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Technical Report II

North-East Corner, Source: Oliver, Glidden, Spina

Largo Medical Office Building

Largo, Florida

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Executive Summary

Alternate structural floor systems and their characteristics are explored in Technical Report II. These alternate structural floor systems are evaluated with the existing system, as well as with each other. Areas of evaluations include but not limited to weight, total floor thickness, cost, and constructability. The typical bay utilized for all systems is 33'-0" x 33'-0." Assumptions were made to expedite and simplify the evaluation process, one of which is no shoring for steel structures. Also covered in Technical Report II, are the site conditions and building characteristics.

Four systems were evaluated, and are as follows:

- Steel Beam and Girder (Existing)
- Composite Joist and Girder
- Girder-Slab
- Two-Way Flat Slab

Structural design of the composite joist and girder system resulted in a 28" structural depth and a total floor depth of 52", assuming 24" space for MEP. In addition this is the least expensive structural floor system. The system utilized 1.5" Vulcraft 1.5VLI20 composite deck with a 2.5" cover. Initially, non-composite joist girders were evaluated but failed the live load deflection criteria, due to 1.3" vs. 1.1." There is a possibility to chamber the non-composite joist girders to achieve 1.1" deflection, but the option was not taken up. As a result W-shapes with shear studs were used instead. The light weight of the system allowed for quicker erection time and smaller foundation sizing. Like many light framed structures fire protection is necessary, for all structural members, to achieve the code required 2 hour rating.

The second system studied is the girder-slab system, which has a maximum structural depth of 22" and total floor depth of 46." In total the system costs 36984.00 USD/bay. Due to the use of modular components, such as hollow core planks and Δ-section, structural erection is relatively quick. 20" deep Δ-Sections were used as girders and have a 8570.5 lb/ft capacity, exceeding the 7669.2 lb/ft demand. Weighing at 106.5 Kips/bay, it is the second heaviest system. The system can easily be modified into a moment frame, requiring no shear walls. In addition, the system's high mass dampens floor vibrations more effectively than steel framed systems. However, fire protection is required for the underside of the girders.

Two-Way flat slab is the heaviest structural floor system evaluated, weighing at 163.6 lb/bay. Though the 12" two-way flat slab with shear capitals is nearly three times the weight of the existing system, it is the thinnest structural system and is intrinsically a moment frame. An additional floor level for additional revenue is possible, while maintaining the same overall building height. The down side of a high mass system are increase foundation size, larger inertia induced loads, and longer construction time. Costing 49715.87 USD/bay the two way flat slab is the most expensive system and only system not feasible.

Building Overview

Largo Medical Office Building (LMOB) is an expansion of the Largo Medical Center complex. Designed in 2007 and completed in 2009, LMOB is managed and constructed by The Greenfield Group. Located in Largo, Florida the six story facility was designed to house improved and centralized patient check-in area. The 155,000 ft^2 facility also houses office space for future tenants, as well as screening and diagnostic equipment.

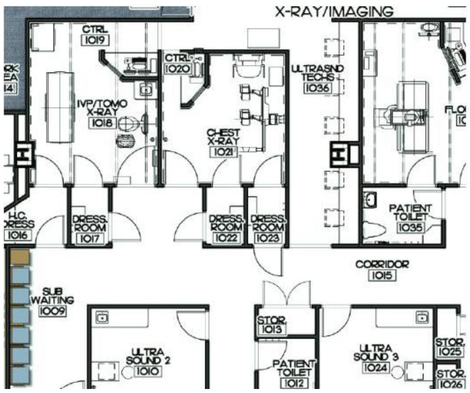

Figure 1.1, Illustrated Floorplans
Source: Oliver, Glidden, Spina & Partners

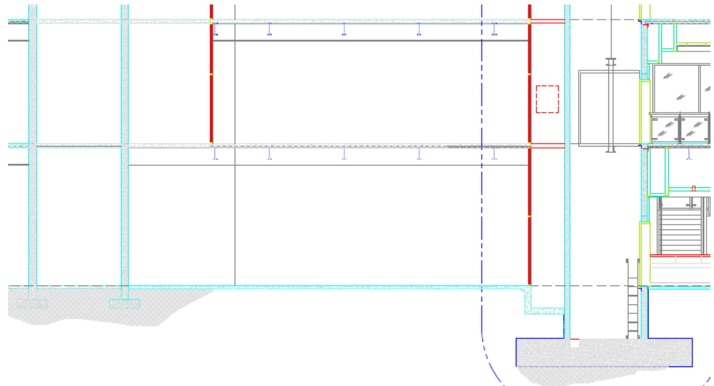

Figure 1.2, Building Section
Source: Oliver, Glidden, Spina & Partners

Patient privacy is a major concern for facilities housing medical related activities. Oliver, Glidden, Spina & Partners answered this by clustering the screening and diagnostic spaces close to the dressing areas (Figure 1.1). The architect went a step further, to preserve privacy by compartmentalizing the building's interior.

LMOB is a 105' tall, steel framed facility with specially reinforced concrete shear walls to resist lateral loads. The shear walls rest on top of strip footings which are at least 27" below grade (Figure 1.2). LMOB's envelope consists of 3-ply bituminous waterproofing with insulating concrete for the roof; impact resistant glazing and reinforced CMU for the façade.

Structural System

Largo Medical Office Building is a 105' tall and 155,000 ft^2 facility which utilizes specially reinforced concrete shear walls and a steel frame.

