Weill Cornell Medical Research Building
413 E. 69th Street
New York, NY

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Structural Option

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Technical Report 2

Submitted: 11/2/11

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Executive Summary

The purpose of this *Pro-Con Structural Study of Alternate Floor Systems* report was to investigate three alternatives to the existing floor system. Each system was analyzed based on structural and non-structural criteria and the feasibility of each system was determined from comparison of the results of the analysis. The systems investigated were:

- Existing Two-Way Flat Plate
- One-Way Pan Joist
- Banded-Beam
- Composite Deck and Beam

The Weill Cornell Medical Research Building has typical bay sizes of 27'-7," 25'-0," and 16'-3" in the North-South direction and 21'-0" in the East-West direction. There is also a 9'-8" cantilever on the front of the building from which the curtain wall is hung.

The only system deemed completely unfeasible was the One-Way Pan Joist system due to large deflections, the depth of the system, and limitations on MEP arrangement. The composite deck and beam system was deemed not feasible based on the preliminary design, but a more economical and viable system could be designed through further investigation. The existing system was deemed the most feasible, but requires a camber on the slab for the cantilever portion. Perhaps a better solution for this would be the banded-beam system, which features post-tensioning. This system also warrants further investigation which could potentially yield a system as viable as the existing one.

Introduction

The Weill Cornell Medical Research Building is the newest addition to the campus of the Weill Cornell Medical College on the upper east side of Manhattan. Located at 413 East 69th Street in New York City, the Medical Research Building is adjacent to other Weill Cornell buildings. The Weill Greenberg Center on its northeast side is an educational facility designed by the same architects as the Medical Research Building. Olin Hall to the east, and the Lasdon House to the north are residential buildings that house students of the medical college. 69th Street slopes down to the east across the site of the Medical Research Building and the utilities run under it. The Con. Edison power vaults are also located under 69th Street and the sidewalk in front of the building.

The $650 million Medical Research Building is approximately 455,000 square feet with three stories below grade and eighteen, plus a penthouse and an interstitial floor, above grade. The total height of the building above grade is 294'-6." Floors 4-16 are dedicated to laboratory space. The first basement level, as well as the interstitial floor between floors 16 and 17, and the 17th and 18th floors are designated as mechanical floors. The bottom two levels of the basement contain the MRB's animal facility. Service and freight elevators and vertical circulation are located on the west side of the building next to the loading docks on the 69th Street side. Passenger elevators and vertical circulation are nearer the center of the building where the two story lobby atrium welcomes people into this hub of scientific exploration.

In the rear of the building, adjoining the second floor, there is a terrace that bridges the gap between the rear façade of the MRB and the Lasdon House. A grand staircase leads from the lobby on the ground floor up to the enclosed lounge on the second floor that opens onto the terrace. There are two entryways from the Lasdon House to the terrace so anyone living in that building and working in the Medical Research Building would have easy access. The terrace also wraps around the side of the Lasdon House and connects to a stairway leading down to the sidewalk on 70th street.

The building is defined visually by the undulating glass sunshade curtain wall across the front of the building. This curtain wall is attached to the floor slabs that are cantilevered out approximately 9'-8" from the exterior row of columns to meet it. The curtain wall itself has two layers. The outer layer features the glass sunshade wall with aluminum mullions. That is tied to the inner layer of insulated glass (also with aluminum mullions) by aluminum. The inner layer is anchored to the slab either directly through the mullion or with a steel outrigger.

Structural Systems

Foundation System

The foundation system consists of spread footings bearing on undisturbed bedrock with strap beams as necessary around the perimeter. This undisturbed bedrock is expected to support 40 tons per square foot. According to the geotechnical report, there are two types of bedrock encountered on the site. One type supports 40 tsf and the other 60 tsf, but it is recommended by Langan Engineering and Environmental Services that the footings be designed to rest on 40 tsf bedrock. The slab on grade is a 6" concrete slab resting on a 3" mud slab on 24" of crushed stone. The perimeter concrete walls of the basement are 20" thick with strip footings. Below, Figure 1 is an image of the foundation plan.

The geotechnical report also states that the water table is approximately 50 feet above the foundation level. This poses the problem of seepage through the rock and also uplift on the foundation. A few different design solutions are presented in the report. The resolution of this problem comes in the form of 4-50 ton rock anchors located at the bottom of Stairwell B on the East side of the building to resist the uplift.

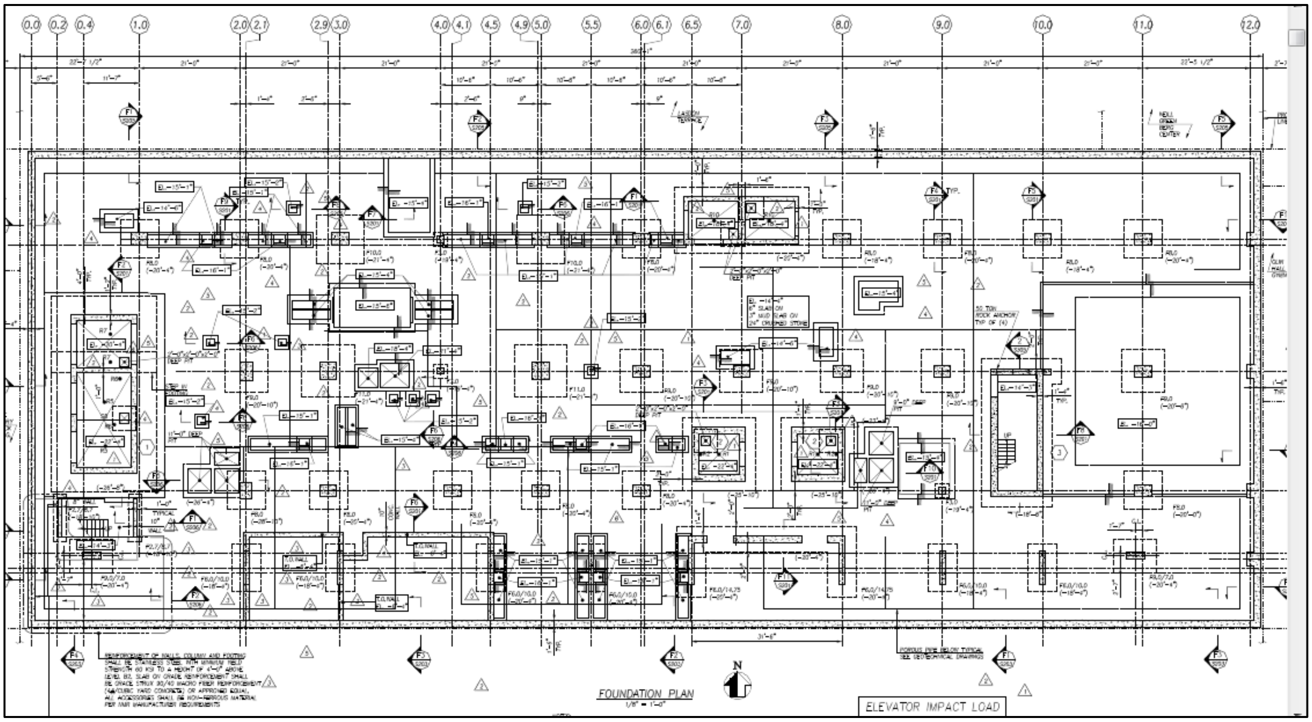


Figure 1: Basement Level 3 – Foundation Plan

Floor System

The floor system in the Medical Research Building is 2 way flat plate concrete slabs. These slabs vary in depth from floor to floor (see Figure 2 below). The bottom reinforcement is typically #5 bars at 12." Top reinforcement and additional bottom reinforcement varies as needed throughout the building. The slabs are especially thick in this building because much of the design was constrained by strict vibration requirements of the medical and research equipment in the building. Laboratory floors were designed to limit vibration velocities to 2000 micro-inches per second. Walking paces were assumed to be moderate (75 footfalls per minute) in the labs and corridors and fast (100 footfalls per minute) only in public areas such as the lobby. There are also vertical HSS members at alternate floors through the middle of the building where the laboratories are located. These members serve no structural load bearing purpose, they are simply meant to tie each floor to another floor to further limit vibrations by forcing any impact to excite vibrations in two floors instead of just one.

Floor	Slab Depth (in)
B3	6
B2	12.5
B1	12.5
1	11
2	12
3	12.5
4	12.5
5	12.5
6	12.5
7	12.5
8	12.5
9	12.5
10	12.5
11	12.5
12	12.5
13	12.5
14	12.5
15	12.5
16	12.5
Interstitial	10.5
17	10.5
18	12.5
19	10.5

Figure 2: Slab Depth per Floor

The front of the building features a cantilever slab extending approximately 9'-8" from the center of column line D. The glass sunshade curtain wall is connected to the edge of the slab. The slab is the same thickness as the rest of the floor, but is cambered up to reduce deflections caused by the curtain wall load. On the second floor, the slab is cambered 1" up. For the third through the interstitial floors, the slab is cambered 5/8" up.

Lateral System

Lateral loads, such as seismic and wind loads, are primarily resisted by 12"-14" concrete shear walls located around the stairwells and elevator cores. A couple of these shear walls step in at the second floor. Extra precautions were taken to make sure that the lateral moment still has a viable path to travel through that step in. Severud, the structural engineers for the project, desired to transfer lateral loads toward the perimeter of the building. In the front of the building there are massive 12/14 x 72 inch columns from which the slabs cantilever out and the glass sunshade curtain wall is hung. These columns also take some of the lateral loads. See the sketch in Appendix E for the location of lateral system elements on a typical floor.

Beams and Columns

There is a very wide variety of beam and column sizes in this building. There are almost forty different sizes of columns with dimensions ranging from 12” to 84,” with the most typical column being 24 x 36, and approximately fifty five different sizes of beams ranging from 8 x24 to 84 x 48. Except on the laboratory floors, which are quite uniform, the column sizes tend to change from floor to floor. Extra precaution was taken during design and reinforcement was provided to ensure the continuity of the load path through these column transfers.

Columns are located on the specified grid of 4 major rows in the East-West direction for the majority of the floors—except the first floor and below grade, which have a fifth row in the back of the building. Bay sizes are 27’-7,” 25’-0,” and 16’-3” in the North-South direction and the typical bay in the East-West direction is 21’-0” with end spans approximately 22’-6.” Beams, however, are only placed where they are needed. They are rarely in the same place from floor to floor and each floor has a different number of beams. The fourth floor has the fewest with 6, and the second floor has the most with 33. Below in Figure 3 is a typical framing plan for the 5^{th}-15^{th} floors.

