TECHNICAL REPORT 2

S.T.E.P.S. Building

Lehigh University

Bethlehem, PA

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Alternate Floor Systems

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Executive Summary

The four floor systems analyze differ in materials, size, and strength. They all have advantages and disadvantages which became clear over the analysis. The existing Composite Beam floor system ended up having the least cost according to RS Means Costworks. In addition, a vibration analysis was performed on this system. In order to effectively compare these floor systems, a vibration analysis based on laboratory equipment would have to be performed.

Least floor depth was associated with the two-way flat plate slab system. However, this system had large minimum column sizes (60”x60”) associated with it. Shear capitals could help to reduce this in further design exploration. In addition, this was the only system with a different sized bay. With the current flat plate in this report, an extra row of columns would need to be added to the building. This has a significant architectural impact.

The concrete systems led to an increased load on the foundation, which the steel system did not.

1. Building Introduction

Lehigh University envisioned the Science, Technology, Environment, Policy, and Society (S.T.E.P.S.) Building as a way to attract new students and retain existing students in the science and engineering fields. The university lacked a modern laboratory building with all the amenities that have come with increases in technology over the years. In an interesting and experimental fashion, the departments have been intermixed by Health, Education & Research Association, Inc. They believe it will lead to increased communication and collaboration among faculty and researchers of various disciplines.

The building is oriented on the corner of East Packer Ave. and Vine St. as shown in the photo below:

Figure 1:

Image Courtesy of Bing.com

Lehigh University slowly purchased the properties which were on the building site as they planned for a building to be put there. The building is also connected to an existing structure through the use of a raised pathway that is enclosed. The building is divided into three wings for the purpose of this analysis. These wings are diagramed in Figure 2 on the following page.

Figure 2:

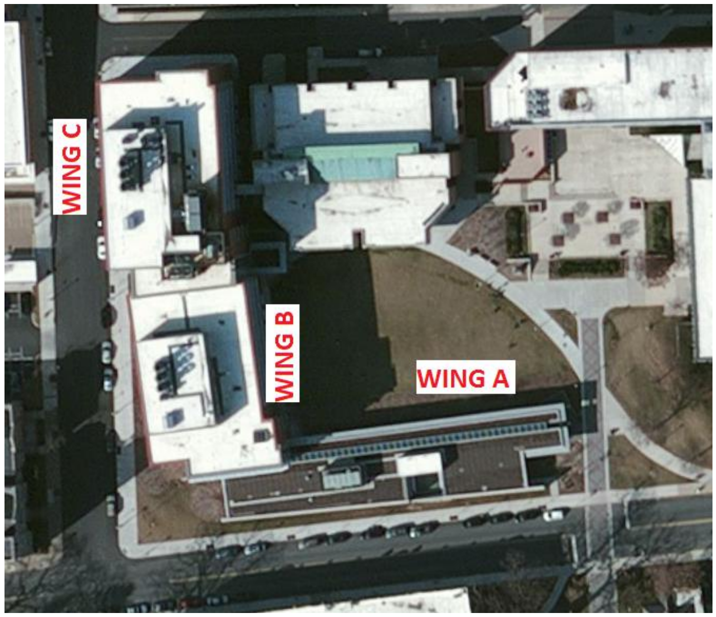


Image courtesy of Bing.com

Wing A is a one story structure with a lounge and entryway. It has raised clearstories to allow for natural daylight to illuminate the space. It also has a 12" deep green roof supported by structural wood which helped in earning LEED Certification. The building is LEED Gold certified by the United States Green Building Council (USGBC). Because of its limited building height, Wing A will not be analyzed in this report.

Wing B is a four story steel framed structure oriented along Packer Ave. Interestingly, Packer Ave. and Vine St. do not meet at a 90 degree angle. So, Wing B is aligned with Packer Ave., and Wing C is aligned with Vine St. There is a large atrium with lounge areas connecting the two structures on each floor.

Wing C is also steel framed and is 5 stories. The building's lateral system consists of moment connections between columns and beams throughout the building. It should be noted that the load resisting elements are one structure as they continue uninterrupted through the atrium.

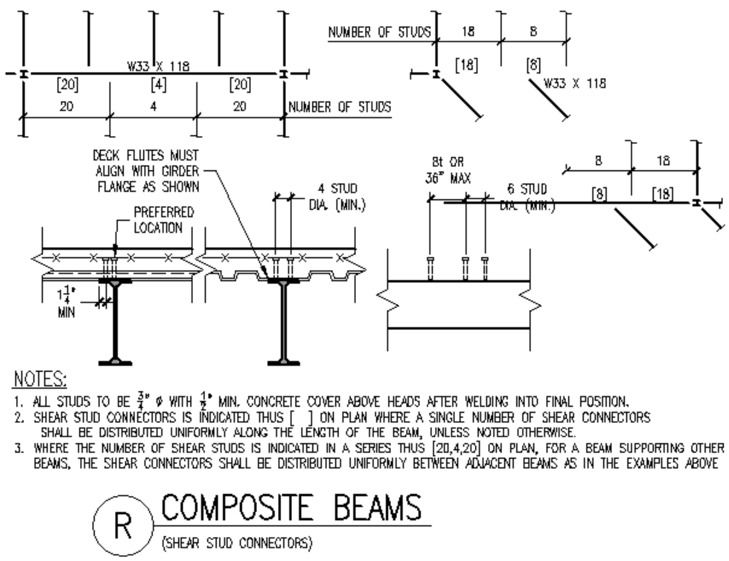
Sustainable features of the building include the green roof, high-efficiency glazing, sun shading, and custom mechanical systems.

2. Structural System

2.1 Floor System

There is a composite steel deck floor system in place for all floors in Wings B & C above grade. Basement floors are slab on grade. Below is a detail of a typical composite beam with shear studs indicated:

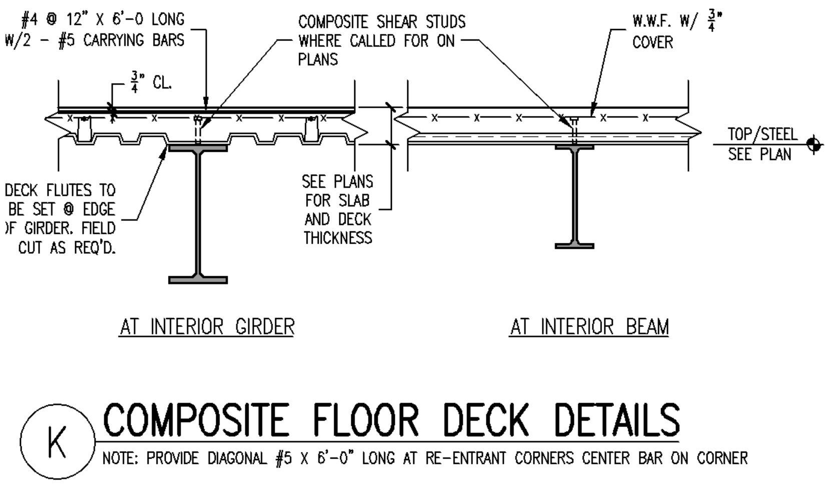
<u>Figure 3:</u>



Along Vine St., which will be considered the longitudinal direction of the building, typical girders have a span of 21'-4" with one intersecting beam at their midpoint. The transverse beams which run parallel to Packer Ave. have a span anywhere from 36'-11" to 42'8".