Concerns about the structural system arose, after looking at the available plans. These concerns include:

1. Effects of drain placement on the rain load
2. Wind loading on the overhang (Figure 2.1)
3. Lack of information due to incomplete drawing set
   - Soil profile
   - Structural member sizes
   - Actual design assumptions and loads

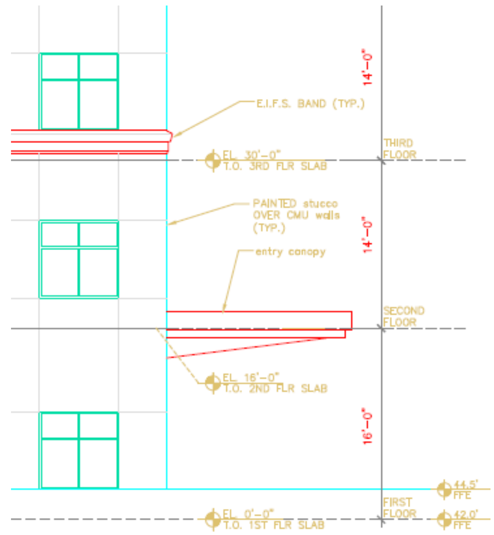

Figure 2.1, Overhang
Source: Oliver, Glidden, Spina & Partners

Due to the lack of information the list of design codes, structural material, and some system details are incomplete. The uncertainty also generated numerous assumptions were made. Assumptions are highlighted in **red** lettering.

Design Codes

Structural engineer consulting firm, McCarthy and Associates, designed the building to comply with the following codes and standards:

1. 2004 Florida Building Code (FBC)
   – Adoption of the 2003 International Building Code (IBC)
2. 13th Edition AISC Steel Manual
3. Design Manual for Floor and Roof Decks by Steel Deck Institute (SDI)
4. ACI 318-05

Codes and standards used for thesis are as follows:

1. 2009 International Building Code (IBC)
2. ASCE 7-05
3. 14th Edition AISC Steel Manual
4. 2008 Vulcraft Decking Manual
5. 2007 Vulcraft Steel Joists and Joist Girders Manual
6. ACI 318-08

Structural Materials Used

Table 2.1, List of Structural Materials	
Steel	
W-Shapes	ASTM A992 Gr. 50
Angles	ASTM A36
Plates	ASTM A36
Reinforcing Bars	ASTM A615
Concrete	
Footings	3000 psi
Slab-on-Grade	3000 psi
Floor Slab	3000 psi

Framing & Lateral System

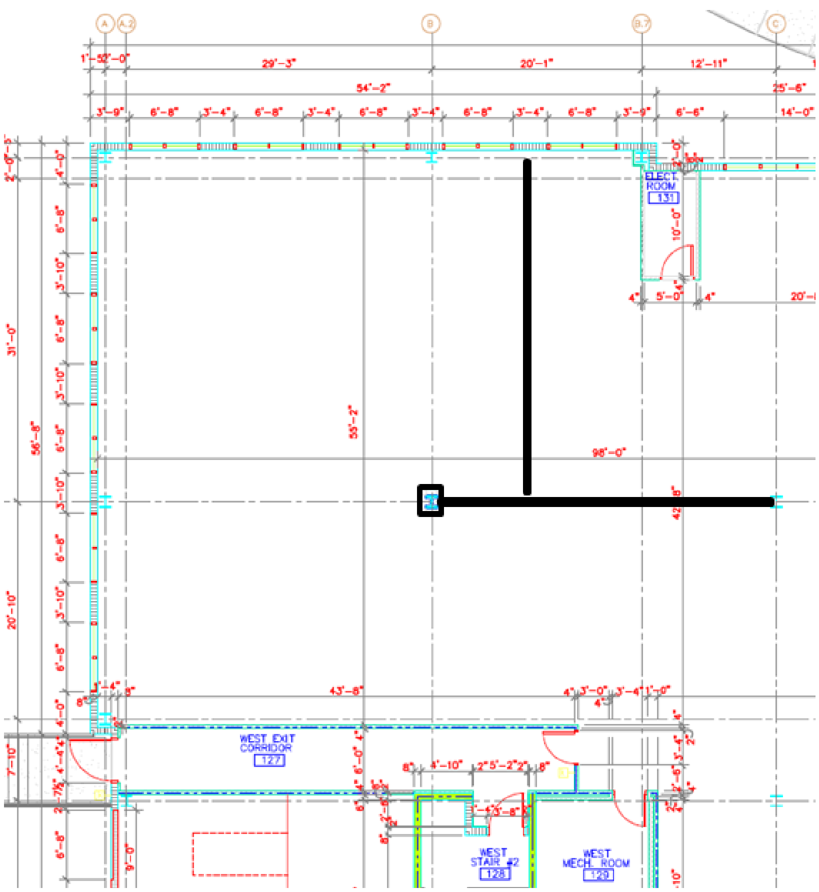

Figure 2.2, Typical Structural Bay
Source: Oliver, Glidden, Spina & Partners

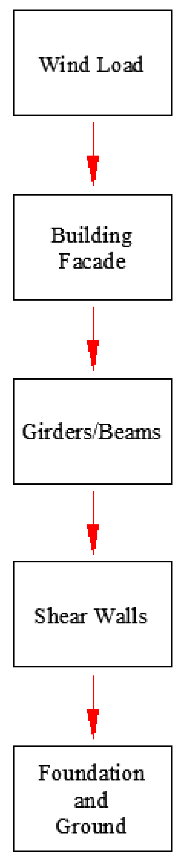

Figure 2.3, Lateral Load Path

The steel frame is organized in the usual rectilinear pattern. There are only slight variations to the bay sizes, but the most typical is 33'-0" x 33'-0" (Figure 2.2). Please refer to Appendix A for typical plans and elevations. Girders primarily span in the East/West (longitudinal) direction.

The only locations where girders are orientated differently include: the overhang above the lobby entrance and the loading dock area. It is assumed that the columns, girders, and beams are fastened together by bearing bolts. As a result, the steel frame only carries gravity loads.

