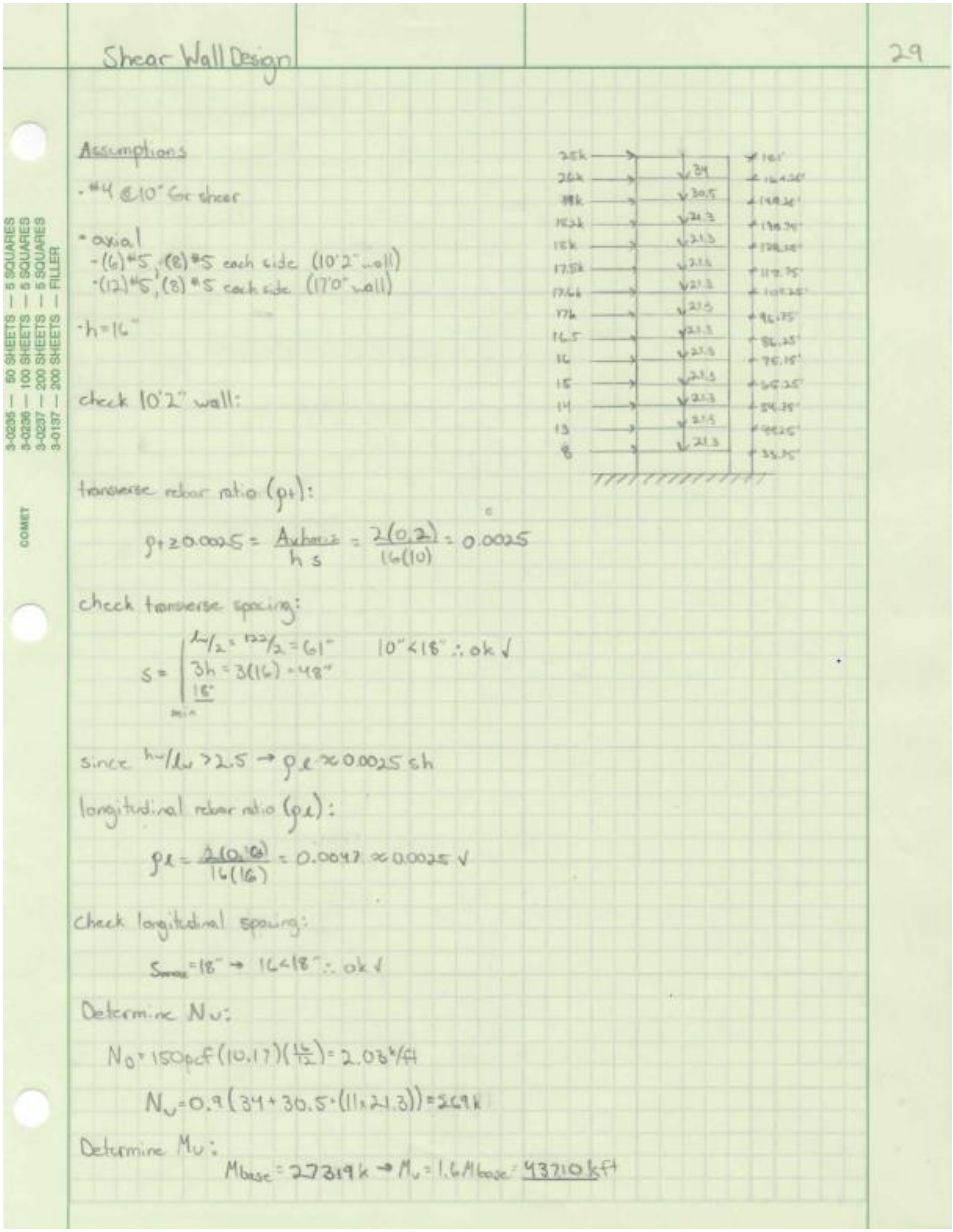
$C_s = \frac{.216}{1.8(\frac{3.5}{1})} = 0.034 > 0.01$ ✓ E-W

[eqn 12.8-1] $V = C_s W$

$V_{N-S} = (.012)(9890) = 118.7k$

$V_{E-W} = (0.034)(9890) = 336.3k$

Shear Wall Design



Shear Wall Design

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Assumptions

- #4 @10" for shear
- axial
  - (6)#5, (8)#5 each side (10'2" wall)
  - (12)#5, (8)#5 each side (17'0" wall)
- h=16"

check 10'2" wall:

transverse rebar ratio (ρ_t):

$$\rho_t \geq 0.0025 = \frac{A_{v,horiz}}{h\,s} = \frac{2(0.2)}{16(10)} = 0.0025$$

check transverse spacing:

$$s = \left| \begin{array}{l} l_w/2 = 122/2 = 61" \\ 3h = 3(16) = 48" \\ \underline{18"} \end{array} \right._{min} \qquad 10" < 18" \therefore ok \checkmark$$

since $h_w/l_w > 2.5 \rightarrow \rho_l \approx 0.0025\,sh$

longitudinal rebar ratio (ρ_l):

$$\rho_l = \frac{2(0.31)}{16(16)} = 0.0047 \approx 0.0025 \checkmark$$

Check longitudinal spacing:

$$S_{max} = 18" \rightarrow 16 < 18" \therefore ok \checkmark$$

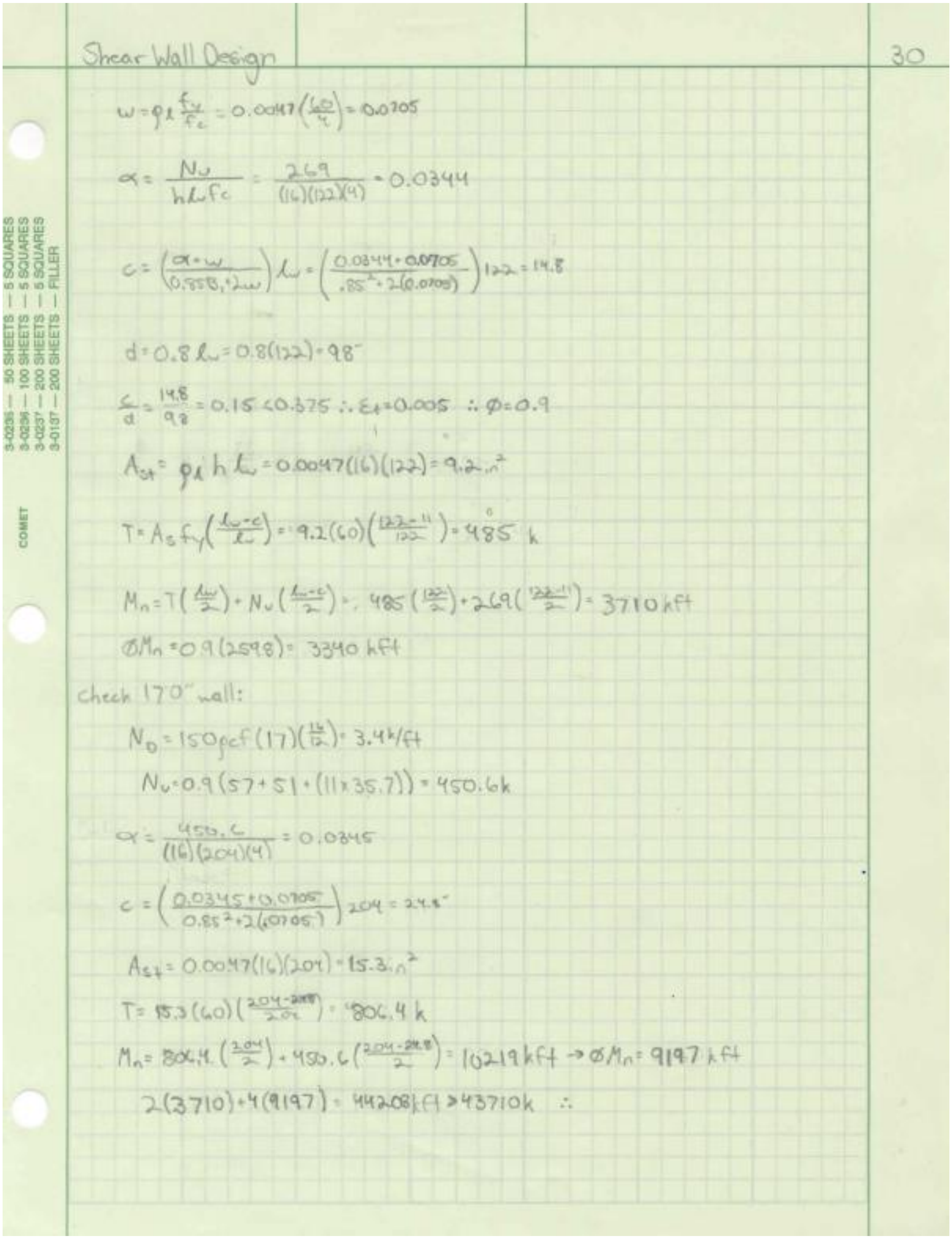
Determine N_u:

$$N_0 = 150pcf(10.17)\left(\tfrac{16}{12}\right) = 2.03^k/ft$$

$$N_u = 0.9(34 + 30.5 + (11 \times 21.3)) = 269k$$

Determine M_u:

$$M_{base} = 27319k \rightarrow M_u = 1.6M_{base} = \underline{43710\,kft}$$



Shear Wall Design

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$\omega = \rho_\ell \frac{f_y}{f_c} = 0.0047\left(\frac{60}{4}\right) = 0.0705$

$\alpha = \frac{N_u}{h \ell_w f_c} = \frac{269}{(16)(122)(4)} = 0.0344$

$c = \left(\frac{\alpha + \omega}{0.85\beta_1 + 2\omega}\right)\ell_w = \left(\frac{0.0344 + 0.0705}{.85^2 + 2(0.0705)}\right)122 = 14.8$

$d = 0.8\ell_w = 0.8(122) = 98"$

$\frac{c}{d} = \frac{14.8}{98} = 0.15 < 0.375 \therefore \varepsilon_t = 0.005 \therefore \phi = 0.9$

$A_{st} = \rho_\ell h \ell_w = 0.0047(16)(122) = 9.2 in^2$

$T = A_s f_y\left(\frac{\ell_w - c}{\ell_w}\right) = 9.2(60)\left(\frac{122-11}{122}\right) = 485 k$

$M_n = T\left(\frac{\ell_w}{2}\right) + N_u\left(\frac{\ell_w - c}{2}\right) = 485\left(\frac{122}{2}\right) + 269\left(\frac{122-11}{2}\right) = 3710 kft$

$\phi M_n = 0.9(2598) = 3340 kft$

check 170" wall:

$N_D = 150pcf(17)\left(\frac{16}{12}\right) = 3.4^k/ft$

$N_u = 0.9(57 + 51 + (11 \times 35.7)) = 450.6k$

$\alpha = \frac{450.6}{(16)(204)(4)} = 0.0345$

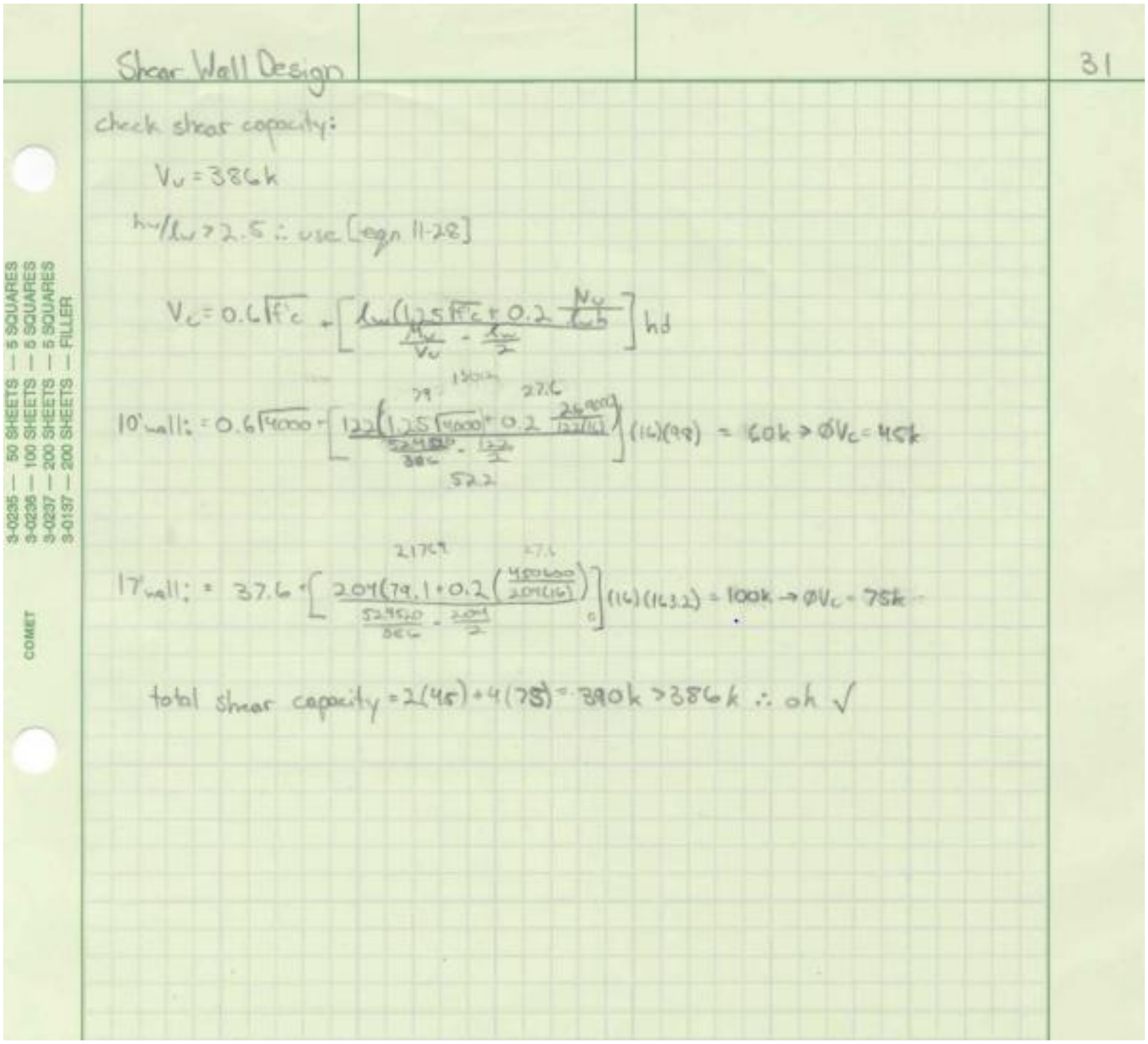
$c = \left(\frac{0.0345 + 0.0705}{0.85^2 + 2(0.0705)}\right)204 = 24.8"$