The decking is a 3" 18 gauge steel deck with 4-1/2" concrete topping and welded wire fabric. The bulk of the decking is run longitudinally throughout Wings B & C and has a clear span of 10'8". The exceptions to this are two bays to the very south of Wing B along Packer Ave. These bays are oriented transversely. The total thickness ends up being 7-1/2" with a 6x6" W2.9 x W2.9 welded wire fabric embedded ¾" from the top of the slab. On the following page is a typical detail of the composite floor slab:

Figure 4:



The floor system is supported by wide flange beams designed as simply supported. A combination of full moment connections, semi-rigid moment connections, and shear connections are used. Typical sizes for transverse beams are W24x55 and W24x76. The girders are W21x44. Most beams have between 28 and 36 studs to transfer shear. Figure 5 shows a typical Full Moment Connection with field welds noted. Figure 6 shows the entirety of the first floor system for Wing B. Figure 8 shows the entirety of the first floor system for Wing C.

Figure 5:

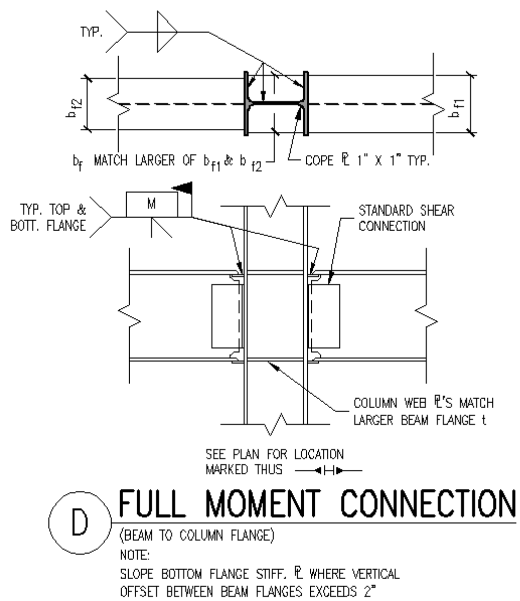


Figure 6:

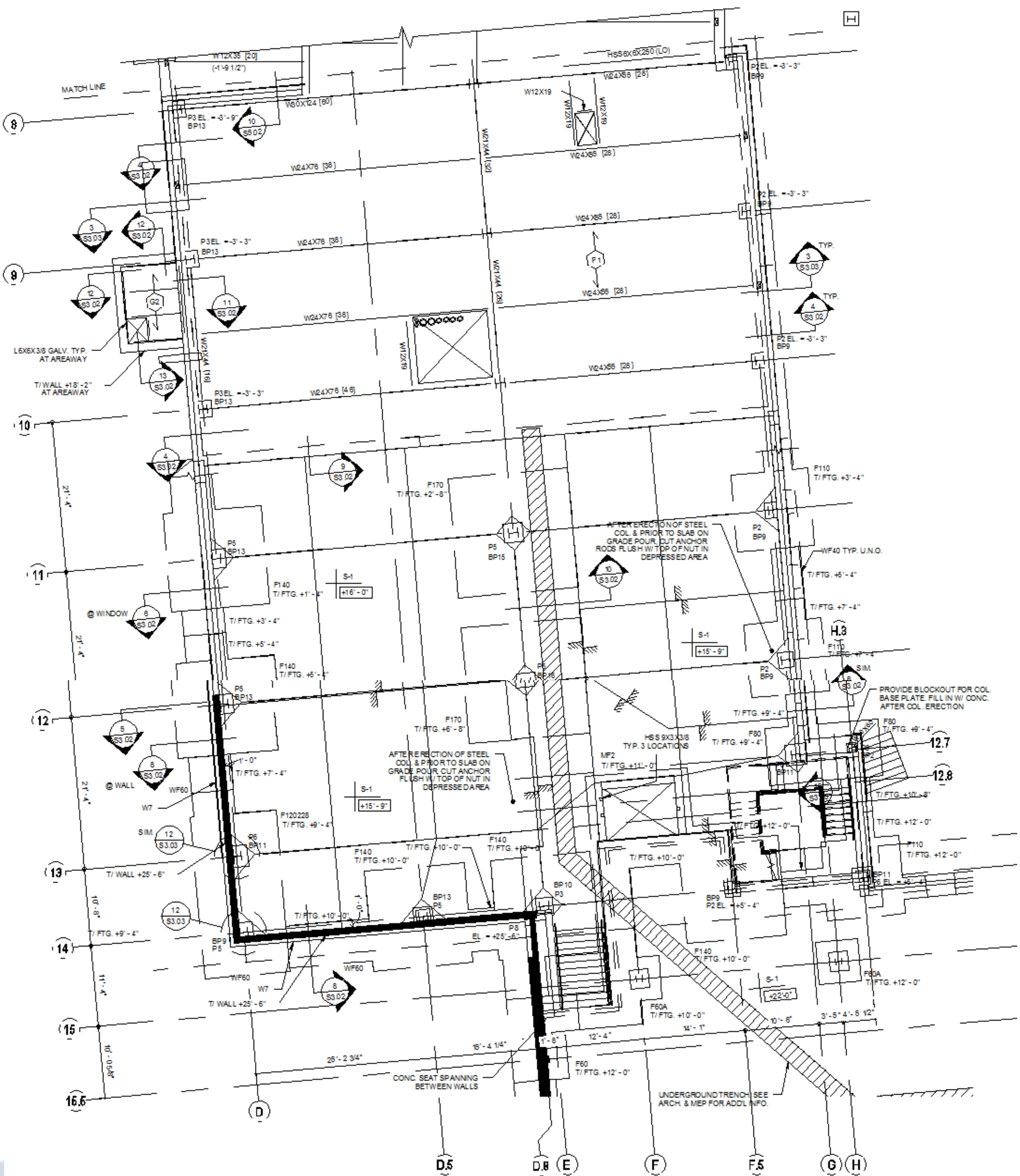
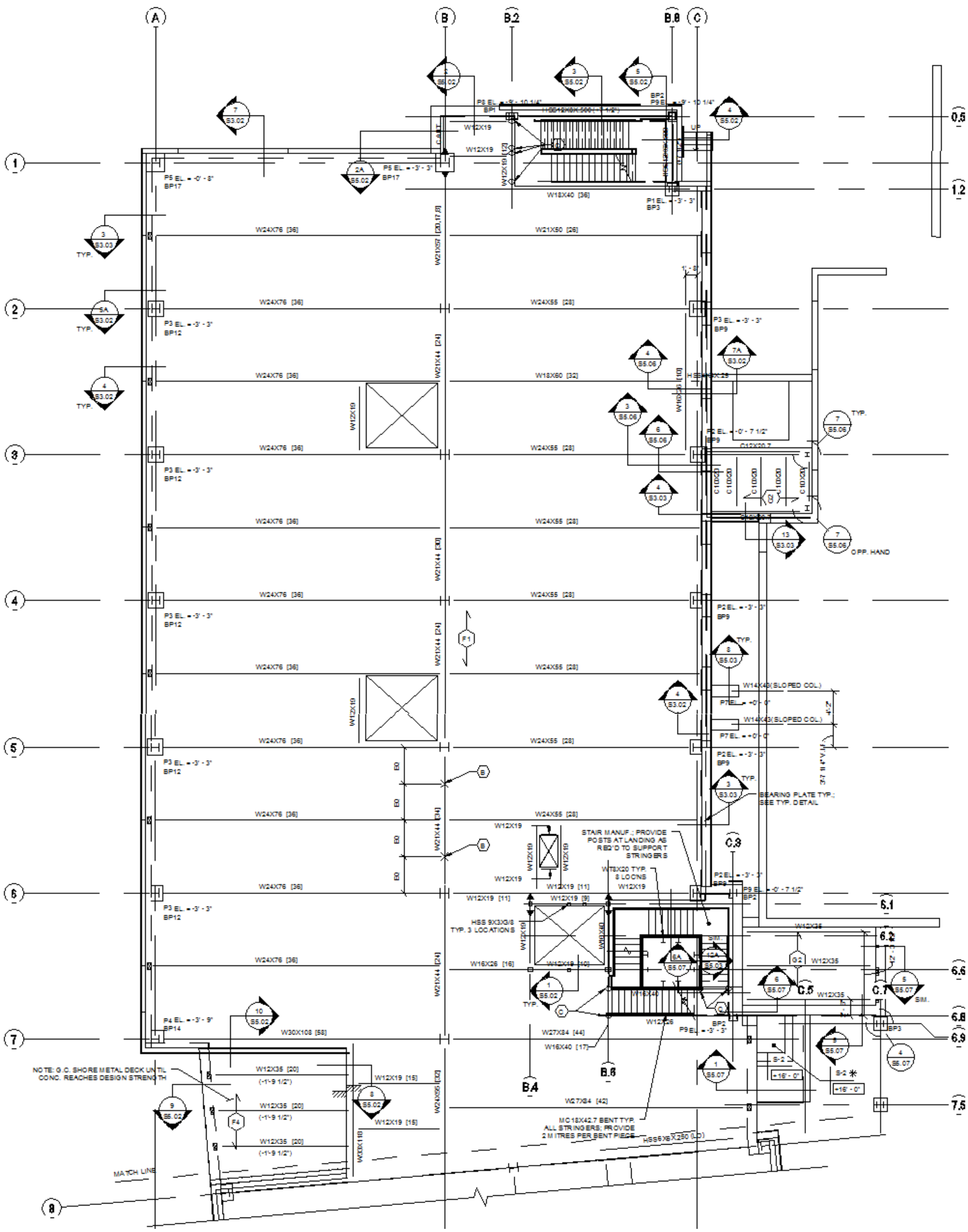


Figure 7:



2.2 Vertical Members

Wide flange columns are used throughout the building for gravity loads. They are arranged for strong axis bending in the transverse direction. Most spans have a column at either end with another at the midpoint.

W14 is the most common section size with weights varying from W14x90 all the way up to W14x192 on the lower floors.

2.3 Foundation

Schnabel Engineering performed a geotechnical analysis of the site in 2007. This concluded that the soil had sufficient bearing capacity to support the loads from the building.

Interior columns are supported by a mat foundation 18' wide and 3' deep. Exterior columns bear on square footings ranging from 11'x11' to 16'x16' with depths from 1'6" to 2'. These are tied into the foundation by base plates with concrete piers.

The reinforced foundation walls have strip footings ranging from 2' to 6' wide with depths between 1' and 2'. These are monolithically cast with the piers for the exterior columns.

2.4 Roof System

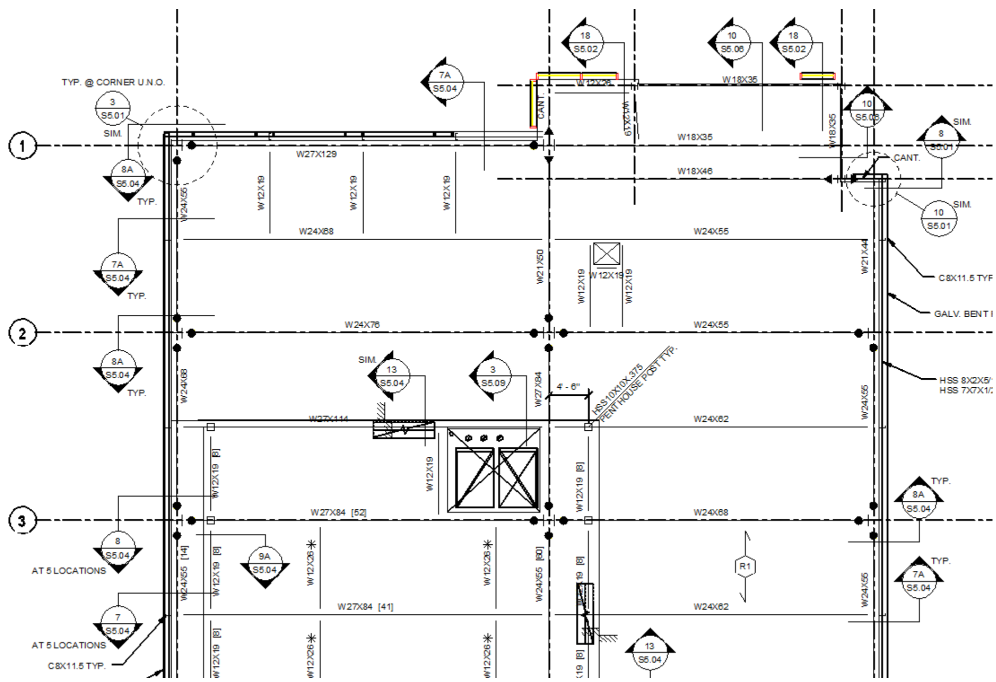
The roof decking consists of a 3" 16 gauge steel roof deck with a sloped roof for drainage. Topping ranges from ¼" to 4-1/2" to achieve a ¼":1' slope. Therefore, total thickness ranges from 3-1/4" to 7-1/2". Framing is similar to floor framing with wide flanges ranging from W24x55 to W24x68.