To deal with the lateral load, specially reinforced shear walls are used. The shear walls help the facility resist wind from the North/South and East/West direction. From the drawings it appears that the shear walls are positioned around the emergency stairwells and the two elevator cores. Typical shear walls span from the ground floor level to the primary roof (86' above ground floor level), highlighted black in Figure 2.2. Only the east emergency stairwell has a greater span due to the need for a direct access to roof level from the interior. Lateral load distribution path is demonstrated in Figure 2.3.

In lieu of using shear walls for the lateral system, brace frames and moment frames could be utilized. There are advantages and drawbacks to each lateral system, see Table 2.2 for a comparison of the systems.

Table 2.2, Comparison of Lateral Systems

System	Shear Walls	Brace Frames	Moment Frames
Lateral Resistance Mechanism	Wall Mass and Solidity	Elongation of Brace	Rigid Connection
Member Size	Large	Small	Large
Footprint and Space Flexibility	Mid	Mid	Small
Weight	Heavy	Light	Mid
Vibration Dampening	High	Low	Low
Cost	High - due to labor	Low	High - due to connection quality control and fastening system

From comparing the various lateral systems with the building's primary function, it appears that the original decision to use shear walls is logical. Throughout the lifetime of the facility will house various tenants with different interior preferences, space flexibility is a significant concern. Both the shear walls and moment frames satisfy the space flexibility criteria. Drift is another concern when evaluating for the optimum lateral system. Greater amounts of drift increases the complexity of joining and fastening the building façade; which in turn leaves room for inadequate construction and rainwater leakage. Shear walls and brace frames are fairly stiff systems which results in reduced story drift when compared to moment frames. In addition the fire rating and safe emergency egress is an equally important criteria. Steel structures require significantly greater fire proofing, in concrete the cover is usually increased and is less labor intensive.

Regional preference also plays a role in choosing a lateral system. In the southern U.S. concrete is the predominant building material, due to the lack of vital ingredients for steel production and steel labor base. As a result, lateral systems requiring special connection methods must be ruled out, such as moment frames.

Flooring System

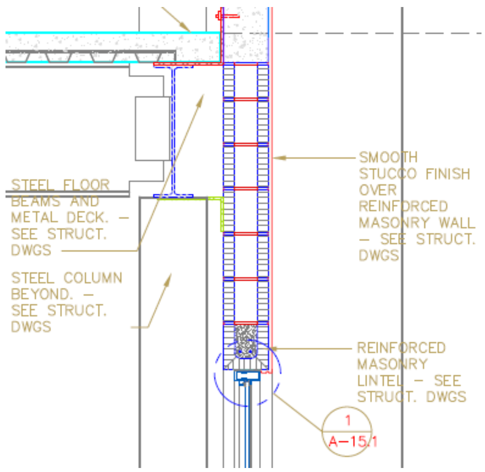

Figure 2.4, Typical Composite Slab
Source: Oliver, Glidden, Spina & Partners

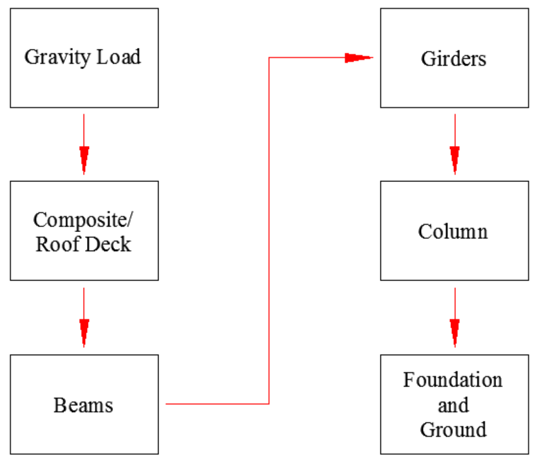

Figure 2.5, Gravity Load Distribution

In general, the structural flooring system is primarily a 5" thick composite slab (Figure 2.4). On all floor levels, except for the ground, the composite slab spans 8'-3". Gravity load distribution path can be followed in Figure 2.5. To satisfy the 2-hour fire rating defined by the FBC, it is likely that the floor assembly received a sprayed cementitous fireproofing. Exposed 2" composite deck with 3" of normal weight (NW) topping only has a 1.5-hour rating, per 2008 Vulcraft Decking Manual.

Hollow core planks and post-tension (pt) slabs are alternatives to the composite slab. PT-slabs do have an advantage in having a thin structural floor, thus allowing greater number of floors when compared to an equally high steel structure. Echoing the frame and lateral system, structural systems for office facilities should allow flexibility in partition and opening placement. Tensioned cables in pt-slabs prevent modification of the slab, like putting an opening into the floor, without first de-stressing the cables and temporary support the floor strip. On the other hand, hollow core planks don't hinder future floor openings. Though pt-slabs aren't easily modified once formed, the system has the advantage in having the thinnest structural floor system. This is advantageous for cities with height limitations since pt-slabs allow greater numbers of floors when compared to an equally high steel structure. In terms of quality control, both pt-slabs and composite slab concrete is typically cast in the field. The results of concrete cast in the field are mix inconsistency and weather induced strength variations. Hollow core planks doesn't have strength inconsistency problems, other than the typical 2" toping.

Roof System

LMOB has three roof levels: main roof, east emergency stairwell roof, and the overhang over the main entrance. There is only one roof type for all three roof levels are the same, consisting of a 3-ply bituminous waterproofing applied over the insulated cast-in-place concrete (Figure 2.6). To ensure adequate rainwater drainage, the insulated cast-in-place concrete is sloped ¼" for every 12" horizontal.

