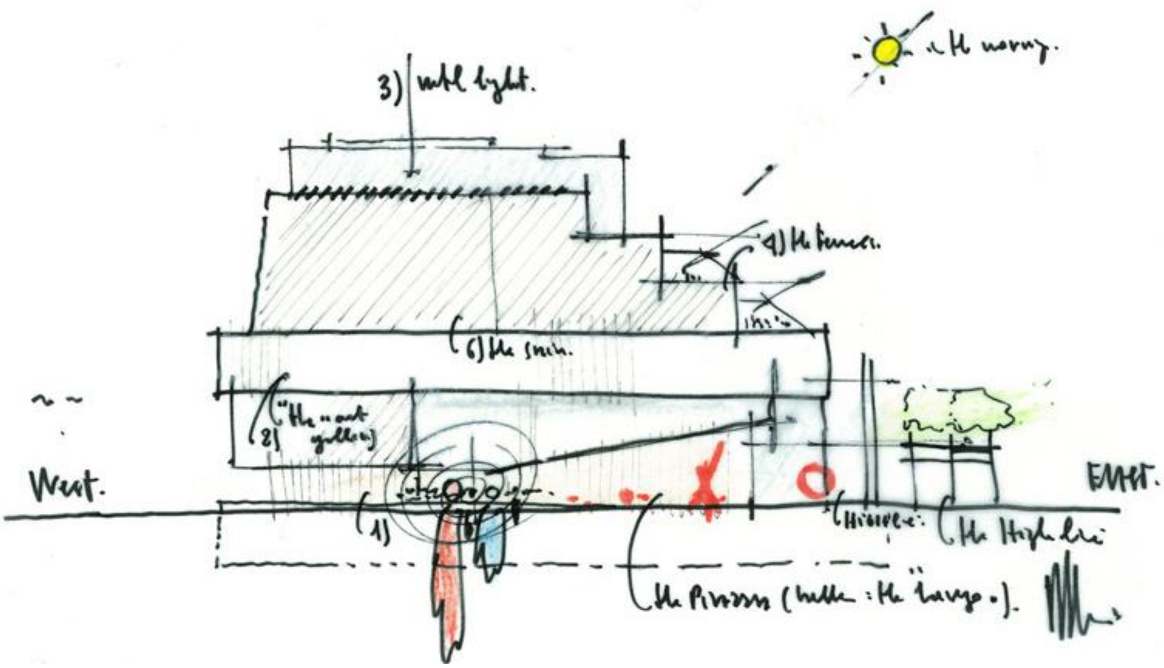
SEAN FELTON | STRUCTURAL



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AMERICAN ART MUSEUM | NORTHEAST UNITED STATES

TECHNICAL REPORT 2

EVALUATION OF ALTERNATE FLOOR SYSTEMS
ADVISOR: HEATHER SUSTERSIC
OCTOBER 12, 2012

EXECUTIVE SUMMARY

Technical Report 2 evaluates the existing composite floor system against the three most viable alternatives that could have been used in the design and construction of the American Art Museum (AAM). Criteria of cost, weight, depth, architectural and structural impacts, MEP coordination, serviceability, and construction considerations were analyzed to find a potential alternative system. Against the lightweight purlin-girder (non-composite), two-way flat slab with drop panels, and one-way with beams and girders, however, the existing steel-composite system proved to be the best such that no considered alternative could effectively replace the current design.

Each system was designed according to a standardized 20' 8" x 20' typical bay for flexural and shear strength, servicability, and constructability considerations(dictated by column lines E-F and 3-4; see S-105 in Appendix A). The loads considered are discussed in the loads section of the report and are found on the dead and live load schedules on drawing S-200.01 in Appendix A. After design, each system was analyzed for weight and cost by a detailed estimate using RS Means Facilities 2012 and compared on a per-square-foot basis.

AAM's architectural design (Figure 1) likely arose from the owner's desire to have an iconic signature building. With that understanding, Renzo Piano Building Workshop (RBPW) most likely established the building's form and function assuming the use of a steel-composite system. If concrete had been a consideration from the beginning, either the flat slab with drop panels or the one-way slab with beams would have been economical alternatives to the existing steel-composite frame system. The form of the building with its large, heavy cantilevers and supports in tension make a concrete frame difficult, if not impossible. Additionally, the large, open art gallery spaces on the upper floors require spans of up to 70', which would be difficult to achieve with concrete.

Figure 1: Rendering of the Building (SW Corner)

The lightweight steel purlin-girder floor system could have been used if that was the desired system. Other lightweight floor systems are all but impossible due to the fact that many manufacturers do not include the span/load combinations required for this building. The steel weight, number of connections, and low resistance to vibrations, however, make the floor system nearly twice as expensive as the existing steel-composite system, offsetting any savings gained from column sizing, which would likely become controlled by the lateral analysis.

Precast concrete systems were not considered for similar reasons as web joists: manufacturers do not include the required span/load combinations required for AAM. Also, post-tensioned floor systems were ignored because the significance of the strength of the tension strands would decrease the flexibility of the gallery spaces.

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TABLE OF CONTENTS

INTRODUCTION

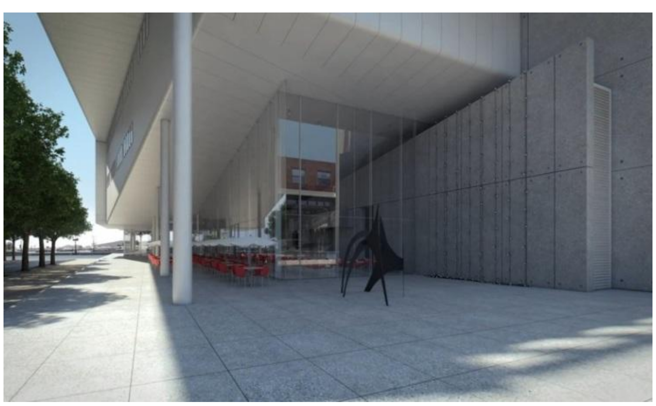
The American Art Museum (AAM) will serve as a replacement to the owner's current facility in New York City. Figure 2 shows AAM's new location in a vibrant district where aging warehouses, distribution centers, and food processing plants are being renovated and replaced by art galleries, shops, and offices. AAM will stand in place of several such warehouses, and will provide a magnificent new southern boundary to the city's recently renovated elevated park, which terminates on the eastern edge of the site.

Figure 2: Arial map showing urban location along river (www.maps.google.com)

Renzo Piano's approach to AAM's design and architecture blends a contemporary architectural style with the historical development of the city. The large cooling towers and outdoor terraces that step back towards the river on the west trace their roots back to the industrial revolution and its local impact. These outdoor terraces will also provide views of the southern skyline and space for outdoor exhibits and tall sculptures while being protected from any wind by the higher portions of the building's west side. Alternately, the large cantilevers, insets, large open spaces, exposed structural steel, and modular stainless plate cladding show no attempt to camouflage AAM with the more historical surrounding buildings.

AAM's façade is comprised of the aforementioned steel plate, pre-cast concrete, and glazing using a standard module of 3'-4" (about 1m) (shown in Figure 3). While most of the façade components are broken at each story, the long steel plates stretch 60' on the southern wall from levels 2 to 6 and from 6 to 9.

This new facility is a multi-use building with gallery and administration space, two café/restaurants, art preservation and restoration spaces, a library, and a 170-seat theater. Public space including the theater, classrooms, restaurants, and galleries are located on the south half of the building on the ground level and levels 5 through 8. Mechanical, storage, conservation, offices, and administration are dispersed on the north side at each level. The 220,000 square-foot AAM will stand 148ft tall and cost approximately $266 million. Construction began in May 2011 and is expected to be complete in December 2014.



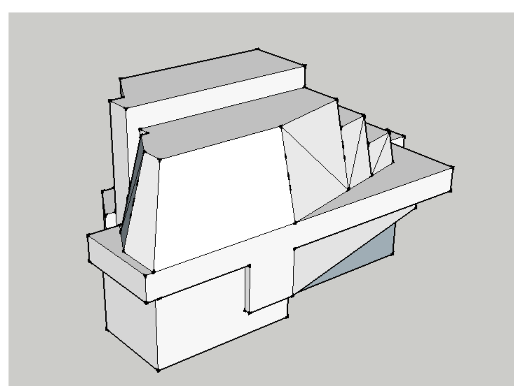


Figure 3 (left): Rendering shows façade at SE corner entrance
Figure 4 (right): Sketchup model shows building's complex geometry from the SW corner

STRUCTURAL SYSTEMS

OVERVIEW

AAM sits on drilled concrete caissons encased in steel with diameters of either 9.875" or 13.375" capped by pile caps. From the foundation level at 32' below grade, 10 levels rise on steel columns and trusses. Each floor will be supported by a steel-composite system. The lateral system consists primarily of braced frames spanning several stories. At some levels however, the floor system uses HSS diagonal bracing between beams and girders to create a rigid diaphragm that also transfers the lateral loads between staggered bracing. Moment frames are used for localized stability purposes. While masonry is used in AAM it is used for fire rating purposes only.

The building classifies as Occupancy Category III. This is consistent with descriptions of "buildings where more than 300 people congregate in one area" and "buildings with a capacity greater than 500 for adult education facilities."

FOUNDATIONS

URS Corporation produced the geotechnical report in February 2011 to summarize the findings of several tests and studies performed between 2008 and 2010. They summarize that while much of the site is within the boundaries of original shoreline, a portion of the western side is situated on fill-in from construction. They explain further that the portion that was formerly river has a lower bedrock elevation and higher groundwater. Due to the presence of organic soils and deep bedrock, URS suggested designing a deep foundation system and provided lateral response tests of 13.375" diameter caissons socketed into bedrock.

The engineers acted on the above suggestions and others. The caissons are specified with a 13.375" diameter of varying concrete fill and reinforcement to provide different strengths to remain consistent with URS Corp's lateral response tests. Low-capacity caissons (9.875" diameter) are individually embedded in the pressure slab, while typical and high-capacity caissons are placed in pile caps consisting of one or two caissons. The high-capacity caissons are always found in pairs and are located beneath areas of high live load or where cantilevers are supported. For a complete layout and caisson schedule, see FO-100 in Appendix A.

A pressure slab and the perimeter secant-pile walls operate in tandem to hold back hydrostatic loads created by the soil and groundwater below grade. The walls vary between 24" and 36" and are set on 6'-6" wall footers and caissons. These are isolated from the pressure slab. The cellar level floor slab consists of a 5" architectural slab-on-grade by a 19" layer of grave on top of a 24" pressure slab (Figure 5).

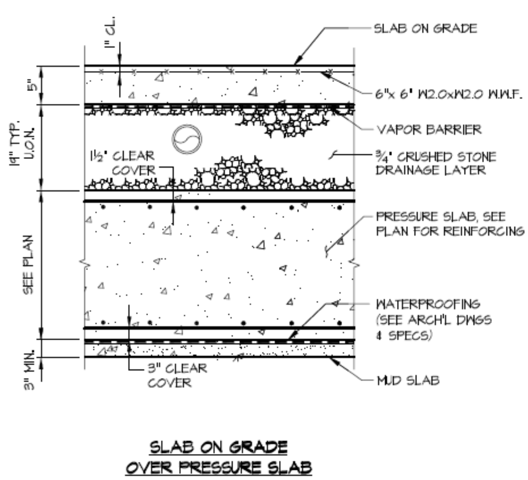


Figure 5: Pressure slab detail (S-201)

Gravity System

Floor System

A surprisingly regular floor layout contrasts the obscure geometry of the building (Figure 6). The engineers managed to create a grid with spacings of roughly 20' (E-W) and 30' (N-S), where the 20' sections are divided by beams which support the floor decking running E-W. Beams that do not align with the typical perpendicular grid indicate a change of building geometry below or above. Each beam is designed for composite bending with the floor slab.

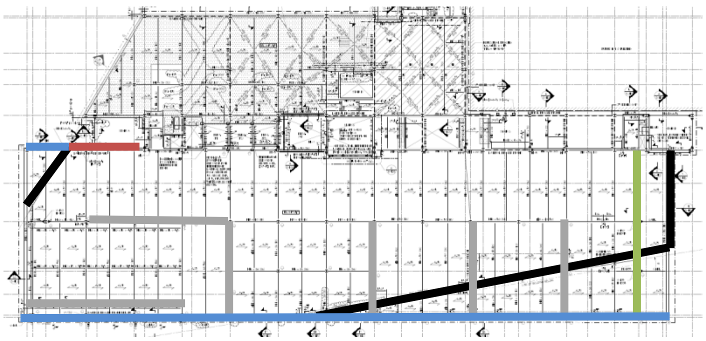


Figure 6: Level 5 framing plan showing regular layout against building footprint (S-105)
Gravity Trusses (above)
Gravity Trusses (below)
Plate Girder (d=46")
Lateral Braced Frames (part of gravity)
Outline of Building Below

Four slab/decking thicknesses are called for depending on deck span and loading, all on 3"-18 gauge composite metal deck. The most common callout is 6.25" (total thickness) lightweight concrete. This provides a 2-hour fire rating. 7.5" normal weight is used on level 1 for outdoor assembly spaces and the loading dock, and 9" normal weight is used for the theater floor. The roof above the level 9 mechanical space calls out 5.5" composite.

While the layout can be considered relatively consistent, the beam sizes and spans selected suggest a much more complicated floor system. Though a typical bay spans 20'-30', the gallery floors (levels 6-8) span over 70'. The shorter spans require filler beams as small as W14x26, but the longer spans supporting the upper gallery levels require beams as large as W40x297s for web openings. In several places welded plate girders are specified at depths from 32.5" to 72." The plate girders are used as transfer large loads and moments as propped cantilevers, especially from gravity trusses and lateral braced frames shown in Figure 7.

Framing System

Cantilevers on the south side of AAM are supported by 1 or 2-story trusses, typically running in the N-S direction. One large gravity truss runs along the southernmost column line between levels 5 and 6 to support the cantilever on the south-eastern corner of the building.

While the vast majority of columns are W12x or W14x shapes, some of the architecturally exposed steel vertical members are HSS shapes, pipes, or solid bars. Furthermore, the gravity load path goes up vertically and horizontally nearly as much as it flows directly down a column to the foundation. Figure 8 shows how large portions of the southern half of AAM's levels 3 and 4 are hung from trusses and beams on the level 5 framing system.

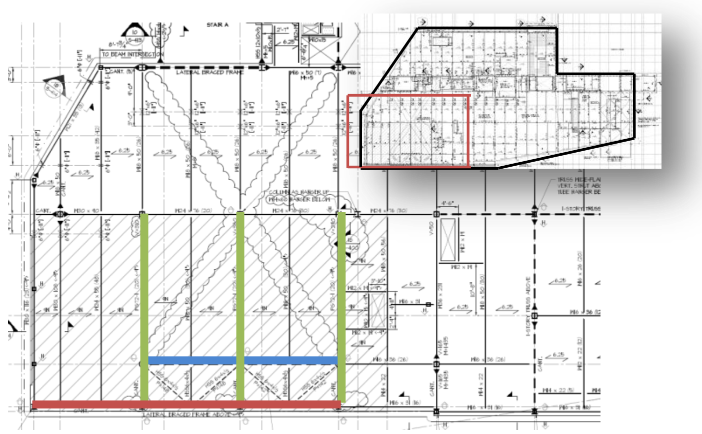


Figure 7: Level 3 framing plan showing transfer girders and lateral braced frames (S-103)
Lateral Braced Frame (above)
Lateral Braced Frame (below)
Plate Girder (d=46")

Renzo Piano's designs often expose structural steel, providing an extra constraint on the design team. One example is column 3-M.5 which supports level 5 from the outdoor plaza below. The foundation column below grade specifies a W14x311, a typical shape for a column, but the architecturally exposed structural steel is called out as 22" diameter solid bar. A unique analysis would be required for a solid bar acting as a column, as AISC XIII does not have provisions for such a selection in its tables or specifications.

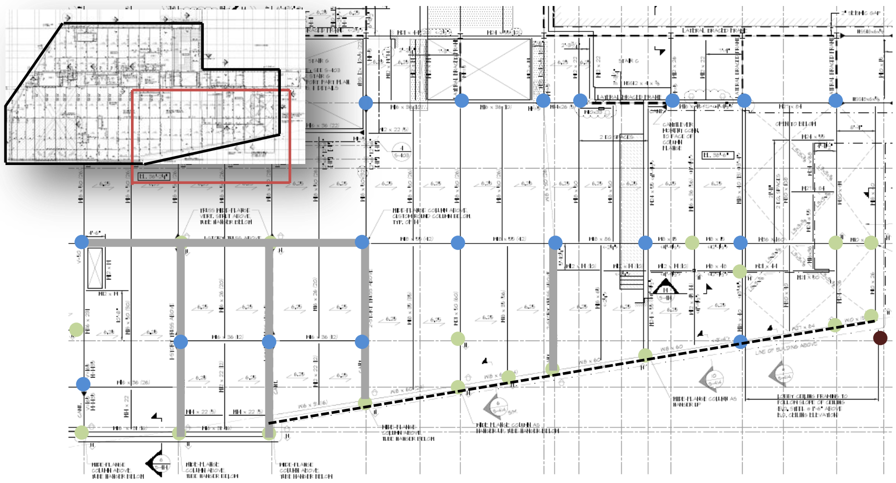


Figure 8: Level 3 framing plan showing hangers and outline of hung/cantilevered portion of building (S-103)

- Gravity Truss (above)
- Compression Support (single below)
- Tension Support (single above)
- Column 3-M.5
- Outline of Building

LATERAL SYSTEM

AAM's lateral system is as complicated as its gravity systems. Concentric braced frames stagger up the building, transferring lateral loads via diagonal bracing within the floor diaphragms on level 3 for the southern portion and 5 for the northern portion as shown in Figure 9. Most of the braced frames terminate at ground level, but three extend all the way down to the lowest level. Those braces that terminate at upper floors transfer uplift through columns that extend underneath them. Bracing members are comprised mostly of W10x, 12x, or 14x shapes in X-braces or diagonals. There are, however, HSS shapes are used with K-braces. An enlarged floor framing plan showing the braced frames at level 5 is provided in Figure 10 below.

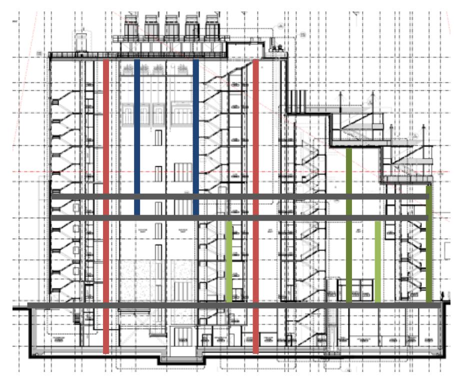


Figure 9: Section cut showing N-S braced frames at staggered heights (A-212)

Figure 10: Level 5 Framing Plan Showing Lateral System (S-105)

- Lateral Braced Frame
- Gravity Truss that Contributes to Lateral System
- Floor System with Diagonal Bracing

Design Codes & Standards

The design codes listed for compliance of structural design can be inferred from drawing S-200.01 and Specification Section 014100.2.B:

- International Code Council, 2007 edition with local amendments including:
  - Building Code
  - Fire Code
- ASCE 7-05: Minimum Design Loads for Buildings and other Structures
- ACI 318 -08: Building Code Requirements for Structural Concrete (LRFD)
- AISC XIII: Specifications for Structural Steel Buildings (LRFD)
- AWS D1.1: American Welding Society Code for Welding in Building Construction

Other codes not applicable to the structural systems of the building can be found in the specifications.