$A_{st} = 0.0047(16)(204) = 15.3 in^2$

$T = 15.3(60)\left(\frac{204-24.8}{204}\right) = 806.4 k$

$M_n = 806.4\left(\frac{204}{2}\right) + 450.6\left(\frac{204-24.8}{2}\right) = 10219 kft \rightarrow \phi M_n = 9197 kft$

$2(3710) + 4(9197) = 44208 kft > 43710k \therefore$



Shear Wall Design

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check shear capacity:

$V_u = 386k$

$h_w / l_w > 2.5 \therefore$ use [eqn 11-28]

$$V_c = 0.6\sqrt{f'_c} + \left[\frac{l_w\left(1.25\sqrt{f'_c} + 0.2\frac{N_u}{l_w h}\right)}{\frac{M_u}{V_u} - \frac{l_w}{2}}\right]hd$$

$$10' \text{wall}: = 0.6\sqrt{4000} + \overset{15014}{\left[\frac{122\left(\overset{79}{1.25\sqrt{4000}} + 0.2\overset{27.6}{\frac{26900}{122(16)}}\right)}{\underset{522}{\frac{524810}{386} - \frac{122}{2}}}\right]}(16)(98) = 60k > \phi V_c = 45k$$

$$17' \text{wall}: = 37.6 + \left[\frac{\overset{21769}{204\left(79.1 + 0.2\overset{27.6}{\left(\frac{450600}{204(16)}\right)}\right)}}{\frac{529520}{386} - \frac{204}{2}}\right](16)(163.2) = 100k \rightarrow \phi V_c = 75k$$

total shear capacity $= 2(45) + 4(75) = 390k > 386k \therefore$ ok ✓

Appendix C (Construction Management Supplementary Info)

Proposed Design Detailed Estimate

Project Name:	AC Hotel Philadelphia	
Location:	230 N 13th St, Philadelphia, Pa	
Category	**Line Number**	**Description**
Forms	03 11 13.20.1150	Forms in Place, Exterior Beam, Job-Built Plywood, 18" Wide, 4 Use
	03 11 13.20.2150	Forms in Place, Interior Beam, Job-Built Plywood, 12" Wide, 4 Use
	03 11 13.20.9000	Forms in Place, Interior Beam, Min. Labor/Equip. Charge
	03 11 13.25.6150	Forms in Place, Column (16"x16") 4 Use
	03 11 13.25.6650	Forms in Place, Column (24"x24") 4 Use
	03 11 13.25.9000	Forms in Place, Column, Min. Labor/Equip Charge
	03 11 13.35.1150	Forms in Place, Elevated Slabs, Flat Plate, Job-built Plywood, up to 15' high, 4 Use
	03 11 13.35.7000	Forms in Place, Elevated Slabs, Edge forms to 6" high, on elevated slab, 4 use
	03 11 13.85.2550	Forms in Place, Walls, Job-Built Plywood 8'-16' high, 4 Use
Rebar	03 21 11.60.0102	Plain Steel Reinforcement Bar, In Place, Beams & Girders (#3-#7)
	03 21 11.60.0152	Plain Steel Reinforcement Bar, In Place, Beams & Girders (#8-#18)
	03 21 11.60.0252	Plain Steel Reinforcement Bar, In Place, Columns (#8-#18)
	03 21 11.60.0402	Plain Steel Reinforcement Bar, In Place, Elevated Slab (#4-#7)
	03 21 11.60.0702	Plain Steel Reinforcement Bar, In Place, Walls (#3-#7)
	03 21 11.60.9000	Plain Steel Reinforcement Bar, In Place, Min. Labor/Equip. Charge
Concrete	03 31 13.35.0300	Heavyweight Concrete, Ready Mix, Delivered, 4000psi
Concrete Placement	03 31 13.70.0050	Placing Concrete, Beams, Elevated, Small Beams, Pumped
	03 31 13.70.0600	Placing Concrete, Columns, 18" Thick, Pumped
	03 31 13.70.0800	Placing Concrete, Columns, 24" Thick, Pumped
	03 31 13.70.1400	Placing Concrete, Elevated Slab, Less than 6" thick, Pumped
	30 31 13.70.5300	Placing Concrete, 15" Thick, Pumped
	30 31 13.80.9000	Placing Concrete, Min. Labor/Equip. Charge
Adjustment Factors		Adjustment for Location: Philadelphia, Pa-
		Adjustment for Time: Mid Project- 08/

RS Means Facilities Construction Cost Data 2014 Edition

Detailed Structure Including: Forms, Rebar & Placement (Bare Costs Include: Bracing & Shoring)