The floor system has increased loads where the mechanical penthouses are situated. The penthouse itself is framed with square HSS tubing. Heavier W27x84 wide flange beams support this area.

2.5 Lateral System

The building resists lateral loads by moment connections at the beam to column locations. They are continuous throughout the building and beams are designed as simply supported for gravity loads. The moment connections are designed only to take lateral loads. Many of these moment connections are semi-rigid connections to give the system more flexibility. An example of the two types of moment connections is shown below in a section of the roof plan for Wing C. The triangles are full moment connections and the dots are semi-rigid.

<u>Figure 7B:</u>



The lateral loads seen in the Penthouse are going to be the greatest based on height. At the highest Penthouse roof level, there are moment connections in the transverse direction and single angle braced frames in the longitudinal direction. The connections to the roof of the building are rigidly connected to the roof framing members. These members then transfer the load to flexible moment connections in the columns supporting the roof. These roof members are a larger W27x102 compared to adjacent members such as W24x68 or W27x84.

3. Design Codes

The Pennsylvania Uniform Construction Code (PUCC) is the code adopted by the city of Bethlehem, Pennsylvania. The PUCC is based on the International Code Council (ICC). When design was completed in 2008, the 2006 PUCC referenced the following codes:

2006 International Building Code

2006 International Electrical Code

2006 International Fire Code

2006 International Fuel Gas Code

2006 International Mechanical Code

ASCE 7-05, Minimum Design Loads for Buildings and Other Structures

AISC Steel Construction Manual, 13th Edition

ACI 318-05, Building Code Requirements for Structural Concrete

ACI 530-05, Building Code Requirements for Masonry Structures

The primary codes employed were the AISC Manual and ASCE 7-05

4. Design Loads

4.1 Live Loads

Table 1: Live Load Values

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)
Office	50 PSF	50 PSF + 15 PSF (Partitions)
Classroom	40 PSF	40 PSF
Laboratory	100 PSF	100 PSF
Storage	125 PSF	125 PSF
Corridors/Lobbies @ Ground Level	100 PSF	100 PSF
Corridors Above Ground Level	80 PSF	80 PSF

4.2 Dead Loads

Table 2: Calculated Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 18 Ga. Composite Deck			2.84
4-1/2" Topping	0.485 CF/SF	150 PCF	72.75
Self-Weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**95.6 PSF**

4.3 Roof Live Load

Table 3: Roof Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Roof	N/A	20 PSF	**20 PSF**

4.4 Roof Dead Load

Table 4: Roof Dead Load

	Dimension	Unit Weight	Load (PSF)
3" 16 Ga. NS Roof Deck			2.46
3" Concrete Topping (Avg.)	0.290 CF/SF	150	43.5
Self-Weight			5
Roofing Allowance			10
TOTAL			**60.96 PSF**

4.5 Roof Snow Load

4.5.1 Uniform Roof Snow Load

Table 5: Uniform Roof Snow Load

Design Factor	ASCE 7-05	Design Value
Snow Load (Pq)	Figure 7-1	30 PSF
Roof Exposure	Table 7-2	Fully Exposed
Exposure Type	Section 6.5.6.2	B
Exposure Factor (Ce)	Table 7-2	.9
Thermal Factor (Ct)	Table 7-3	1.0
Building Type	Table 1-1	III
Importance Factor (I)	Table 7-4	1.1
Flat Roof Snow Load (Pf)	Equation 7-1	20.8 PSF
Minimum Snow Load (Pf,min)	Section 7.2	22 PSF
Design Snow Load	Section 7.2	**22 PSF**

$Pf = 0.7(Ce)(Ct)(I)(Pq)$

$Pf = 0.7(.9)(1.0)(1.1)(30) = 20.8$ PSF

$20.8 < Pf,min = 22$ → Use 22 PSF as the Design Snow Load

5.5.2 Drift Snow Load

NOTE: For simplification of this analysis, snow drift was not considered. However, it will be necessary to consider snow drift later.

4.6 Penthouse Live Load

Table 6: Penthouse Live Load

Occupancy	Design Load on Drawings	ASCE 7-05 Load (Tables 4-1, C4-1)	Design Load
Mechanical Room	N/A	200 PSF	**200 PSF**

5.7 Penthouse Dead Load

Table 7: Penthouse Dead Load

	Dimension	Unit Weight	Design Load (PSF)
3” 18 Ga. Composite Deck			2.84
4-1/2” Concrete Topping	0.485 CF/SF	150 PCF	72.75
Self-weight			5
MEP Allowance			10
Ceiling Allowance			5
TOTAL			**95.6 PSF**

5.8 Brick Façade Load

Table 8: Brick Façade Load (Per Level)

	Height	Unit Weight (PSF)	Design Load (PLF)
Brick Veneer	10’-3”	35	357.8
2” Rigid Insulation	10’-3”	3	30.7
Steel Framing	10’-3”	6	61.3
Gypsum Wall Board	10’-3”	2	20.5
Window (Glass, Frame, Sash) (ASCE 7-05 Table C3-1)	5’-1”	8	40.8
TOTAL			**510.6 PLF**

5.9 Glass Curtain Wall Load

Table 9: Glass Curtain Wall Load (Per Level)

	Dimension	Unit Weight (PSF)	Design Load (PLF)
Window (Glass, Frame, Sash) (ASCE 7-05 Table C3-1)	15'-4"	8	**122.4 PLF**