Materials Specifications

The different materials specifications are summarized in Figure 11 below. Additional information can be found on drawing S-200.01 in Appendix A.

Materials Specifications						
Concrete & Reinforcement			Structural Steel			
Wt	Use	f'c (psi)	Shape	ASTM	Gr.	Fy (ksi)
LW	Floor Slabs (typ)	4000	Wide Flange	A992	-	50
NW	Foundations (walls, slab, pile caps, grade beams)	5000	Hollow Structural	A500	B	46
			Structural Pipe	A501/A53	-/B	30
NW	Composite Column Alternate	8000	Channels	A36	-	36
NW	Other	5000	Angles	A36	-	36
			Plates	A36	-	36
Gr.	Use	ASTM	Connection Bolts	A325-SC	-	80
70	Reinforcement	A185	(3/4") Anchor Bolts	F1554	36	36
70	Welded Wire Fabric	A185				

Figure 11: Summary of Structural Materials Specifications in AAM

Building Gravity Loads

Loads Summary

Dead Loads

Because the live loads are so high, special care seems to have been taken by the design engineers to be very precise in their dead load calculations. Similar to the live loads, the diversity of different use types and load requirements have led to a congruent variety of dead load arrangements in structural steel weight, concrete density, MEP requirements, partitions, pavers, roofing, and other finishes. A total of 37 different dead load requirements, arranged by use and location, are listed in the Dead Load Schedule on drawing S-200.01 in Appendix A. These range from 76 PSF to 214 PSF. In all, the building has a dead weight of 23,084 k (11,500 tons) from level 1 through level 9 Roof North.

Live Loads

Typically, one would expect to see Live Loads calculated from ASCE 7 minimums (ASCE 7 Table 4-1). The structural narrative explains that much of AAM does not fit with any ASCE 7 descriptions of use types, so the engineers have provided their own design loads summarized in Figure 12. Additionally the engineers created a live load plan on S-200.01 in Appendix A which shows areas of equal live load on each floor.

The engineers, in a desire for maximum flexibility of the gallery spaces, elected to conservatively design the AAM-specific spaces for live loads, while being consistent with ASCE 7 minimums for more common areas.

LL Schedule Designation		ASCE 7 Designation	
Use	LL	LL	Description
Gallery - Typical	100	100	Assembly Area - Typical
Gallery - Level 5	200	100	Assembly Area - Typical
Testing Platform	200	150	Stage Floors
Offices	50	50	Offices
Private Assembly/Museum Use	60	n/a	n/a
Auditorium - Movable Seating	100	100	Theater - Moveable Seats
Compact Storage	300	250	Storage Warehouse - Heavy
Art Handling & Storage	150	125	Storage Warehouse - Light
Largo and Loading Dock	AASHTO HS-20	250	Vehicular Driveways
Stairs and Corridors	100	100	Stairs and Exit Ways
Lobby and Dining	100	100	Assembly Area - Lobby
Mech Spaces Levels 2, 9	150	n/a	n/a
Mech Spaces Cellar	200	n/a	n/a
Roof - Typical	22 + S	20	Roof - Flat

Figure 12: Comparison of design live loads and ASCE 7 minimum live loads

Floor System Analysis

Overview

Technical Report 2 analyzes and compares AAM's existing floor systems with three alternates. Each system was evaluated based on criteria such as system weight, overall depth, cost, feasibility, and impact on both the lateral and foundation systems. Other considerations unique to each system were also considered. A table summarizing these findings is in the Summary section following the four system descriptions.

Figure 13 indicates the bay that was considered "typical" for the purposes of Technical Report 2; having dimensions of 20' (E-W) x 20'-8" (N-S). The following systems are discussed below:

- Steel-Composite System
- Purlin-Girder (Non-Composite)
- One-way Slab with Beams
- Two-way Flat Slab with Drop Panels

This study does not include any precast concrete systems because the manufacturers do not include the span/loading combinations required for this building. A pre-stressed hollow-core plank, for example, would require a unique design where shear controls. Similarly, post-tensioned slab systems were not included because of the need for flexibility in the spaces above. If ever the museum were to build partitions, anchors drilled into the slab could damage the post-tensioning tendons. In an effort to accommodate a flexible use of the space per the project requirements, post-tensioned systems had to be neglected.

The weight and cost estimates were calculated as carefully as possible using the RS Means Facilities estimates (detailed) 2012. Each common assembly from the assembly book was altered to match the project and design requirements and itemized into a detailed report seen in Appendix F.

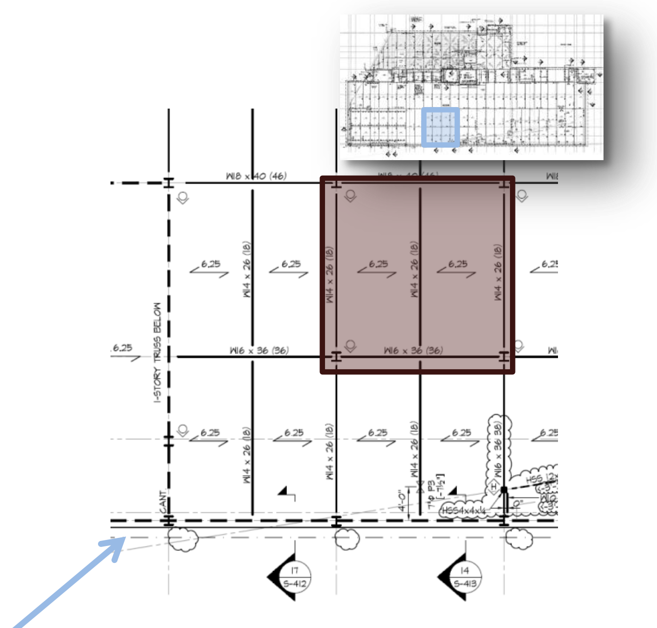


Figure 13: Typical bays supporting level 5

Steel-Composite System

Description

The existing floor system, shown in Figure 14, is a composite system utilizing concrete, composite metal deck, and wide-flange steel beams and girders. A technical analysis for this system was executed as member spot-checks in Technical Report 1, and the relevant calculations have been included in Appendix B of this report.

3 ¼" of lightweight concrete sits atop 3" – 18 ga. Composite decking for a total of 6 ¼". Per the calculations performed in Technical Report 1, Vulcraft specified 3VLI 18 is sufficient for the superimposed dead and live loads required, and provides a 2-hour fire rating. N-S running W14x26s with 18 shear studs support the decking every 10', while W16x36s with 36 shear studs run between the columns in the E-W direction, supporting the W14s.

Advantages

Technical Report 2 does not address issues such as the building's form or the a-typical 70' span supporting the gallery on Level 6 technically (see Figures 3 & 4 in Introduction), but a composite system likely addressed those challenges more than those for the typical bay analyzed. Drawing S-106 in Appendix A shows that many of those longer spans are supported by W40x249s with over 200 shear studs and web openings to accommodate MEP coordination. Additionally, a lighter non-composite system would have less stiffness; the floor would be much more susceptible to vibration problems. Ultimately, the composite system was likely chosen because it costs about 55% of its congruent non-composite purlin-girder system.

Disadvantages

While the composite system described may have the advantage in cost, it weighs about 50% more than its non-composite counterpart. Also, the composite system is the second-deepest of the four analyzed. Furthermore, Vulcraft specifies the composite assembly as 2HR inherently, but the beams and girders still require fireproofing.

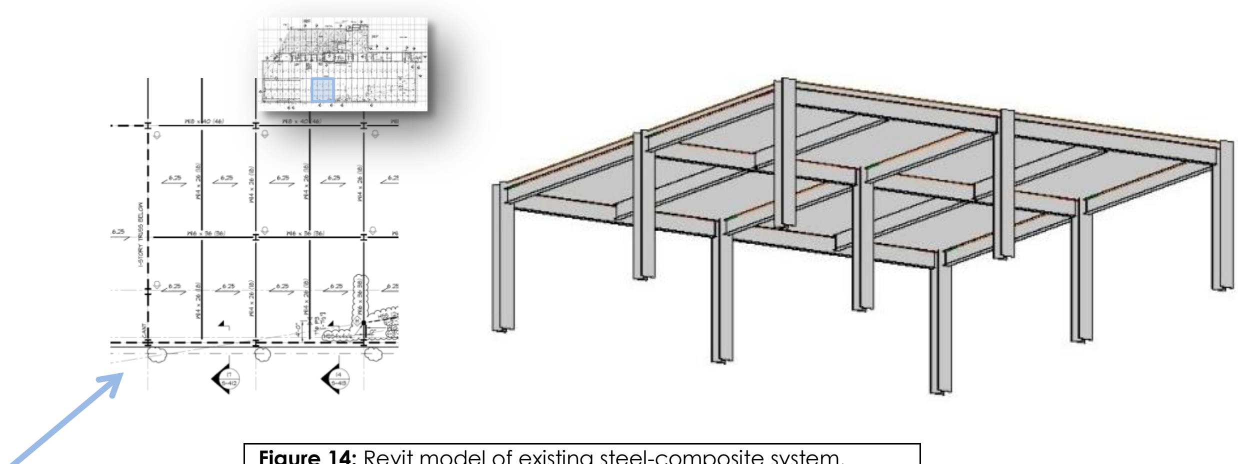


Figure 14: Revit model of existing steel-composite system.

ALTERNATE 1: PURLIN-GIRDER NON-COMPOSITE SYSTEM

DESCRIPTION

Figure 15 displays the layout of the purlin-girder alternate designed for Technical Report 2. This analysis chose to consider a purlin-girder system as opposed to other, manufactured, lightweight flooring systems because of the large live loads. Similar to the reasoning presented against precast concrete, manufacturers do not include the load/span combinations desired for this building. In order to reduce the impact to architectural layout (changing column spacing), a purlin-girder system emerged as the solution for lightweight floor analysis. In the full design calculations in Appendix C, only the channels are assumed to be fully braced. The channels and the W18s are controlled by deflection.

The load path alters slightly from that of the existing composite system above. The 1"-24 ga. Floor decking with 2 ½" topping, specified as Vulcraft 1.0C24, still runs E-W, but spans 3' instead of 10' (see Figure 14). Next in the load path lays C8x11.5 channel sections. These channels run 10' 4" and are supported by a W18x55, which span 20' (E-W). Finally, a W24x84 distributes the loads from its midpoint to the columns, running 20' 8" (N-S).

ADVANTAGES

A steel purlin-girder system is the only alternate that would allow for the geometry of the building to remain similar. The hangers and steel trusses could remain in the design scheme for a lightweight steel floor where the concrete alternatives could not. Objectively, the lightweight purlin-girder system has few additional advantages other than weight. If, for some reason, the architect and owner required minimal column profile, or the foundations had size or depth constraints, or if there was a stipulation to minimize the concrete used, the purlin-girder system may have been a viable solution. Also, it is possible that an architect or owner might insist on using this system explicitly.

DISADVANTAGES

The purlin-girder system is the deepest of the four analyzed, and with an 11' 6" floor-to-floor height, the 27" floor depth leaves only 3" for MEP on a 9' ceiling. This constraint would have to lead to either a change in floor-to-floor or floor-to-ceiling height. Also, because the system uses the most amount of steel by volume, it is by far the most expensive of the systems. Additionally the labor required for the connections drives the cost significantly. Furthermore, the lost mass of the system results in greater susceptibility to vibration issues. To meet the fireproofing requirement of 2 hours, the girders, beams, channels, and decking would all need to be encased in fibrous-spray protection. Finally, these types of systems are not often constructed and would result in more risk, and thus even greater expense for the contractor and owner.

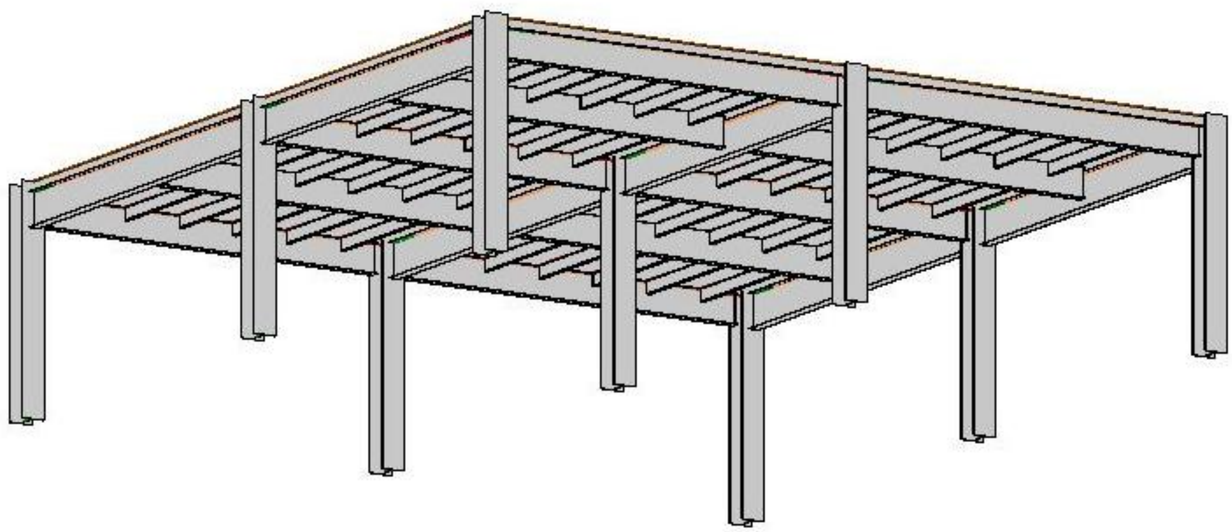


Figure 15: Revit model of proposed purlin-girder alternate system

ALTERNATE 2: TWO-WAY FLAT SLAB WITH DROP PANELS

DESCRIPTION

The two-way flat slab with drop panels shown in Figure 16 below was analyzed as the second alternative floor system. Similar to the purlin girder system above, the bay size was not changed in order to control Architectural impact. Switching the floor from steel to concrete requires the frame system to change as well. Calculations found in Appendix D explore how an 18"-diameter spiral-reinforced column was designed to replace a "typical", median column capacity found in AAM's column schedule (S-120.01 in Appendix A). Once a minimum slab thickness of 6.7" was established, the flat slab system was evaluated by hand via direct design method as outlined in ACI 318-11 and using spSlab using equivalent frame method.

Calculations used f'_c = 4000psi (lightweight) and f_y = 70ksi to remain consistent with the project requirements (see Figure 11 in Materials Specifications). Because the bay is nearly square, the hand analysis designed for the most extreme moments along the column lines and detailed the reinforcement to match the most extreme conditions throughout the slab. Differences in design assumptions arose in spSlab where ACI 318-08 was the latest version available and f_y = 60ksi was the maximum stirrup strength allowed by the program. The specified reinforcement is as follows:

- Top Reinforcement: no. 6 @ 8" O.C. both directions at support
- Bottom Reinforcement: no. 4 @ 8" O.C. both directions at mid-span

Two-way shear was checked at the columns without the use of drop panels in an iteration not included in the calculations in Appendix D. These finalized calculations provided establish the 7" slab depth with 9" drops are sufficient without additional reinforcement for wide-beam shear and two-way shear at the critical locations. Additionally, minimum drop panel dimensions of 8.75" thick and 3.45' wide were rounded up to 9" and 3' 6" (3.5') for constructability.

ADVANTAGES

Amongst the four systems considered, the flat slab with drop panels has the lowest overall depth and is the least expensive per square-foot. A 9" overall depth would allow for both reduction of floor-to-floor height and greater ease in MEP coordination. Also, if all of the floors were as typical as the bay considered, this would greatly reduce the cost of the structure compared to existing composite system. Furthermore, the two-way flat slab with drop panels is a very common construction.

DISADVANTAGES

The building's geometry with large cantilevers and suspended hanger supports makes the switch to a concrete frame nearly impossible. Altering the material of AAM would result in a complete change of the building's form, layout, and gravity scheme. Also, the lateral loads would need to be considered using either moment-frame analysis or shear walls would need to replace the existing concentric braced frames. Although a less significant consideration, the two-way flat slab system weighs the most of the four systems analyzed and would significantly impact the foundation systems.

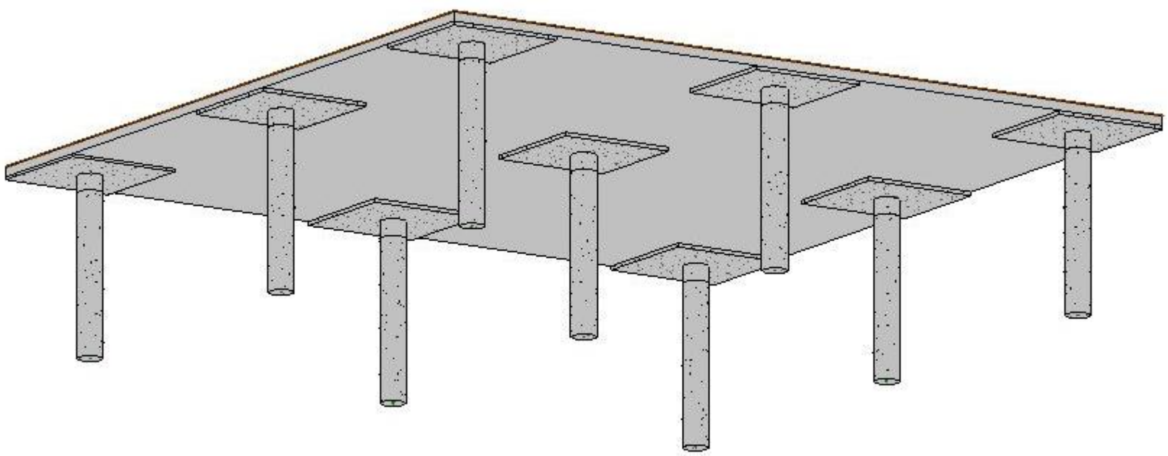


Figure 16: Revit model of proposed flat-slab with drop panels

Alternate 3: One-way Slab with Beams and Girders

Description

This report analyzes a one-way slab with beams as a tertiary alternative to the existing composite system for AAM. To be consistent with the above investigations, the one-way slab calculations did not alter the bay dimensions. The same design assumptions were used alongside the two-way system, and the beam width was assumed to be 18" to match the concrete column diameter. Deflections were considered to be non-critical as ACI 318-11 9.5.2.1, with the appropriate adjustment factors, permits omission of these calculations.