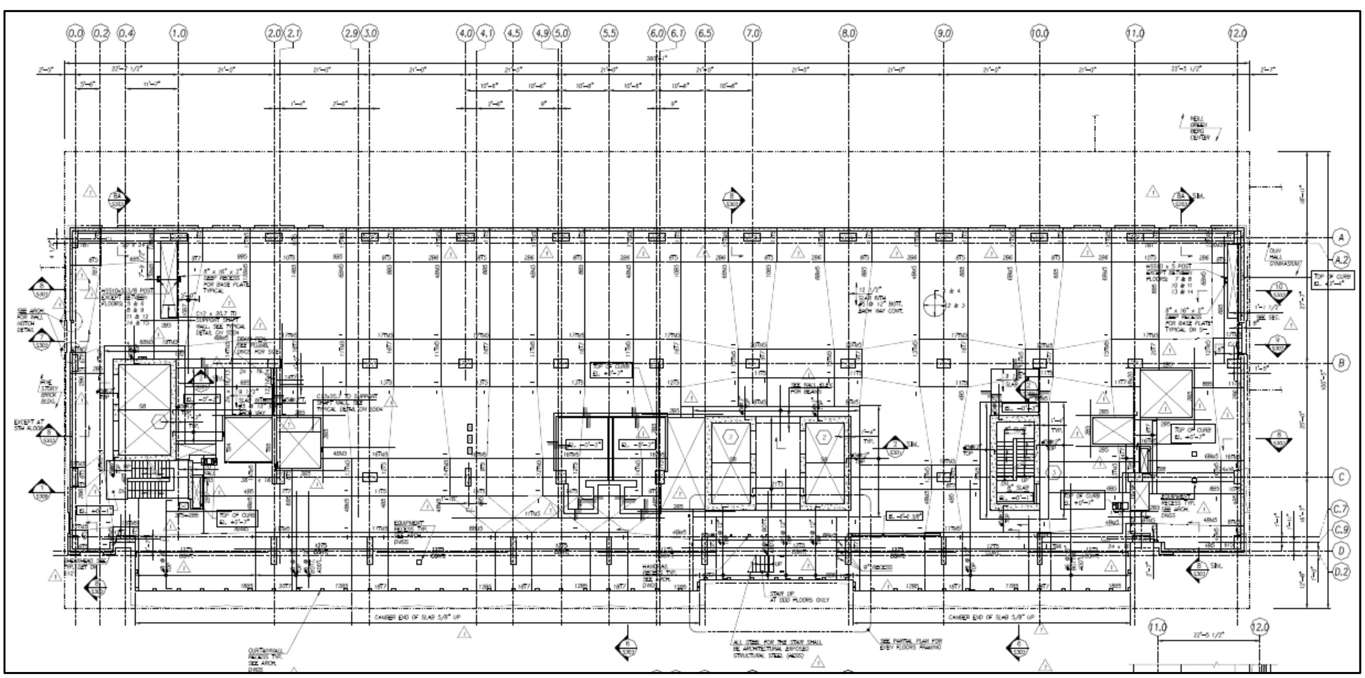


Figure 3: Typical Framing Plan – 5^{th}-15^{th} Floors

Design Codes and Standards

The Weill Cornell Medical Research Building was designed according to the 1968 New York City Building Code based on the UBC. In 2008 New York City updated their building code, which is now based on the IBC. For this report, the new 2008 code for analysis and design is being used; which references ASCE 7-02, ACI 318-02, etc. For relevance, ASCE 7-05, ACI 318-08, and the AISC Steel Construction Manual 14th ed. will be referenced in this report. The design for the Medical Research Building was submitted in 2008 and the project team decided to file under the old code. The MRB is located in New York City's zoning district R8, the use group is 3 (college), the construction class is I-C, and the occupancy group is D-2.

Structural Materials

The Medical Research Building is a predominantly concrete structure. The f'_c of the concrete varies throughout. See the table below in Figure 4 for the strength of concrete per floor.

On the roof and penthouse levels, there are structural steel members that frame platforms for mechanical equipment (cooling towers on the roof level), and also the window washing platform on the penthouse level. This penthouse level platform provides the means from which the window washing apparatus are hung and operated.

Steel members include W14s as horizontal framing members and HSS 10x8x5/8 for the perimeter. Columns, some of which extend down to the 19th floor (on the west side of the building) and some which continue to the 18th floor (on the east side) are HSS 8x8x3/8. The cooling tower platform consists of horizontal members ranging from W8s – W18s and HSS 8x8s as the columns. Figures 5 and 6 show the window washing platform and 19th floor framing plans.

Floor	f'$_c$ Beams and Slabs(psi)	f'c Columns (psi)
B3	4000	8000
B2	5950	8000
B1	5950	8000
1	5950	8000
2	5950	8000
3	5950	8000
4	5950	8000
5	5950	8000
6	5000	5950
7	5000	5950
8	4000	5000
9	4000	5000
10	4000	4000
11	4000	4000
12	4000	4000
13	4000	4000
14	4000	4000
15	4000	4000
16	4000	4000
Interstitial	4000	4000
17	4000	4000
18	4000	4000
19	4000	4000

Figure 4: Concrete Strength per floor

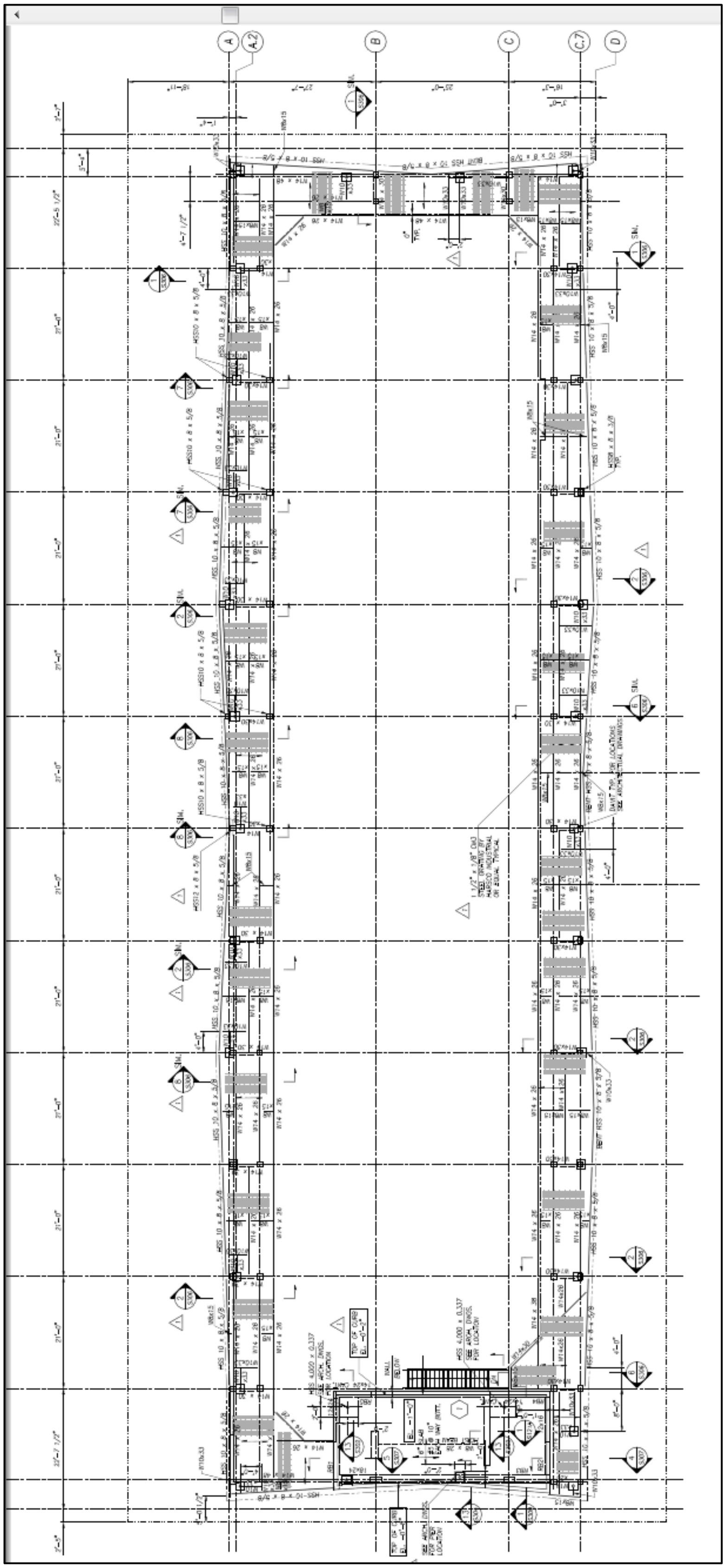


Figure 5: Window Washing Platform Framing Plan

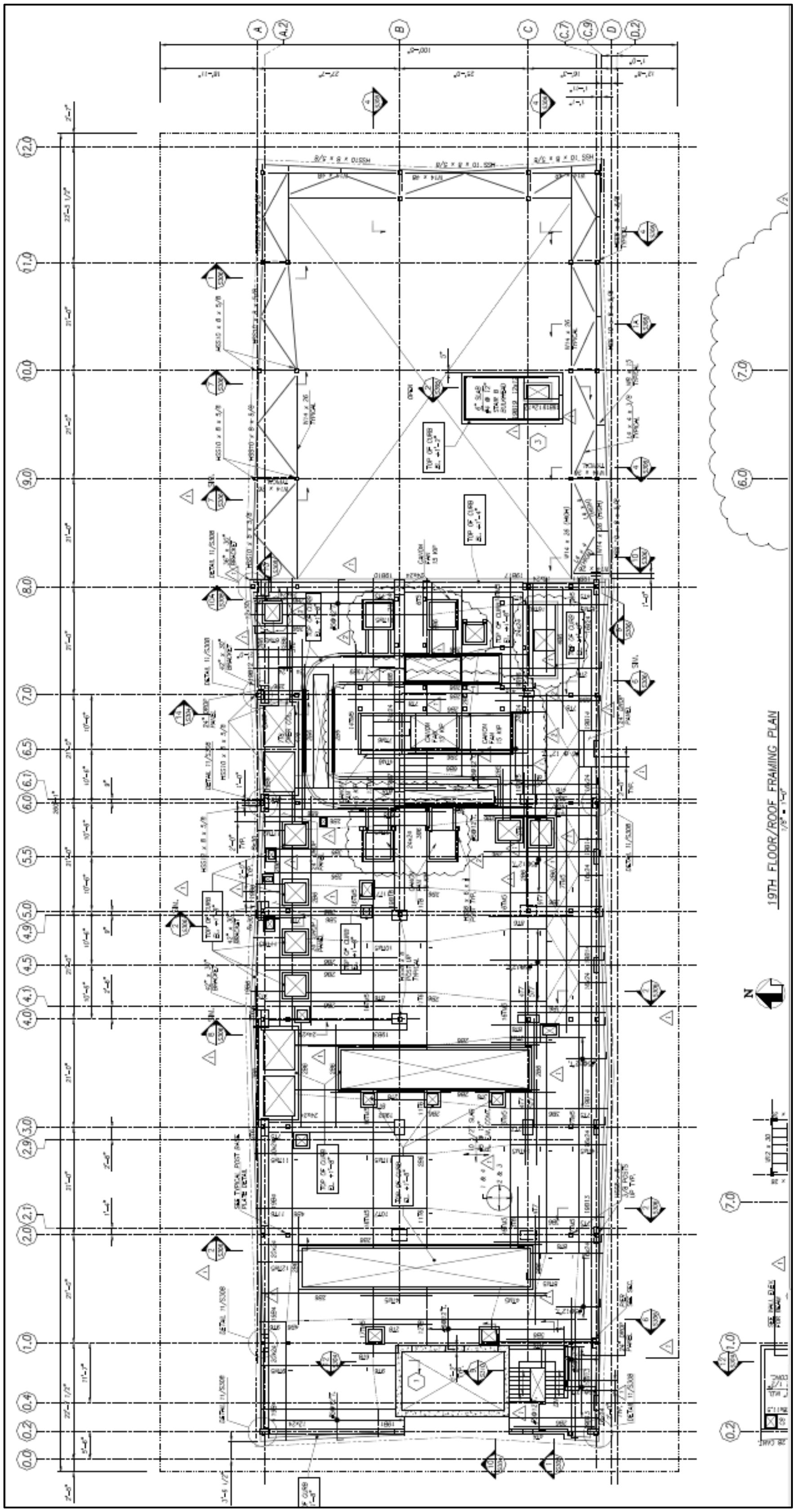


Figure 6: 19th Floor/Roof Framing Plan

Building Loads

Dead and Live Loads

There are a number of different occupancies within this building. The lower floors feature more business and office-like occupancies while the labs and mechanical rooms present more unique circumstances. The table below in Figure 7 shows some typical loads seen throughout the building. Unique loads for this building include the vivarium, which is located on the third basement level in the animal facility. It is an enclosed facility that acts as a recreation of an ecosystem for the study of plants and animals.