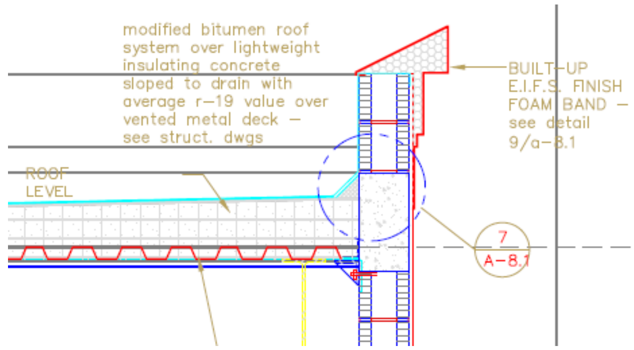

Figure 2.6, Roof Detail
Source: Oliver, Glidden, Spina & Partners

The insulated cast-in-place concrete was used in-lieu of rigid insulation with stone ballast. One reason is that the facility is in a hurricane zone. What it means is, loose material can potentially become airborne projectiles and cause damage when there is a hurricane. The insulated concrete has sufficient mass to resist becoming airborne. In addition, the added mass counters the uplift wind force.

Gravity Loads

Dead, live, rain, and snow loads were calculated for verification of the gravity system. ASCE 7-05 was utilized to factor the loads, using the LRFD method, to determine the size gravity members and check adequacy of actual system. Figure 2.2 shows the typical members, highlighted, which were checked.

Due to the lack of sufficient information, stemming from incomplete drawing set and specifications, a direct comparison of member sizes and design loads was not achieved. Instead actual member sizes were taken by measuring the member depth on the CAD architectural files.

Gravity load and member size calculations can be referenced in Appendix A and Appendix C, respectively.

Dead Loads

Before any dead load calculations were performed, quantity takeoffs and research in material weight were implemented. Take-offs was organized by floor level, which allowed ease of future analysis and design of alternate structural systems. The division by floor level has flexibility built in, where changes in materials can be easily tracked without having to decipher the entire building load equation. Items included in the take offs are: slab concrete volume, floor finish areas, areas of roofing layers/components, volume and area of façade components. See Table 3.1 and Table 3.2 for the material weights and total un-factored dead load by floor level.

Table 3.1, Weight of Building Materials		
Material	Weight	Reference
Normal-Weight (NW) Concrete	150 lb/ft^3	AISC 14th Edition – Table 17-13
Light-Weight (LW) Concrete	113 lb/ft^3	Arch. Graphics Standards 11 Edition
Vinyl Composition Tile (VCT)	1.33 lb/ft^2	Arch. Graphics Standards 11 Edition
Ceramic/Porcelain Tile	10 lb/ft^2	AISC 14th Edition – Table 17-13
3-Ply Roofing	1 lb/ft^2	AISC 14th Edition – Table 17-13
0.8" Laminated Glass	8.2 lb/ft^2	
MEP	15 lb/ft^2	

Table 3.2, Unfactored Dead Load	
Floor Level	Load (kip)
Ground	2425.2
1	3325.7
2	3289.7
3	3289.7
4	3289.7
5	3289.7
Roof	3248.9

Once material quantities and material weight were determined, floor weight was determined. Items not included in the floor weight are the metal decking, joists, and structural steel members. Only after sizing the metal decking, joists, and structural steel members were the items included in the floor weight. A collateral load, of 5 lb/ft^2, was included in the dead load to account for unforeseen items.

Assumptions were made to accelerate and simplify the take-offs and load determination. The assumptions are as follows:

1. Metal deck has equal rib volume
2. All beams are identical to the beam in the typical bay
3. All girders identical to the girder in the typical bay
4. Glazing and concrete are the only façade materials
5. All floors except for the roof use the same type of concrete

Live Loads

LMOB is classified as a type B occupancy, by the 2009 IBC. The outcome of the classification is the use of office live loads. The other live load used to analyze the gravity system is associated with emergency egress. Due to the lack of access to the actual live loads used by the structural consultant, the 2003 IBC live loads were compared to the ASCE 7-05 live loads. Comparison of the live loads is on Table 3.3.

Table 3.3, Live Load Comparison		
Description	2003 IBC	ASCE 7-05
Stairs	100 lb/ft^2	100 lb/ft^2
Lobby & First Floor Corridor	100 lb/ft^2	100 lb/ft^2
Corridors Above First Floor	80 lb/ft^2	80 lb/ft^2
Ordinary Flat Roofs	To Be Calculated	20 lb/ft^2
Partitions	20 lb/ft^2	15 lb/ft^2

The option to use live load reductions was not taken up. Primary reason is that there is a likelihood that the busy hospital will expand its use of facility. Already the hospital occupies 39700 ft^2 of LMOB and has added a parking garage to accommodate additional patients. Another reason, it is likely that the facility will see new equipment, un-foreseen by the designers, in the future.

Table 3.4, Unfactored Live Load	
Floor Level	Load (kip)
Ground	2313.6
1	2001.7
2	2103.9
3	2103.9

4	2103.9
5	2103.9
Roof	528.8

Like the dead load calculations, live loads are broken down by floor level (Table 3.4).

Rain & Snow Loads

Location of LMOB was the deciding factor in whether rain or snow loads controlled. Being that the facility is in Largo, Florida; Figure 7-1 in ASCE 7-05 indicates that the ground snow load is zero. The result is no snow roof loads. Rain load was determined through the use of ASCE 7-05 and the International Plumbing Code (IPC). A ponding instability investigation was not required by ASCE 7-05, because the roof slope is a 1/4" rise for every 12" horizontal. Thus there was no study of ponding potential on the roof.

The hourly rain rate for Largo, Florida wasn't in the standards; the closest city's hourly rain rate was used. Tampa, Florida is the closest city to Largo, Florida. It was determined that the rain load is greater than the live roof load. In many calculations, the rain load (27.89 lb/ft^2) substituted the live roof load (20 lb/ft^2).