Figure 17 below shows the framing of the one-way slab system. Its framing system is congruent to that of the purlin-girder non-composite system analyzed in alternate 1. Like the channel sections, the 5 1/2" slab runs 10' 4" (N-S) and is supported by a beam 14" deep by 18" wide. These primary beams run the 20' span to girders 16" deep by 18" wide, which span N-S to the columns. The controlling reinforcement for the one-way slab system is as follows:

- Slab: no. 3 @ 6" O.C. with no additional shear reinforcement
- 14" x 18" Beam: (4) no. 8 with (8) no. 3 stirrups @ 6" O.C. from 2" from face
- 16" x 18" Beam: (3) no. 9 with (8) no. 3 stirrups @ 6" O.C. from 6" from face

Advantages

The advantages of the one-way slab with beam system are similar to those of the two-way. Its overall depth is significantly less than that of the current system. Compared to the flat-slab system, it weighs nearly 10 PSF less and is only $.05 more expensive. Also, the mass and arrangement of this system makes it the least susceptible to vibration problems. Finally, like the two-way and composite systems, the one-way slab is commonly built and would require no additional or unique scheduling considerations in a typical frame.

Disadvantages

The disadvantages of this system are also similar to those of the two-way slab. The one-way system still weighs 15 PSF more than the current system, and would still require significant foundation alterations. Likewise, the switch to a concrete system would so drastically affect the architecture of the building's form, layout, and gravity scheme. As mentioned above, it would also require a change to a concrete lateral system. One criterion that exceeds the two-way's disadvantage is its overall depth. While MEP coordination would not be a problem for the typical bay examined, the 70' spans supporting Level 6 (S-106 in Appendix A) would likely be much deeper than the 40" for the composite system, especially if they required similar web openings.

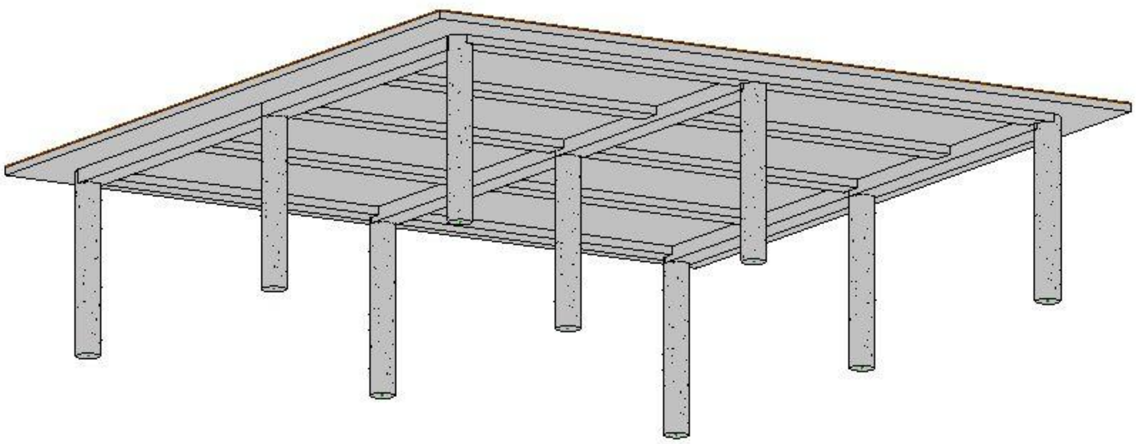


Figure 17: Revit model of proposed one-way slab with beams

SUMMARY

A side-by-side comparison is in Figure 18 below. Figure 19 indicates the impact of the RS Means location factor on the overall cost of the systems. Detailed calculations of the system statistics can be found in Appendices B-D, while the weight and cost evaluations can be found in Appendix F.

Criterion		Existing	Alternatives		
		Concrete on Composite Deck and Composite Beams	Purlin-Girder (non-composite)	Two-Way Flat Slab with Drop Panels	One Way Slab on Beams
General					
	Weight (PSF)	40.8	26.8	64.2	55.7
	Overall Depth	22 1/4"	27"	9"	16"
	Slab Depth	6 1/4"	3"	7"	5.5"
	Cost ($/SF)	$23.24	$44.01	$18.95	$19.00
Architectural					
	Fire Rating	2HR - Beams protected	2HR - beams and deck protected	2HR	2HR
	MEP Coordination	Easy	More difficult	More difficult	Most difficult
	Other	No impact	Reduce floor-to-ceiling height	SE corner geometry extremely difficult	SE corner geometry extremely difficult
Structural					
	Gravity	No impact	Reduce column sizes due to substantial DL decrease	18"-dia. CIP columns, substantial DL increase; reconfigure cantilevers due to loss of hanging supports	18"-dia. CIP columns, substantial DL increase; reconfigure cantilevers due to loss of hanging supports
	Foundation	No impact	Reduce caisson capacity	Increase caisson capacity	Increase caisson capacity
	Lateral	No impact	Reduce diaphragm stiffnessness	Moment frame/shear walls	Moment frame/shear walls
Serviceability					
	Vibration	Minimal	Most likely	Less likely	Least likely
Construction					
	Formwork	Minimal	Minimal	Yes	Yes
	Constructability	1	2	4	3
	Lead Time	Long	Long	Short	Short

Figure 18 (above): Side-by-side comparison
Figure 19 (below): Cost summary

Cost Summary				
System	Location Factor	Material Cost ($/SF)	Installations Cost ($/SF)	Total Cost ($/SF)
Compsoite Beam	131.9	15.14	2.47	$23.24
Purlin-Girder	131.9	20.86	12.51	$44.01
Two-way Slab	131.9	8.48	5.88	$18.95
One-Way Slab & Beam	131.9	7.17	7.24	$19.00

CONCLUSION

Technical Report 2 evaluates the existing composite floor system against the three most viable alternatives that could have been used in the design and construction of the American Art Museum. The three alternative systems analyzed were:

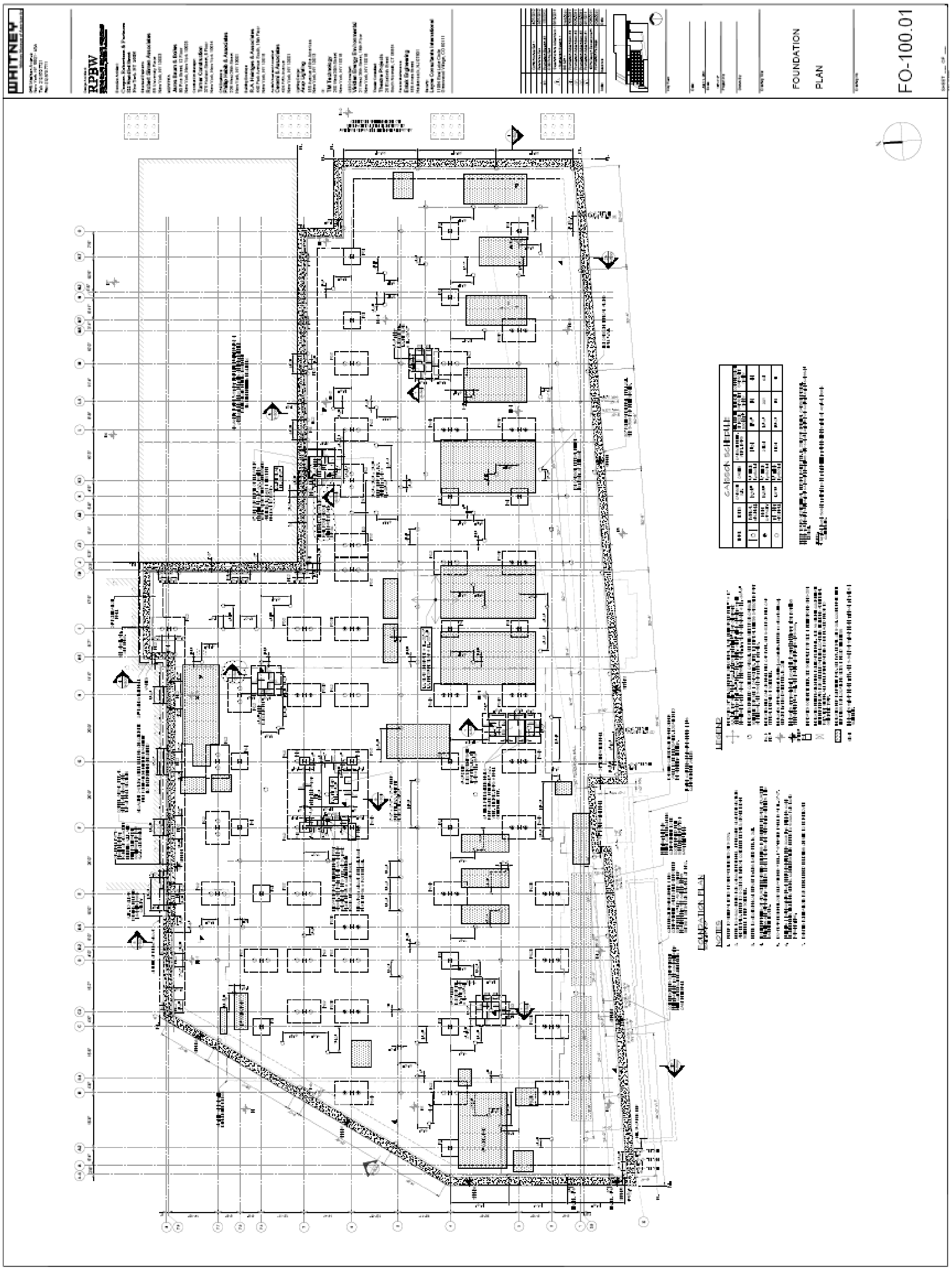
- Purlin-Girder (Non-Composite)
- One-way Slab with Beams
- Two-way Flat Slab with Drop Panels

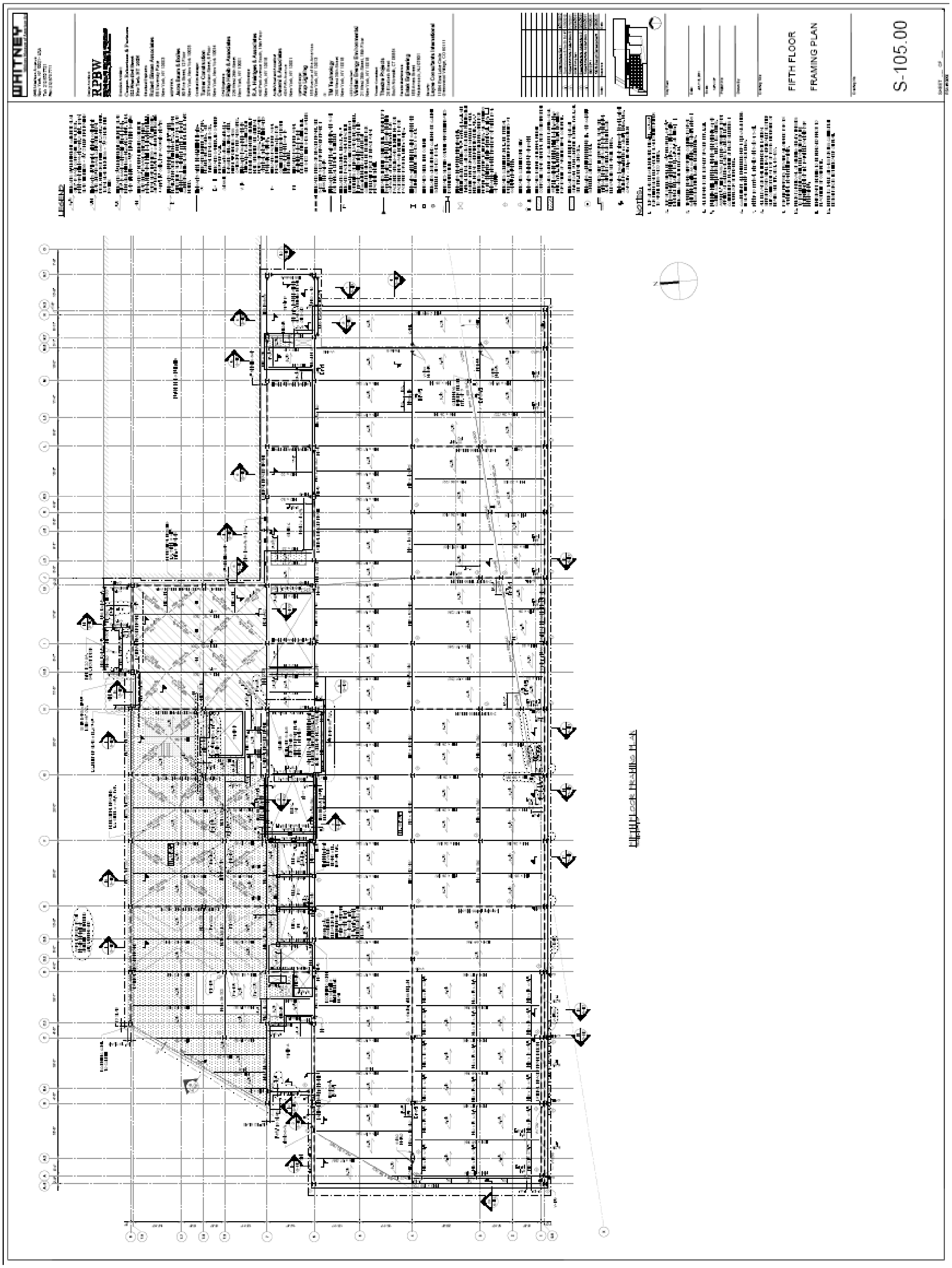
In an effort to create the most equivalent comparison, each system was evaluated on the current 20' 8" x 20' typical bay under criteria of cost, weight, depth, architectural and structural design impact, serviceability, and construction considerations.

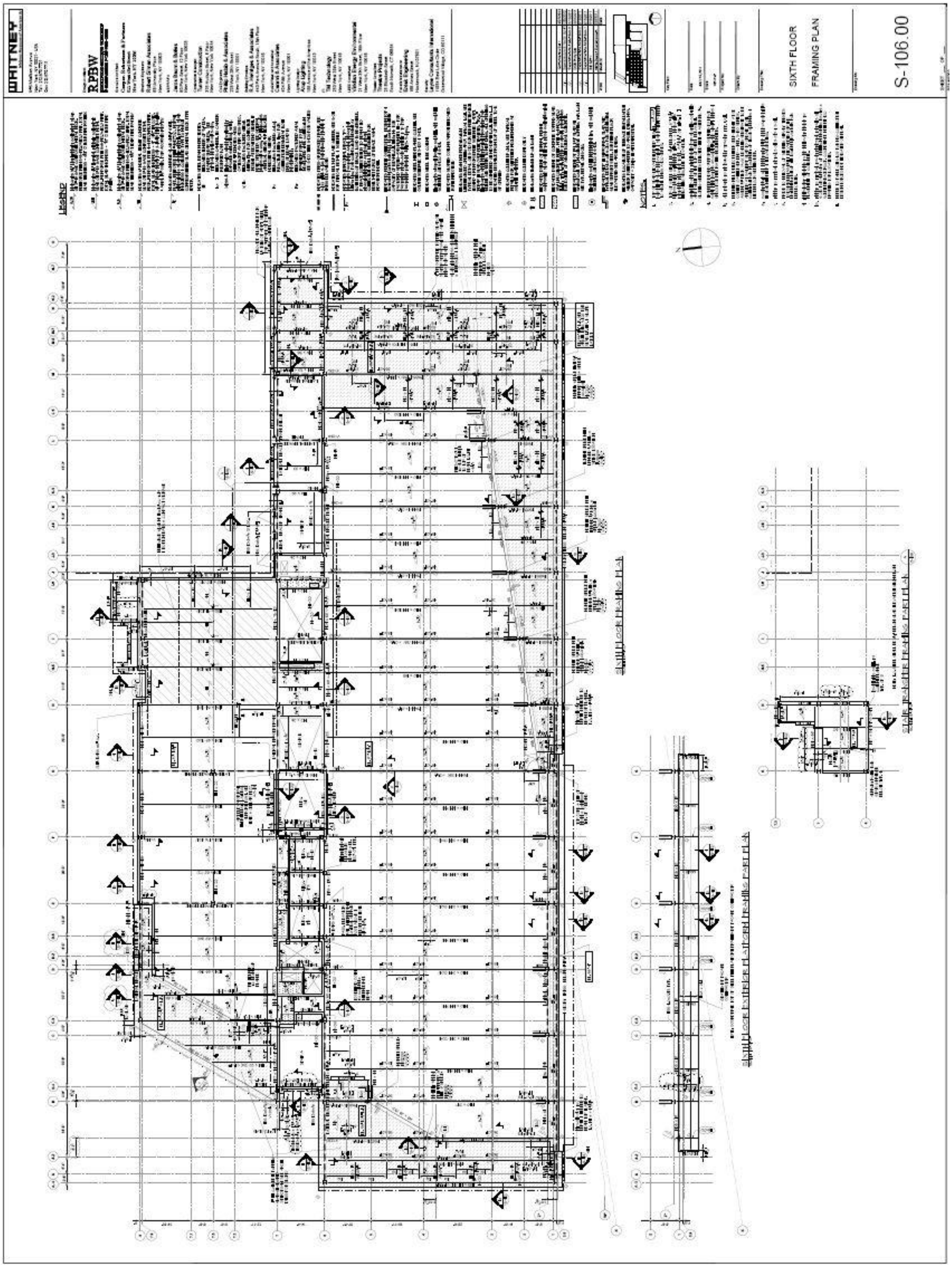
After a thorough investigation, the existing steel-composite system emerged as the only truly viable option for this project because of considerations outside of the typical floor framing system analyzed. The form and gravity structural scheme of AAM dictate a steel frame system be used, and the lightweight floor system, though to code, would deflect 4 times that of the existing system and be highly susceptible to vibration issues. Also, the lightweight floor system could nearly double the cost of the structural steel.