LOADING SCHEDULE (PSF)

LEVEL	SLAB	CEILING AND MECH.	PART'N.	MISC. DL.	LIVE LOAD	TOTAL LOAD	REMARKS
VIVARIUM	160	20	60	5	60	305	–
VIVARIUM MEZZ.	VARIES	10	–	15	50	VARIES	OR EQUIP.
B1	VARIES	30	10	15	150	VARIES	OR EQUIP.
LOADING DOCK	150	10	60	5	400	625	+4" TOPPING SLAB
SIDEWALK	150	10	–	50	600	810	–
LOBBY	140	10	–	25	100	275	–
AUDITORIUM	140	10	12	15	100	277	–
LABORATORY	160	10	12	5	60	247	–
OFFICES	160	10	12	5	50	237	–
MECHANICAL	160	30	12	5	150	357	OR EQUIP.
CORRIDOR	VARIES	10	12	5	100	VARIES	–
INTERSTITIAL	130	30	–	5	50	195	–
DATA CENTER	150	10	12	15	300	487	–
ROOF	130	30	–	15	30	205	OR EQUIP.
STORAGE	VARIES	10	12	5	150	VARIES	–

FACADE LOADS:
BLOCK AND BRICK 95 PSF
DOUBLE GLASS CURTAIN WALL 46 PSF
3

Figure 7: Loading Schedule

Floor System Analysis

Four different floor systems including the existing system were analyzed for this report. Calculations were performed for gravity loads and deflections were checked in order to arrive at preliminary sizes for the main structural components of the various systems. The Medical Research Building has four typical bay sizes (Figure 8). The exact bays used for design span between column lines 2.0 and 3.0. It was assumed that member sizes should be the same throughout the floor for ease of construction and, therefore, either Bay AB or the cantilever bay would control the design for both flexural strength and deflections.

Bay	N-S Dimension	E-W Dimension
AB	27'-7"	21'-0"
BC	25'-0"	21'-0"
CD	16'-3"	21'-0"
Cantilever	9'-8"	21'-0"

Figure 8: Typical Bay Dimensions

Existing Two-Way Flat Plate System

The existing two-way flat plate system was analyzed based on the Direct Design Method. The stipulations for the use of this method were met as shown in Appendix A. The design of this system consists of a 12.5" slab with typical top and bottom reinforcement of #5 bars at 12" O.C. with additional reinforcement in the column and middle strips where needed. The analysis showed the existing design to be adequate for flexure, wide beam and two-way punching shear, and deflections. For a typical detail of the existing flat plate system see Figure 9 below.

Advantages:

A two-way flat plate system results in a thin assembly which allows for lower floor to floor heights. This reduces constructions costs by decreasing the necessary vertical runs of MEP equipment. This system requires simple and reusable formwork, which minimizes the construction effort and cost. Due to the thickness of the slab provided, a fire rating of at least 3 hours can be expected. The shallowness of the slab reduces the weight of concrete, which also reduces cost.

Disadvantages:

Vibrations were not specifically analyzed, however, due to the thinness of the system, it may not be adequate to meet the strict vibration requirements of the laboratory spaces. In the existing design, vibration is dampened by the HSS members connecting every other floor.

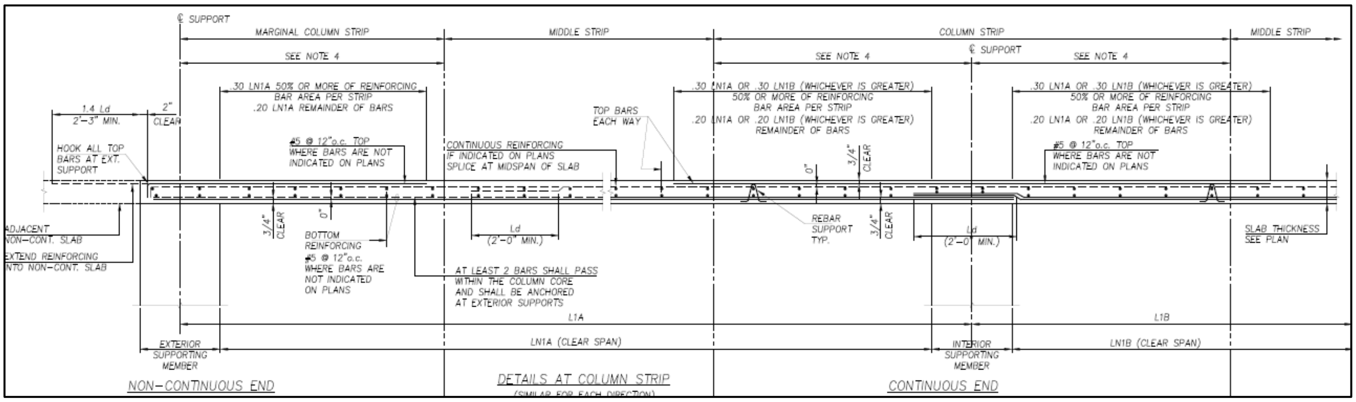


Figure 9: Typical Detail of Existing Flat Plate system

One-Way Pan Joist System

A one-way pan joist system is comprised of evenly spaced concrete joists, a one-way reinforced concrete slab cast integrally with the joists, and beams spanning between columns perpendicular to the joists. For this design, a 40" pan was used. The joists are 8" wide. The slabs, joists, and beams were designed according to ACI 318-08. The slab depth was determined to be 14" and the overall pan depth 26." Flexural reinforcement for the slab was determined to be #5 bars at 8" O.C. For the joists, the flexural reinforcement is two #8 bars.

Two beams were designed. One beam, which would be used for bays AB, BC, and CD, is 24"x26" while the other beam would be used in the cantilever bay and is 24"x38." Both beams have three #10 bars for flexural reinforcement. For the layout of the system, see Figure 10 below.

Advantages:

The advantages of a pan joist system are typically that the dead load is reduced by the pan voids, they are economical for long spans, and MEP equipment can be run neatly in between the joists. This pan joist system can be expected to have at least a 2 hour fire rating based on cover and thickness.

Disadvantages:

Due to the demands of this building, there are a number of flaws with a pan joist system. The overall depth of the system is much greater than a flat slab and due to the required depth of the slabs spanning between the joists, there isn't very much room for mechanical equipment. Another consequence of a deeper system is higher floor to floor heights in a building that is already nearly 300 feet tall. Formwork is complex and expensive. Deflections are also an issue with this system.

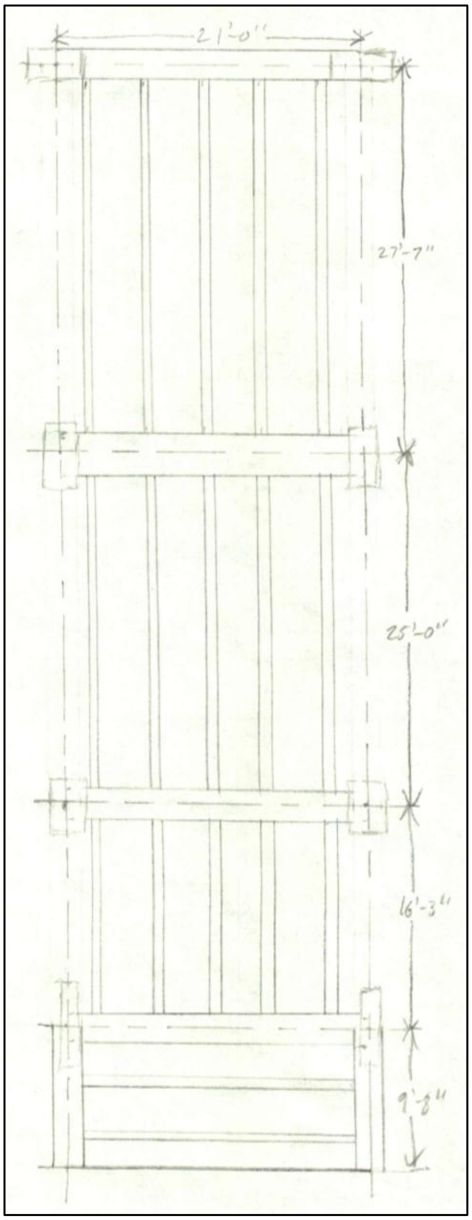


Figure 10: One-Way Pan Joist Layout

Banded-Beam System

A banded beam system consists of a uniform slab with thickened portions along column lines (usually in the long direction). These thickened portions are typically post-tensioned and called "band-beams." For this evaluation, preliminary sizes for the slabs and band-beams were arrived at using ultimate strength design. The one way slabs between the beams would follow the same design as the slabs of the one-way pan joist system (which utilized the "unit strip method" and could therefore be extrapolated for a longer span).The assumption was made that the band-beams would have a base equal to the column strip width of the two way slab, 10'-5." A height of three feet for the beams was used for a starting point for calculations which arrived at a design of twelve 7/16" Grade 250 strands spaced 6" O.C. This design would be adequate for flexure and deflections for all typical spans. For the band-beam layout, see Figure 11.