Gravity Spot Checks

Deck & Joist

Determining the building weight was the primary reason to size the deck and joist. All decks and joist shall use of cementitious fire protection, to achieve a 2-hour fire rating required by the FBC. There were only two assumptions made concerning decks; as follows: the deck has equal rib sizes, and all decks are 3 spans. Figure 3.1 and 3.2 shows the deck and joist placement.

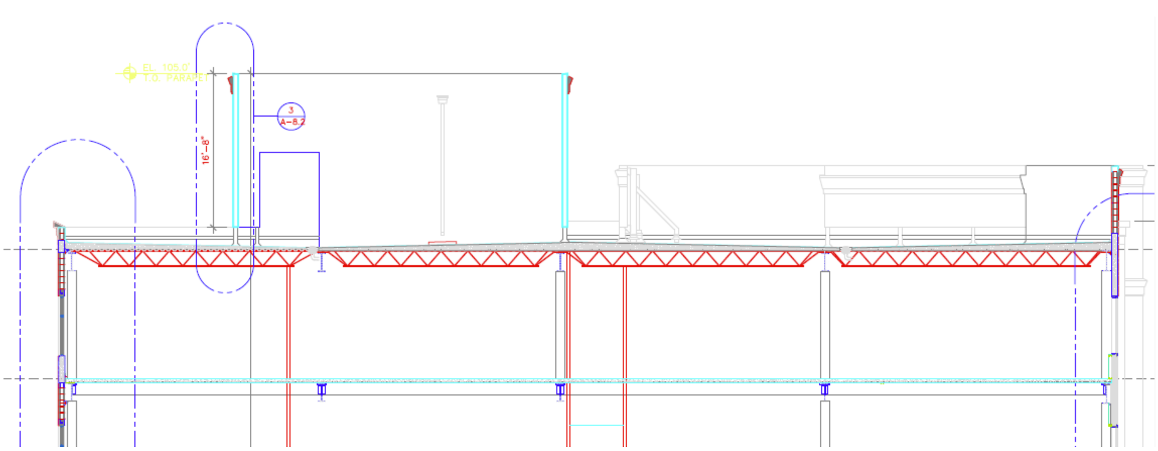

Figure 3.1, Roof Structure
Source: Oliver, Glidden, Spina & Partners

Rain and dead load was used to size the metal roof deck instead of recommended the roof live and dead load. The 27.89 lb/ft^2 rain load is greater than 20 lb/ft^2 live roof load. From the spot check, the original 1.5" thick metal roof deck spanning 5'-6" is sufficient to resist the superimposed rain and dead load.

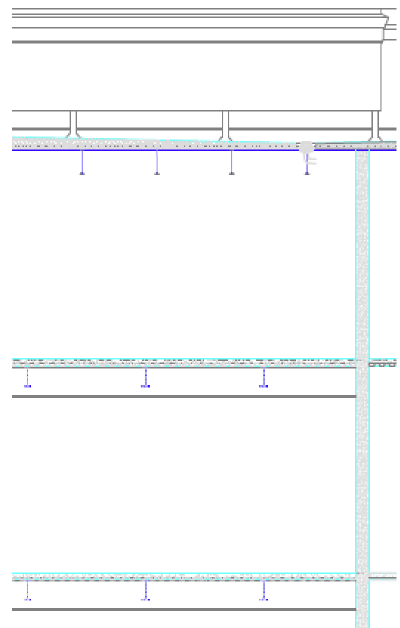
Figure 3.3, Joist and Beam Offsets
Source: Oliver, Glidden, Spina & Partners

The only deviation with the original deck and joist design, appears to be the joist. The spot check showed that a 22K6 joist, also the lightest, is required to support the rain and dead load. Depth of the designed joist is 20" deep, this is a 10 percent difference with the spot check. The difference can be due to a number of factors:

1. Actual rainfall rate could be smaller than the substitute (Tampa, Florida)
2. Use of the prescribed live roof load instead of the rain load
3. Selection of heavier member but with less depth

See Table 3.5 for comparison of the decks and joists used in the original design and spot check.

Table 3.5, Comparison of Original Decks and Joist with Spot Check		
Component	Original	Spot Check
Roof Deck	1.5B	1.5B24
Floor Deck	2VLI	2VLI22
Roof Joist	20" Depth	22K6

Beam & Girder

Beams and girders spanning the largest typical bay, 33'-0"x33'-0", were used for the floor system spot check. In addition to spot checking, the calculated size of the beams and girders were factored into the weight of the building. The members were evaluated for flexural capacity and deflection. It was assumed that the girders use shear studs to have composite action and that shear is completely transferred from the composite slab to the girder. Comparison of the typical beams and girders can be referenced in Table 3.6.

Table 3.6, Comparison of Original Beams and Girders with Spot Check		
Component	Original	Spot Check
Beam	W16	W14x74
Girder	W24	W24x76

There are slight differences between the original beam sizes. The difference is approximately 14 percent, some possible explanations for the difference are:

1. Vibration criteria not evaluated in the spot check
2. Use of economical and predominate sections
3. Greater gravity load due to additional mechanical equipment

Column

Spot check calculations of the typical column, at the intersection of lines B and 2, were implemented once the other structural steel members were sized according to the ASCE 7-05 loads. Column, B-2, was selected because it is an interior column not part of the lateral system. As a result it does not experience lateral loads, as the exterior columns. In terms of bracing, beams and girders prevent the column from having an un-braced length greater than 16'.

Due to the existence of the specially reinforced shear walls, it was assumed that the typical column is pin base. Also, it was assumed that the column did not change size to suit the changing gravity loads. Instead all columns are the same size, to ensure ease of construction and reduce complex column splice connections.