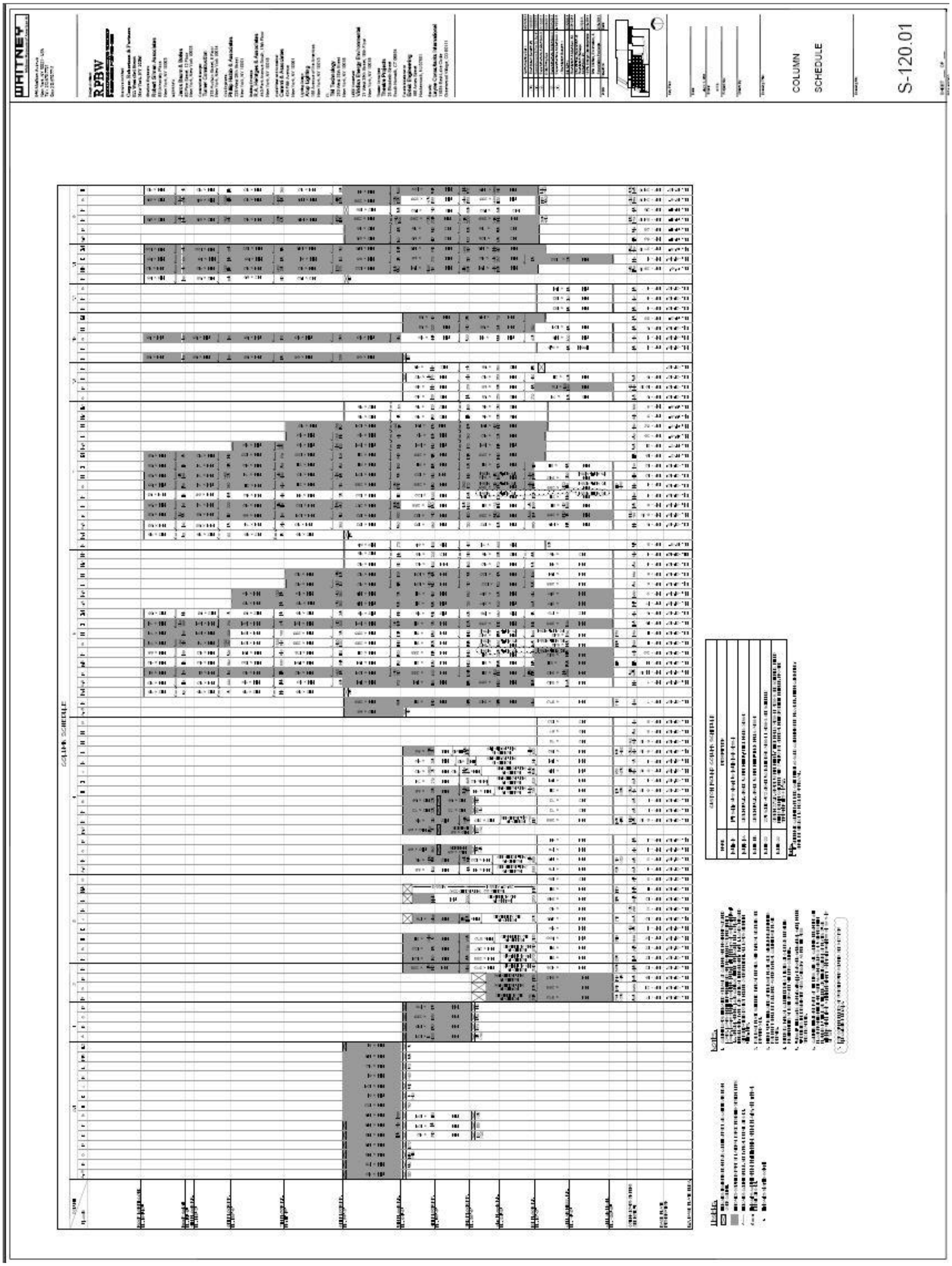
If a concrete frame had been considered as part of the form and geometry of the building, the two-way flat slab with drop panels may have been considered. It is unlikely, however, that the architect and owner would have chosen a concrete system that would not provide the spans required for such open and flexible art gallery spaces – a problem for both concrete systems. Again, the drive for an iconic building with large open spaces on elevated slabs and a unique form necessitate a steel frame.

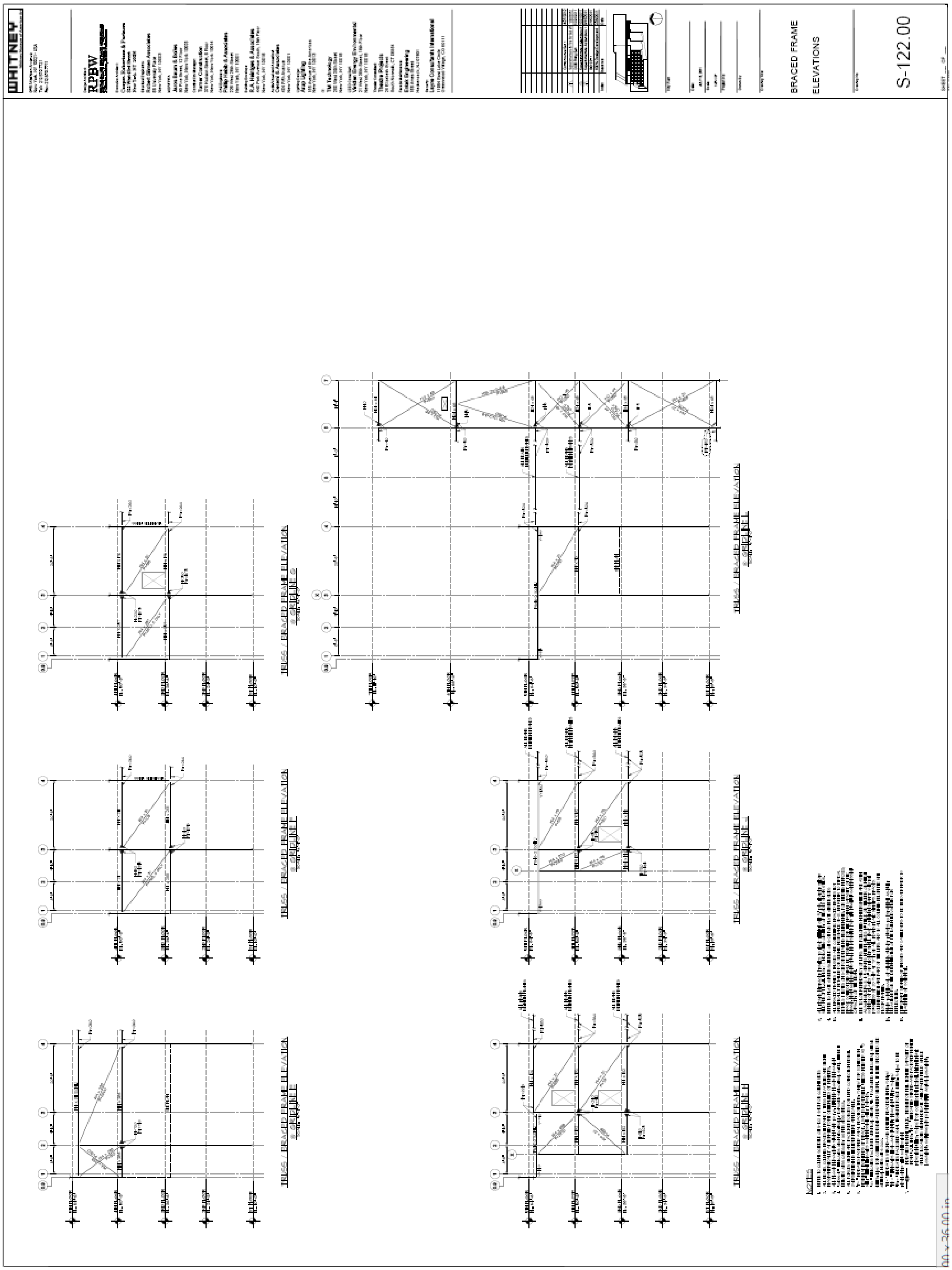
Appendix A: Reference Drawings & Documents

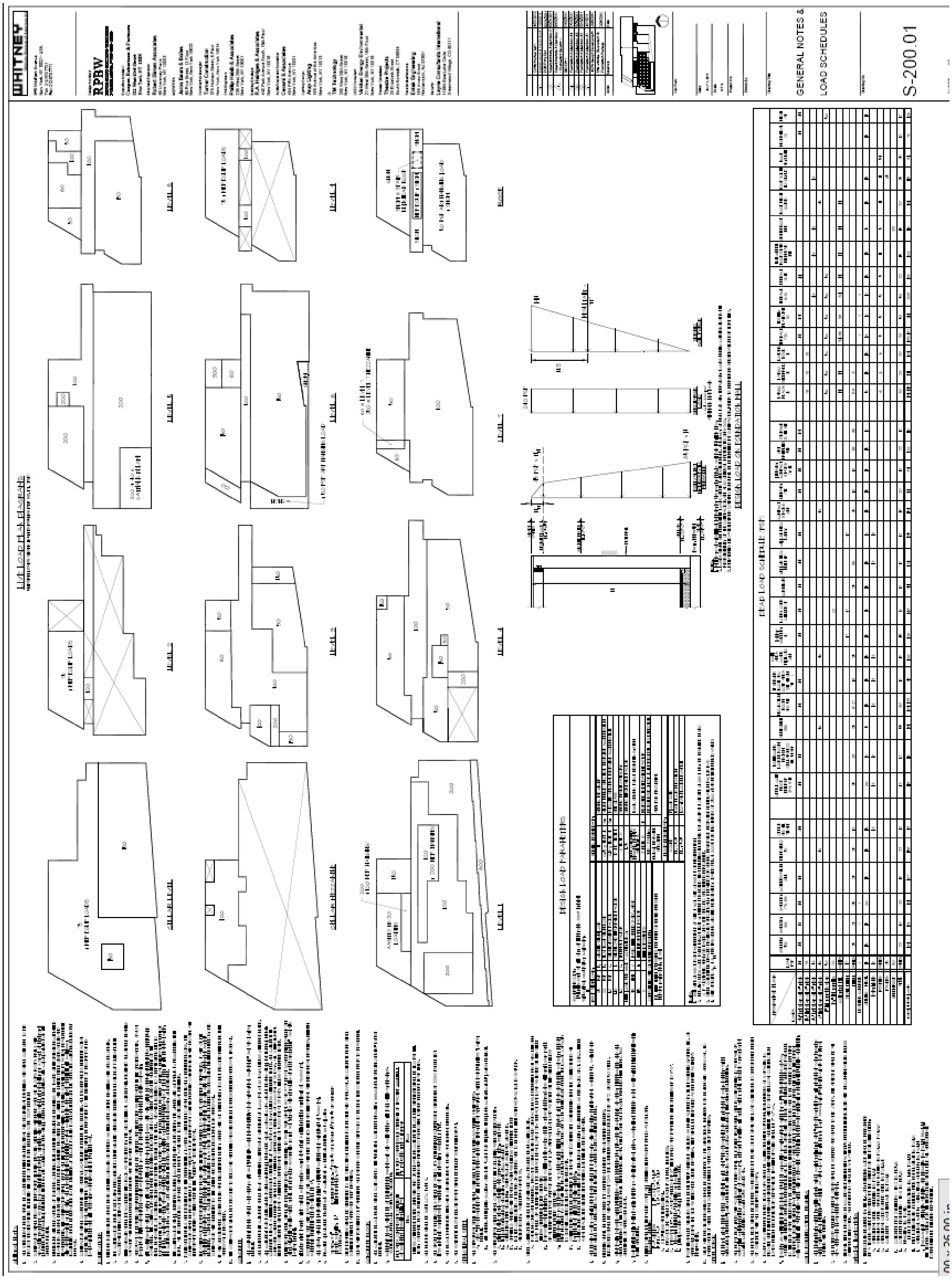


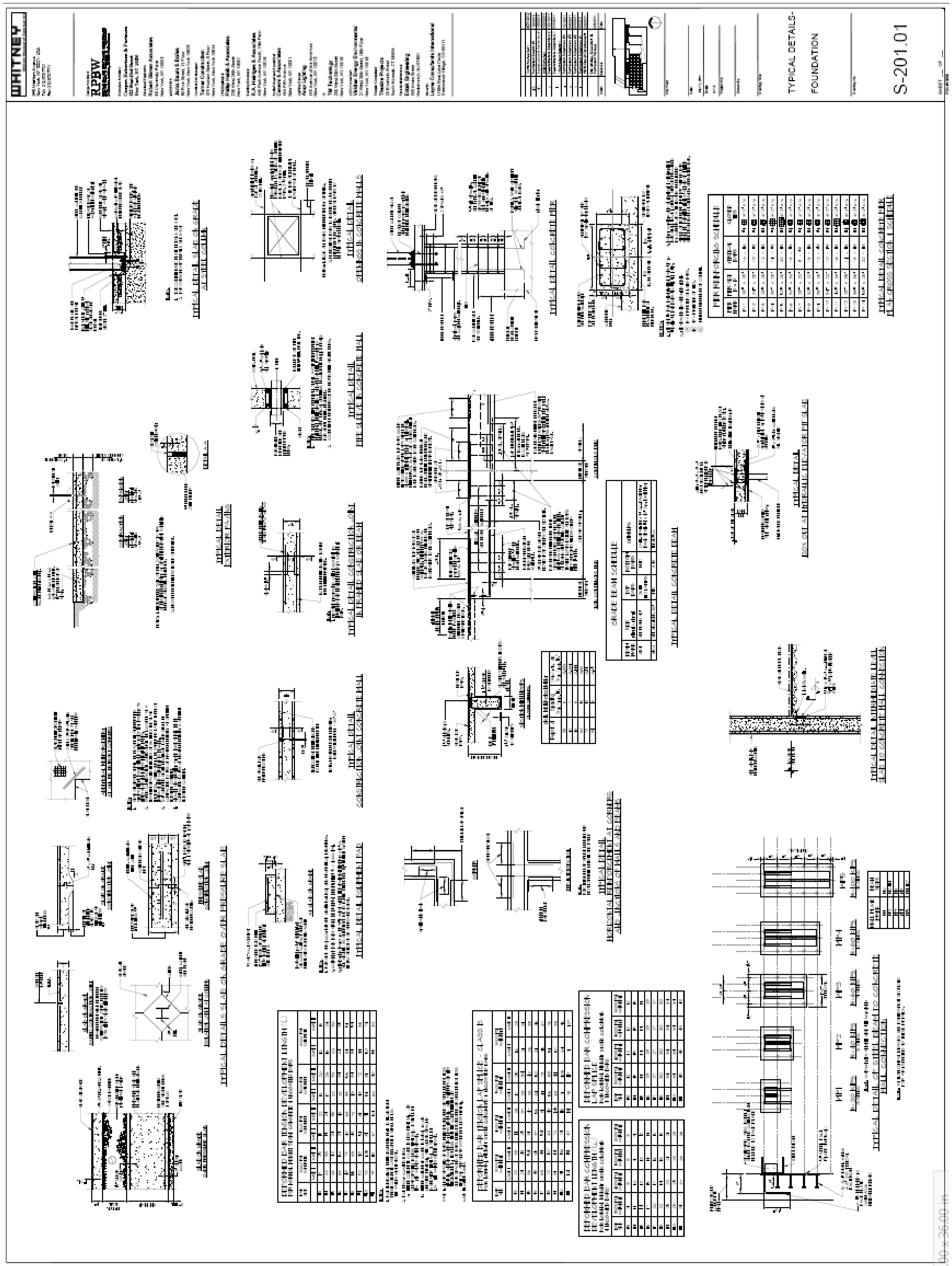

WHITNEY
RPBW
FIFTH FLOOR
FRAMING PLAN
S-105.00


WHITNEY
RPBW
SIXTH FLOOR
FRAMING PLAN
S-106.00


WHITNEY
RPBW
COLUMN
SCHEDULE
S-120.01


WHITNEY
RPBW
BRACED FRAME
ELEVATIONS
S-122.00


WHITNEY
RPBW
GENERAL NOTES & LOAD SCHEDULES
S-200.01

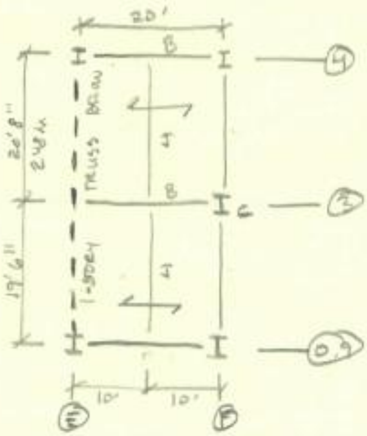

WHITNEY
RPBW
TYPICAL DETAILS-
FOUNDATION
S-201.01

APPENDIX B: STEEL-COMPOSITE SYSTEM

1/11

GRAVITY SPOT-CHECK

TYP. DECK, JOIST, COLUMN, TRUSS SYSTEM AT LEVEL 5
FLOOR FRAMING



A: DECKING

SUPERIMPOSED DL
MECH/CEILING : 15 PSF
FLOOR FINISH : 25 PSF
SUPERIMPOSED LL : 200 PSF
240 PSF

* LOAD ASSUMPTIONS TAKEN FROM PAGE S-200.01 LL PLANS, DL SCHEDULE

* DECKING CALLOUT ON P. S-105 NOTES 3 1/4" CONCRETE (L.W.) ON 3" - 18 GA METAL DECK

* USING 3-SPAN WHERE POSSIBLE, UNSHORED (SPEC 053000 1.4 B,D)

* VULCRAFT 1 OF 4 POSSIBLE MFG (SPEC 053000 2.2.B.3)

DECK SPANS 10'-0"
→ USE VULCRAFT DECK CATALOG

L.W. COMPOSITE 3VLI:
$t = 3.25$
3VLI 18:

UNSHORED			SPAN
1 SPN	2 SPN	3 SPN	10'-0"
12'-9"	15'-0"	15'-0"	235 PSF
OK	OK	OK	

235 PSF $\not\geq$ 240 PSF NG

$\frac{5 \text{ PSF}}{240 \text{ PSF}}$ RESULTS IN A 2.1% DEFICIENCY

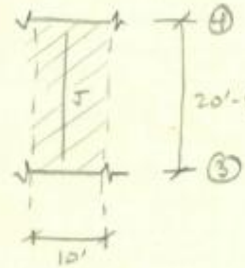
1.) POSSIBLY ROUNDING (i.e. 235 → 240 = 240 OK)
OR
2.) DIFFERENT MFG COMPANY MAY HAVE HIGHER CAPACITY (242 ≥ 240 ∴ OK)

2/11

B: JOIST 3-4 : 20'-8"

DL : FLOOR TYPE 1 = 109 PSF (S-200.01)
DECK/SLAB = 49 PSF
FLOOR FINISH = 25 PSF
CEILING/MECH = 15 PSF
STL SELF WT = 20 PSF

LL = 200 PSF



FIND beff

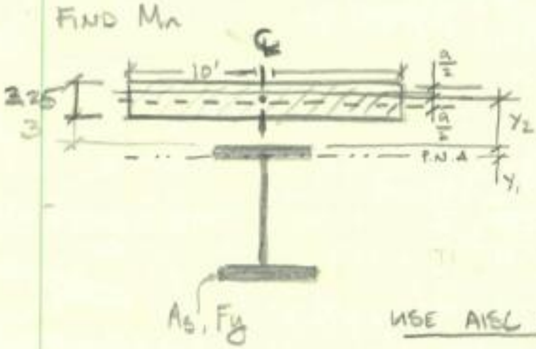
$b_{eff} = \max \begin{cases} S/2 = 10' \\ \ell/8 = 2.6' \end{cases}$

FIND Mu

$w_u = 10[(1.2 \cdot 109) + (1.6 \cdot 200)]/1000 = \underline{4.5\ klf}$

$M_u = \dfrac{w_u \ell^2}{8} = \dfrac{4.5 \cdot (20.67^2)}{8} = \underline{240\ k\text{-}ft}$

FIND Mn



f'_c = 4000 psi (S-200.01)
F_y = 50 ksi (S-200.01)

Assume continuous bracing
Assume a ≈ 1" ∴ Y_2 = 6.25 - 0.5 in
Y_2 = 5.75 in, use $\underline{Y_2 = 5.5 in}$

USE AISC XIV TABLE 3-19 TO SELECT:

TRY W14x SECTION FOR CONSISTENCY (Y_2 = 5.5 ; Y_1 w/in Flange)
ASSUME 3/4" SHEAR STUDS : 1/RIB, WEAK LOCATION PER TABLE 3-21.