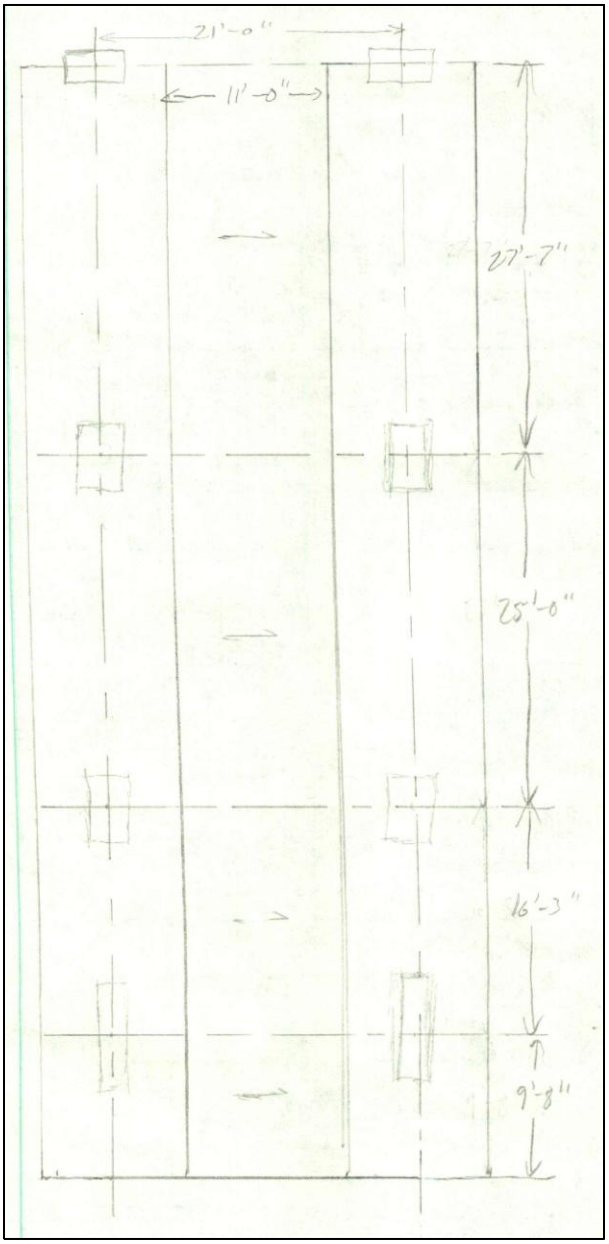


Figure 11: Banded-Beam System Layout

Advantages:

In this design the post-tensioning is adequate without needing any non-prestressed or post-tensioned reinforcement. Post-tensioning is useful for long spans and heavy loads by providing flexural strength and deflection control. The stiffness of this system diminishes vibrations. This system also requires simple and reusable formwork, which saves on cost. Band-beams can be considered two-way slabs for fire ratings, which means this particular system can be expected to achieve a rating of at least three hours.

Disadvantages:

This system features a lot of dead load from self-weight and concrete material. Consequences of the amount of material used include more weight on columns and the foundation as well as less room for MEP equipment.

Composite Deck and Beam System

For the composite deck a 3” 20 gage deck was chosen from the 2008 Vulcraft Steel Roof and Floor Deck catalog. It was determined that 3VLI20 with a 7.5” slab would be necessary based on deflections in the cantilever bay. It was assumed that there are two evenly spaced beams between each of the column lines. Using the 14^{th} edition of the AISC Steel Construction Manual beams were designed as W14x22’s with 24 shear studs, girders for bays AB, BC, and CD were designed as W16x31's with 26 shear studs, and the girders for the cantilever bay was designed as W21x44's with 32 shear studs. For the layout of the composite steel system, see Figure 12 below.

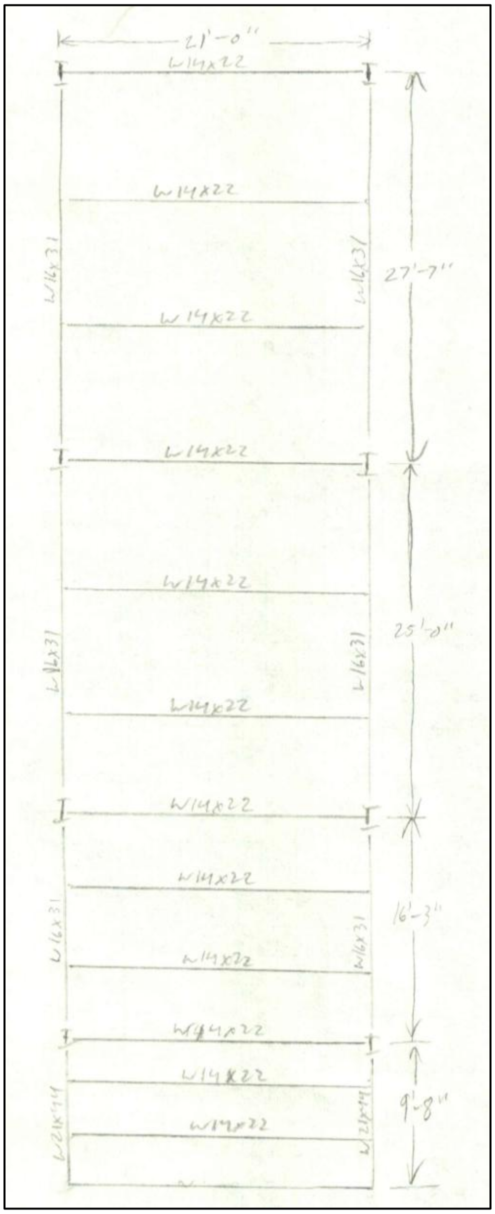


Figure 12: Composite Deck and Beam System Layout

Advantages:

The composite steel system is a lightweight system with standard construction methods for ease of construction and minimization of costs. This system also leaves plenty of room below the slab and between the beams for MEP equipment. Deflections are also moderate.

Disadvantages:

Steel is much more susceptible to vibrations than concrete, which is a major consideration for this building. Also, steel is more vulnerable to fire than concrete even with spray-on fire proofing. The overall depth of the system at almost two and a half feet would have an impact on floor to floor heights.

Conclusion

This study has given some insight into the feasibility of four floor systems for use in the Weill Cornell Medical Research Building. The systems investigated include the existing two-way flat plate, as well as one-way pan joist, banded-beam, and composite deck and beams.

All of the systems perform well when it comes to carrying the gravity loads analyzed. However, deflections are a major criteria for the viability of a floor system in this building due not only to the vibration requirements but also the cantilever bay. The banded-beam system performed incredibly well for deflections. The two-way flat plate system also had minimal deflections. The composite deck and beam system saw deflections that would be permissible by code but perhaps too great for this building. The one-way joist system showed the worst deflections.

Floor to floor height is also a concern in the Medical Research Building because of the MEP equipment running throughout the building as well as the number of stories of the building. Although total building height isn't necessarily a zoning issue in New York City, it is a cost issue because a taller building means more vertical runs of MEP equipment as well as more façade material. For this criterion, the two-way flat plate system provides the least depth of the four floor systems. The other systems are comparable as far as depth and space provided within the system.

From the results of this study, the composite steel and one-way joist systems are deemed the least feasible for this building. Both systems have high deflections, lower fire ratings, and would result in larger floor to floor heights. The one-way joist system should most likely be ruled out entirely, but the composite steel system could potentially be investigated further by additional design iterations and a vibration investigation to produce a more economical system and feasible system.

The two-way flat plate and banded-beam systems were the most viable. The existing two-way flat plate system appears to be the most feasible based on this study, reinforcing the decision by Severud Associates to use the system in the Medical Research Building. A flat plate slab provides the lowest floor to floor heights and the most freedom for the arrangement and coordination of MEP as well as simple construction methods with reusable formwork. The only possible disadvantage of the existing design is the solution for the cantilever slab which calls for a camber of 5/8" for most of the floors. Cambering is a delicate design tool because you can only really design a camber for dead load deflections (assuming the camber is successfully produced in the field).

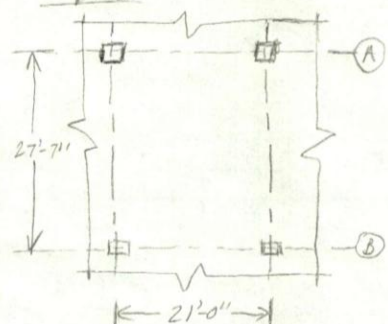
Another solution for this cantilever slab is post-tensioning, which is the definitive feature of the banded-beam system. The preliminary sizing of the system yielded excellent results for deflections but needed a lot of concrete and reinforcement to do so. The strength of the preliminary system also far exceeded the necessary strength requirements. Further design iterations might lead to a more economical and feasible system which closer rivals that of the existing two-way flat plate.

Appendix A: Two-Way Flat Plate System

Jonathan Coan | AE Senior Thesis | Existing System: Two-way flat plate slab | 1

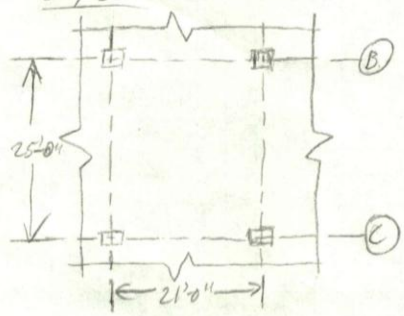
Two Way Flat Plate Slab

Bay AB:



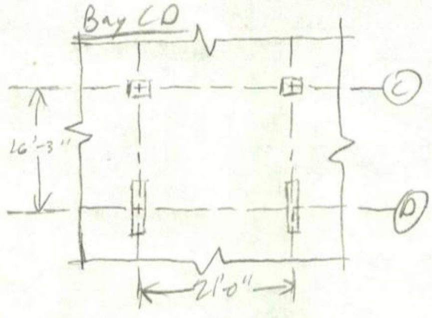
Columns: Row A – 24x20
Row B – 36x24

Bay BC:



Columns: 36x24

Bay CD



Columns: Row C – 36x24
Row D – 14X72

Direct Design Method

Requirements: ACI 318-08 §13.6.1

13.6.1.1
E-W Dir. > 3 spans
N-S Dir. = 3 spans + cantilever
ok

13.6.1.2
AB $\frac{\ell_2}{\ell_1} = \frac{27+\frac{7}{12}}{21} = 1.31 < 2$ ok

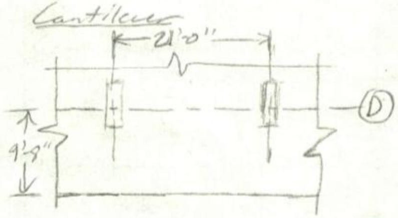
BC $\frac{\ell_2}{\ell_1} = \frac{25}{21} = 1.19 < 2$ ok

CD $\frac{\ell_2}{\ell_1} = \frac{21}{16.25} = 1.29 < 2$ ok

13.6.1.3
$(27+\frac{7}{12}) - 25 \le \frac{1}{3}(27+\frac{7}{12})$
$2.583 \le 9.19$ ok
$25 - 16.25 \le \frac{1}{3} 25$
$8.25 \le 8.33$ ok

13.6.1.5
This design only addresses gravity loads

Cantilever



Columns: 14X72

Notes: Assume f'c = 4000 psi

Bay AB:

Minimum Thickness: ACI 318-08 Table 9.5(c)
w/o drop panels
Ext. Panel
w/o edge beam
fy = 60,000psi

$$\frac{l_n}{30} = t_{min}$$

$$l_n = (27 + \tfrac{7}{12}) - \tfrac{1}{2}(\tfrac{20}{12}) - \tfrac{1}{2}(\tfrac{24}{12}) = 25.75 \text{ ft.}$$

$$t_{min} = \frac{25.75(12)}{30} = 10.3 \text{ in}$$

Existing System: t = 12.5 in > 10.3 in ok

Moments:
Loads: (From Loading Diagrams)
Dead: SDL = 27 psf
Selfweight = $\frac{12.5}{12}(150) = 156.25$ psf
Live: 60 psf

$$w_u = 1.2(27 + 156.25) + 1.6(60) = 316 \text{ psf}$$

ACI 318-08 §13.6.2

N-S Frame: $M_o = \frac{w_u l_2 l_n^2}{8} = \frac{.316(21)(25.75)^2}{8} = 550$ kip-ft

E-W Frame: $M_o = \frac{w_u l_2 l_n^2}{8} = \frac{.316(27\frac{7}{12})(21 - \frac{1}{2}(\frac{36}{12}) - \frac{1}{2}(\frac{44}{12}))^2}{8} = 340$ kip-ft

ACI 318-08 §13.6.3
13.6.3.3 – Ext. Edge Unrestrained

N-S Frame:
$M^-_{int} = .75(550) = 412.5$ kip-ft
$M^+ = .63(550) = 346.5$ kip-ft
$M^-_{ext} = 0(550) = 0$

E-W Frame:
$M^-_{int} = .75(340) = 255$ kip-ft
$M^+ = .63(340) = 214.2$ kip-ft

ACI 318-08 §13.6.4
$\alpha = 0$ (no beams)

N-S Frame:
−412.5 < 75% to col. strip = −309 kip-ft; 25% to mid. strip = −103 kip-ft
+346.5 < 60% to col. strip = +208 kip-ft; 40% to mid. strip = +139 kip-ft

E-W Frame:
−255 < 75% to col. strip = −191 kip-ft; 25% to mid. strip = −64 kip-ft
+214.2 < 60% to col. strip = +129 kip-ft; 40% to mid. strip = +86 kip-ft

Reinforcement Design:

N-S Frame: Col. Strip Width $= \frac{\ell_1}{2} = 10.5 ft = 126 in$ Mid. Strip = 10.5 ft = 126 in
E-W Frame: Col. Strip width $= \frac{\ell_1}{2} = 10.5 ft = 126 in$ Mid. Strip = 17.1 ft = 205 in
Slab t = 12.5 in Assume: #5 bars (Existing design)

$$d_{short} = 12.5 - .75 - \frac{.625}{2} = 11.4375 \text{ in}$$

$$d_{long} = 11.4375 - .625 = 10.8125 \text{ in}$$

N-S Frame:

Item	Column Strip M^-	Column Strip M^+	Middle Strip M^-	Middle Strip M^+
Moment (M_u) (k-ft)	-307	+208	-103	+139
Width of Col. Strip (b) / Width of MS	126"	126"	126"	126"
Effective Depth (d)	10.825"	10.8125"	10.8125"	10.8125"
$\frac{M_u(12)}{b}$ ($\frac{k-in}{in}$)	-29.4	+19.8	-9.81	+3.71
$M_n = M_u/\phi = \frac{M_u}{.9}$ (k-ft)	-343	+231	-114	+154
$R = \frac{M_n}{bd^2}$ ($\frac{lb}{in^2}$)	279	188	92.9	125
ρ (Table A.5a)	.0049	.0032	.0016	.0021
$A_s = \rho bd$ (in^2)	6.68	4.36	2.18	2.86
$A_{s,min} = .0018bt$	2.84	2.84	2.84	~~2.84~~
$N = \frac{A_s}{.31}$	21.5 ⇒ 22	14.06 ⇒ 15	9.16 ⇒ 10	9.22 ⇒ 10
$N_{min} = \frac{b}{2t}$	5.04 ⇒ 6	6	6	6

(Table A.4) $\rho_{max} = .0206$

$bd^2 = 4250$
Table A.5a from Nilson et al 2004

E-W Frame:

Item	Column Strip M^-	Column Strip M^+	Middle Strip M^-	Middle Strip M^+
Moment (M_u) (k-ft)	-191	+129	-64	+86
Width of Col. Strip (b) (in)	126	126	205	205
Effective Depth (d) (in)	11.4375	11.4375	11.4375	11.4375
$\frac{M_u(12)}{b}$ ($\frac{k-in}{in}$)	-18.1	12.3	-3.75	5.03
$M_n = M_u/\phi = \frac{M_u}{.9}$ (k-ft)	-212	+143	-71.1	+95.6
$R = \frac{M_n}{bd^2}$ ($\frac{lb}{in^2}$)	154	104	51.8	69.6
ρ (Table A.5a)	.0027	.0023	.0008	.0013
$A_s = \rho bd$ (in^2)	3.89	3.31	1.88	~~3.04~~
$A_{s,min} = .0018bt$	2.84	2.84	4.61	4.61
$N = \frac{A_s}{.31}$	12.5 ⇒ 13	10.7 ⇒ 11	14.9 ⇒ 15	15
$N_{min} = \frac{b}{2t}$	5.04 ⇒ 6	6	8.2 ⇒ 9	9

(Table A.4) $\rho_{max} = .0206$

Jonathan Coan | AE Senior Thesis | Existing System: Two Way Flat Plate Slab | 4

Shear Check

Wide Beam Action:

$w_u = 316$ psf

$a = 20$ in $d_{avg} = \frac{(11.4375 + 10.8125)}{2} = 11.125$ in

$\frac{(27+\frac{7}{12})}{2} - \frac{20}{2(12)} - \frac{11.125}{12} = 12.03$ ft

$V_u = (.316)(12.03)(21) = 79.8$ k

$V_n = V_c = 2\sqrt{f'_c}\, b_w d$
$= 2\sqrt{4000}(21)(12)(11.125)(\frac{1}{1000})$
$= 354.6$ kips

$\phi V_n = .75(354.6) = 266$ kips $> V_u = 79.8$k ok

Two Way Action (Punching Shear):

$d = 11.4375$ $d/2 = 5.719$ in

$V_u = w_u A = (.316)\left[(21)(27+\frac{7}{12}) - (\frac{20(44)}{144})\right] = 181$ kips

$b_o = 2(44) + 2(20) = 128$ in

$V_c = 4\sqrt{4000}(128)(11.4375)(\frac{1}{1000}) = 370$ kips

$\phi V_c = .75(370) = 278$ kips > 181 kips ok

Bay BC:

Minimum Thickness: ACI 318-08 Table 9.5(c)

w/o drop panels
Int. panel
$f_y = 60000\,psi$

$$\left.\right\} \frac{l_n}{33} = t_{min}$$

$$l_n = 25 - \frac{24}{12} = 23\,ft$$

$$t_{min} = \frac{23(12)}{33} = 8.4\,in$$

Existing System: $t = 12.5\,in > 8.4\,in$ ok

Moments:

Loads: Dead: $SDL = 27\,psf$

Self weight $= \frac{12.5}{12}(150) = 156.25\,psf$

Live: $60\,psf$

$$w_u = 1.2(27 + 156.25) + 1.6(60) = 316\,psf$$

ACI 318-08 §13.6.2

N-S Frame: $M_o = \frac{w_u l_2 l_n^2}{8} = \frac{.316(21)(23)^2}{8} = 439\,kip\text{-}ft$

E-W Frame: $M_o = \frac{w_u l_2 l_n^2}{8} = \frac{.316(25)(21 - \frac{34}{12})^2}{8} = 320\,kip\text{-}ft$

ACI 318-08 §13.6.3

N-S Frame: $M^- = .65(439) = 285.35\,kip\text{-}ft$

$M^+ = .35(439) = 153.65\,kip\text{-}ft$

E-W Frame: $M^- = .65(320) = 208\,kip\text{-}ft$

$M^+ = .35(320) = 112\,kip\text{-}ft$

ACI 318-08 §13.6.4

$\alpha = 0$ (no beams)

N-S Frame: -285.35
- 75% to col. strip $= -214\,kip\text{-}ft$
- 25% to mid. strip $= -71.3\,kip\text{-}ft$

$+153.65$
- 60% to col. strip $= +92.2\,kip\text{-}ft$
- 40% to mid. strip $= +61.5\,kip\text{-}ft$

E-W Frame: -208
- 75% to col. strip $= -156\,kip\text{-}ft$
- 25% to mid. strip $= -52\,kip\text{-}ft$

$+112$
- 60% to col. strip $= +67.2\,kip\text{-}ft$
- 40% to mid. strip $= +44.8\,kip\text{-}ft$

Shear

Wide Beam

$w_u = 316 psf$

$a = 2 ft$ $\quad d_{avg} = 11.125 in$

$\frac{(25)}{2} - \frac{2}{2} - \frac{11.125}{12} = 10.57 ft$

$V_u = (.316)(10.57)(21) = 70.1 Kips$

$V_n = V_c = 2\sqrt{4000}(21)(12)(11.125)(\frac{1}{1000})$
$= 354.6 Kips$

$\phi V_n = .75(354.6) = 266 Kips > V_u = 70.1 Kips$ OK

Two Way:

$d = 11.4375$

$V_u = w_u A = (.316)\left[(21)(25) - \frac{(24)(36)}{144}\right] = 164 Kips$

$b_o = 2(24)+2(36) = 120 in$

$V_c = 4\sqrt{4000}(120)(11.4375)(\frac{1}{1000}) = 347 Kips$

$\phi V_c = .75(347) = 260 Kips > 164k$ OK

Bay CD

Minimum Thickness:

w/o drop panels, Int. panels, $f_y = 60000$ psi $\Bigg\}\ \frac{l_n}{33} = t_{min}$

$l_n = 21 - \frac{14}{12} = 19.8$ ft

$t_{min} = \frac{19.8(12)}{33} = 7.2$ in

Existing System: $t = 12.5$ in > 7.2 in ok

Moments:

Loads: Dead: SDL = 60 psf
Selfweight = 156.25 psf
Live: 100 psf

$w_u = 1.2(60 + 156.25) + 1.6(100) = 420$ psf

N-S Frame: $M_o = \frac{.42(21)\left(16 + \frac{3}{12} - \frac{1}{2}\left(\frac{72}{12}\right) - \frac{1}{2}\left(\frac{24}{12}\right)\right)^2}{8} = 165$ kip-ft

E-W Frame: $M_o = \frac{.42(16.25)(19.8)^2}{8} = 334.5$ kip-ft

N-S Frame: $M^- = .65(165) = 107.25$ kip-ft
$M^+ = .35(165) = 57.75$ kip-ft

E-W Frame: $M^- = .65(334.5) = 117$ kip-ft
$M^+ = .35(334.5) = 217$ kip-ft

N-S Frame: -107.25 < 75% to col. strip = -80.4 kip-ft; 25% to mid. strip = -26.8 kip-ft

$+57.75$ < 60% to col. strip = $+34.7$ kip-ft; 40% to mid. strip = $+23.1$ kip-ft

E-W Frame: -117 < 75% to col. strip = -87.8 kip-ft; 25% to mid. strip = -29.3 kip-ft