Crew	Daily Output	Labor Hours	Unit	Material	Labor	Equipment	Unit Total	Unit Total (O&P)	Quantity	Total
C-2	315	0.152	SFCA	0.88	6.80	0.00	7.68	12.15	31460	$241,613
C-2	377	0.127	SFCA	1.15	5.70	0.00	6.85	10.60	67718	$463,868
2 CARP	2	8	JOB	0.00	365.00	0.00	365.00	605.00	1	$365
C-1	235	0.136	SFCA	0.83	5.95	0.00	6.78	10.65	24048	$163,045
C-1	238	0.134	SFCA	0.93	5.85	0.00	6.78	10.65	5487	$37,202
2 CARP	2	8	JOB	0.00	0.00	365.00	365.00	605.00	1	$365
C-2	560	0.086	SF	1.18	3.83	0.00	5.01	7.60	167308	$838,213
C-1	500	0.064	LF	0.18	2.79	0.00	2.97	4.78	2729	$8,105
C-2	395	0.122	SFCA	0.73	5.45	0.00	6.18	9.70	22546	$139,334
4 Rodm	3200	0.01	LB	0.50	0.51	0.00	1.01	1.38	190294	$192,197
4 Rodm	5400	0.006	LB	0.50	0.30	0.00	0.80	1.04	56741	$45,393
4 Rodm	4600	0.007	LB	0.50	0.35	0.00	0.85	1.13	67380	$57,273
4 Rodm	5800	0.006	LB	0.50	0.28	0.00	0.78	1.01	55881	$43,587
4 Rodm	6000	0.005	LB	0.50	0.27	0.00	0.77	0.99	14343	$11,044
1 Rodm	4	2	JOB	0.00	101.00	0.00	101.00	166.00	1	$101
N/A	N/A	N/A	CY	104.00	N/A	N/A	104.00	114.00	2763	$287,352
C-20	60	1.067	CY	0.00	42.00	12.95	54.95	82.50	138	$7,583
C-20	90	0.711	CY	0.00	28.00	8.65	36.65	55.00	334	$12,241
C-20	92	0.696	CY	0.00	27.50	8.45	35.95	54.00	93	$3,343
C-20	140	0.457	CY	0.00	18.00	5.55	23.55	35.00	1364	$32,122
C-20	120	0.533	CY	0.00	21.00	6.45	27.45	41.00	450	$12,353
C-6	2	24	JOB	0.00	915.00	33.00	948.00	1525.00	1	$948
									Subtotal	$2,597,648
t:98.9%, Inst:133.4%, Total: 113.9%									Location	1.139
-31months=2.58yrs, 3% interest									Time	1.079
									Grand Total	**$3,193,185**

Existing Design Detailed Estimate

Project Name:	AC Hotel Philadelphia		Source:
Location:	230 N 13th St, Philadelphia, Pa		Estimate:
Category	**Line Number**	**Description**	**Notes**
Slab	03 41 13.50.0100	Precast Structural Concrete, Slab, Hollow-Core Planks, 8" Thick	
Columns	05 12 23.17.4550	Structural Steel Framing, Columns, HSS6"x6"x1/4"x12'	for HSS6x6x3/8,6x6x1/2
	05 12 23.17.4600	Structural Steel Framing, Columns, HSS8"x8"x3/8"x14'	for HSS8x8x1/2
	05 12 23.17.5700	Structural Steel Framing, Columns, HSS12"x8"x1/2"x16'	for HSS18x6x1/2,12x6x1/2
	05 12 23.17.7000	Structural Steel Framing, Columns, W10x45	for W10x33,49,54
	05 12 23.17.7050	Structural Steel Framing, Columns, W10x68	for W10x60,77
	05 12 23.17.7150	Structural Steel Framing, Columns, W12x50	for W12x40,50,58,65
	05 12 23.17.7200	Structural Steel Framing, Columns, W12x87	for W12x72,79,96
	05 12 23.17.7350	Structural Steel Framing, Columns, W14x74	for W14x43,53,61,74,90,99
	05 12 23.17.7400	Structural Steel Framing, Columns, W14x120	for W14x109,120,145
	05 12 23.17.7450	Structural Steel Framing, Columns, W14x176	for W14x176,211,257,
	05 12 23.17.8090	Structural Steel Framing, Columns, For Projects 75-99 tons add	234 tons steel
	05 12 23.17.9000	Structural Steel Framing, Columns, Min. Labor/Equip. Charge	
Framing	05 12 23.75.0502	Structural Steel Framing, Members, W8x31	DB8x37=W8x35 DB8x61=W8x58 DB8x65=W8x67 W8x13,15
	05 12 23.75.0702	Structural Steel Framing, Members, W10x22	for W10x15,22,33
	05 12 23.75.0902	Structural Steel Framing, Members, W10x49	
	05 12 23.75.1102	Structural Steel Framing, Members, W12x16	for W12x19
	05 12 23.75.1502	Structural Steel Framing, Members, W12x26	for W12x30
	05 12 23.75.1902	Structural Steel Framing, Members, W14x26	for W14x22
	05 12 23.75.2702	Structural Steel Framing, Members, W16x26	for W16x31,36,57
	05 12 23.75.2902	Structural Steel Framing, Members, W16x31	
	05 12 23.75.3102	Structural Steel Framing, Members, W16x40	for W16x36,57
	05 12 23.75.3502	Structural Steel Framing, Members, W18x40	
	05 12 23.75.3902	Structural Steel Framing, Members, W18x55	for W18x60,71
	05 12 23.75.4302	Structural Steel Framing, Members, W21x50	for W21x68
	05 12 23.75.5302	Structural Steel Framing, Members, W24x68	for W24x55
	05 12 23.75.6102	Structural Steel Framing, Members, W30x99	for W30x90
	05 12 23.75.6902	Structural Steel Framing, Members, W33x130	
	05 12 23.75.7502	Structural Steel Framing, Members, W36x150	for W36x170
	05 12 23.75.8102	Structural Steel Framing, Members, W36x302	for W36x330,360,361,652, W40x211,593
	05 12 23.75.8490	Structural Steel Framing, Members, For Projects 75-99 tons add	222 tons steel
	05 12 23.75.9000	Structural Steel Framing, Members, Min. Labor/Equip Charge	
Shear Studs	05 05 23.85.0300	Weld Shear Connectors, 3/4" dia, 4 3/16" long	for 1/2" dia, 4" long
	05 05 23.85.9000	Weld Shear Connectors, Min. Labor/Equip. Charge	
Crane	01 54 33.5000	Construction Aids, Equip. Rental, Hoist & Tower, Personnel, Electric, 6000lb, 100' @275 fpm	rent (2) cranes, rent each for 4 months
Adjustment Factors	Adjustment for Location: Philadelphia, Pa- State/Zip: 19		
	Adjustment for Time: Mid Project- 08/16, Time From		

RS Means Facilities Construction Cost Data 2014 Edition

Detailed Structure Including: Slab, Structural Steel & Shear studs (Bare Costs Include: Structural Bolts, Delivery, Installation/Erection)