5.10 Penthouse Wall Load

Table 10: Penthouse Wall Load

	Dimension	Unit Weight (PSF)	Load (PLF)
Metal Wall Panel	16'-4"	5	81.7
Steel Framing	16'-4"	7	114.3
Bracing Allowance	16'-4"	3	49
TOTAL			**246 PLF**

6. Alternate Floor Systems Analysis

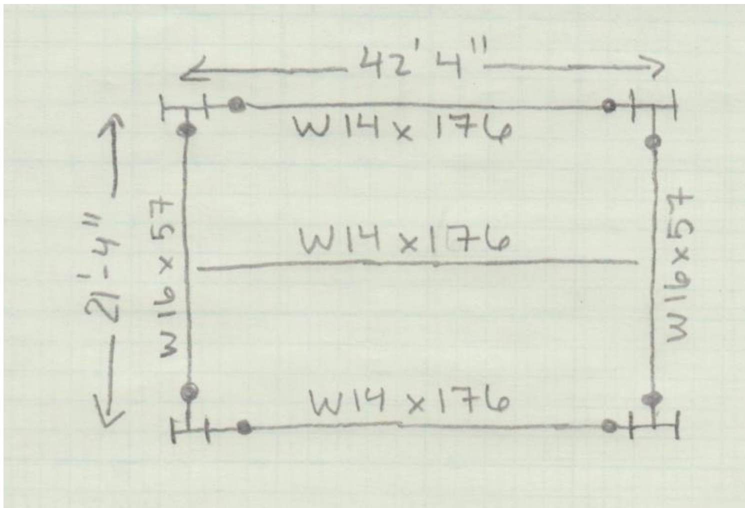
6.1 Floor System 1: Non-Composite Steel Decking on Steel Framing

The first alternate proposed floor system consists of a Vulcraft 3C18 deck sitting on a W14x176 beam. The beam was selected from the Plastic Section Modulus (Zy) Table in the AISC Steel Manual (Table 3-4). It was the most economical for the required moment capacity. The connection was modeled as a simply supported beam as the current beam in the same bay is simply supported. The beam is framed into a W16x57 girder which is modeled with fixed connections. This is a simplification of the semi-rigid Wind Clip connections that is in the existing system. The bay size was kept the same as the existing bay size, approximately 21.33' x 42.25', for easy comparison. The system passed deflection serviceability requirements. Figure 8 contains a floor plan with sizes of beams and girders indicated.

Calculations can be found in Appendix A.

Section 7 contains The Comparison Table for Alternate Floor Systems

Figure 8: Non-Composite Floor Plan



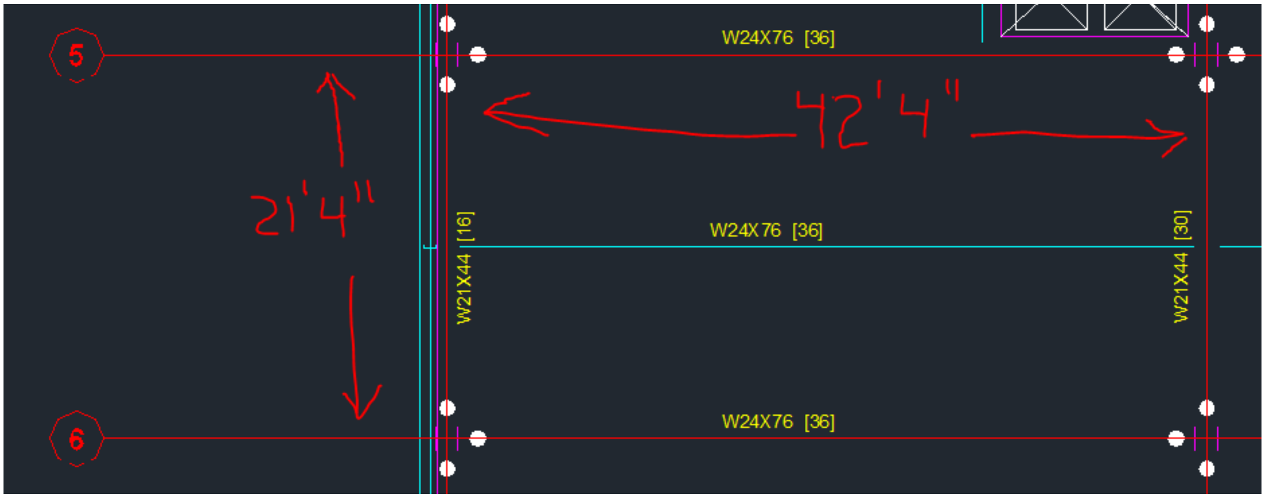
6.2 Floor System 2: Existing Composite Deck on Steel Framing

The existing steel system contains a 3" 18 Ga. composite deck. Vulcraft 3VLI18 was selected as an appropriate way to analyze the current floor. This rests on a W24x76 [36] typically spaced 10'-8" on center. The bay size is 21.33' x 42.25' and approximately 900 sq. ft. These beams are framed into a W21x44 [30] girder that also takes full advantage of composite action. Shoring was not required by the beam, however it was required by the girder. Serviceability requirements were met by all members. A plan view is provided in Figure 9.

Calculations can be found in Appendix B.

Section 7 contains The Comparison Table for Alternate Floor Systems.

Figure 9: Plan view of Composite Floor System:



6.3 Floor System 3: Solid Concrete Two-Way Flat Plate

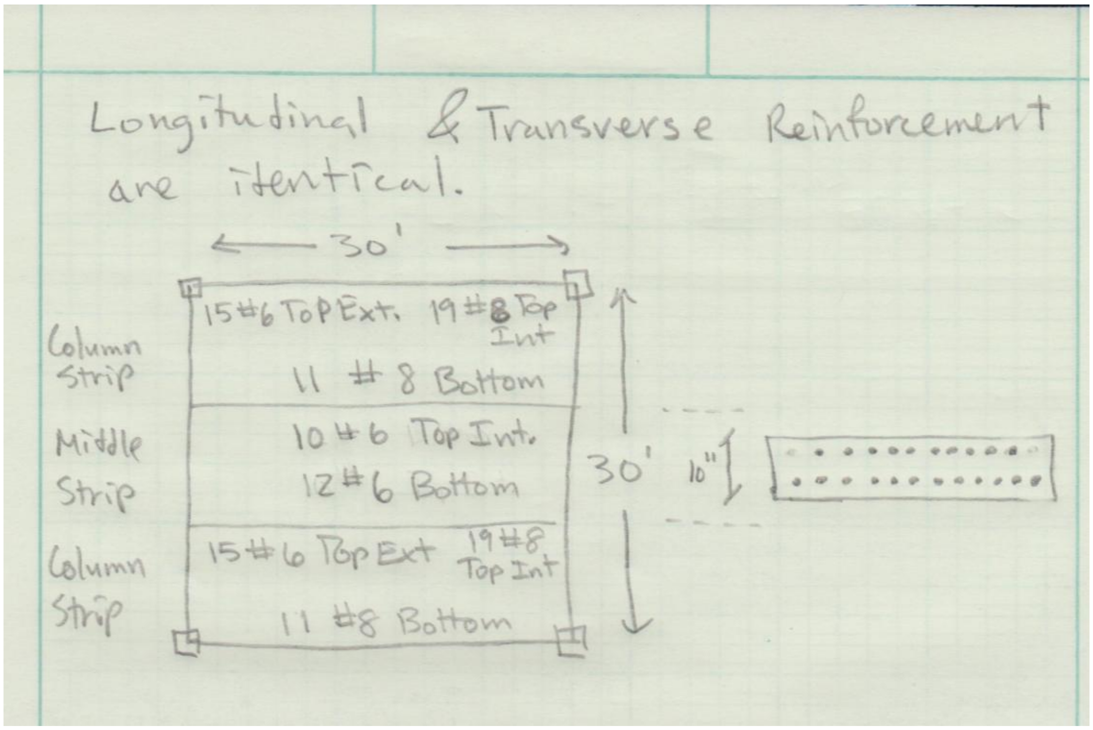
Two-way flat plate construction is a good way to lower the cost of a floor system according to the CRSI 2008 Design Handbook. They help to minimize time and field work because of simple formwork and reinforcing steel layout. A 30'x30' square edge slab was selected for comparison purposes. It has a 900 sq. ft. plan which is equivalent to the current system. It should be noted that this system would require the addition of columns in order to effectively span the current transverse width of the building when compared to the two steel systems. Minimum square column size was 60" x 60". In future analysis, the use of shear caps or increased reinforcement should be investigated to lower this value.

Page 9-35 of the CRSI 2008 Design Handbook was used for the calculation. Depth of the slab was 10" which is significantly less than the existing system. Reinforcement can be seen in plan view on Figure #. Deflection calculations were not required by the code since ACI 9.5.3.2 was satisfied. A plan view of this system is in Figure 10.

Calculations can be found in Appendix C.

Section 7 contains The Comparison Table for Alternate Floor Systems.

Figure 10: Two-Way Flat Plate



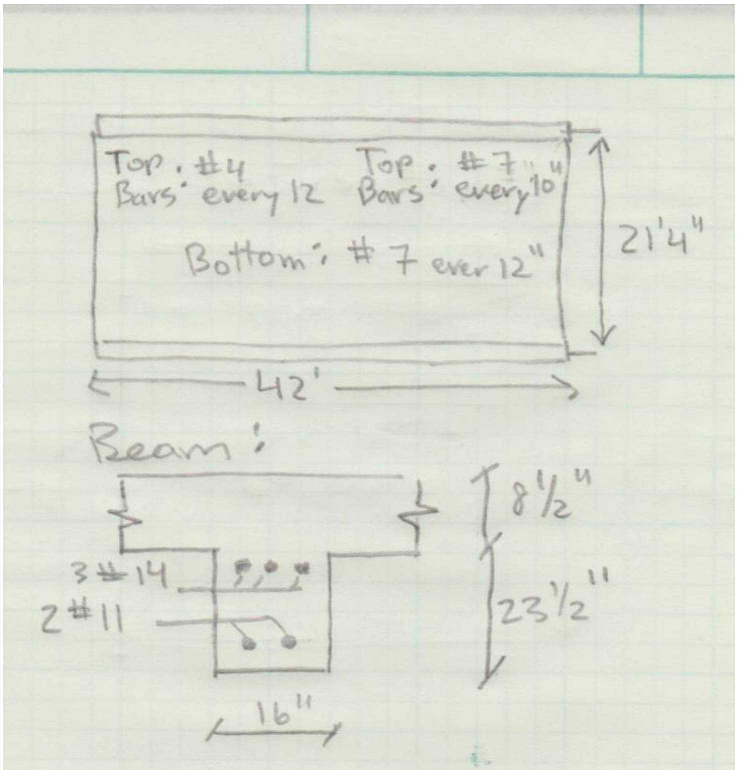
6.4 Floor System 4: One-Way Slab with T-beams

Since the two-way flat plate required extra columns, it was decided that a one-way slab should be explored as well. This maintained a bay size (21'-4" x 42') relatively equal to the bay of the steel framed system (21'-4" x 42'-4"). An 8-1/2" slab with a clear span between beams of 20' was selected. The beams ran 42' from column face to column face and dropped 23-1/2" below the slab with a width of 16". The total depth was 32" for the beams. This is over 3 times the total depth of the two-way slab, but comparable to the existing structure's total floor depth of 31.5". A plan view can be found in Figure 11.

Calculations can be found in Appendix C.

Section 7 contains The Comparison Table for Alternate Floor Systems.

Figure 11: One-Way Slab with T-Beam



7. Comparison Table for Alternate Floor Systems

Cost per square foot was determined using the latest RS Means Costworks 2012 for Allentown, PA. This is the neighboring city to Bethlehem, which is where the building is located. The cost of fireproofing was not included and should be considered an additional cost to the steel framed systems.

	Floor System 1: Non-Composite	Floor System 2: Fully Composite (Existing)	Floor System 3: Two-Way Slab	Floor System 4: One-Way Slab
Total Weight (kips)	78.7	79.4	112.8	115.4
Total Depth (in.)	22.4”	31.5”	10”	32”
Bay Size	42’-4” x 21’-4”	42’-4” x 21’-4”	30’-0” x 30’-0”	42’-0” x 21’-4”
Fire Assembly	2 Hour Rated Sprayed Fiber	2 Hour Rated Sprayed Fiber	2 Hour Rated	2 Hour Rated
Cost per S.F.	$49.75	$33.66	$35.64	$25.65
Arch. Impact	Reduced floor depth	N/A	Reduced floor depth; additional columns; reduced span	Minimal
Impact on Foundation	Slightly Reduced	N/A	Increased Load	Increased Load

8. Conclusion

A vibration analysis is critical to the actual comparison of these systems, because the existing floor system has been designed for vibrations. The two-way flat plate should be out of the running unless shear capitals are considered.

The one-way flat plate should be eliminated because it led to a significantly higher load on the building foundation compared to the steel systems.