$+217$ < 60% to col. strip = $+130.2$ kip-ft; 40% to mid. strip = $+86.8$ kip-ft

Shear

wide Beam:

$w_u = 420$ psf

$a = 3$ ft $\quad d_{avg} = 11.125$ in

$\frac{21}{2} - \frac{3}{2} - \frac{11.125}{12} = 8.07$ ft

$V_u = (.42)(8.07)(16.25) = 55$ Kips

$V_n = V_c = 2\sqrt{4000}(16.25)(12)(11.125)(\frac{1}{1000})$
$= 274$ Kips

$\phi V_n = .75(274) = 205$ kips > 55 k OK

Two Way:

$d = 11.4375$

$V_u = w_u A = (.316)\left[(16.25)(21) - \left(\frac{1}{2}\left(\frac{24(36)}{144}\right) + \frac{1}{2}\left(\frac{14(72)}{144}\right)\right)\right]$
$= 106$ kips

$b_o = 120$ in

$V_c = 4\sqrt{4000}(120)(11.4375)(\frac{1}{1000}) = 347$ kips

$\phi V_c = .75(347) = 260$ kips > 106 k OK

Deflections
from Wight & MacGregor 2009)

Assume: - Avg % of mom. taken by col. strip = 67.5%
- DL consists of selfweight of slab

Immediate Defl
Column Strip:

$w_D = \frac{12.5}{12}(21)(150)(.675) = 2215$ plf

Table 9-2:

$\Delta_{D_{max}} = .0026 \frac{wl^4}{EI_g}$

$= .0026 \frac{(2.215)(21)^4(1728)}{3605(41015)}$

$I_g = \frac{1}{12}bh^3 = \frac{1}{12}(21)(12)(12.5)^3 = 41015 \text{ in}^3$

$E_c = 57000\sqrt{f'_c} = 3605$ ksi

$\Delta_{D_{max}} = .0131$ in

Middle strip:

$w_D = \frac{12.5}{12}(21)(150)(.325) = 1066$ plf

$\Delta_{D_{max}} = .0026 \frac{(1.066)(21)^4(1728)}{3605(41015)}$

$\Delta_{D_{max}} = .0063$ in

Additional DL Defl. After a Long Time:
pg 742: $\lambda = 3$

Col. Strip: $\Delta_{long\ term} = 3(.0131) = .0393$ in
Mid. Strip: $\Delta_{long\ term} = 3(.0063) = .0189$ in

Immediate LL Deflections:
LL = 60 psf
Column Strip:

$w_L = 60(21)(.675) = 850.5$ plf

Table 9-2:

$\Delta_{L_{max}} = .0048 \frac{(.851)(21)^4(1728)}{(3605)(41015)} = .0093$ in

Middle Strip:

$w_L = 60(21)(.325) = 410$ plf

$\Delta_{L_{max}} = .0048 \frac{(.41)(21)^4(1728)}{(3605)(41015)} = .0045$ in

Worst case: $\Delta = .0093 + .0393 = .0486$ in

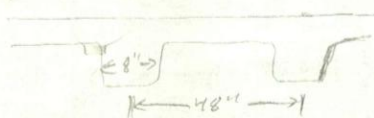
$\frac{l}{480} = \frac{21(12)}{480} = .525$ in $\gg .0486$ in ok

Appendix B:
One-Way Pan Joist System

Jonathan Coan | AE Senior Thesis | One-way Pan Joist System | 1

Use: 48/8 joist space

ACI 318-08 § 8.3.4
Design as slabs & beams



Assume: joists span N-S dir.

Slab Design:

Min. Thickness: ACI 318-08 Table 9.5(a)

Bay AB: $h_{min} = \frac{\ell}{24} = \frac{(27+\frac{7}{12})12}{24} = 13.79$ in $\Rightarrow h_{min} = 14$ in

Bay BC: $h_{min} = \frac{\ell}{28} = \frac{25(12)}{28} = 10.71$ in $\Rightarrow h_{min} = 11$ in

Bay CD: $h_{min} = \frac{\ell}{28} = \frac{16.25(12)}{28} = 7$ in

Cantilever: $h_{min} = \frac{\ell}{10} = \frac{9.67(12)}{10} = 11.6$ in $\Rightarrow h_{min} = 12$ in

$h = 14$ in $\Rightarrow$ Assume: $d = 12.5$ in

Loads: Dead: SDL = 27 psf

Self = $\frac{14}{12}(150) = 175$ psf

Live: 60 psf

$w_u = 1.2(27+175) + 1.6(60) = 338.4$ psf

Assume: ends fixed: simply supported

$M_u = \frac{w_u \ell^2}{12} = \frac{.3384(27+\frac{7}{12})^2}{12} = 21.5$ k-ft/ft

$A_s = \frac{M_u}{4d} = \frac{21.5}{4(12.5)} = .43$ in²/ft $\Rightarrow$ Try #5 @ 8" o.c. ($A_s = .46$ in²/ft)

Actual $d = 14 - .75 - \frac{5}{16} = 12.94$ in

Assume $\varepsilon_s > \varepsilon_y$:

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.46(60)}{.85(4)(12)} = .676$ in $\qquad c = \frac{a}{\beta_1} = .795$ in

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{.003}{.795}(12.94 - .795) = .046 > .005 \Rightarrow \phi = .9$

$\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = .9(.46)(60)(12.94 - \frac{.676}{2})$

$\phi M_n = 26$ k-ft/ft $> M_u = 21.5$ k-ft/ft OK

Use: $h = 14$ in, #5 @ 8" o.c.

Joist Design:

Assume: $h = 26$ in $\Rightarrow d \approx 23.5$ in

Loads: from slab: $w_u = 338.4$ psf (4 ft) = 1353.6 plf

Joist trib. width

Joist Selfweight = $1.2\left[\frac{26(8)}{144}(150)\right] = 260$ plf

$w_{u_{JST}} = 1614$ plf $\qquad M_u = \frac{w_u \ell^2}{12} = \frac{1.614(27+\frac{7}{12})^2}{12} = 102$ k-ft

$A_s = \frac{M_u}{4d} = \frac{102}{4(23.5)} = 1.09$ in² $\qquad$ Use: (2) #8 ($A_s = 2(.79) = 1.58$ in²)

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{1.58(60)}{(.85)(4)(8)} = 3.49$ in $\qquad c = 4.1$ in $\qquad \varepsilon_s = \frac{.003}{4.1}(23.5 - 4.1) = .014 > .005$ OK

$\phi M_n = .9(1.58)(60)(23.5 - \frac{3.49}{2})$

$\phi M_n = 155$ kip-ft $> M_u = 102$ k-ft OK

Joist: $\boxed{8 \times 26 \text{ w/ (2) \#8}}$

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Area & Spacing Requirements:

$$A_{s,min} = \frac{3\sqrt{4000}}{60{,}000}(8)(23.5) \geq \frac{200(8)(23.5)}{60000}$$
$$A_{s,min} = .595\,in^2 \geq .627\,in^2$$
$$A_{s,min} = .627\,in^2 < 1.58\,in^2 \quad \text{ok}$$

$$\rho_{max} = .85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.007}\right) = .0206$$
$$A_{s,max} = .0206(8)(23.5) = 3.87\,in^2 > 1.58\,in^2 \quad \text{ok}$$

Table A.8 Min # of bars $(b_w = 8) \Rightarrow$ 1 bars < 2 ok
Table A.7 Max # of bars $(b_w = 8) \Rightarrow$ 2 bars $\geq$ 2 ok

$$b_{min} = 2(1.5) + \underbrace{2(.5)}_{stirrup} + \underbrace{2(1) + 1.33}_{min\ spacing} = 7.33\,in < 8\,in \quad \text{ok}$$

Deflections:

Immediate: $\bar{y} = \frac{bh\left(\frac{h}{2}\right) + (n-1)A_s d}{bh + (n-1)A_s} = \frac{8(26)\left(\frac{26}{2}\right) + (2-1)(1.58)(23.5)}{8(26) + (2-1)(1.58)} = 13.08\,in$

$$I_{ut} = \frac{bh^3}{12} + bh\left(\frac{h}{2} - \bar{y}\right)^2 + (n-1)A_s(d - \bar{y})^2$$
$$= \frac{(8)(26)^3}{12} + (8)(26)\left(\frac{26}{2} - 13.08\right)^2 + (2-1)(1.58)(23.5 - 13.08)^2$$
$$= 5506\,in^4$$

$$E_c = 57000\sqrt{4000} = 3605\,ksi$$

$$M_{cr} = \frac{f_r I_{ut}}{y_t} = \frac{7.5\sqrt{4000}(5506)}{23.5 - 4.1} = 11.22\,kip\text{-}ft$$
$$w_{D+L} = 808 + 240 = 1048\,plf$$
$$M_a = M_{D+L} = \frac{1048\left(27 + \frac{7}{12}\right)^2}{12} = 66.45\,kip\text{-}ft$$

$$\rho = \frac{A_s}{bd} = \frac{1.58}{(8)(23.5)} = .0084$$

$$k = -\rho n + \sqrt{(\rho n)^2 + 2\rho n} = -.0084(2) + \sqrt{(.0084(2))^2 + 2(.0084)(2)} = .167$$
$$kd = 3.93\,in$$
$$I_{cr} = \frac{b(kd)^3}{12} + b(kd)\left(\frac{kd}{2}\right)^2 + nA_s(d - kd)^2$$
$$= \frac{8(3.93)^3}{12} + 8(3.93)\left(\frac{3.93}{2}\right)^2 + 2(1.58)(23.5 - 3.93)^2$$
$$= 1372\,in^4$$
$$I_g = \frac{1}{12}bh^3 = \frac{1}{12}(8)(26)^3 = 11717\,in^4$$

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \leq I_g$$
$$= \left(\frac{11.22}{66.45}\right)^3(11717) + \left[1 - \left(\frac{11.22}{66.45}\right)^3\right](1372) \leq 11717$$
$$= 1422\,in^4$$

Table 10-3 ⇒ k = .6

$$\Delta_i = K\left(\frac{5}{48}\right)\frac{M_a \ell^2}{E_c I_e} = .6\left(\frac{5}{48}\right)\frac{(66.45)\left(27 + \frac{7}{12}\right)^2(1728)}{3605(1422)} = 1.07\,in > \frac{\ell}{360} \quad \text{No Good}$$

Beam Design:

Joists provide point loads on beams
>5 point loads => model as distributed load

Loads:

$w_u = 1.614\,klf(27+\frac{7}{12}ft)(5)/21ft = 10.6\,klf$

$M_u = \frac{w_u l_n^2}{11} = \frac{10.6(21-3)^2}{11} = 312\ kip\text{-}ft$

Assume $\rho = .0125$, $b = \frac{4}{3}d$

$M_u \le \frac{bd^2}{20}$

$20(312) = bd^2$

$6240 = \frac{4}{3}d^3$

$d = 19.93\,in$

Because of joist depth, use: $d = 23.5\,in$, $h = 26\,in$, $b = 24\,in$

$A_s = \frac{M_u}{4d} = \frac{312}{4(23.5)} = 3.32\,in^2$ Try: (3) #10 ($A_s = 3(1.27) = 3.81\,in^2$)