W14x22 : ϕM_n = 248 k-ft @ ΣQ_n = 199 k
$n = \dfrac{199k}{17.2k}$ (3/4" , wk, ⊥ DECK, LWf'c=4ksi) = 12 → 24 STUDS XNG
(> 1 stud per rib ∴ NG)

W14x26 : ϕM_n = 250 k-ft @ ΣQ_n = 135k
$n = \dfrac{135}{17.2}$ = 8 → 16 STUDS TW = 20.67×26 + 160 = 697 lb

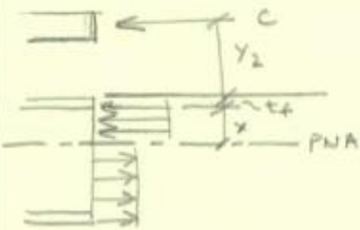
W14x30 : ϕM_n = 263 k-ft @ ΣQ_n = 111 k
$n = \dfrac{111}{17.2}$ = 7 → 14 STUDS TW = 20.67×30 + 140 = 760 lb

3/11

USE W14x26 w/ 16 STUDS

CHECK $\phi M_n \geq M_u$

$b_{eff} = 10'$ (per gravity calcs p.2)

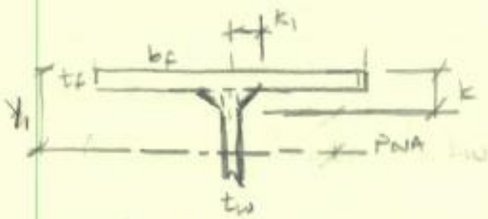


$\Sigma Q_n = 17.2 \cdot 8 = 137.6 \text{ k}$

$\Sigma Q_n = 0.85 f'_c b a$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{137.6}{0.85(4)(10' \times 12)}$$

$a = 0.337 \text{ in}$



$Y_1 = ?$

$b_f = 5.03 \text{ in}$

$t_f = 0.42 \text{ in}$

$t_w = 0.255 \text{ in}$

$k_1 = 0.75 \text{ in}$

$k = 0.82 \text{ in}$

FIND AREA OF STEEL IN COMPRESSION; A_{sc} ASSUME WEB

$$A_{sc} = b_f t_f + (Y_1 - t_f) t_w + (k - t_f) k_1$$
$$= (5.03)(0.42) + [(Y_1 - 0.42)0.255] + [(0.82 - 0.42)0.75]$$
$$= 2.113 + 0.255 Y_1 - 0.107 + 0.3$$
$$= 2.306 + 0.255 Y_1$$

FIND Y_1: $T = C$

$$A_s F_y = 0.85 f'_c b_{eff} a + 2 A_{sc} F_y$$

$$\frac{A_s F_y - 0.85 f'_c b_{eff} a}{2 F_y} = A_{sc}$$

$$\underbrace{\frac{7.69}{2}}_{=3.845} - \frac{\overbrace{0.85(4)(120)(0.337)}^{=137.5}}{\underbrace{2(50)}_{=100}} = 2.306 + 0.255 Y_1$$

$$\frac{2.470 - 2.306}{0.255} = Y_1$$

$Y_1 = 0.643 \text{ in} \gg 0.42 \text{ in}$ ∴ PNA IN WEB

4/11

FIND A_{sc} CENTROID

$$\bar{y} = \frac{\Sigma A_i y_i}{\Sigma A_i} = \frac{2.113\left(\frac{0.42}{2}\right) + (0.255 \cdot 0.643)\left(\frac{0.643-0.42}{2}\right) + 0.3\frac{(0.82-0.42)}{2}}{[2.306 + 0.255(0.443) = 2.47]}$$

$\bar{y} = 0.211$ in FROM TOP OF STEEL

FIND M_n FROM CONCRETE COMPRESSION LOCATION:

$$y_2 = 6.25 - \frac{0.337}{2} = 6.082$$

$$A_s f_y \left(y_2 + \frac{d}{2}\right) - 2 A_{sc} F_y (y_2 + \bar{y}) = M_n$$

$$7.69(50)\left(6.082 + \frac{13.9}{2}\right) - 2(2.47)(50)(6.082 + 0.211) = M_n$$

$$M_n = 3456 \text{ k·in} = 288 \text{ k·ft}$$

$\phi M_n \geq M_u$

$0.9 \cdot 288 \geq 240$

$\boxed{259 \text{ k·ft} \geq 240 \text{ k·ft} \therefore \text{OK STRENGTH}}$

* NOTE: DRAWINGS SELECT W14x26 WITH 18 SHEAR STUDS, WHICH IS ONE (1) ADDITIONAL ON EACH END.

$\Sigma Q_n (9) = 17.2 \cdot 9 = 155$ k

FIND y_1 WHEN $\Sigma Q_n = 155$ k. $\quad a = \frac{155}{0.85 \cdot 4 \cdot 120} = 0.38$ in

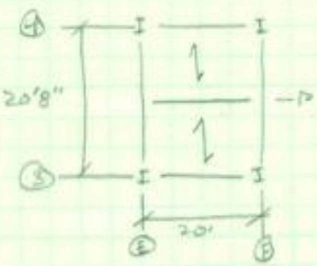
$$\frac{3.815 - 1.5504 - 2.306}{0.255} = y_1$$

$y_1 = -0.0447$; PNA IS NOT IN WEB

APPENDIX C: PURLIN-GIRDER SYSTEM

PURLIN 1

DESIGN PURLIN FLOOR SYSTEM : TRY 3'-0" SPCG



PICK DECK w/ 2HR FIRE RATING

- REQUIRES FIREPROOFING (SPRAY FIBER)
VULCRAFT NON-COMPOSITE

$2\frac{1}{2}$" TOPPING G801 | G804

$\gamma_{CONCR} = 110 \cdot 2.5/12 = 23$ PSF

Allowable q_0 = LL 200
15 MEP / CEILING
25 FLR
23 CONCR
263 PSF

USE VULCRAFT 1.0C24 AT 2-3 SPAN @ 3'0"

STR : 272 PSF ≥ 263 PSF ∴ ok
DEFLECTION ∴ ok (DOES NOT CONTROL)
CONSTRUCTION : 8'-7" ≥ 3'-0" ok

wt = 29 PSF

$q_u = 1.2(15 + 25 + 29) + (200)1.6 = 403$ PSF × 3'0" SPN = 1210 PLF

$w_u = \underline{1.21 \text{ klf}}$; 10' 3" length = 12.4 k

USE C7 × 9.8 : 12.4k ≥ 12.4 k ∴ ok

CHECK DEFLECTION

$$\ell/360 \geq \frac{5}{384EI} \cdot w\ell^4$$

$$\frac{10.25 \times 12}{360} \geq \frac{5}{384\,(29000)(21.2)} \cdot \frac{1.21}{12}\,(10.25 \times 12)^4$$

(ksi, in^4, klf)

0.34 in ≥ 0.489 in NG

$$I_{min} = \frac{5 \cdot 360}{384}\,\frac{w\ell^3}{E} = 4.6875 \cdot \frac{1.21}{12} \cdot \frac{(10.25 \times 12)^3}{29000}$$

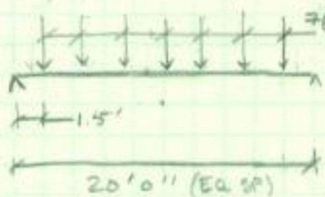
$I_{min} = 30.4\,in^4$

C8 × 11.5 , I = 32.5 in^4 ≥ 30.4k ok

Max LOAD = 20.3k ≥ 12.4k ∴ ok

PURLIN 2

DESIGN BM P



$12.4 + (1.2(10.25 \times 11.50)) = 12.54\,k$

ASSUME BRACED BY DECK

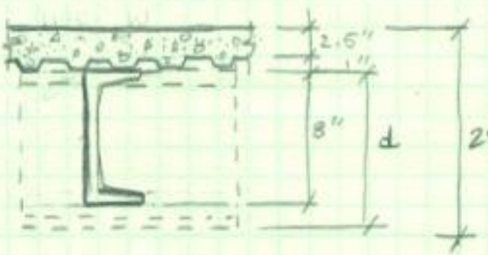
FIND Mu

$w_u = \frac{7(12.54\,k)}{20} = 4.4\,klf$

$M_u = \frac{4.4(20^2)}{8} = 220\,k\text{-}ft$

USE TABLE 3-10 TO SELECT BM SIZE

FLR HT = 11'6" ∴ CONSIDER STRUCT. DEPTH



SELECT W16 × 36 : $\phi M_n = 240\,k\text{-}ft \geq 220\,k\text{-}ft$ ok

CHECK DEFL. $I = 448\,in^4$

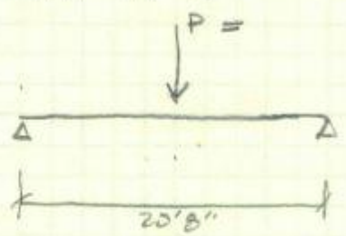
$I_{min} = \frac{5 \cdot 360}{384} \cdot \frac{w l^3}{E} = 4.6875 \cdot \frac{4.4}{12} \cdot \frac{(20 \times 12)^3}{29000}$

$I_{min} = 819\,in^4$ ∴ NG ∴ TRY W18×55 $I = 890\,in^4$

$\phi M_n = 420\,k\text{-}ft \geq 220\,k\text{-}ft = M_u$ TABLE 3-10 ok

PURLIN 3

DESIGN BM G:



$P = \frac{2(4.4 \cdot 20')}{2} = 44k \times 2 = 88k$

$M_{max} = \frac{PL}{4} = \frac{88 \cdot 20.67}{4}$

$M_{max} = 456$ k-ft

TABLE 3-10 AISC XIV:

SELECT W24 x 84

$\left.\begin{array}{l}\phi M_n\ 20' = 521\ ft\text{-}k \\ \phi M_n\ 22' = 453\ ft\text{-}k\end{array}\right\} \phi M_{n\,20.67'} = 475.5\ k\text{-}ft$

CHECK SELF-WT.

84 PLF = 0.084 KLF

$M_u = \frac{0.084(20.67^2)}{8} = 4.5\ k\text{-}ft \longrightarrow w_{eq} = 8.63\ klf$

$M_u = 456 + 4.5 = 461\ k\text{-}ft$

$\phi M_n = 475\ k\text{-}ft \geq 461\ k\text{-}ft \therefore ok$

CHECK DEFLECTION

$I_{min} = \frac{3 \cdot 360}{384} \cdot \frac{w l^3}{E} = 4.6875 \cdot \left(\frac{8.63}{12}\right) \frac{(20.67 \times 12)^3}{29000}$

$I_{min} = 1774\ in^4$

$I_{24\times84} = 2370 \geq 1774$ ok

CHECK V:

$\phi V_n = 340k$

$V_u = 22 + 0.084(10.33) = 23k \therefore ok$

VULCRAFT

SLAB INFORMATION

Total Slab Depth, in.	Theo. Concrete Volume Yd³ / 100 ft²	Theo. Concrete Volume ft³ / ft²	Recommended Welded Wire Fabric
2 1/2	0.62	0.157	6x6 - W1.4xW1.4
3	0.77	0.208	6x6 - W1.4xW1.4
3 1/2	0.93	0.250	6x6 - W1.4xW1.4
3 3/4	1.00	0.271	6x6 - W1.4xW1.4
4	1.08	0.292	6x6 - W2.1xW2.1
4 1/2	1.23	0.333	6x6 - W2.1xW2.1
4 3/4	1.31	0.354	6x6 - W2.1xW2.1
5	1.39	0.375	6x6 - W2.1xW2.1

SECTION PROPERTIES

Deck Type	Design Thickness in.	Deck Weight psf	Section Properties I_p in⁴/ft	I_n in⁴/ft	S_p in³/ft	S_n in³/ft	V_a lbs/ft	F_y ksi
1.0C26	0.0179	0.96	0.040	0.042	0.067	0.071	2216	60
1.0C24	0.0239	1.28	0.057	0.059	0.098	0.103	3867	60
1.0C22	0.0295	1.57	0.073	0.073	0.130	0.134	4803	60
1.0C20	0.0358	1.91	0.088	0.088	0.167	0.165	5744	60

NON-COMPOSITE

ALLOWABLE UNIFORM LOAD (PSF)

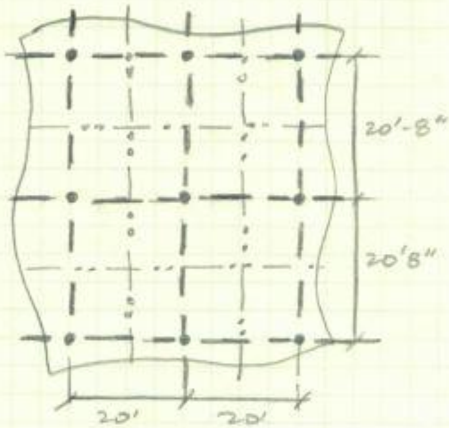
TYPE NO.	NO. OF SPANS	DESIGN CRITERIA	CLEAR SPAN (ft-in) 3-0	3-3	3-6	3-9	4-0	4-6	5-0	5-6	6-0	6-6	7-0	7-6	8-0
1.0C26	1	Fb = 36,000	178	152	131	114	100	79	64	53	45	38	33	29	25
		Defl. = l/240	97	77	61	50	41	29	21	16	12	10	8	6	5
		Defl. = l/180	130	102	82	66	55	38	28	21	16	13	10	8	7
	2	Fb = 36,000	187	159	138	120	106	84	68	56	47	40	35	30	27
		Defl. = l/240	240	189	151	123	101	71	52	39	30	24	19	15	13
		Defl. = l/180	320	252	202	164	135	95	69	52	40	31	25	20	17
	3	Fb = 36,000	232	198	171	149	132	104	84	70	59	50	43	38	33
		Defl. = l/240	188	148	118	96	79	56	41	30	23	18	15	12	10
		Defl. = l/180	250	197	158	128	106	74	54	41	31	25	20	16	13
1.0C24	1	Fb = 36,000	[illegible]	222	192	167	147	116	94	78	65	56	48	42	37
		Defl. = l/240	139	109	87	71	58	41	30	22	17	14	11	9	7
		Defl. = l/180	185	145	116	95	78	55	40	30	23	18	15	12	10
	2	Fb = 36,000	272	232	200	174	153	121	98	81	68	58	50	44	39
		Defl. = l/240	340	267	214	174	143	101	73	55	42	33	27	22	18
		Defl. = l/180	453	356	285	232	191	134	98	73	57	45	36	29	24
	3	Fb = 36,000	[illegible]	289	249	218	191	151	123	102	85	73	63	55	48
		Defl. = l/240	266	209	167	136	112	79	57	43	33	26	21	17	14
		Defl. = l/180	354	279	223	181	149	105	77	58	44	35	28	23	19
1.0C22	1	Fb = 36,000	346	295	254	221	195	154	125	103	86	74	64	55	49
		Defl. = l/240	178	140	112	91	75	53	38	29	22	17	14	11	9
		Defl. = l/180	237	186	149	121	100	70	51	38	30	23	19	15	12
	2	Fb = 36,000	353	301	260	227	200	158	128	106	89	76	65	57	50
		Defl. = l/240	427	336	269	219	180	127	92	69	53	42	34	27	23
		Defl. = l/180	570	448	359	292	240	169	123	92	71	56	45	36	30
	3	Fb = 36,000	440	375	324	283	249	197	160	132	111	95	82	71	63
		Defl. = l/240	334	263	211	171	141	99	72	54	42	33	26	21	18
		Defl. = l/180	446	351	281	228	188	132	96	72	56	44	35	29	24
1.0C20	1	Fb = 36,000	444	379	327	284	250	198	160	132	111	95	82	71	63
		Defl. = l/240	214	168	135	110	90	63	46	35	27	21	17	14	11
		Defl. = l/180	285	224	180	146	120	85	62	46	36	28	22	18	15
	2	Fb = 36,000	435	371	320	279	246	194	158	130	109	93	81	70	62
		Defl. = l/240	515	405	324	264	217	153	111	84	64	51	41	33	27
		Defl. = l/180	687	540	433	352	290	204	148	111	86	68	54	44	36
	3	Fb = 36,000	541	462	399	348	306	242	197	163	137	117	101	88	77
		Defl. = l/240	403	317	254	206	170	119	87	65	50	40	32	26	21
		Defl. = l/180	538	423	339	275	227	159	116	87	67	53	42	34	28

27

APPENDIX D: TWO-WAY SLAB SYSTEM
Hand Calculations: Direct Design Method

2-WAY 1

DESIGN TWO-WAY SLAB ON SPIRAL COLUMNS



FOR CONSISTENCY WITH PROJECT REQUIREMENTS:

$f'_c = 4000$ psi (LT-WT)
$\lambda = 0.85$
$f_y = 70$ ksi

- ASSUME NO EDGE OR INTERIOR BEAMS.
- ASSUME DROP PANELS

ASSUME COL. DIA = 18" (CHECK P. 2)

FIND h_{min}:

TABLE 9.5(c) - interpolate for $f_y = 70$ ksi

$h_{min} = l_n/35 = (21-1.5)/35 \times 12 = 6.6$ in

use SLAB h = 7" > 6.6 in ok

7" > 4" (9.5.3.2) ok

2-WAY 2

PRELIMINARY SIZE OF TYPICAL COLUMNS: CHECK 18" ASSUMPTION SIZE ACCORDING TO TYPICAL STEEL COL:

P ≈ 1500 k . ASSUMES DL OF 69 PSF FOR COMPOSITE CONSTRUCTION

7" LWT 2-WAY RC SLAB: 65 PSF ∴ ASSUME P STAYS SAME

ASSUME NO FLEXURE EXPOSURE, SPIRAL COLUMNS TO MATCH BAR ROUNDS (ARCH. EXPOSED STL)

$\phi P_n \geq 0.85\phi\left[0.85 f'c\, A_c + f_y A_s\right]$ EQN 10-1 ACI 318-11

$\phi = 0.85$ SPIRAL

$1500 \geq 0.85^2\left[0.85 f'c\, A_c + f_y A_s\right]$

$2076 \geq 0.85(4) A_c + 70 A_s$

$2076 \geq 3.4 A_c + 70 A_s$

ASSUME 18" - DIA. COL.