Crew	Daily Output	Labor Hours	Unit	Material	Material Incr. From Tonnage	Labor	Equipment	Unit Total	Unit Total (O&P)	Quantity	Total
C-11	3200	0.023	SF	7.10	N/A	1.13	0.57	8.80	10.45	87098	$766,462
E-2	54	1.037	EA	360.00	396.00	52.00	28.50	476.50	520.00	278	$132,467
E-2	50	1.12	EA	775.00	852.50	56.00	30.50	939.00	990.00	34	$31,926
E-2	48	1.167	EA	1425.00	1567.50	58.50	32.00	1658.00	1725.00	5	$8,290
E-2	1032	0.054	LF	65.50	72.05	2.72	1.48	76.25	78.50	1190	$90,738
E-2	984	0.057	LF	99.00	108.90	2.86	1.55	113.31	116.00	411	$46,514
E-2	1032	0.054	LF	73.00	80.30	2.72	1.48	84.50	86.50	378	$31,912
E-2	984	0.057	LF	127.00	139.70	2.86	1.55	144.11	146.00	131	$18,806
E-2	984	0.057	LF	108.00	118.80	2.86	1.55	123.21	126.00	628	$77,376
E-2	960	0.058	LF	175.00	192.50	2.93	1.59	197.02	199.00	1093	$215,343
E-2	912	0.061	LF	257.00	282.70	3.08	1.68	287.46	289.00	1135	$326,267
N/A	N/A	N/A	ALL	10%	N/A	N/A	N/A	N/A	N/A	N/A	N/A
1 SSwk	1	8	JOB	0.00	0.00	410.00	0.00	410.00	750.00	1	$410
E-2	550	0.102	LF	45.00	49.50	5.10	2.78	57.38	61.50	2205	$126,523
E-2	600	0.093	LF	32.00	35.20	4.68	2.55	42.43	46.50	145	$6,152
E-2	550	0.102	LF	71.50	78.65	5.10	2.78	86.53	90.50	0	$0
E-2	880	0.064	LF	23.50	25.85	3.19	1.74	30.78	33.00	2357	$72,548
E-2	880	0.064	LF	38.00	41.80	3.19	1.74	46.73	49.00	104	$4,860
E-2	990	0.057	LF	38.00	41.80	2.84	1.54	46.18	48.00	1797	$82,985
E-2	1000	0.056	LF	38.00	41.80	2.81	1.53	46.14	48.00	566	$26,115
E-2	900	0.062	LF	45.00	49.50	3.12	1.70	54.32	57.00	80	$4,346
E-2	800	0.07	LF	58.50	64.35	3.51	1.91	69.77	72.50	1597	$111,423
E-5	960	0.083	LF	58.50	64.35	4.22	1.74	70.31	73.50	598	$42,045
E-5	912	0.088	LF	80.00	88.00	4.44	1.83	94.27	98.00	163	$15,366
E-5	1064	0.075	LF	73.00	80.30	3.81	1.57	85.68	88.50	533	$45,667
E-5	1110	0.072	LF	99.00	108.90	3.65	1.51	114.06	117.00	466	$53,152
E-5	1200	0.067	LF	144.00	158.40	3.38	1.39	163.17	167.00	139	$22,681
E-5	1134	0.071	LF	189.00	207.90	3.57	1.47	212.94	216.00	47	$10,008
E-5	1170	0.068	LF	219.00	240.90	3.46	1.43	245.79	248.00	51	$12,535
E-5	1035	0.077	LF	440.00	484.00	3.92	1.61	489.53	495.00	169	$82,731
N/A	N/A	N/A	ALL	10%	N/A	N/A	N/A	N/A	N/A	N/A	N/A
E-2	2	28	JOB	0.00	0.00	1400.00	765.00	2165.00	3325.00	1	$2,165
E-10	935	0.017	EA	0.63	0.00	0.89	0.51	2.03	2.83	5429	$11,021
1 SSwk	2	4	JOB	0.00	0.00	204.00	0.00	204.00	375.00	1	$204
			EA					11900.00	11900.00	8	$95,200
										Subtotal	$2,574,239
0-191, Mat:98.9%, Inst:133.4%, Total: 113.9%										Location	1.139
n Jan 2014-31months=2.58yrs, 3% interest										Time	1.079
										Grand Total	**$3,164,409**

Time Factor & System Breakdown

Time Factor		
Variable	**Value**	**Unit**
Projected Project Timeline	10/2015-06/2017	
Mid Project	8/1/2016	
RS Means Data	1/1/2014	
Adjustment	31	months
	2.58	yrs
Inflation	3	%
Time Factor	1.08	

System Breakdown				
Component	**Steel**	**%**	**Concrete**	**%**
Slab	$942,182	30%	$1,133,412	35%
Framing	$886,669	28%	$1,169,050	37%
Columns	$1,204,735	38%	$336,165	11%
Miscellaneous	$130,824	4%	$554,558	17%
Total	**$3,164,409**	100%	**$3,193,185**	100%

Proposed Structure Takeoffs

Concrete Slab Takeoffs

Concrete	Area	Openings	Applicable Area	# Floors	Total Area [sf]	Thickness [ft]	Total Area [cf]	Total Area [cy]	Contact Area [sf]	Perimeter [lf]	Perimeter (Openings) [lf]	Contact Area [lf] (from per. & opens)
Roof 5" Mild Reinforced	4002	750	3252	1	3252	0.42	1355.0	50.2	6504	288	191	479
Typ Floor (4-14) 5" Mild Reinforced	5657	488	5169	11	56859	0.42	23691.3	877.5	113718	308	159	467
Floor 3 6" Mild Reinforced	8662	718	7944	1	7944	0.50	3972.0	147.1	15888	365	227	592
Floor 2 6" Mild Reinforced	8662	938	7724	1	7724	0.50	3862.0	143.0	15448	392	248	640
Floor 1 6" Mild Reinforced	8662	787	7875	1	7875	0.50	3937.5	145.8	15750	392	159	551
								1364	**167308**	N/A	N/A	**2729**

Concrete Column Takeoffs

Column Location	Dimension [in]	Area [sf]	Contact Area [sf/ft]	Height [ft]	Column Line						Quantity	Total Area [cf]
					A	B	C	D	E	F		
Interior	18	2.3	6.0	115.7	0	0	4	1	0	0	5	1301.6
	18	2.3	6.0	99.0	0	0	0	2	0	0	2	445.3
	20	2.8	6.7	21.0	0	0	4	3	0	0	7	408.3
	22	3.4	7.3	28.6	0	0	4	3	0	0	7	672.9
	22	3.4	7.3	18.1	0	0	0	1	0	0	1	60.8
	24	4.0	8.0	15.7	0	0	4	4	0	0	8	502.4
Exterior	18	2.3	6.0	15.7	5	1	0	0	0	0	6	212.0
	18	2.3	6.0	33.8	0	2	1	0	2	6	11	836.6
	18	2.3	6.0	147.2	0	0	0	1	0	0	1	331.2
	18	2.3	6.0	164.3	0	0	0	1	5	0	6	2218.1
	18	2.3	6.0	181.0	0	5	1	0	0	0	6	2443.5
	18	2.3	6.0	16.8	0	0	2	2	1	0	5	188.4
Elevator	18	2.3	6.0	115.7	0	0	0	0	0	0	4	1041.3
	20	2.8	6.7	21.0	0	0	0	0	0	0	4	233.3
	22	3.4	7.3	28.6	0	0	0	0	0	0	4	384.5
	24	4.0	8.0	15.7	0	0	0	0	0	0	4	251.2