$c = \frac{3.81(60)}{.85(4)(24)} = 2.8\,in \Rightarrow c = 3.29$

$\phi M_n = .9(3.81)(60)(23.5 - \frac{2.8}{2})$

$\phi M_n = 378.9\ kip\text{-}ft > 312\ k\text{-}ft$ ok

$\varepsilon_s = \frac{.003}{3.29}(23.5-3.29) = .018 > .005$ ok

$\boxed{\text{Beam: } 24\text{x}26 \text{ w/ (3) \#10}}$

Area & Spacing Requirements:

$A_{s,min} = \frac{3\sqrt{4000}}{60000}(24)(23.5) \ge \frac{200(24)(23.5)}{60000}$

$A_{s,min} = 1.78\,in^2 \ge 1.88\,in^2$

$A_{s,min} = 1.88\,in^2 < 3.81\,in^2$ ok

$\rho_{max} = .85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.007}\right) = .0206$

$A_{s,max} = .0206(24)(23.5) = 11.6\,in^2 > 3.81\,in^2$ ok

Table A.8 min # of bars ($b_w = 24$) => 3 bars ok

Table A.7 max # of bars ($b_w = 24$) => 8 bars > 3 ok

min bar spacing = max $\begin{cases} 1'' \\ d_b \\ \frac{4}{3}A_{agg} \end{cases} = 1.33\,in$

$b_{min} = 2(1.5) + \underbrace{2(.5)}_{stirrup} + 3(1.27) + \underbrace{2(1.33)}_{min\ spacing} = 10.47\,in < 24\,in$ ok

Deflections:

$\bar{Y} = \frac{24(26)(13) + (3-1)(3.81)(23.5)}{24(26) + (3-1)(3.81)} = 13.13\,in$

$I_{ut} = \frac{(24)(26)^3}{12} + 24(26)\left(\frac{26}{2} - 13.13\right)^2 + (3-1)(3.81)(23.5-13.13)^2$

$= 35982\,in^4$

$M_{cr} = \frac{7.5\sqrt{4000}(35982)}{13.13} = 108\ kip\text{-}ft$

$\rho = \frac{A_s}{bd} = \frac{3.81}{24(23.5)} = .00676$

$I_g = \frac{1}{12}bh^3 = \frac{1}{12}(24)(26)^3 = 35,152\,in^4$

$w_{D+L} = 6.88\,klf$

$M_a = \frac{6.88(27+\frac{7}{12})^2}{12} = 436\ kip\text{-}ft$

$k = -.00676(3) + \sqrt{(.00676(3))^2 + 2(.00676)(3)}$

$= .182 \Rightarrow kd = 4.28\,in$

$I_{cr} = \frac{24(4.28)^3}{12} + 24(4.28)\left(\frac{4.28}{2}\right)^2 + 3(3.81)(23.5-4.28)^2$

$= 4850\,in^4$

$I_e = \left(\frac{108}{436}\right)^3(35152) + \left[1 - \left(\frac{108}{436}\right)^3\right](4850) \le 35152 \Rightarrow I_e = 5311\,in^4$

$\Delta_i = .6\left(\frac{5}{48}\right)\frac{(436)(27+\frac{7}{12})^2(1728)}{3605(5311)} = 1.87\,in > \frac{l}{360}$ No Good

Cantilever Bay Beam Design

Assume: Joists span E-W dir

Deflection Check:

Loads: Dead: $SDL = 57(21)(4) = 4.788$ kips

Slab self-weight $= \frac{14}{12}(150)(21)(4) = 14.7$ kips

Joist self-weight $= \frac{22(8)}{144}(150)(21) = 2.1$ kips

$P_D = 21.588$ kips

Live: $50(21)(4) = 4.2$ kips

$P_L = 4.2$ kips

$P_{D+L} = 21.588 + 4.2 = 25.788$ kips

$M_a = 25.788(9+\frac{4}{12}) + 25.788(5+\frac{4}{12}) + \frac{1}{2}(25.788)(\cancel{1}+\frac{4}{12}) = 395$ kip-ft

$\Delta_{allow} = \frac{l}{360} = \frac{(9+\frac{8}{12})(12)}{360} = .322$ in

Table 10.3 ⇒ $K = 2.4$

$\Delta_i = K\left(\frac{5}{48}\right)\frac{M_a l^2}{E_c I_e}$

$.322 = 2.4\left(\frac{5}{48}\right)\frac{(395)(9+\frac{8}{12})^2(1728)}{3605\, I_e} \Rightarrow I_{e_{req}} = 13{,}736\ in^4$

Assume: $I_e \approx 13.5\%(I_g)$ ⇒ $I_{g_{req}} = 101{,}748\ in^4$

Assume: $b = 24''$ $\quad 101{,}748 = \frac{1}{12}bh^3$

$h = 37.05$ ⇒ use $h = 38$ in

Try: 24" x 38" beam

LRFD Design

Loads: $P_U = 1.2(21.588) + 1.6(4.2) = 32.6$ kips

$M_u = 32.6(9+\frac{4}{12}) + 32.6(5+\frac{4}{12}) + \frac{1}{2}(32.6)(1+\frac{4}{12}) = 500$ kip-ft

$A_s = \frac{M_u}{4d} = \frac{500}{4(35.5)} = 3.52\ in^2$ $\quad$ Try: (3) #10 ($A_s = 3.81\ in^2$)

$d = 38 - 1.5 - .5 - \frac{1.27}{2} = 35.365$

$a = \frac{3.81(60)}{.85(4)(24)} = 2.8$ in ⇒ $c = 3.29$

$\varepsilon_s = \frac{.003}{3.29}(35.365 - 3.29)$

$= .029 > .005$ ok

$\phi M_n = .9(3.81)(60)(35.365 - \frac{2.8}{2})$

$\phi M_n = 582$ kip-ft $> M_u = 500$ k-ft ok

Area & Spacing

$A_{s_{min}} = \frac{3\sqrt{4000}}{60000}(24)(35.365) \geq \frac{200(24)(35.365)}{60000}$

$A_{s_{min}} = 2.68\ in^2 \geq 2.83\ in^2$

$A_{s_{min}} = 2.83\ in^2 < 3.81\ in^2$ ok

$\rho_{max} = .85(.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.007}\right) = .0206$

$A_{s,max} = .0206(24)(35.365) = 17.5\ in^2 > 3.81\ in^2$ ok

Re-check Deflections:

$$\bar{y} = \frac{24(38)(\frac{38}{2}) + (3-1)(3.81)(35.365)}{24(38) + (3-1)(3.81)} = 19.14 \text{ in}$$

$$I_{ut} = \frac{24(38)^3}{12} + (24)(38)(\frac{38}{2} - 19.14)^2 + (3-1)(3.81)(35.365 - 19.14)^2$$
$$= 111768 \text{ in}^4$$

$$M_{cr} = \frac{7.5\sqrt{4000}(111768)}{35.365 - 3.29} = 137.7 \text{ kip-ft}$$
$$M_i = 395 \text{ k-ft}$$

$$\rho = \frac{3.81}{24(35.365)} = .0045$$
$$K = -.0045(3) + \sqrt{(.0045(3))^2 + 2(.0045)(3)} = .151$$
$$Kd = 5.35 \text{ in}$$

$$I_{cr} = \frac{24(5.35)^3}{12} + 24(5.35)(\frac{5.35}{2})^2 + 3(3.81)(35.365 - 5.35)^2$$
$$= 11522 \text{ in}^4$$

$$I_g = \frac{1}{12}(24)(36)^3 = 109{,}744 \text{ in}^4$$

$$I_e = (\frac{137.7}{395})^3(109744) + [1 - (\frac{137.7}{395})^3](11522) \le 109744$$
$$I_e = 15{,}683 \text{ in}^4$$

$$\Delta_i = 2.4(\frac{5}{48})\frac{(395)(9+\frac{2}{3})^2(1728)}{3605(15683)} = .282 \text{ in} < \frac{l}{360} = .322 \text{ in} \quad \underline{OK}$$

Cantilever Beam: 24"x38" w/(3) #10

Appendix C:
Banded-Beam System

Jonathan Coan | AE Senior Thesis | Band Beam System | 1

Bay AB: Span: 27'-7"

$b = 10'\text{-}6''$ (col. strip width)

Assume: $h = 36$ in $\Rightarrow d_p = 33$ in

Loads: Dead: SDL = 27 psf (21') = 567 plf — trib width

Slab Self Weight $= \frac{14}{12}(150)(10.5) = 1837.5$ plf — $w_D = 7130$ plf

Beam Self Weight $= \frac{36}{12}(10.5)(150) = 4725$ plf

Live: $60(21) = 1260$ plf

$$w_u = 1.2(7.13) + 1.6(1.26) = 10.57 \text{ klf}$$

Assume: ends fixed

$$M_u = \frac{w_u l_n^2}{12} = \frac{10.57(27+\frac{7}{12})^2}{12} = 670 \text{ kip-ft}$$

Beam Design: Assume: Gr 250 STL ($f_{pu} = 250{,}000$ psi), $f_{pc} = 150{,}000$ psi, bonded tendons

$f_{py} = 215{,}000$ psi

$f'_c = 4000$ psi

$$\rho_p = \frac{A_p}{36(10.5)(12)} = .0002205 A_p \qquad \frac{f_{py}}{f_{pu}} = \frac{215}{250} = .86 > .85 \Rightarrow \gamma_p = .4$$

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\frac{\rho_p f_{pu}}{f'_c}\right)\right] = 250{,}000\left[1 - \frac{.4}{.85}\left(\frac{.000205 A_p (250{,}000)}{4000}\right)\right]$$

$$f_{ps} = 250000 - 1507 A_p \quad < f_{py}$$

$$a = \frac{A_p f_{ps}}{.85 f'_c b} = \frac{A_p(250000 - 1507 A_p)}{.85(4000)(10.5)(12)} = \frac{250{,}000 A_p - 1507 A_p^2}{428400}$$

$$M_u \le \phi M_n = \phi A_p f_{ps}\left(d_p - \frac{a}{2}\right)$$

$$670(12) = .9 A_p (250{,}000 - 1507 A_p)\left(33 - \frac{250{,}000 A_p - 1507 A_p^2}{2(428400)}\right)$$

Try: 12 – $\frac{7}{16}$" Dia. Strands @ 6" o.c.

$$A_p = 10.5(2)(12)(.108) = 27.2 \text{ in}^2$$

$\rho_p = .005576$

$f_{ps} = 209010$ psi $< f_{py}$

$a = 13.3$ in $\Rightarrow c = 15.6$ in

$$\varepsilon_s = \frac{.003}{15.6}(33 - 15.6) = .0033 \Rightarrow \phi = .65 + .25\left(\frac{.0033 - .002}{.005 - .002}\right) = .758$$

$\phi M_n = 9467$ kip-ft > 670 k-ft ok

ACI 318-08 §18.9.2

$$A_{s,min} = .004 A_{ct} = .004(10.5)(12)(15) = 7.56 \text{ in}^2 < 27.2 \text{ in}^2 \text{ ok}$$