$A_c = A_g - A_s$

$= \pi(9^2) - A_s$

$= 254.5 - A_s$

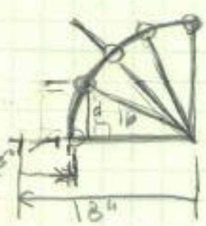
$2076 \geq 3.4(254.5 - A_s) + 70 A_s$

$1210.7 \geq 66.6 A_s$

$A_s = 18.17 \text{ in}^2$

(16) REINF BARS $\Rightarrow A_{si} = \frac{18.17}{16} = 1.13 \text{ in}^2$

USE (16) no. 10 } ok - REASONABLE FOR NOW
(8) no. 14



$= 1" + 0.38" + 1.27/2 = 2"$

CHECK SPCG: $16 \sin(22.5) = d$

$= 6.1" - 1.27"$

$= 4.8"$ ok

$\boxed{\text{USE (16) no 10s}}$

$\boxed{\therefore 18" \text{ COL IS OK}}$

2-WAY 3

CHECK SHEAR CAPACITY W/O SHEAR REINF.

7" SLAB ; d = 6"

FIND LOADS :

DL : SLAB : (7/12 = 0.58) · 110PCF = 65 PSF

MEP/CEILING : 15 PSF

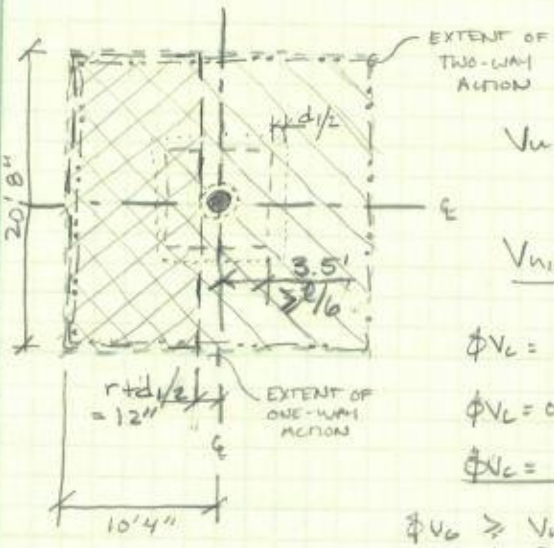
FLOOR FINISH : 25 PSF

$\boxed{105 \text{ PSF}}$

LIVE LOAD : LEVEL 5 GALLERY : $\boxed{200 \text{ PSF}}$

$q_u = 1.2(105) + 1.6(200) = \underline{445 \text{ PSF}}$

CHECK 1-WAY SHEAR : TYPICAL INT. COL



$V_{u\,1\text{-}way} = q_u A$

$= .445[20.67 \times (10.33 - 12/12)]$

$\underline{V_{u\,1\text{-}way} = 85.8 \text{ k}}$

$\phi V_c = \phi \lambda 2 \sqrt{f'_c}\, b_o d$

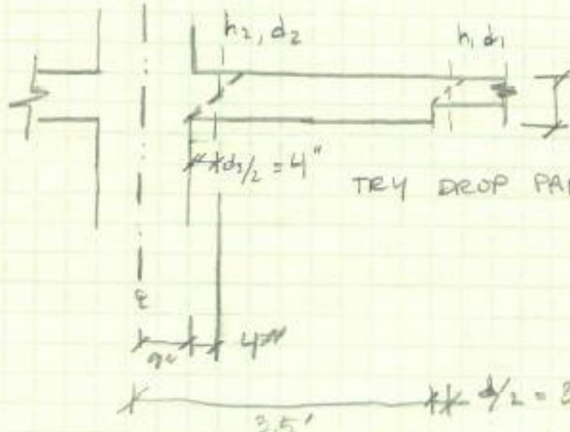
$\phi V_c = 0.75(0.85)\, 2\sqrt{4000} \cdot 20.67 \cdot 6$

$\underline{\phi V_c = 120\text{k}}$

$\phi V_c \geq V_u$

$120\text{k} \geq 85.8\text{k}$ ∴ OK

CHECK 2-WAY SHEAR



$h_2 \geq 1.25 h_1 = 8.75$

TRY DROP PANEL @ 9"

2-WAY 4

$b_{o1} = 4(2(3.5) + 6") = 30' = \underline{360"}$

$b_{o2} = 4(9" + 9") = \underline{72"}$

$\beta = 1$ (CIRCLE COL)

$\alpha_s = 40$ (INT)

CHECK h, d, FOR TWO-WAY SHEAR

$V_u = 0.445(20.67^2 - 7^2) = \underline{168.3\ k}$

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}\, d b_o \cdot \min \begin{vmatrix} \frac{4}{\beta} + 2 = \cancel{6} \\ \frac{\alpha_s d}{b_o} + 2 = \frac{40 \cdot 6}{360} = \underline{2.7} \\ \cancel{4} \end{vmatrix}$$

$\phi V_c = 0.75(.857)\sqrt{4000} \cdot 360 \cdot 6\ (2.7) / 1000$

$\underline{\phi V_c = 235.1\ k}$

$$\boxed{\begin{array}{l} \phi V_c \geq V_u \\ 235.1k \geq 168.3k \ \therefore \text{OK} \end{array}}$$

CHECK 9" DROP PANEL

$\Delta q_u = 2/12 \cdot 110 = 20\text{PSF} \rightarrow 0.465$ KSF

$V_u = 168.3 + 0.465\left(7^2 - \left(\frac{\pi(1.18^2)}{144}\right)\right)$ $= 3.68$

$= 168.3 + 21.1 = \underline{189.4\ k}$

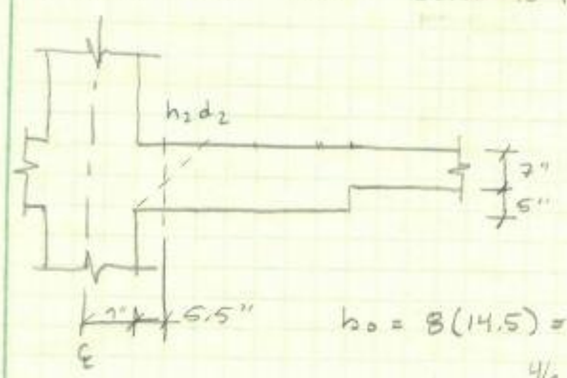
$$\phi V_c = \phi 2\lambda\sqrt{f'_c}\, b_o d \cdot \min \begin{vmatrix} 4/\beta + 2 \\ \frac{\alpha_s d}{b_o} + 2 \\ 4 \end{vmatrix}$$

$$= 0.75 \cdot 0.85\sqrt{4000} \cdot 72 \cdot 8\ \min \begin{vmatrix} \frac{40 \cdot 8}{72} + 2 = \cancel{6.4} \\ 4 \end{vmatrix}$$

$\underline{\phi V_c = 92.9\ k}$ $\therefore$ NG

2-WAY 5

INCREASE DROP PANEL DEPTH TO 12", $d = 11"$



$\Delta q_u = 5/12 \cdot 110 = 46$ PSF

$q_u = 0.491$ KSF

$V_u = 168.3 + 0.491\left(7^2 - \frac{\pi(14.5^2)}{144}\right)$

$V_u = 168.3 + 21.8$

$\underline{V_u = 190.1\ k}$

$b_o = 8(14.5) = \underline{116\ in}$

$$\phi V_c = \phi 2 \lambda \sqrt{f'_c}\, b_o d \cdot \min \begin{cases} 4/\beta + 2 & = 6 \\ \frac{\alpha_s d}{b_o} + 2 & = \frac{40 \cdot 11}{116} = \cancel{5.9} \\ 4 & = \cancel{4} \end{cases}$$

$\phi V_c = 0.75(0.85)(\sqrt{4000})\, 116 \cdot 11 \cdot 4$

$\phi V_c = 205.8\ k$

$\phi V_c \geq V_u$

$205.8k \geq 190.1k \therefore OK$

2-WAY 7

FIND FLEXURAL REINF. TO CHECK DEFL.

FOR TECH 2, USE DDM TO FIND MOMENTS FOR REINF, DEFLECTIONS CAN BE CHECKED AFTER

EQUIVALENT SQ. COL.

$c_1 = 16.02\text{ in}$ (P.1 2-WAY)

$l_{n_{NS}} = 20'8'' - 16'' = 19'4'' = 19.33'$

$l_{n_{EW}} = 20' - 16'' = 18'8'' = 18.67'$

N/S DIRECTION

$M_o = \dfrac{q_u l_2 l_n^2}{8}$

$= \dfrac{0.445 \cdot 20(19.33^2)}{8}$

$M_o = \underline{416\text{ k-ft}}$

E/W DIRECTION

$M_o = \dfrac{q_u l_2 l_n^2}{8}$

$= \dfrac{0.445(20.67)(18.67^2)}{8}$

$M_o = \underline{401\text{ k-ft}}$

ASSUME A TYPICAL INTERIOR SPAN

$\alpha f_1 = 0$ (NO BEAMS; W/IN 15 ft-k; DESIGN BOTH DIRECTIONS

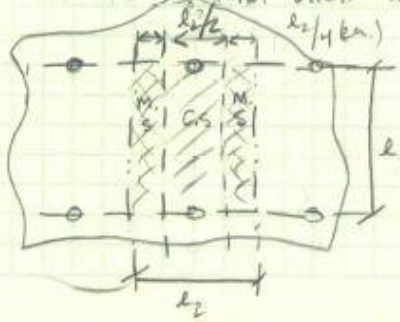
FOR $M_o = \underline{416\text{ ft-k}}$

$M_u^- = 0.65 M_o = 271\text{ ft k}$ (13.6.3.2)

$M_u^+ = 0.35 M_o = 146\text{ ft k}$

LONG DIRECTION: COL. STRIP IS 10' WIDE

MID. STRIP IS 5' ON EA. SIDE



$l_2/2 = 10'$

$l_2/4 = 5'$

2-WAY 8

13.6.4.1, 13.6.4.4 - COL. STRIP, MIDDLE STRIP MOMENT RESISTANCE

$\ell_2/\ell_1 \approx 1$, $\alpha_{f1} = 0$

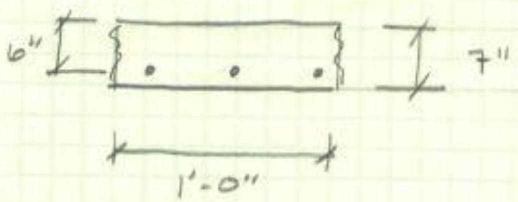
$M_u^- = 271$ ft k

	FACTOR	MOMENT
C.S.	0.75	204 ft k
M.S.	0.25	68 ft k

$M_u^+ = 146$ ft k

	FACTOR	MOMENT
C.S.	0.60	88 ft k
M.S.	0.40	58 ft k

DESIGN (+) REINF. AT COL. STRIP (CONTROLS)
ASSUME 1'-0" WIDE SECTION



$M_u = \dfrac{88 \text{ ft-k}}{10 \text{ ft}} = 8.8$ ft k

FIND $A_{s,min}$:

$$A_{s,min} = \frac{b_w d}{f_y} \max \begin{vmatrix} 3\sqrt{f'_c} \\ 200 \end{vmatrix}$$

$$= \frac{12 \cdot 6}{70000} \max \begin{vmatrix} 3\sqrt{4000} = \cancel{189} \\ 200 = 200 \end{vmatrix}$$

$A_{s,min} = 0.206 \text{ in}^2 / \text{ft}$

FIND $A_{s,req'd} = \dfrac{M_u}{\phi f_y jd}$ (Assume $jd = 0.95$)

$$A_{s,req'd} = \frac{88 \,(\text{k} \xrightarrow{} 12{,}000)}{0.9(70{,}000)(0.95 \cdot 6)} = 0.293 \text{ in}^2$$

kft → lb·in

$0.293 \text{ in}^2 > 0.206 \text{ in}^2$; $\underline{A_{s,req'd} \text{ controls}}$

2-WAY 9

SPACING REQUIREMENTS:

$$s_{min} \le s \le s_{max} \qquad (7.6)$$

$$\text{Max}\left|\begin{array}{l} d_b \\ 1" \end{array}\right. \le s \le \text{min}\left|\begin{array}{l} \cancel{3h = 21"} \\ 18 = 18" \end{array}\right.$$

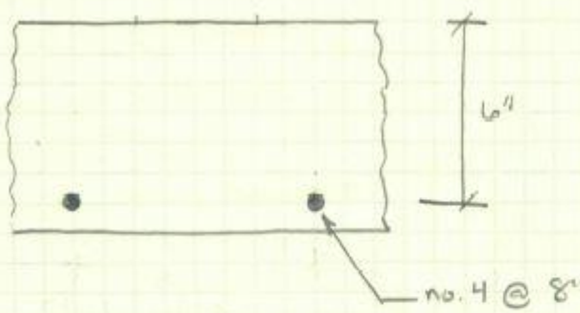
no. (4)s @ 8" spacing

$$b/s \times A_s \ge A_{s\,min}$$

$$12/8 \cdot 0.2\,in^2 \ge 0.293\,in^2$$

$$0.3\,in^2 \ge 0.293\,in^2$$

$$\cancel{\text{max}\left|\begin{array}{l} 0.5 \\ 1" \end{array}\right.} \le 8" \le 18" \quad \text{ok}$$

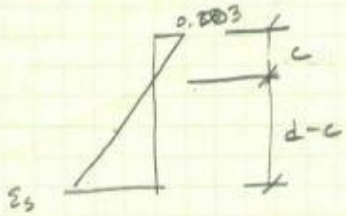


Assume $\varepsilon_s \ge 0.005$

$$A_s f_y = 0.85\, f'_c\, b a$$

$$a = \frac{0.3 \cdot 70000}{0.85(4)(12)} = \underline{0.515\,in}$$

2-WAY 10



$\beta_1 = 0.85$ ($f'_c = 4000$)

$a = \beta_1 c$

$c = \frac{a}{\beta_1} = \frac{.515}{0.85} = 0.61$ in

$\frac{\varepsilon_s}{d-c} = \frac{\varepsilon_u}{c} \rightarrow \varepsilon_s = (6 - 0.61) \cdot \left(\frac{0.003}{0.61}\right)$

$\varepsilon_s = 0.026 >> 0.005$ ∴ ok

FIND ϕM_n

$\phi = 0.9$ ($\varepsilon_s >> 0.005$)

$M_n = A_s f_y (d - a/2)$

$= 0.3(70)\left(6 - \frac{0.515}{2}\right)$

$M_n = 120.6$ k.ft

$\phi M_n = 0.9 \cdot 120.6$

$\boxed{\phi M_n = 109 \text{ k.ft} \geq 8.8 \text{ k.ft} = M_u^{(+)} \therefore \text{ok}}$

CHECK A_s req'd : (ASSUMPTION $jd = 0.95$

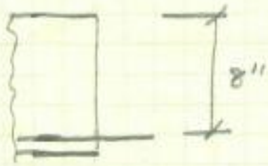
$jd = d - a/2 = 6 - \frac{0.515}{2} = 5.74$

jd assumed $= 0.95 \cdot 6 = 5.70$

$A_{s\,req'd} = 0.3 \cdot \frac{5.70}{5.74} = 0.29 \leq 0.3$ ∴ ok

2-WAY

DESIGN REINF. FOR M_u^- AT DROP PANEL



$M_u^- = 204$ ft·k TOTAL

$M_u^- = 20.4$ ft·k /ft

FIND $A_{s\,min}$: Assume b is same, f_y is same

$A_{s\,min} = 0.206 \times \frac{8}{6} = \underline{0.275\ in^2}$

FIND $A_{s\,req'd} = \frac{M_u}{\phi f_y jd}$ Assume $jd = 0.95d = 7.6$

$A_{s\,req'd} = \frac{20.4 \times 12000}{0.9 \cdot 70000 \cdot 7.6} = \underline{0.51\ in^2}$

$A_{s\,req'd} > A_{s\,min}$ ∴ use $0.51\ in^2$ /ft

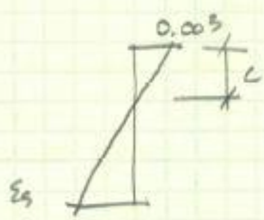
use no. 6 bars @ 8'' o.c

$12/8 \cdot 0.44 = 0.66\ in^2 > 0.51\ in^2$ ∴ ok

$1'' \le 8'' \le 18''$ ∴ spacing ok

Assume $\varepsilon_s \geq 0.005$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.66 \cdot 70}{0.85 \cdot 4 \cdot 12} = 1.13\ in$



$c = \frac{1.13}{0.85} = 1.33$

$\varepsilon_s = (8 - 1.33)\left(\frac{0.003}{1.33}\right)$

$\varepsilon_s = 0.015 > 0.005$ ∴ ok

$\phi = 0.9$

2-WAY 12

CHECK jd : $d - a/2 = 8 - \left(\frac{1.33}{2}\right) = 7.44$

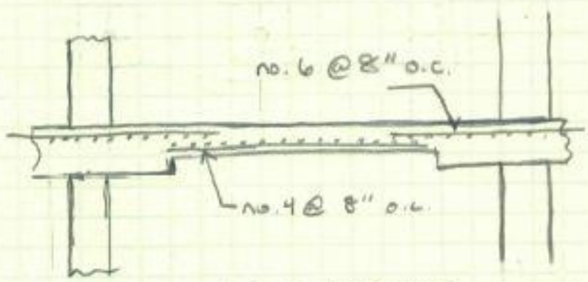
$A_s\ reqd = \frac{7.6}{7.44} = 0.51 , \frac{7.6}{7.44} = 0.52 < 0.66\ in^2$ ∴ ok

FIND ϕM_n :