Total Area [cy]	Contact Area [sf]	#Bars/Col	# Rebar used	Weight [lb/ft]	Total Rebar [lb]
48.2	3471.0	4	9	3.4	7868
16.5	1187.4	4	9	3.4	2691
15.1	980.0	4	9	3.4	1999
24.9	1468.1	4	10	4	3446
2.3	132.7	4	10	4	312
18.6	1004.8	4	11	5	2669
7.9	565.2	4	9	3.4	1281
31.0	2230.8	4	9	3.4	5056
12.3	883.2	4	9	3.4	2002
82.2	5914.8	4	9	3.4	13407
90.5	6516.0	4	9	3.4	14770
7.0	502.5	4	9	3.4	1139
38.6	2776.8	4	9	3.4	6294
8.6	560.0	4	9	3.4	1142
14.2	838.9	4	10	4	1969
9.3	502.4	4	11	5	1335
427	**29535**				**67380**

Floor	Beam Size		Beam Area [sf]	Beam Length [lf]	Beam Contact Area [sfca]
	Width [in]	Depth [in]			
Typ Floor (4-14) & Roof	10	12	0.8	23	84
	10	12	0.8	24	88
	10	12	0.8	21	77
	16	20	2.2	59	354
Floor 3	10	14	1.0	23	92
	10	14	1.0	24	96
	10	14	1.0	21	84
	10	14	1.0	17.5	70
	12	14	1.2	23	100
	12	14	1.2	24	104
	12	14	1.2	21	91
	12	14	1.2	17.5	76
	12	16	1.3	10	47
	12	16	1.3	24	112
	12	16	1.3	21	98
	12	16	1.3	17.5	82
	18	22	2.8	73	487
Floor 2	10	14	1.0	23	92
	10	14	1.0	24	96
	10	14	1.0	21	84
	10	14	1.0	17.5	70
	12	14	1.2	23	100
	12	14	1.2	24	104
	12	14	1.2	21	91
	12	14	1.2	17.5	76
	12	16	1.3	10	47
	12	16	1.3	24	112
	12	16	1.3	21	98
	14	20	1.9	24	136
	18	22	2.8	73	487
Floor 1	10	14	1.0	23	92
	10	14	1.0	24	96
	10	14	1.0	21	84
	10	14	1.0	17.5	70
	12	14	1.2	23	100
	12	14	1.2	24	104
	12	14	1.2	21	91
	12	14	1.2	17.5	76
	12	16	1.3	10	47
	12	16	1.3	24	112
	12	16	1.3	21	98
	12	16	1.3	17.5	82
	18	22	2.8	73	487

Concrete Beam & Girder Takeoffs

Quantity	Beam Area [cf]	Beam Area [cy]	Total Contact Area [sfca]	Total Beam Area/Floor [cy]	Total Contact Area/Floor [sfca]	# Floors	Total Beam Area [cy]	Total Contact Area [sfca]
12	19.17	0.71	1012	7.0	5642	12	83.5	67704
22	20.00	0.74	1936					
12	17.50	0.65	924					
5	131.11	4.86	1770					
4	22.36	0.83	368	17.8	9903	1	17.8	9903
7	23.33	0.86	672					
4	20.42	0.76	336					
4	17.01	0.63	280					
7	26.83	0.99	698					
13	28.00	1.04	1352					
6	24.50	0.91	546					
8	20.42	0.76	607					
4	13.33	0.49	187					
8	32.00	1.19	896					
4	28.00	1.04	392					
2	23.33	0.86	163					
7	200.75	7.44	3407					
4	22.36	0.83	368	18.7	10715	1	18.7	10715
7	23.33	0.86	672					
4	20.42	0.76	336					
4	17.01	0.63	280					
7	26.83	0.99	698					
8	28.00	1.04	832					
7	24.50	0.91	637					
5	20.42	0.76	379					
6	13.33	0.49	280					
16	32.00	1.19	1792					
5	28.00	1.04	490					
4	46.67	1.73	544					
7	200.75	7.44	3407					
4	22.36	0.83	368	17.8	10857	1	17.8	10857
7	23.33	0.86	672					
4	20.42	0.76	336					
4	17.01	0.63	280					
7	26.83	0.99	698					
10	28.00	1.04	1040					
7	24.50	0.91	637					
7	20.42	0.76	531					
6	13.33	0.49	280					
16	32.00	1.19	1792					
5	28.00	1.04	490					
4	23.33	0.86	327					
7	200.75	7.44	3407					
							138	**99178**

Concrete Slab Rebar Takeoffs

Slab	Slab Area	Slab Length (N-S) [ft]	Eff Slab Length (E-W) [ft]	# Rows #4 (@12")	Wt #4 Rebar [plf]	Rebar Wt [lb]
Roof	3252	59	55.1	55	0.668	2172
Typ Floor (11)	56859	59	963.7	964	0.668	37982
3rd	7944	73	108.8	109	0.668	5307
2nd	7724	86.5	89.3	89	0.668	5160
1st	7875	86.5	91.0	91	0.668	5261
						55881

Concrete Beam & Girder Rebar Takeoffs

Floor	Beams & Girders	Rebar #	# Bars	Length [ft]	Weight [plf]	# Beams/Girds	# Floors	Total Rebar Wt [lb]	#3-#7 [lb]	#8-#18 [lb]
Typ. Floor	Beam	7	4	92.0	2.04	12	12	108316	108316	
	Gird	7	2	59.0	2.04	12	12	34732	34732	
		9	2	59.0	3.40	6	12	28886		28886
3rd	Beam	7	5	109.5	2.04	11	1	12310	12310	
		7	2	78.0	2.04	4	1	1275	1275	
		8	2	78.0	2.67	4	1	1666		1666
	Gird	9	5	86.5	3.40	3	1	4412		4412
		9	5	73.0	3.40	2	1	2482		2482
		9	5	59.0	3.40	1	1	1003		1003
2nd	Beam	7	5	109.5	2.04	12	1	13429	13429	
		7	2	78.0	2.04	6	1	1913	1913	
		8	2	78.0	2.67	6	1	2499		2499
	Gird	9	5	86.5	3.40	3	1	4412		4412
		9	5	73.0	3.40	2	1	2482		2482
		9	5	59.0	3.40	1	1	1003		1003
1st	Beam	7	5	109.5	2.04	12	1	13429	13429	
		7	5	63.0	2.04	3	1	1932	1932	
		7	5	96.5	2.04	3	1	2959	2959	
	Gird	9	5	86.5	3.40	3	1	4412		4412
		9	5	73.0	3.40	2	1	2482		2482
		9	5	59.0	3.40	1	1	1003		1003
									190294	**56741.1**

Concrete Wall Takeoffs (Stair & Elevator Walls)

Wall Type	Height [ft]	Length [ft]	Width [ft]	# walls	Contact Area [sfca]	Total Area [cf]	Total Area [cy]
1	191	17.0	1.17	2	6941	3799	141
2	191	10.2	1.17	4	4332	2273	84
3	191	19.3	1.17	2	7800	4302	159
					19073		**384**

Concrete Wall Takeoffs (Shear walls + Stair & Elevator Walls)