Neither the live load nor live roof load were reduced. All floor levels, other than the roof, were loaded with 80 lb/ft^2 live load. The spot check resulted in W14x120 as the lightest column size to resist gravity loads. McCarthy Associates used a W12 column, the difference is 14%. Reason for a slightly smaller original column can be attributed to:

1. Smaller live load assumption due to either different load criteria or use of live load reduction
2. Use of predominant sections

Structural Floor Systems

Largo Medical Office Building (LMOB) has a typical bay size of 33 ft. x 33 ft. The facility has a regular column arrangement, where the difference in column spacing is no more than 33 percent different. At the facility's north-east and north-west corners the bays are much larger, due to the 3 ft. architectural extrusions.

Four structural systems were analyzed, including the existing/current floor structure. Weight, total floor thickness, cost, and constructability were used in the structural comparison. Items not designed and calculated in this technical report are as follows: columns, foundations, lateral resisting systems, torsion in structural members, structural member connections, and reinforcement development length. Hand calculations can be referenced in Appendix D, Appendix E, Appendix F, and Appendix G.

Parameters which all four structural systems share includes:

1. Typical Bay – 33 ft. x 33 ft.
2. Dead and live loads
3. Maximum structural beam, girder, or slab shall not exceed 2 ft. depth
4. Relative ease in future modification of the structural floor system, such as floor openings
5. Two hour fire rating

In addition to the hand calculations, structural computer modeling of two structural systems were implemented. Structural computer modeling served to reinforce the hand calculations. The two structural systems chosen are the composite joist & girder, as well as the two-way flat slab system.

Existing Floor Structure

Steel beam and composite girder is the existing/current floor structural system at LMOB. Steel beams spaced at 8 ft 3 in. supports the 5 in. composite slab. No structural floor member in LMOB exceeded a depth of 2 ft.

As a result of incomplete structural drawings, assumptions about the structure and materials were made. These assumptions are as follows:

1. Slabs are compositely attached to the girder
2. No shoring during construction
3. Metal decking, for floors, have equal sized corrugations
3. Concrete strength is 3000 psi
4. Wide Flanges use A992 Gr. 50 steel
5. All member connections are bearing and hold no moment

From the assumptions and available drawings, the structural floor system was determined. See Appendix C for calculation details of the current system.

It was determined that the composite metal decking used is equivalent to 2VLI22. The 3 in. cover is insufficient, per Vulcraft 2008 Decking Manual, as a result spray cementitious or fiber fire protection on the underside of the deck is necessary to achieve the required 2 hour rating.

Beam and composite girder sizes are W14x74 and W24x76 respectively. Moment was the controlling factor for the composite girder and the primary reason for using 3 rows of shear studs. Each 33 ft. composite girder requires 94 shear studs (3/4 in. diameter). The total depth of the current floor system is 53 inches, including the assumption that MEP requires a 24 in. depth allowance. Typical beam and composite girder system is illustrated in Figure 4.1 and Figure 4.2.