$\phi M_n = \phi A_s f_y (d - a/2)$

$= 0.9(0.66)(70)(7.44)$

$\boxed{\phi M_n = 309\ ft\cdot k \geq 20.4 = M_u \therefore ok}$



IN BOTH DIRECTIONS

spSlab Data: Equivalent Frame

```
====================================================================================================================
[1] INPUT ECHO
====================================================================================================================

General Information
===================
   File name: C:\TEMP\felton_2way.slb
   Project: Tech 2 - Two-Way Flat Slab with Drops
   Frame:
   Engineer:
   Code: ACI 318-08
   Reinforcement Database: ASTM A615
   Mode: Design
   Number of supports = 4
   Floor System: Two-Way

   Live load pattern ratio = 75%
   Minimum free edge for punching shear = 4 times slab thickness
   Deflections are based on cracked section properties.
   In negative moment regions, Ig and Mcr DO NOT include flange/slab contribution (if available)
   Long-term deflections are calculated for load duration of 60 months.
   30% of live load is sustained.
   Compression reinforcement calculations NOT selected.
   Default incremental rebar design selected.
   User-defined slab strip widths NOT selected.
   User-defined distribution factors NOT selected.
   One-way shear in drop panel NOT selected.
   Distribution of shear to strips NOT selected.
   Beam T-section design NOT selected.
   Longitudinal beam contribution in negative reinforcement design over support NOT selected.
   Transverse beam contribution in negative reinforcement design over support NOT selected.

Material Properties
===================
            Slabs|Beams       Columns
           ------------   ------------
   wc    =          110            110 lb/ft3
   f'c   =            4              4 ksi
   Ec    =       2407.9         2407.9 ksi
   fr    =      0.35576        0.35576 ksi

   fy    =           70 ksi, Bars are not epoxy-coated
   fyt   =           60 ksi
   Es    =        29000 ksi

Reinforcement Database
======================
   Units: Db (in), Ab (in^2), Wb (lb/ft)
   Size       Db       Ab       Wb    Size       Db       Ab       Wb
   ---- -------- -------- --------    ---- -------- -------- --------
     #3     0.38     0.11     0.38      #4     0.50     0.20     0.67
     #5     0.63     0.31     1.04      #6     0.75     0.44     1.50
     #7     0.88     0.60     2.04      #8     1.00     0.79     2.67
     #9     1.13     1.00     3.40     #10     1.27     1.27     4.30
    #11     1.41     1.56     5.31     #14     1.69     2.25     7.65
    #18     2.26     4.00    13.60

Span Data
=========
   Slabs
   -----
   Units: L1, wL, wR (ft); t, Hmin (in)
   Span Loc       L1        t       wL       wR     Hmin
   ---- ---- -------- -------- -------- -------- --------
      1 ExtL   20.670     7.00   10.000   10.000     7.33   *b
      2 Int    20.670     7.00   10.000   10.000     6.70
      3 ExtR   20.670     7.00   10.000   10.000     7.33   *b
   NOTES:
   *b - Slab thickness is less than minimum. Deflection check required.
```

```
Support Data
============
   Columns
   -------
   Units: c1a, c2a, c1b, c2b (in); Ha, Hb (ft)
   Supp      c1a      c2a       Ha        c1b      c2b       Hb  Red%
   ---- -------- -------- --------   -------- -------- -------- ----
      1    16.02    16.02   11.050      16.02    16.02   11.500  100
      2    16.02    16.02    0.000      16.02    16.02   11.500  100
      3    16.02    16.02    0.000      16.02    16.02   11.500  100
      4    16.02    16.02   11.050      16.02    16.02   11.500  100

   Drop Panels
   -----------
   Units: h (in); L1, L2, W1, W2 (ft)
   Supp        h       L1       L2       W1       W2
   ---- -------- -------- -------- -------- --------
      1     9.00    0.000    3.500    3.500    3.500 *d
      2     9.00    3.500    3.500    3.500    3.500 *d
      3     9.00    3.500    3.500    3.500    3.500 *d
      4     9.00    3.500    0.000    3.500    3.500 *d
   *d - Excessive drop thickness will not be used for flexural design.

   Boundary Conditions
   -------------------
   Units: Kz (kip/in); Kry (kip-in/rad)
   Supp    Spring Kz   Spring Kry Far End A Far End B
   ---- ------------ ------------ --------- ---------
      1            0            0     Fixed     Fixed
      2            0            0     Fixed     Fixed
      3            0            0     Fixed     Fixed
      4            0            0     Fixed     Fixed

Load Data
=========
   Load Cases and Combinations
   ---------------------------
   Case      SELF    Super     Live
   Type      DEAD     DEAD     LIVE
   ---- -------- -------- --------
   U1       1.200    1.200    1.600

   Area Loads
   ----------
   Units: Wa (lb/ft2)
   Case/Patt Span           Wa
   --------- ---- ------------
   SELF         1        64.17
                2        64.17
                3        64.17
   Live         1       200.00
                2       200.00
                3       200.00
   Super        1        40.00
                2        40.00
                3        40.00
   Live/Odd     1       150.00
                3       150.00
   Live/Even    2       150.00
   Live/S1      1       150.00
   Live/S2      1       150.00
                2       150.00
   Live/S3      2       150.00
                3       150.00
   Live/S4      3       150.00

   Line Loads
   ----------
   Units: Wa, Wb (lb/ft), La, Lb (ft)
   Case/Patt Span           Wa           La           Wb           Lb
   --------- ---- ------------ ------------ ------------ ------------
   SELF         1       577.50        0.000       577.50        3.500
                1       577.50       17.170       577.50       20.670
                2       577.50        0.000       577.50        3.500
                2       577.50       17.170       577.50       20.670
                3       577.50        0.000       577.50        3.500
                3       577.50       17.170       577.50       20.670
```

```
Reinforcement Criteria
======================
   Slabs and Ribs
   --------------
                 _____Top bars___  __Bottom bars___
                    Min      Max      Min      Max
                 -------  -------  -------  -------
     Bar Size         #4       #8       #4       #8
     Bar spacing    1.00    12.00     1.00    12.00 in
     Reinf ratio    0.14     5.00     0.14     5.00 %
     Cover          0.75              0.75          in
     There is NOT more than 12 in of concrete below top bars.


Beams
   -----
                 _____Top bars___  __Bottom bars___    ____Stirrups____
                    Min      Max      Min      Max        Min      Max
                 -------  -------  -------  -------    -------  -------
     Bar Size         #5       #8       #5       #8         #3       #5
     Bar spacing    1.00    18.00     1.00    18.00       6.00    18.00 in
     Reinf ratio    0.14     5.00     0.14     5.00 %
     Cover          1.50              1.50          in
     Layer dist.    1.00              1.00          in
     No. of legs                                              2        6
     Side cover                                            1.50          in
     1st Stirrup                                           3.00          in
     There is NOT more than 12 in of concrete below top bars.
```

```
=================================================================================================================
[2] DESIGN RESULTS*
=================================================================================================================

*Unless otherwise noted, all results are in the direction of analysis only. Another analysis
 in the perpendicular direction has to be carried out for two-way slab systems.

Strip Widths and Distribution Factors
=====================================
   Units: Width (ft).
               __________Width___________   ______ Moment Factor______
   Span Strip    Left**  Right**  Bottom*     Left**  Right**  Bottom*
   ---- ------ -------- -------- --------   -------- -------- --------
      1 Column    10.17    10.00    10.17      1.000    0.750    0.600
        Middle     9.83    10.00     9.83      0.000    0.250    0.400

      2 Column    10.00    10.00    10.00      0.750    0.750    0.600
        Middle    10.00    10.00    10.00      0.250    0.250    0.400

      3 Column    10.00    10.17    10.17      0.750    1.000    0.600
        Middle    10.00     9.83     9.83      0.250    0.000    0.400
   *Used for bottom reinforcement.   **Used for top reinforcement.

Top Reinforcement
=================
   Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)
   Span Strip  Zone     Width       Mmax     Xmax    AsMin    AsMax    SpReq    AsReq     Bars
   ---- ------ ------ -------- ---------- -------- -------- -------- -------- -------- -------
      1 Column Left      10.17      67.19    0.667    2.419   18.854    9.385    0.889   13-#4  *3
               Middle    10.17      21.70   13.235    1.318   11.334   11.092    0.696   11-#4  *3 *5
               Right     10.00     138.69   17.170    1.296   11.147    5.000    4.722   24-#4

        Middle Left       9.83       0.24    1.140    1.274   10.960   11.799    0.008   10-#4  *3 *5
               Middle     9.83       4.21   13.235    1.274   10.960   11.799    0.134   10-#4  *3 *5
               Right     10.00     110.77   20.003    1.296   11.147    6.316    3.714   19-#4

      2 Column Left      10.00     121.14    3.500    1.296   11.147    5.000    4.084   24-#4
               Middle    10.00      45.81   13.235    1.296   11.147   12.000    1.486   10-#4  *5
               Right     10.00     121.14   17.170    1.296   11.147    5.000    4.084   24-#4

        Middle Left      10.00      93.08    0.667    1.296   11.147    6.316    3.091   19-#4
               Middle    10.00      15.27   13.235    1.296   11.147   12.000    0.488   10-#4  *3 *5
               Right     10.00      93.08   20.003    1.296   11.147    6.316    3.091   19-#4

      3 Column Left      10.00     138.69    3.500    1.296   11.147    5.000    4.722   24-#4
               Middle    10.17      21.70    7.435    1.318   11.334   11.092    0.696   11-#4  *3 *5
               Right     10.17      67.19   20.003    2.419   18.854    9.385    0.889   13-#4  *3

        Middle Left      10.00     110.77    0.667    1.296   11.147    6.316    3.714   19-#4
               Middle     9.83       4.21    7.435    1.274   10.960   11.799    0.134   10-#4  *3 *5
               Right      9.83       0.24   19.530    1.274   10.960   11.799    0.008   10-#4  *3 *5
   NOTES:
   *3 - Design governed by minimum reinforcement.
   *5 - Number of bars governed by maximum allowable spacing.

Top Bar Details
===============
   Units: Length (ft)
               _____________Left______________   __Continuous__   ____________Right_____________
   Span Strip     Bars  Length     Bars  Length     Bars  Length     Bars  Length     Bars  Length
   ---- ------ ------- ------- ------- -------   ------- -------   ------- ------- ------- -------
      1 Column    2-#4    7.05      ---             11-#4   20.67      7-#4    7.05    6-#4    4.54
        Middle     ---              ---             10-#4   20.67      9-#4    4.92     ---

      2 Column    7-#4    7.17     7-#4    4.54     10-#4   20.67      7-#4    7.17    7-#4    4.54
        Middle    9-#4    4.92      ---             10-#4   20.67      9-#4    4.92     ---

      3 Column    7-#4    7.05     6-#4    4.54     11-#4   20.67      2-#4    7.05     ---
        Middle    9-#4    4.92      ---             10-#4   20.67       ---             ---
```

Bottom Reinforcement
====================

Units: Width (ft), Mmax (k-ft), Xmax (ft), As (in^2), Sp (in)

Span	Strip	Width	Mmax	Xmax	AsMin	AsMax	SpReq	AsReq	Bars	
1	Column	10.17	113.12	8.222	1.318	11.334	6.422	3.793	19-#4	
	Middle	9.83	75.41	8.222	1.274	10.960	9.076	2.484	13-#4	
2	Column	10.00	60.76	10.211	1.296	11.147	12.000	1.985	10-#4	
	Middle	10.00	40.50	10.211	1.296	11.147	12.000	1.310	10-#4	*5
3	Column	10.17	113.12	12.448	1.318	11.334	6.422	3.793	19-#4	
	Middle	9.83	75.41	12.448	1.274	10.960	9.076	2.484	13-#4	

NOTES:
*5 - Number of bars governed by maximum allowable spacing.

Bottom Bar Details
==================

Units: Start (ft), Length (ft)

		Long Bars			Short Bars		
Span	Strip	Bars	Start	Length	Bars	Start	Length
1	Column	19-#4	0.00	20.67	---		
	Middle	7-#4	0.00	20.67	6-#4	2.10	15.47
2	Column	10-#4	0.00	20.67	---		
	Middle	7-#4	0.00	20.67	3-#4	3.10	14.47
3	Column	19-#4	0.00	20.67	---		
	Middle	7-#4	0.00	20.67	6-#4	3.10	15.47

Flexural Capacity
=================

Units: x (ft), As (in^2), PhiMn (k-ft)

Span	Strip	x	AsTop	AsBot	PhiMn-	PhiMn+
1	Column	0.000	2.60	3.80	-193.54	113.30
		20.670	4.80	3.80	-350.51	113.30
	Middle	0.000	2.00	1.40	-61.17	43.20
		2.099	2.00	1.40	-61.17	43.20
		3.861	2.00	2.60	-61.17	78.80
		7.435	2.00	2.60	-61.17	78.80
		15.807	2.06	2.60	-62.95	78.80
		17.551	3.80	1.41	-113.20	43.58
		20.670	3.80	1.40	-113.20	43.20
2	Column	0.000	4.80	2.00	-350.51	61.20
		20.670	4.80	2.00	-350.51	61.20
	Middle	0.000	3.80	1.40	-113.20	43.22
		3.101	3.80	1.40	-113.20	43.22
		3.421	3.80	1.56	-113.20	48.02
		4.309	2.73	2.00	-82.78	61.20
		16.361	2.73	2.00	-82.78	61.20
		17.249	3.80	1.56	-113.20	48.02
		17.570	3.80	1.40	-113.20	43.22
		20.670	3.80	1.40	-113.20	43.22
3	Column	0.000	4.80	3.80	-350.51	113.30
		20.670	2.60	3.80	-193.54	113.30
	Middle	0.000	3.80	1.40	-113.20	43.20
		3.119	3.80	1.41	-113.20	43.58
		4.863	2.06	2.60	-62.95	78.80
		16.809	2.00	2.60	-61.17	78.80
		18.571	2.00	1.40	-61.17	43.20
		20.670	2.00	1.40	-61.17	43.20

Slab Shear Capacity
==================

Units: b, d (in), Xu (ft), PhiVc, Vu(kip)

Span	b	d	Vratio	PhiVc	Vu	Xu	
1	240.00	6.00	1.000	102.46	102.97	19.50	*EXCEEDED
2	240.00	6.00	1.000	102.46	83.21	19.50	
3	240.00	6.00	1.000	102.46	102.97	1.17	*EXCEEDED

Flexural Transfer of Negative Unbalanced Moment at Supports
===

Units: Width (in), Munb (k-ft), As (in^2)

Supp	Width	Width-c	d	Munb	Comb	Pat	GammaF	AsReq	AsProv	Add Bars
1	64.02	64.02	14.50	109.91	U1	Odd	0.680	0.992	1.364	---
2	64.02	64.02	14.50	152.95	U1	Odd	0.600	1.222	2.561	---
3	64.02	64.02	14.50	152.95	U1	Odd	0.600	1.222	2.561	---
4	64.02	64.02	14.50	109.91	U1	Odd	0.680	0.992	1.364	---

Punching Shear Around Columns
=============================

Critical Section Properties

Units: b1, b2, b0, CG, c(left), c(right) (in), Ac (in^2), Jc (in^4)

Supp	b1	b2	b0	CG	c(left)	c(right)	Ac	Jc
1	15.51	31.02	62.04	11.63	11.63	3.88	930.6	32044
2	31.02	31.02	124.08	0.00	15.51	15.51	1861.2	3.1594e+005
3	31.02	31.02	124.08	0.00	15.51	15.51	1861.2	3.1594e+005
4	15.51	31.02	62.04	-11.63	3.88	11.63	930.6	32044

Punching Shear Results

Units: Vu (kip), Munb (k-ft), vu (psi), Phi*vc (psi)

Supp	Vu	vu	Munb	Comb	Pat	GammaV	vu	Phi*vc
1	73.14	78.6	37.79	U1	All	0.320	96.2	142.3
2	205.57	110.4	-83.96	U1	All	0.400	130.2	142.3
3	205.57	110.4	83.96	U1	All	0.400	130.2	142.3
4	73.14	78.6	-37.79	U1	All	0.320	96.2	142.3

Punching Shear Around Drops
===========================

Critical Section Properties

Units: b1, b2, b0, CG, c(left), c(right) (in), Ac (in^2), Jc (in^4)

Supp	b1	b2	b0	CG	c(left)	c(right)	Ac	Jc
1	44.88	89.75	179.50	33.66	33.66	11.22	1077	2.2753e+005
2	90.00	90.00	360.00	0.00	45.00	45.00	2160	2.9192e+006
3	90.00	90.00	360.00	0.00	45.00	45.00	2160	2.9192e+006
4	44.88	89.75	179.50	-33.66	11.22	33.66	1077	2.2753e+005

Punching Shear Results

Units: Vu (kip), vu (psi), Phi*vc (psi)

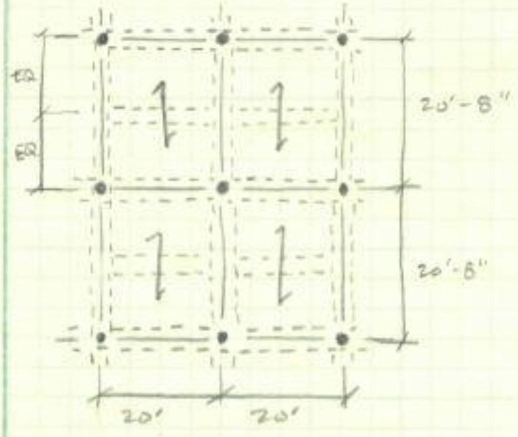
Supp	Vu	Comb	Pat	vu	Phi*vc
1	62.18	U1	All	57.7	94.9
2	183.51	U1	All	85.0	94.9
3	183.51	U1	All	85.0	94.9
4	62.18	U1	All	57.7	94.9

```
Deflections
===========
   Section properties
   ------------------
   Units: Ig, Icr, Ie (in^4), Mcr, Mmax (k-ft)                                  _______________Load Level______________
        ________Ie,avg________                                                  _________Dead_______ ______Dead+Live_____
   Span       Dead   Dead+Live Zone          Ig         Icr      Mcr     Mmax          Ie     Mmax          Ie
   ---- ----------- ----------- ------ ----------- ----------- -------- -------- ----------- -------- -----------
      1      12631        5218 Middle        6860        1818    58.11    44.98        6860   129.07        2278
                               Right        45331       13857   224.63  -124.34       45331  -354.30       21879
      2      18401       13054 Left         45331       13857   224.63  -104.15       45331  -296.78       27506
                               Middle        6860        1238    58.11    10.64        6860    31.60        6860
                               Right        45331       13857   224.63  -104.15       45331  -296.78       27506
      3      12631        5218 Left         45331       13857   224.63  -124.34       45331  -354.30       21879
                               Middle        6860        1818    58.11    44.98        6860   129.07        2278

   Maximum Instantaneous Deflections - Direction of Analysis
   ---------------------------------------------------------
   Units:  D (in), Ig (in^4)
        ___________Frame__________ ____________________________Strips_____________________________
   Span    Ddead    Dlive   Dtotal Strip             Ig    LDF  Ratio    Ddead    Dlive   Dtotal
   ---- -------- -------- -------- ------- ------------ ------ ------ -------- -------- --------
      1    0.108    0.488    0.596 Column       3487.45  0.738  1.451    0.157    0.708    0.865
                                   Middle       3372.55  0.262  0.534    0.058    0.261    0.318
      2    0.011    0.052    0.063 Column          3430  0.675  1.350    0.016    0.070    0.085
                                   Middle          3430  0.325  0.650    0.007    0.034    0.041
      3    0.108    0.488    0.596 Column       3487.45  0.738  1.451    0.157    0.708    0.865
                                   Middle       3372.55  0.262  0.534    0.058    0.261    0.318

   Maximum Long-term Deflections - Direction of Analysis
   -----------------------------------------------------
   Time dependant factor for sustained loads = 2.000
   Units:  D (in)
        ___________________Column Strip____________________ ____________________Middle Strip___________________
   Span    Dsust Lambda      Dcs   Dcs+lu    Dcs+l   Dtotal    Dsust Lambda      Dcs   Dcs+lu    Dcs+l   Dtotal
   ---- -------- ------ -------- -------- -------- -------- -------- ------ -------- -------- -------- --------
      1    0.369  2.000    0.739    1.234    1.447    1.604    0.136  2.000    0.272    0.454    0.533    0.590
      2    0.036  2.000    0.073    0.122    0.143    0.158    0.018  2.000    0.035    0.059    0.069    0.076
      3    0.369  2.000    0.739    1.234    1.447    1.604    0.136  2.000    0.272    0.454    0.533    0.590

Material Takeoff
================
   Reinforcement in the Direction of Analysis
   ------------------------------------------
   Top Bars:      1205.0 lb   <=>  19.43 lb/ft    <=>  0.972 lb/ft^2
   Bottom Bars:   1105.7 lb   <=>  17.83 lb/ft    <=>  0.892 lb/ft^2
   Stirrups:         0.0 lb   <=>   0.00 lb/ft    <=>  0.000 lb/ft^2
   Total Steel:   2310.7 lb   <=>  37.26 lb/ft    <=>  1.863 lb/ft^2
   Concrete:       833.7 ft^3 <=>  13.44 ft^3/ft <=>  0.672 ft^3/ft^2
```

APPENDIX E: ONE-WAY SLAB/BEAM SYSTEM

1 - WAY

DESIGN 1-WAY SLAB / BEAM SYSTEM



$f'_c = 4000$ psi

$f_y = f_{yt} = 70$ksi

$\lambda = 0.85$ (LTWT)

$w_c = 110$ PCF

ASSUME 18" SPIRAL COLUMNS FROM TWO-WAY CALCS

SUPERIMPOSED

DL = 40 PSF

LL = 200 PSF

FIND MIN SLAB THICKNESS FROM TABLE 9.5 (a)

$h_{min} = l/28 = 10.33 \times 12/28 = 4.4$ in

- ADJUST FOR w_c, f_y

$F_{wc} = (1.65 - 0.005(115)) = 1.08$

$F_{fy} = (0.4 + 70000/100000) = 1.1$

$4.4 \times 1.08 \times 1.1 = 5.5$ in slab, $d = 4.5"$

DEFLECTION CALCS UNNECESSARY.