Possible remaining systems to be further studied include the Two-Way slab with Shear Capitals, the Fully Composite system, and the Non-Composite system. A vibration analysis should be performed on all alternate designs.

Appendix A-1:

Floor System 1

Non-Composite Floor System

- Max Superimposed Total Load

Maintain Existing

125 psf live + 20 psf misc. dead = 145 psf

- Center to Center span = 10'-8"
  - Conservative Assumption
- Minimum number of spans = 3

Try Vulcraft 3C Conform

Use clear span of 11'-0" > 10'8" OK

Try 3C18 with Allowable psf = 160 psf

145 + 2.84 (Deck Wt.) = 147.84 psf

160 > 147.84 psf OK

Check $\Delta/240$ = 116 psf

$147.84/1.33$ = 111.15 psf

116 psf > 111.15 psf OK for $\Delta/240$

Note:

Total Slab Depth = 6" (t = 3.0")

Weight = 57 psf (145 pcf NWC)

Bay Size = (21.33)(42.25) = 901.2 ft^2

Total Weight = 57(901.2) + 2.84(901.2)

= 53,927.8 lb. = 53.93 k

Floor System 1

Beam for Non-Composite System

Reduce Live Load:

For typ. Beam (No cantilevers)

$K_{LL} = 2$

$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$A_T = (10'8")(42'3") = 450.67\ ft^2$

$L = 125\left(0.25 + \frac{15}{\sqrt{2 \cdot 450.7}}\right) = 93.7\ psf$

$1.2D + 1.6L = 1.2(57+20) + 1.6(93.7)$

$= 242.3\ psf$

$w_u = 242.3\ (10.67') = 2585.3\ plf$

Beam is pinned-pinned with shear connections.

$M_u = \frac{w_u l^2}{8} = \frac{2.585\ (42.25)^2}{8}$

$= 576.8\ k\text{-}ft$

- Fully braced due to decking

Try W14x176 from Zy Table (3-4) AISC spec.

$\phi M_n = 611\ k\text{-}ft > M_u = 576.8\ k\text{-}ft$

- Weight = 176 lb/ft

OK

- Check $L/360 = \frac{42.25(12)}{360} = 1.41"$

$I_x = 2140\ in^4$

$\Delta_{LL} = \frac{5 w_L l^4}{384 EI} = \frac{5(1.0)(42.25)^4(1728)}{384(29000)(2140)}$

$\Delta_{LL} = 1.16" < L/360 = 1.41"$ OK

Appendix 2:

Floor System 1

Girder for Non-Composite System

$$A_T = \left(\frac{79'2''}{2}\right)(21.33') = 844.31\ ft^2$$

$$L = 125\left(.25 + \frac{15}{\sqrt{2 \cdot 844.3}}\right) = 76.88\ psf$$

$$1.2(77) + 1.6(76.88) = 215.4\ psf$$

$$w_u = 215.4(39.58) = 8525.5\ plf$$

Girder is fixed - fixed (moment connection)

Max

$$M_{u,max} = \frac{w\ell^2}{12} = \frac{8.526(21.33)^2}{12} = 323.3\ k\text{-}ft.$$

$$\text{Unbraced length} = 21.33/2 = 10.67'$$

W16x57 has $\phi M_n = 330$ k-ft with Unbraced length of 11'

$$\phi M_n = 330 > M_u = 323.3\ k\text{-}ft$$

ok

Weight = 57 lb./ft

$$\text{Check } L/360 = \frac{21.33(12)}{360} = 0.711''$$

$$I_x = 758\ in^4$$

$$\Delta_{LL} = \frac{w\ell^4}{384EI} = \frac{(3.04)(21.33)^4(1728)}{384(29000)(758)}$$

$$\Delta_{LL} = 0.129'' < L/360 = 0.711''$$

OK

$$\text{Total Wt.} = \left(53{,}928 + 3(176)(42.25) + 2(57)(21.33)\right)/1000$$

$$= 78.7\ k$$

$$\text{Cost} = \underset{\text{slab}}{\$7.78} + \underset{\text{Beam + Girder}}{\$41.97} = \$49.75/\text{S.F.}$$

Floor System 2

Composite Beam:

$K_{LL} = 2$

$$L = 125\left(.25 + \frac{15}{\sqrt{2 \cdot 450.7}}\right) = 93.7 \text{ psf}$$

$$1.2(77) + 1.6(93.7) = 242.3 \text{ psf}$$

$$w_u = 242.3\,(10.67) = 2.585 \text{ klf}$$

Pinned - Pinned Connection

$$M_u = \frac{w_u l^2}{8} = \frac{2.585(42.25)^2}{8}$$
$$= 576.8 \text{ k-ft}$$

- Existing Beam is W 24 x 76 [36]
- Shear Stud Strength Assumed = 17.4 k

$$\Sigma Q_n = 18\,(17.4) = 313.2 \text{ k}$$

$$b_{eff} = \left| \begin{array}{l} \frac{42.25(12)}{8}(2) = 126.75'' \\ \frac{10.67(12)}{2}(2) = 128.04'' \end{array} \right.$$

$$b_{eff} = 126.75''$$

$A_s = 22.4 \text{ in}^2$ for W24x76

$$A_s f_y = 22.4\,(50) = 1120 \text{ k}$$

$$V'_c = 0.85(126.75)(4000)(4.5)/1000$$
$$= 1939.3 \text{ k}$$

$$\Sigma Q_n \begin{array}{l} < V'_c \\ < A_s f_y \end{array} \Rightarrow Y_2 = 7.5 - a/2$$

Floor System 2

$$Y_2 = 7.5 - \frac{313.2}{(2)(.85)(4)(126.75)}$$

$= 7.5 - 0.3636 = 7.136'' \Rightarrow 7''$

Conservative and in Table 3-19 ✓

For a W24x76 w $\Sigma Q_n = 313.2$ k, and $Y_2 = 7''$,

ΣQ_n	ϕM_n
394	1180
280	1080

$$\frac{1180-1080}{394-280} = \frac{1180-x}{394-313.2}$$

$x = 1109.1$ k-ft

$\phi M_n = 1109.1$ k-ft $> M_u = 576.8$ k-ft

OK

- Does it need shoring?