Wall	Height [ft]	Length [ft]	Width [ft]	Contact Area [sfca]	Rebar #	Weight [lb/ft]	# Bars	Total Rebar [lb]	Total Area [cf]	Total Area [cy]
SW-1	191	17.0	1.17	6941	5	1.043	24	4781	3799	141
SW-2	191	10.2	1.17	4332	5	1.043	12	2391	2273	84
SW-3	191	10.2	1.17	4332	5	1.043	12	2391	2273	84
SW-4	191	17.0	1.17	6941	5	1.043	24	4781	3799	141
				41619				**14343**		**834**

Existing Structure Takeoffs

Hollow-Core Precast Structural Slab Takeoffs

Level	Area [ft^2]	Openings [ft^2]	Applicable Area [ft^2]	# Floors	Total Area [ft^2]
Roof	4002	750	3252	1	3252
Typ Floor	5925	488	5437	11	59807
Floor 3	8662	718	7944	1	7944
Floor 2	9770	938	8832	1	8832
Floor 1	8050	787	7263	1	7263
					87098

Steel Braced Frames Takeoffs

Braced Frames	BF-1	BF-2	BF-3	BF-4	BF-5	BF-6	BF-7	BF-8	Total Length of Specific Beam [lf]	Quantity (adj for length)
HSS6x6x3/8,6x6x1/2	520	520	520	520	520	352	189	189	3330	278
HSS8x8x1/2	80	80	80	80	80	0	39	39	478	34
HSS18x6x1/2,12x6x1/2	96	0	0	0	0	0	0	0	96	6

Steel Beam Takeoffs

Beams	Typ. Floor	3rd Floor	2nd Floor	1st Floor	Total [lf]	Amount of Steel [tons]
W8x13,15,31 DB8x37=W8x35 DB8x61=W8x58 DB8x65=W8x67	173	33.8	75.4	20	2205	34.2
W10x15,22,33	0	24.6	51.2	68.8	145	1.6
W12x16,19	55.3	770.2	593.4	329.6	2357	22.4
W12x26,30	0	0	103.8	0	104	1.3
W14x22,26	108	326.4	123.1	51.9	1797	23.4
W16x26,31,36,57	20.2	242.4	43.8	37.4	566	8.8
W16x31	0	79.8	0	0	80	1.2
W16x36,40,57	126.8	23.7	11.1	40.4	1597	31.9
W18x40	0	114.5	254.2	229	598	12.0
W18x55,60,71	0	0	144.5	18.3	163	4.9
W21x50,68	19.2	19.2	150.2	133	533	18.1
W24x55,68	0	97.6	160	207.9	466	12.8
W30x90,99	0	0	58.8	80.5	139	6.3
W33x130	0	17.8	29	0	47	3.0
W36x150,170	0	33.9	17.3	0	51	4.4
W36x302,330,360,361,652 W40x211,593	0	146	23.2	0	169	30.5
Shear Stud Count	241	742	970	825	5429	5.4
						222.1

Steel Column Takeoffs

Columns	B-1	B-3	B-4	B-6.9	B-8	B-11	B-12	B-13	B-14	B-3.1	C-1	C-3	C-4	C-6.9	C-8	C-11	C-12	C-13	C-14	C.7-7	C.7-9	D-1	D-5	D-13	D.3-7	D.3-9	D.3-13	D.3-14	E-1	E-1.3	E-2	E-3.5	E-6	E-10	E-12.8	E-13	E-14	E.8-14	F-3.5	F-6	F-7	F-11.4	F-13	F-13.8	Total Length of Specific Column [LF]	Amount of steel [tons]
W10x33,39,45,49,54	0	0	0	0	0	0	0	181	33.75	0	147.25	0	0	0	0	0	0	0	33.75	0	0	0	0	0	0	0	0	33.75	0	130.5	130.5	0	0	99	164.25	0	33.75	33.75	33.75	33.75	0	33.75	33.75	33.75	**1190**	26.8
W10x60,68,77	165.34	0	0	0	0	0	0	0	0	0	33.75	0	0	0	0	0	0	0	0	0	0	0	0	130.5	0	0	0	0	0	0	0	0	0	65.25	0	0	0	0	0	0	15.66	0	0	0	**411**	14.0
W12x40,50,58,65	0	0	0	0	115.75	0	0	0	0	15.66	0	0	0	0	0	0	0	147.25	0	0	0	0	99	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	**378**	9.4
W12x72,79,87,96	0	0	0	0	31.5	0	0	0	0	0	0	0	0	0	0	0	0	33.75	0	0	0	0	31.5	0	0	0	0	0	0	0	0	33.75	0	0	0	0	0	0	0	0	0	0	0	0	**131**	5.2
W14x43,53,61,74,90,99	0	0	0	0	0	0	0	0	0	0	0	0	0	0	147.25	0	0	0	0	62.75	62.75	130.5	0	0	62.75	62.75	0	0	0	0	0	0	99	0	0	0	0	0	0	0	0	0	0	0	**628**	19.1
W14x109,120,145	0	84.25	84.25	33.75	0	84.25	84.25	0	0	0	0	84.25	84.25	0	0	84.25	84.25	0	0	63	63	33.75	0	0	63	63	0	0	33.75	0	0	0	65.25	0	0	0	0	0	0	0	0	0	0	0	**1093**	59.5
W14x176,211,233,257	0	96.75	96.75	0	0	96.75	96.75	0	0	0	0	96.75	96.75	33.75	0	96.75	96.75	0	0	65.25	65.25	0	0	0	65.25	65.25	33.75	0	0	0	0	0	0	0	0	33.75	0	0	0	0	0	0	0	0	**1136**	100.0
Totals	165.34	181	181	33.75	147.25	181	181	181	33.75	15.66	181	181	181	33.75	147.25	181	181	181	33.75	191	191	164.25	130.5	130.5	191	191	33.75	33.75	33.75	130.5	130.5	33.75	164.25	164.25	164.25	33.75	33.75	33.75	33.75	33.75	15.66	33.75	33.75	33.75	4965	**234.0**

Appendix D (Lighting Supplementary Info)