$q_{sw} = (110 \times 5.5/12) = 50$ PSF

$q_u = 1.2(D + q_{sw}) + 1.6(L)$

$= 1.2(90) + 1.6(200) = 428$ PSF $= \underline{0.428 \text{ KSF}}$

1-WAY 2

DESIGN FOR SHEAR WILL CONTROL OVER MOMENT CAPACITY

FIND V_u:

ASSUME BEAMS ARE 18" WIDE TO MATCH COL. DIA.

$V_u = \frac{1}{2} \cdot 0.428 \cdot (20' - 1.5') \cdot (\frac{20.67'}{2} - 1.5') = 0.214(18.5')(8.835')$

$V_u = 35\ k$

FIND ϕV_c:

$V_c = 2\lambda\sqrt{f'_c} \cdot b_w \cdot d = 2(0.85)\sqrt{4000} \cdot 18.5^{\times 12} \cdot 4.5$

$V_c = 107\ k \quad ; \phi = 0.75$

$\phi V_c = 80k \geq 35\ k \quad \therefore ok$

DESIGN FOR MOMENT CAPACITY:

1-WAY 3

FIND A_s

$$A_{s,min} = 200 \frac{bd}{f_y} \quad \text{(PER 2-WAY CALCS)}$$

$$= 200 \frac{12" \times 4.5"}{70000} = 0.154 \, in^2 / ft \text{ width}$$

$A_{s,req'd}$: Assume $jd = 0.95 \cdot 4.5 = 4.275$ in

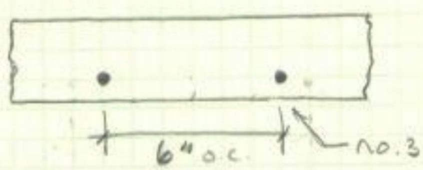
$$A_{s,req'd} = \frac{M_u}{\phi f_y jd} = \frac{\frac{(0.429 \, ksf)(10.25)^2}{12} \times (12000)}{0.9 \cdot 70000 \cdot 4.275}$$

$A_{s,req'd} = 0.168 \, in^2$ / ft width *controls

use no. 3 @ 6" o.c.

$\frac{12}{6} \cdot 0.11 \geq 0.168$

$0.22 \geq 0.168$ ok

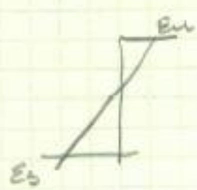


$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.22 \cdot 70000}{0.85 \cdot 4 \cdot 12} = 0.377 \text{ in}$$

$$jd = d - a/2 = 4.5 - \frac{0.377}{2} = 4.3 \text{ in}$$

$$A_{s,req'd} = 0.168 \cdot \frac{4.275}{4.3} = 0.167 \leq 0.220 \therefore \text{ok}$$

$$c = \frac{a}{\beta} = \frac{0.377}{0.85} = 0.444 \text{ in}$$



$$\varepsilon_s = (4.5 - 0.444)\left(\frac{0.003}{0.444}\right)$$

$$\varepsilon_s = 0.027 \gg 0.005 \therefore \phi = 0.9$$

1-WAY 4

FIND M_n :

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$\phi M_n = 0.9(0.22)(70000)(4.3)$$

$$\phi M_n = 59.6 \text{ k-in} = 5 \text{ k-ft}$$

FIND M_u :

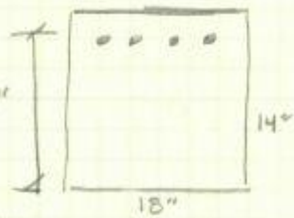
$$M_u^+ = \frac{0.429 \times (10.25 - 1.5)^2}{16} \quad (8.3.3)$$

$$M_u^+ = 2.1 \text{ k-ft}$$

$$\boxed{\begin{array}{l} M_u^+ \leq \phi M_n \\ 2.1 \text{ kft} \leq 5 \text{ kft} \therefore \text{ok} \end{array}}$$

1-WAY 5

WITH 5.5" SLAB : DESIGN BM



FIND h_{min}

TABLE 9.5a

$h_{min} = \ell/21 = 20 \times 12/21 = 11.5$ in

$11.5 \cdot 1.08 \cdot 1.1 = 13.6 \rightarrow$ USE 14" BM , $d = 12$"

ASSUME $b \times h$: 18" x 14"

$w_{SW} = 110 \times 1.5 \times 14/12 = 193$ plf $\rightarrow$ 200 plf

$w_u = 1.2(0.2) + \left(\frac{20.67}{2} \times 0.428\right) = 4.7$ klf

DESIGN FOR SHEAR :

$V_u = \frac{1}{2} w_u \ell_n = \frac{1}{2} \cdot 4.7 \cdot 18.5 = \underline{43.5k}$

FIND V_c :

$V_c = 2\lambda\sqrt{f'_c}\, bd = 2(0.85)\sqrt{4000} \cdot 18 \cdot 12 / 1000$

$\underline{V_c = 23.2k}$

$V_s = V_u/\phi_s - V_c = 43.5/0.9 - 23.2$

$\underline{V_s = 25.1\ k}$ $\leq V_{s\,max} = 8\sqrt{f'_c}\, b_w d = 8\sqrt{4000} \cdot 18 \cdot 12 = 109\ k \therefore ok$

1-WAY 6

CHECK AGAINST $4\sqrt{f'_c}\, b_w d = {}^{109}/_{2} = \underline{54.5k}$

$25.1k \leq 54.5k$

$$S_{max} = \min \left| \begin{array}{lll} d/2 & = & 6 \\ 0.75h & = & 0.75 \cdot 14 = \cancel{10.5} \\ 24 & = & \cancel{24} \end{array} \right.$$

*

$S_{max} = 6''$

$$A_{v\,min} = \frac{b_w s}{f_{yt}} \cdot \max \left| \begin{array}{l} 0.75\sqrt{f'_c} = \cancel{47.1} \\ 50 \end{array} \right.$$

$$= 50 \cdot \frac{18 \cdot 6}{70000} = 0.077\ in^2 \ |\ \text{stirrup}$$

use no. 3 ; $A_v = 0.22\ in^2$ | stirrup

FIND SPACING:

$$s = \frac{d\, A_v f_{yt}}{V_s} = \frac{12 \cdot 0.22 \cdot 70}{25.1} = 7.4\ in.$$

FIND END OF SHEAR REINF.

$V_c = V_{max} - w_u x$

$23.2 = 40 - 4.7x$

$x = 3.6$ ft

$V_{max} = V_u - w_u \cdot b/2$

$= 43.5 - 4.7(0.75)$

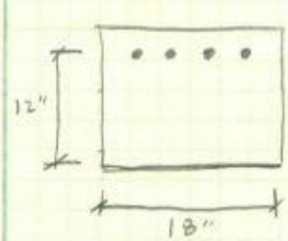
$V_{max} = 40k$

use (8) no. 3 x [ ] @ 6" o.c FROM 2"

1-WAY 7

DESIGN FOR $M_u(-)$

$w_u = 4.7$ klf (1-WAY P.5)



$M_u^- = \dfrac{w_u \, l_n^2}{11} = \dfrac{4.7 \cdot (18.5)^2}{11}$

$M_u^- = 146.25$ k.ft

ESTIMATE $A_s = \dfrac{M_u}{4d} = \dfrac{146}{4(12)} = 3.0 \text{ in}^2$ *

$A_{s\,min} = 200 \cdot {18 \cdot 12}/{70000} = 0.617 \text{ in}^2$

FOR 2-WAY CALCS

SELECT (4) no. 8 bars ; $A_s = 3.16 \text{ in}^2 \geq 3.0 \text{ in}^2$ ∴ ok

FIND ϕM_n :

$a = \dfrac{3.16 \cdot 70}{0.85 \cdot 4 \cdot 18} = 3.6$ in

$(d - a/2) = 12 - 3.6/2 = 10.2$ in

$c = \dfrac{3.6}{0.85} = 4.25$ in

$\varepsilon_s = (d-c)\dfrac{\varepsilon_{cu}}{c} = (12-4.25)\dfrac{0.003}{4.25}$

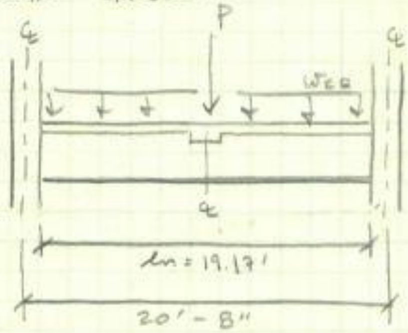
$\varepsilon_s = 0.0055 \geq 0.005$ ∴ $\phi = 0.9$

$\phi M_n = A_s f_y (d - a/2) = 3.16 \cdot 70 \cdot 10.2 / 12$

$\phi M_n = 188$ k.ft ≥ 146.25 k.ft ∴ ok

1-WAY 8

DESIGN GIRDER



$P = 2 \times 43.5\,k$ (1WAY P.5)

$P = 87\,k$

$w_{EQ}' = P/\ell n = 87/19.17$

$w_{EQ}' = 4.54$ klf

$h_{min} = \ell n / 21 = 19.17 \times 12 / 21 = 11$ in

$11 \times 1.08 \times 1.1 = 13$ in

ASSUME $18 \times 16''$ bm ; $d = 14''$

$w_{SW} = 110 \times 18 \times 16 \times 1/144 = 250$ plf $= 0.25$ klf

$w_{EQ} = 1.2(0.25) + 4.54 = 4.9$ klf

$M_u^- = \dfrac{w_u \ell_n^2}{11} = \dfrac{4.9(19.17^2)}{11} = 163.7$ k-ft

FIND ϕM_n:

$As_{req'd} = \dfrac{M_u}{4d} = \dfrac{163.7}{4(14)} = 2.92\,in^2$ ✩

$As_{min} = 200 \dfrac{bd}{f_y} = 200 \cdot \dfrac{18 \cdot 14}{70000} = 0.72\,in^2$

(2-WAY P 8)

SELECT (3) no. 9 $\rightarrow$ $A_s = 3.0\,in^2$

1-WAY 9

$$a = \frac{A_s f_y}{0.85 f'_c\, b} = \frac{3 \cdot 70}{0.85 \cdot 4 \cdot 18} = 3.43 \text{ in}$$

$$c = \frac{a}{\beta_1} = \frac{3.43}{0.85} = 4.03 \text{ in}$$

$$\varepsilon_s = (d-c)\frac{\varepsilon_u}{c} = (14-4.03)\frac{0.003}{4.03} = 0.0074 > 0.005$$

$$\phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2) = 0.9(3)(70)(14 - 3.43/2)$$

$$\phi M_n = 193 \text{ ft}\cdot\text{k}$$

$\phi M_n = 193 \text{ ft k} \geq 163.7 \text{ k ft} = M_u$ ∴ ok Flexure

CHECK SHEAR:

$$V_u = \frac{w\, l_n}{2} = \frac{4.9(19.17)}{2} = 47k$$

FIND V_c

$$V_c = 2\lambda\sqrt{f'_c}\, bd = 2(0.85)\sqrt{4000}\cdot 18 \cdot 14 / 1000$$

$$V_c = 27k$$

$$V_s = V_u/\phi - V_c = 47/0.9 - 27 = 25k$$

$V_s \leq V_{s\,max}$? $V_{s\,max} = 8\sqrt{f'_c}\, bd = 27/0.85 \times 4 = 127k$ ∴ ok

1-WAY 10

CHECK AGAINST $4\sqrt{f'c}\,bd = 127/2 = \underline{63.5\text{ k}}$

$V_s = 25\text{ k} \leq 63.5\text{k} \therefore$

$$S_{max} = \min \left| \begin{array}{lll} d/2 & = & 7'' \; * \\ 0.75h & = & 12'' \\ 24 & = & 24'' \end{array} \right.$$

$S_{max} = 7''$

$$A_{v\,min} = \frac{b_w s}{f_{yt}} \cdot \max \left| \begin{array}{l} 0.75\sqrt{f'c} = \cancel{47.4} \\ 50 \end{array} \right.$$

$$= \frac{50 \cdot 18 \cdot 7}{70000} = 0.09\text{ in}^2/\text{stirrup}$$

use no. 3; $A_v = 0.22\text{in}^2/\text{stirrup}$

FIND SPACING:

$$s = \frac{dA_v f_{yr}}{V_s} = \frac{14 \cdot 0.22 \cdot 70}{25} = 8.624\text{ in}$$

use spacing = 6"

FIND END OF SHEAR REINF.

$V_c = V_u - w_u x$

$27 = 47 - 4.9x$

$x = 4.1'$

USE (8) no. 3 x ⊔ @ 6" O.C. FROM 6"

Appendix F: Weight and Cost Takeoffs

Weight Takeoff

Composite BM					
Material	Unit	Wt	QTY	n	Total Mat
W14x26	PLF	26.00	20.00	8.00	4160.00
W16x40	PLF	40.00	20.00	4.00	3200.00
Shr Studs 3/4"x 4 7/8"	Ea.	10.00	18.00	1.00	180.00
3" x 18 ga. Comp. Deck	PSF	2.55	400.00	1.00	1020.00
4000 psi LtWt Conc*	PCF	110.00	70.37	1.00	7740.74
				Totals	16300.74
				PSF	40.75

Purlin-Girder					
Material	Unit	Wt	QTY	n	Total Mat
C8x11.5	PLF	11.5	10.33	12.00	1425.54
W18x55	PLF	55	20.00	2.00	2200.00
W24x84	LF	84	20.00	1.00	1680.00
1" x 24 ga. Comp. Deck	PSF	1.31	400.00	1.00	524.00
4000 psi LtWt Conc*	PCF	110	44.44	1.00	4888.89
				Totals	10718.43
				PSF	26.80

Two-way w/Drops					
Material	Unit	Wt	QTY	n	Total Mat
7" Slab	PCF	110	196.58	1.00	21624.17
9" Drop	PCF	110	36.75	1.00	4042.50
		Sum	233.3	Totals	25666.67
		CY	8.6	PSF	64.17

One-way w/Beams					
Material	Unit	Wt	QTY	n	Total Mat
5.5" Slab	PCF	110	97.83	1.00	10761.67
18"x14" Beam	PCF	110	32.38	2.00	7122.50
18"x16" Beam	PCF	110	40	1.00	4400.00
		Sum	203.6	Totals	22284.17
		CY	7.5	PSF	55.71

Cost Takeoff

Composite BM						
Material	Unit	Mat	Inst.	QTY	Total Mat	Total Inst.
W14x26	LF	37.50	2.73	40.00	1500.00	109.20
W16x40	LF	55.00	3.38	40.00	2200.00	135.20
Shr Studs 3/4"x 4 7/8"	Ea.	1.10	0.87	54.00	59.40	46.98
3" x 18 ga. Comp. Deck	SF	2.55	0.56	400.00	1020.00	224.00
WWF 6x6 - W2.0*	CSF	21.50	25.50	4.00	86.00	102.00
4000 psi LtWt Conc*	CY	149.35		5.86	875.82	0.00
Place & Vib > 6"	CY		15.10	5.86	0.00	88.55
Spray on Fireproofing	SFin	0.79	0.71	400.00	316.00	284.00
1.85 for fy				Totals	6057.22	989.93
1.4 for LtWt				$/SF	15.14	2.47

Two-way w/Drops						
Material	Unit	Mat	Inst.	QTY	Total Mat	Total Inst.
Forms <15', drops	SF	1.16	3.79	400.00	464.00	1516.00
Reinforcing #4 to #7*	lb	0.53	0.27	745.20	394.96	201.20
4000 psi LtWt Conc*	CY	149.35		8.64	1290.68	0.00
Place & Vib > 6"	CY		15.10	8.64	0.00	130.49
18" Dia. Column	LF	5.00	9.55	23.00	115.00	219.65
Reinforcing #10*	Ton	2821.25	715.00	0.40	1128.50	286.00
1.85 for fy				Totals	3393.14	2353.35
1.4 for LtWt				$/SF	8.48	5.88

Purlin-Girder						
Material	Unit	Mat	Inst.	QTY	Total Mat	Total Inst.
C8x11.5	LF	12	33.50	124.02	1488.24	4154.67
W18x55	LF	76.75	4.28	40.00	3070.00	171.20
W24x84	LF	114	3.14	20.00	2280.00	62.80
1" x 24 ga. Comp. Deck	SF	1.31	0.41	400.00	524.00	164.00
WWF 6x6 - W1.4	CSF	13.8	22.50	4.00	55.20	90.00
4000 psi LtWt Conc*	CY	149.35		3.70	553.15	0.00
Place & Vib <6"	CY		17.25	3.70	0.00	63.89
Spray on Fireproofing	SFin	0.79	0.71	400.00	316.00	284.00
Connections	n/a	2.3125	0.61	24.00	55.50	14.66
1.85 for fy				Totals	8342.09	5005.22
1.4 for LtWt				$/SF	20.86	12.51

One-way w/Beams						
Material	Unit	Mat	Inst.	QTY	Total Mat	Total Inst.
Forms <15', flat	SF	1.03	3.68	400.00	412.00	1472.00
Reinforcing #4 to #7*	lb	0.53	0.27	1.14	0.60	0.31
4000 psi LtWt Conc*	CY	149.35		7.54	1126.12	0.00
Place & Vib < 6"	CY		17.25	7.54	0.00	130.07
Forms Int Bm 18"w	SFCA	0.72	6.55	120.00	86.40	786.00
18" Dia. Column	LF	5.00	9.55	23.00	115.00	219.65
Reinforcing #10*	Ton	2821.25	715.00	0.40	1128.50	286.00
1.85 for fy				Totals	2868.62	2894.02
1.4 for LtWt				$/SF	7.17	7.24