Deflections:

$$\bar{y} = \frac{(126)(36)(\frac{36}{2}) + (20)(27.2)(33)}{126(36) + 20(27.2)} = 19.6 \text{ in}$$

$$I_{ut} = \frac{126(36)^3}{12} + 126(36)(18-19.6)^2 + 20(27.2)(33-19.6)^2$$
$$= 599{,}181 \text{ in}^4$$

$$M_{cr} = \frac{7.5\sqrt{4000}\,(599181)}{33-15.6} = 1361 \text{ kip-ft}$$

$$I_g = \frac{126(36)^3}{12} = 489\,888 \text{ in}^4$$

$$w_{D+L} = 7.13 + 1.26 = 8.39 \text{ klf}$$

$$M_a = \frac{8.39(27+\frac{7}{12})^2}{12} = 532 \text{ kip-ft}$$

$$\rho = \frac{27.2}{126(36)} = .006$$

$$k = -.006(20) + \sqrt{(.006(20))^2 + 2(.006)(20)} = .384 \Rightarrow kd = 12.7 \text{ in}$$

$$I_{cr} = \frac{126(12.7)^3}{12} + 126(12.7)\left(\frac{12.7}{2}\right)^2 + 21(27.2)(33-12.7)^2$$
$$= 321418 \text{ in}^4$$

$$I_e = \left(\frac{1361}{532}\right)^3 (489888) + \left[1 - \left(\frac{1361}{532}\right)^3\right](321418) \leq 489888$$
$$= 3{,}172{,}151 \text{ in}^4 \leq 489888$$

$$I_e = I_g = 489{,}888 \text{ in}^4$$

$$\Delta_i = .6\left(\frac{5}{48}\right)\frac{532(27+\frac{7}{12})^2}{3605(489888)} = .0000143 \text{ in} < \frac{\ell}{360} \quad \underline{\text{ok}}$$

Use: $10.5' \times 3'$ w/ (12) $\frac{7}{16}''$ strands @ 6" o.c.

Cantilever Bay: Span: 9'-8"

$b = 10'-6''$

Assume: $h = 36 \text{ in} \Rightarrow d_p = 33 \text{ in}$

Loads: Dead: $SDL = 57(21) = 1197 \text{ plf}$

Slab-self weight $= \frac{14}{12}(150)(10.5) = 1837.5 \text{ plf}$ $\qquad w_D = 7760 \text{ plf}$

Beam self weight $= \frac{36}{12}(10.5)(150) = 4725 \text{ plf}$

Live: $50(21) = 1050 \text{ plf}$

$$w_u = 1.2(7.76) + 1.6(1.05) = 10.99 \text{ klf}$$

$$M_u = \frac{10.99(9+\frac{8}{12})^2}{12} = 86 \text{ kip-ft}$$

Use: $10.5' \times 3'$ w/ (12) $\frac{7}{16}''$ strands @ 12" o.c.

Appendix D:
Composite Deck and Beam System

Jonathan Coan | AE Senior Thesis | Composite Deck + Beams | 1

Composite Decking – 3"-20 Gage Composite Deck

N-S x E-W [using Vulcraft Steel Roof and Floor Deck (2008)]

Bay AB (27'-7" x 21")

Superimposed Live Load = LL + SDL = 60+27 = 87 psf

Assume beams spaced approx. 9'-2" ⇒ 3 spans (N-S) for Bay AB

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3VLI20 w/5" slab: SLL_{max} = 150 psf > 87 psf ok

Max Unshored Clr span = 13'-4" > 9'-2" ⇒ No shoring needed

Total Depth = 5 in

topping = 2 in

Self-weight = 45 psf

Assume concrete f'_c = 4000 psi

Composite Beams

[using AISC Steel Construction Manual 14th ed.]

Bay AB

Loads: Dead: slab/deck: 45 psf, SDL: 27 psf > 72 psf

Live: 60 psf

Assume beams spaced 9.194'

$$w_u = (1.2(72) + 1.6(60))(9.194) = 1677 \text{ plf}$$

$$M_u = \frac{w_u l^2}{8} = \frac{1.677(21)^2}{8} = 93 \text{ kip-ft}$$

$$b = \min \begin{cases} \text{span}/8 = \frac{21(12)}{8} = 31.5 \text{ in} \leftarrow \text{controls} \\ \frac{1}{2}\text{ adj. width} = \frac{9.194(12)}{2} = 55 \text{ in} \end{cases}$$

b_{eff} = 2b = 63 in

Assume a=1: $a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} \overset{a=1}{\Rightarrow} \Sigma Q_{n,req} = .85(4)(63) = 214.2$ kips

$Y_2 = 5 - \frac{1}{2} = 4.5$ in

From Table 3-19 (w/ Y_2 = 4.5, ΣQ_n < 214)

Try: W14x22 w/ ΣQ_n = 199 kips & M = 233 kip-ft

$a = \frac{199}{.85(4)(63)} = .93$ in < 1" ok

d = 13.7 in

A_s = 6.49 in²

Studs: $\frac{199}{17.1} = 11.7 \Rightarrow 12$

24 studs per bm

$$M_n = A_s F_y \left(\frac{d}{2} + t - \frac{a}{2}\right)$$

$$\phi M_n = .9(6.49)(50)\left(6.85 + 5 - \frac{.93}{2}\right)$$

ϕM_n = 222 kip-ft > M_u = 93 k-ft ok

Use: W14x22 w/24 studs

Deflections

Wet Concrete Deflection:

$w_{wc} = 45(9.194) + 22 = 436$ plf

$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(.436)(21)^4(1728)}{384(29000)(199)} = .331$ in

$\Delta_{wc_{max}} = \frac{l}{240} = \frac{21(12)}{240} = 1.05$ in $> .33$ in ok

Live Load Deflection:

$w_{LL} = 60(9.194) = 552$ plf

$I_{LB} = 577$ (Table 3-20 w/ $Y_2 = 4.5$ & $\Sigma Q_n = 199$)

$\Delta_{LL} = \frac{5(.552)(21)^4(1728)}{384(29000)(517)} = .384$ in

$\Delta_{LL_{max}} = \frac{21(12)}{360} = .7$ in $> .384$ in ok

Composite Girders

Bay AB

Loads: Dead: $w_d = 72(9.194) + 22 = 684$ plf

Live: $w_L = 60(9.194) = 552$ plf

$P_u = (1.2(684) + 1.6(552))(21) = 35.8$ kips

$M_u = P_a = 35.8(9.194) = 329$ kip-ft

$b = \min \begin{cases} \text{span}/8 = \frac{(27+\frac{7}{12})(12)}{8} = 41.4 \text{ in} \leftarrow \text{controls} \\ \frac{1}{2}\text{ adj width} = \frac{25(12)}{2} = 150 \text{ in} \end{cases}$ $b_{eff} = 2b = 82.8$ in

Assume: $a = 1$ $\Sigma Q_{n\,req} = .85(4)(82.8) = 282$ kips

From Table 3-19 (w/ $Y_2 = 4.5$, $\Sigma Q_n < 282$)

Try: W16X31 w/ $\Sigma Q_n = 213$k & $M = 340$k-ft

$a = \frac{213}{.85(4)(82.8)} = .76$ in $< 1''$ ok

$d = 15.9$ in

$A_s = 9.13$ in^2

Studs:
$\frac{213}{17.1} = 12.5 \Rightarrow 13$
26 studs per girder

$M_n = A_s F_y \left(\frac{d}{2} + t - \frac{a}{2}\right)$

$\phi M_n = .9(9.13)(50)\left(\frac{15.9}{2} + 5 - \frac{.76}{2}\right)$

$\phi M_n = 430$ kip-ft > 329 k-ft ok

Use: W16X31 w/ 26 studs

Live Load Deflection Check:

$$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{11.6(27+\frac{7}{12})^3(1728)}{28(29000)(825)} \quad I_{LB} = 825\ in^3$$

$\Delta_{LL} = .628\ in$

$\Delta_{LLmax} = \frac{L}{360} = \frac{(27+\frac{7}{12})(12)}{360} = .919\ in > .628\ in$ ok

Cantilever Bay

Girders Cantilever 9'-8" beam spacing = 4.83 ft

Loads: Dead: $P_D = (57(4.83) + 45(4.83) + 22)(21) = 10.81$ kips (SDL; slab/deck)

Live: $P_L = 50(4.83)(21) = 5.07$ kips

$P_U = 1.2(10.81) + 1.6(5.07) = 21.1$ kips

$P_{extwall} = 46(13)(21) = 12.55$ kips

$M_U = P_ub + P_ul + P_{ext}l = 21.1(4.83) + 21.1(9.67) + 12.55(9.67)$

$M_U = 427$ kip-ft

W16x31 w/ 26 studs $\phi M_n = 430$ kip-ft still works for this bay

Deflections

$$\Delta_{LL} = \frac{P_ub^2}{6EI}(3l-b) + \frac{P_ul^3}{3EI} + \frac{P_{ext}l^3}{3EI}$$

$$\Delta_{LL} = \frac{21.1(4.83)^2(144)}{6(29000)(825)}(3(9.67) - 4.83) + \frac{21.1(9.67)^3(1728)}{3(29000)(825)} + \frac{12.55(9.67)^3(1728)}{3(29000)(825)}$$

$\Delta_{LL} = .74\ in$

$\Delta_{LLmax} = \frac{9.67(12)}{360} = .322\ in < \Delta_{LL}$ No Good

$I_{req} = 1907.5\ in^3$

Girder Design

$$b_{min} = \begin{cases} span/8 = \frac{9.67(12)}{8} = 14.5\ in \leftarrow \text{controls} \\ \frac{1}{2}\ adj\ width = \frac{16.25(12)}{2} = 97.5\ in \end{cases}$$

$b_{eff} = 2b = 29\ in$

Assume a = 1 $\quad \Sigma Q_{n_{req}} = .85(4)(29) = 58$ kips

From Table 3-19 ($w/Y_2 = 4.5$, $\Sigma Q_n < 58$)

Nothing fits that also meets I_{req}

Solution ⇒

Use : 3VLI20 w/ 7.5" slab

$a = 4.5$ $\Sigma Q_{n,req} = .85(4)(29)(4.5) = 261$ kips

Table 3-19 ($Y_2 = 7$, $\Sigma Q_n < 261$)

Try W21X44 w/ $\Sigma Q_n = 260k$ & $M = 625$ k-ft, $I_{LB} = 1960 in^3 > 1907 in^3$

$d = 20.7 in$

$A_s = 13 in^2$

Studs

$\frac{260}{17.1} = 15.2 \Rightarrow 16$

32 studs per girder

$M_n = A_s F_y \left(\frac{d}{2} + t - \frac{a}{2}\right)$

$\phi M_n = .9(13)(50)\left(\frac{20.7}{2} + 7.5 - \frac{4.5}{2}\right)$

$\phi M_n = 760.5$ kip-ft $> M_u = 427$ k-ft ok

Use : W21X44 w/ 32 studs