Finish & Glazing Schedules

Area	Key	Material	Manufacturer	Style/Name	Product No.	Color	Finish	Material Reflectance	Size	Notes
Elev. Lobby Floor	CT-1	Ceramic Tile	Stone Source/Florgres	Floortech 9.0	N/A	Floortech 9.0	Matte (DÉCOR)	**0.3**	12"x36"	Floor Tile & Base
Lobby Floor	CT-2	Ceramic Tile	Stone Source/Florgres	N/A	N/A	Floortech 9.0	Matte	**0.3**	12"x36"	Floor Tile & Base
Reception	CT-2	See Above	See Above	See Above	See Above	See Above	See Above		See Above	N/A
Lobby Lounge Floor	CPT-1	Carpet	Shaw- Custom Carpet	N/A	N/A	N/A	Match Custom Strike-Off	**0.12**	Broadloom	Lounge & Elevator
Lobby Vestibule	WM-1	Walk-Off Mat	Shaw- Custom Carpet	N/A	N/A	N/A	N/A	**0.2**	N/A	N/A
North Wall	WD-2	Wood	Custom Fabricated	Custom Fabricated	N/A	N/A	Custom Fabricated	0.16	N/A	Black Walnut Veneer
	MP-1	Metal Panel	Cambridge Architectural	Tidal	79B	N/A	N/A	**0.42**	N/A	Wall Accent
	S-1	Stone	Stone Source	Polished Quartzite, Slabs		Luce De Luna	Polished	**0.55**	1.25" thick	Bar Countertop
	S-2	Stone	Existing	Reclaimed Stone	N/A	N/A	N/A	**0.35**	N/A	N/A
	WC-1	Wall Covering	Design Tex	Keshi	6585-104	Metallic Brown	Vinyl	**0.1**		Lounge
	GL-1	Glass	See Chart Below	Optiwhite/PPG Starphire	See Chart Below	See Chart Below	See Chart Below		See Chart Below	N/A
	PT-4	Paint	Benjamin Moore	N/A	2108-40	Stardust	N/A	0.24	N/A	Bar Ceiling & Door Frame
	WC-3	Wall Covering	Design Tex	Eri Silk	62562-806	Blackberry	Vinyl	**0.4**	N/A	Wall Accent
	PL-1	Plastic Laminate	Wilsonart	N/A	7965-60	Walnut Heights	N/A	**0.4**	N/A	Doors/Millwork/Base
	WD-1	Wood	Custom Fabricated	Custom Fabricated	N/A	N/A	Match Architect's Sample	0.3	N/A	Walnut Veneer
East Wall	CT-6	Ceramic Tile	Design & Direct Source	Cascade Series	N/A	Dew	Glazed	**0.6**	2"x8"	Wall Tile & Base
	PT-1	Paint	Benjamin Moore	N/A	2108-50	Silver Fox	N/A	0.44	N/A	Lobby/Lounge Walls & Ceilings
South Wall	WD-2	Millwork Fins	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall		See North Wall	Millwork Fins
	WD-1 Base	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall		See North Wall	N/A
West Wall	WD-1	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall		See North Wall	N/A
	MP-1	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall		See North Wall	N/A
	WC-1	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall	See North Wall		See North Wall	N/A

Area	Key	Description	Details	Note	Basis of Design	Transmittance
North Wall	GL-1	Ultra Clear Low Iron Fully Tempered Float Glass	6mm Min. Thickness, Monolithic	Entrance Canopy	Pilkington North America Inc.	TBD…GL-6 (46%)
	Fixed glazing & framing areas shall have a U-factor of no more than 0.30 Btu/SF					

Existing & Proposed Luminaire Schedules

Project Name: AC Hotel Philadelphia | Source: IES Handbook
Location: 230 N 13th St, Philadelphia, Pa | Estimate: Lighting Analysis for Lobby/Vestibule/Reception Areas

Proposed Lighting (Compact Fluorescent)

	Type	Manufacturer	Catalog #	Luminaire Wattage	Voltage	# Luminaires	Luminaire Lumen Output	Unit Cost	Total Cost	Total Wattage [kW]
F	4" Triple Tube Square Downlight	Indy	FDSQ24	26	277	29	1800	$96	$2,784	0.754
G	4" Triple Tube Square Wallwash	Indy	FWSQ14	26	277	1	2400	$50	$50	0.026
H	4" Triple Tube Square Downlight	Indy	FDSQ24	26	277	8	1800	$96	$768	0.208
H.2	4" Spec 5 Wall Wash	Zumtobel	S5D4311	13	120	3	1250	$28	$84	0.039
L.1	Microsquare Surface Fluorescent	Phillips	MQ11F10AN	32	120	1	2600	$150	$150	0.032
L.3	Microsquare Surface Fluorescent	Phillips	MQ11F10AN	32	120	7	2600	$150	$1,050	0.224
L.31	Microsquare Surface Fluorescent	Phillips	MQ11F10AN	32	120	9	2600	$150	$1,350	0.288
X.1	MP1 Fluorescent Mini Pendant	Pathway	SF70.1 F13SPR/GU24/27K	13	120	17	900	$88	$1,496	0.221
								Totals	$7,732	1.792

Project Name: AC Hotel Philadelphia | Source: IES Handbook
Location: 230 N 13th St, Philadelphia, Pa | Estimate: Lighting Analysis for Lobby/Vestibule/Reception Areas

Existing Lighting (LED)

Mark	Type	Manufacturer	Catalog #	Luminaire Wattage	Voltage	# Luminaires	Luminaire Lumen Output	Unit Cost	Total Cost	Total Wattage [kW]
F	Recessed 4" Square Downlight	Phillips Lightolier	C4X4L10NUVBLD	14	U 277	42	1100	$150	$6,300	0.588
G	Recessed 4" Square Wallwash	Phillips Lightolier	C4X4L10NUVBLD	14	U 277	1	700	$150	$150	0.014
H	3" Downlight Pinhole	Phillips Lightolier	C3L085N1Z10V	20	120	8	1000	$85	$680	0.160
H.2	3" Adjustable Accent	Phillips Lightolier	CL08NA1Z10V	20	120	3	800	$280	$840	0.060
L.1	Interior Linear LED 4' Grazing Uplight	Phillips	523-000076-05	27	120	1	2000	$442	$442	0.027
L.3	Interior Linear LED 4' Grazing Downlight	Phillips	523-000065-17	50	120	7	2600	$200	$1,400	0.350
L.31	Interior linear LED 1' Grazing Downlight	Phillips	523-000065-09	12.5	120	9	650	$50	$450	0.113
X.1	Mini Canopy Light	Bocci	BOC-28-1	1.8	12	15	47	$375	$5,625	0.027
								Totals	$15,887	1.339

Electricity Prices

2015 Avg. Price for Electrictiy (Philadelphia, Pa)	
Month	**$$/kWh**
January	$0.159
February	$0.160
March	$0.156
April	$0.157
May	$0.156
June	$0.160
July	$0.159
August	$0.159
September	$0.158
October	$0.155
November	$0.155
December	$0.155
Average Price	**$0.157**

20-Year Cost Comparison

	System	
Year	**LED**	**Fluorescent**
0	$15,887	$7,732
1	$17,728	$10,196
2	$19,568	$12,661
3	$21,409	$15,125
4	$23,250	$17,589
5	$25,090	$20,054
6	$26,931	$22,518
7	$28,772	$24,982
8	$30,613	$27,447
9	$32,453	$29,911
10	$34,294	$32,375
11	$36,135	$34,840
12	$37,975	$37,304
13	$39,816	$39,768
14	$41,657	$42,233
15	$43,497	$44,697
16	$45,338	$47,162
17	$47,179	$49,626
18	$49,020	$52,090
19	$50,860	$54,555
20	$52,701	$57,019