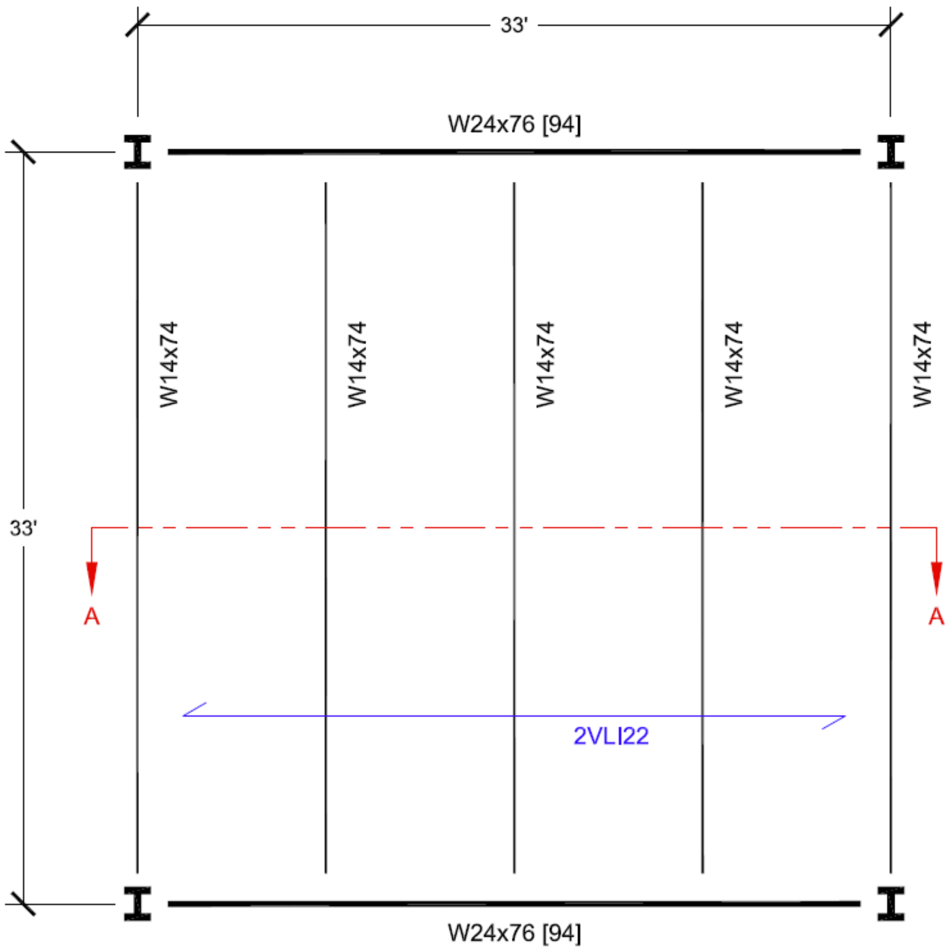

Figure 4.1, Structural Members of Typical Bay – Steel Beam and Girder

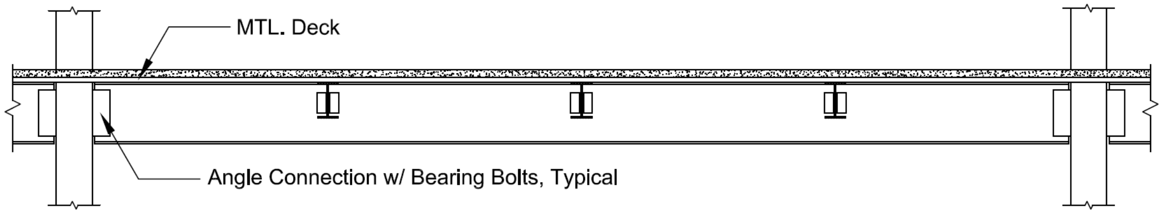

Figure 4.2, Section A-A

The structural floor of the typical bay weights 68.5 kips, which translates to 62.9 lb/ft^2. Most of the weight is due to the 50 lb/ft^2 composite slab. Weight of the lateral load resisting system wasn't factored into the weight of the typical bay.

Advantages

1. Relatively light weight construction, compared to concrete structural systems
2. Low soil bearing pressure
3. Reduced inertia load when exposed seismic activity
4. Creep resistance
5. No shoring or formwork necessary
6. Erection speed
7. Weather and climate doesn't significantly impact strength

Disadvantages

1. Deep floor system
2. Reduction of rentable space and stories, compared similar height concrete buildings
3. Resistance to overturning moments due to building weight is reduced
4. Fire protection for all structural floor members including beams and girders
5. Region doesn't specialize or have sufficient labor pool for steel construction

Composite Joist & Girder

Composite joist and girder structural floor system was chosen due to structural efficiency. Structural efficiency reduces the quantity and size of members. This allows for shorter erection time, reduced building weight and foundation demand. Composite joist design is based on the prescribed method in the Vulcraft 2009 Composite and Noncomposite Floor Joist Manual. Hand calculations can be referenced in Appendix E.

Assumptions used in the design of the composite joist and girder system are as follows:

1. No shoring during construction
2. Metal decking, for floors, have equal sized corrugations
3. Concrete strength is 3000 psi

4. Wide Flanges use A992 Gr. 50 steel
5. All member connections are bearing and hold no moment
6. All shear studs (3/4 in. diameter) are installed in the field

Three composite joist spacing were evaluated to determine the lightest arrangement; which includes 5 ft. 6 in., 6 ft. 7in., and 8 ft. 3 in. spans. There are two ways to evaluate the lightest joist arrangement. One is the actual weight, which doesn't factor in the degree of work necessary to install the shear studs. Effective weight method includes the degree of work necessary to install the shear studs. Installation of each shear stud is equivalent to installing 10 lbs. of steel.

In the end, effective weight and fire protection was the deciding factor on the joist spacing. Actual weight wasn't used due to the small variation, 0.78 percent, between the three spans. It was determined that the 8 ft. 3 in. span had the smallest effective weight and requires less volume and work on fire protection.