$w_u = 2.585$ klf

Total Uniform Load (kips) $= 2.585(42.25)$

$= 109.22$ k < 143 k allowable for 42' long beam (Table 3-6)

OK

- Shoring not needed

- Check wet concrete deflection
  - Assume concrete = 57 psf

$$W_{wc} = \frac{57(10.67)+76}{1000} = 0.684 \text{ klf}$$

$$\Delta_{wc} = \frac{5(.684)(42.25)^4(1728)}{384(29000)(2100)} \qquad I_x = 2100 \text{ in}^4$$

$= 0.81''$

$$\ell/240 = \frac{42.25(12)}{240} = 2.11'' > 0.81'' \quad \text{OK}$$

Floor System 2

· Check LL Deflection:

$I_{LB} =$

ΣQ_n	
394	4190
280	3710

$$\frac{4190-3710}{394-280} = \frac{4190-x}{394-313.2}$$

$$I_{LB} = 3850 \text{ in}^4$$

$$\ell/360 = 1.41''$$

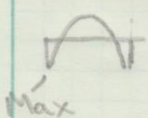
$$\Delta_{LL} = \frac{5(1.0)(42.25)^4(1728)}{384(29000)(3850)}$$

$$= 0.646'' < \ell/360 = 1.41'' \quad \underline{OK}$$

Floor System 2

Composite Girder:

$w_u = 8.526$ klf



$M_{u_{max}} = 323.3$ k-ft

Existing Girder is W 21x44 [30]

$\Sigma Q_n = 15(17.4) = 261$ k

$$b_{eff} = \begin{cases} \frac{21.33(12)}{8}(2) = 63.99'' \\ \frac{39.58(12)}{2}(2) = 474.96'' \end{cases}$$

$b_{eff} = 63.99''$

$A_s = 13$ in^2 $\quad I_x = 843$ in^4

$A_s f_y = 13(50) = 650$ k

$V'_c = 0.85(63.99)(4000)(4.5)/1000$

$= 979$ k

$\Sigma Q_n \begin{matrix} < V'_c \\ < A_s f_y \end{matrix} \Rightarrow Y_2 = 7.5 - a/2$

$$Y_2 = 7.5 - \frac{261}{(2)(.85)(4)(63.99)} = 6.9''$$

Use $Y_2 = 6.5''$ (Conservative)

ΣQ_n	$Y_2 = 6.5$
260	615 ft-k

Use $\Sigma Q_n = 260$ (Conservative)

$\phi M_p = 615$ k-ft

Floor System 2

$\phi M_p = 615 > M_u = 323.3$ k-ft

OK

Does it need Shoring?

$w_u = 8.526$ klf

Total Uniform Load (kips) $= 8.526(21.33)$

$= 181.86$ k > 136 k allowable for 21' long beam (Table 3-6)

* Girder must be shored or sized up

• Since it is the existing design, provide Shoring

• Check wet concrete deflection

$$W_{wc} = \frac{57(39.58)+44}{1000} = 2.3 \text{ klf}$$

$$\Delta_{wc} = \frac{2.3(21.33)^4(1728)}{384(29000)(843)} = 0.088''$$

$$\ell/240 = \frac{21.33(12)}{240} = 1.07'' > \Delta_{wc} = 0.088''$$

OK

Check LL Deflections:

$I_{LB} = 1350 \text{ in}^4$

$\ell/360 = 0.711''$

$$\Delta_{LL} = \frac{(21.33)^4(1728)(3.04)}{384(29000)(1350)} = 0.072'' < \ell/360 = 0.711''$$

OK

Floor System 2

Total Weight:

$$67.9 + \frac{\left[(3)(76)(42.25)+(2)(44)(21.25)\right]}{1000} = 79.4 \text{ k}$$

Appendix 3:

Floor System 3

Solid Two-Way Flat Plate (Concrete)

Bay Size = 21.33' x 42.25' ≈ 900 sq. ft.
↑ Existing.

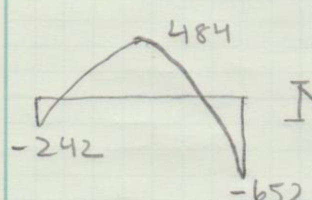
For comparison to the steel framing systems, a 30' x 30' (900 sq. ft) square slab will be designed.
This is especially convenient for the cost analysis.

- Used CRSI Design Handbook 2008

$w_u = 1.2(20) + 1.6(125)$
↑ super imposed

$w_u = 224$ psf

Page 9-35 of CRSI:
- Edge Panel
- 10" thick — 0.833 c.f./s.f.
- Use 250 psf row (conservative)
- Min Square column = 60"



M (ft-kip)

- Column Strip
  - Top Ext: 15 #6
  - Bottom = 11 #8
  - Top Int = 19 #8

Floor System 3

- Middle Strip

12 #6 Bottom

10 #6 Top Int.

$$\frac{\text{End Panel}}{\text{Steel (psf)}}$$

Location:

Edge = 4.57 psf

Edge Corner (EC) = 4.64 psf

Corner = 4.81 psf

- No Drop Panels
- No shear heads
- Deflection Calculations not required by Code, because ACI 9.5.3.2 was satisfied (Min. Thickness).

Total Weight:

$$\left[0.833\,(145\text{ pcf})(900) + (4.57)(900)\right]/1000$$
$$= 112.82\text{ k}$$

Appendix 4:

Floor System 4

One-Way Slab

- Factored Superimposed Load
  w_u = 224 psf
- Used CRSI Design Handbook 2008

  Clear Span = 20'-0"
- Use 8 1/2" thick slab from page 7-18

  for Exterior Span $\rho = 0.0075$

  Factored Usable Load = 382 psf > 224 psf.
  - Designed for Servizability OK

  Top Bars = #7 every 10"

  Bottom Bars = #7 every 12"

  Top Bars (Free End) = #4 every 12"

  T-S Bars = #3 every 7"

  As Top = 0.720 in^2

  As Bottom = 0.6 in^2

  Slab Wt. = 106 psf

  Steel Wt. = 3.46 psf

Floor System 4

Beam:

$w_u = \frac{224(21.33)}{1000} = 4.78$ k-ft

$l_n = 42'$

$h = 32''$

$M_u = \frac{wl^2}{12} = \frac{(4.78)(42.25)^2}{12} = 711$ k-ft

$\phi M_n = 725$ k-ft

$b = 16''$

Bottom Bars = 2 #11

Top Bars = 3 #14

Stirrup Ties = 185 M

Steel Weight = 1775 lb.

- Designed for Deflection Control
- Referenced Page 12-43 of CRSI 2008

Total Weight:

$\text{Slab} = \frac{106(892.5) + 3.46(892.5)}{1000} = 97.69$

$\text{Beam} = 1.775 + \left(\frac{23.5}{12}\right)\left(\frac{16}{12}\right)(42)(145)/1000 = 17.68\text{k}$

$\text{Total} = 115.4\text{k}$