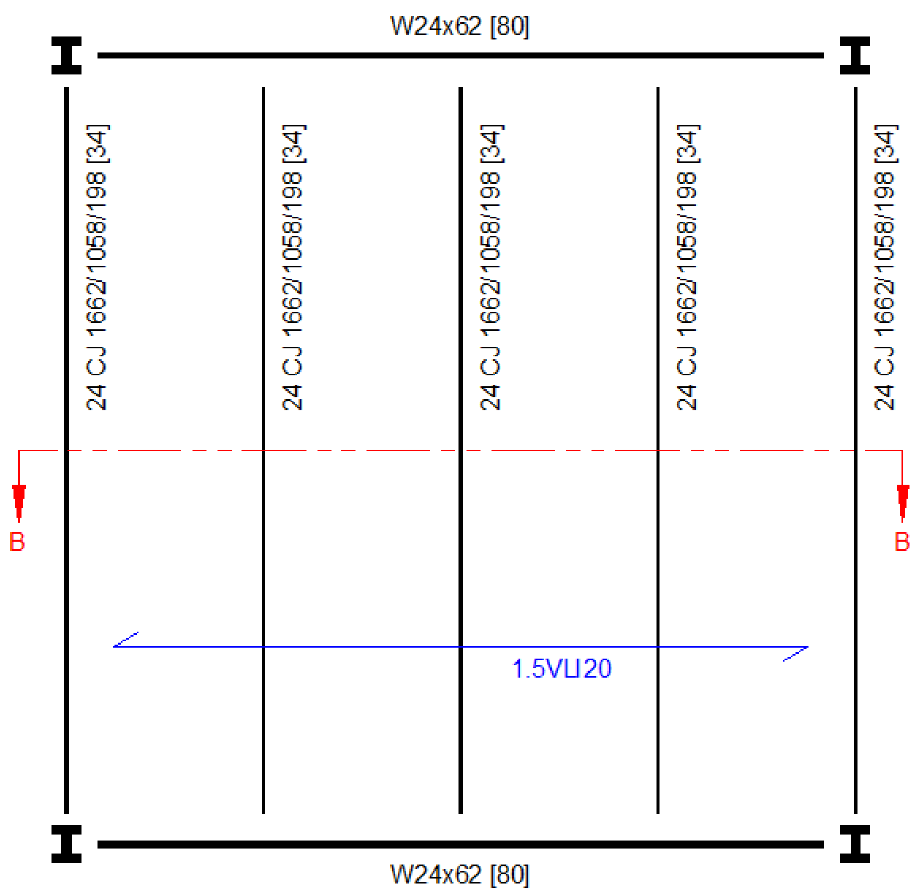

Figure 4.3, Structural Members of Typical Bay – Composite Joist & Girder

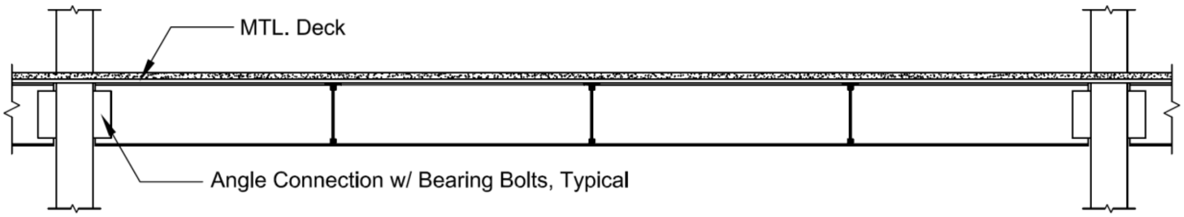

Figure 4.4, Section B-B

Instead of re-using the composite slab in the existing system, a lighter composite slab was selected. The 39 lb/in^2 slab, with 1.5VLI20 and a 2.5 in. concrete topping, was selected. Like the 2 in. metal deck, fire protection is necessary. Vulcraft 2008 Steel Deck Manual recommends that either sprayed cementitious or fiber fire protection can used.

All composite joists and girders require a minimum of 2 rows and 3 rows of shear studs, respectively. Only then will shear be transferred from the slab to the joists and girders. Figure 4.3 and Figure 4.4 are illustrations of the composite joist and girder system. Initially non-composite joist-girders were considered in lieu of the composite girders. As it turned out, the non-composite joist-girders didn't satisfy the live load deflection criteria. It is possible to chamber the joist-girders, to meet the deflection criteria, but this option wasn't taken since the joist-girders are 48 in. deep. Please refer to Appendix E for details of the joist-girder deflection calculation.

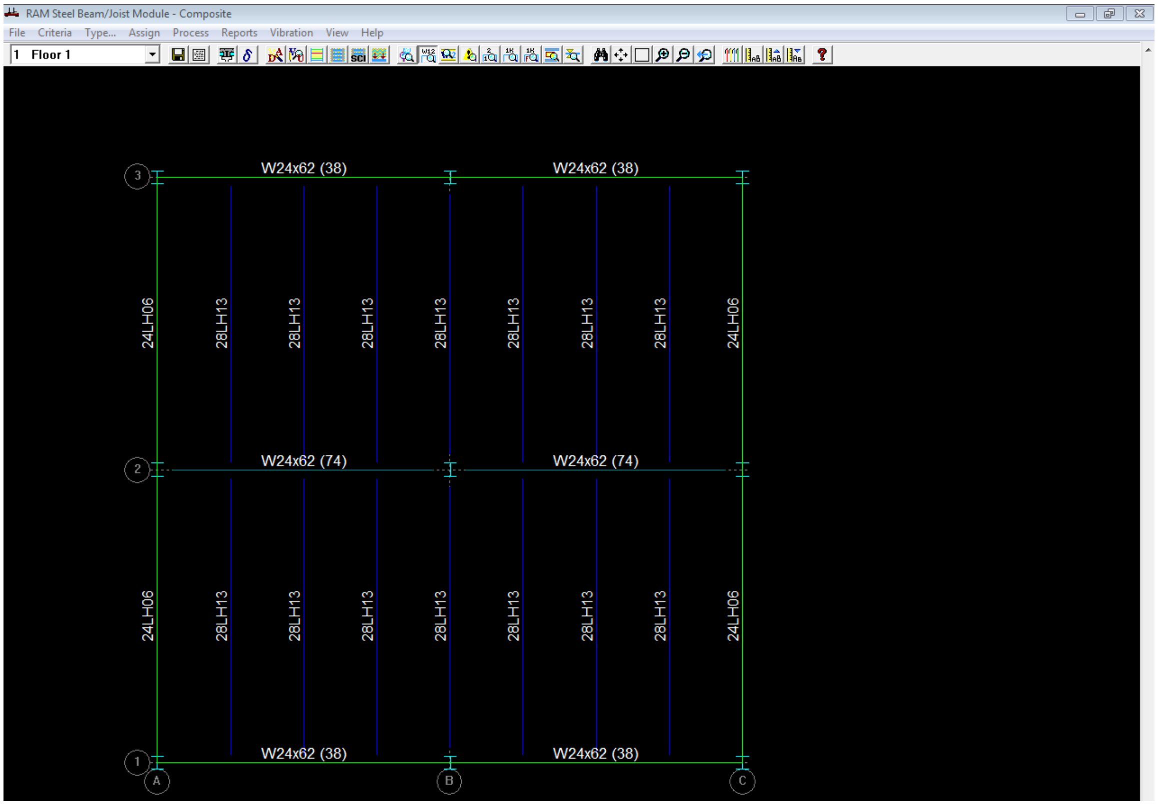

Figure 4.5, *RAM* Model

RAM computer structural modeling software was used to verify the hand calculation. Composite joist were not available in RAM, as a result non-composite joists were used in-lieu. The impact is a deeper and heavier joist. Also it was assumed that 80 percent is the minimum acceptable percentage of full composite.

It was not surprising to determine that the number of shear studs is 74, provided that the system modeled in RAM has greater self-weight. Plus the neutral axis more deeply imbedded in the steel girder. The reduction in the number of shear studs can be also attributed to the assumption that 80 percent is the minimum acceptable percentage of full composite. See Figure 4.5 for the structural design in RAM.

The total depth and effective weight of the composite joist and girder system are respectively 52 in. and 53.4 kips per bay.

Advantages

1. Relatively light weight construction, compared to concrete structural systems
2. Low soil bearing pressure
3. Reduced inertia load when exposed seismic activity
4. Creep resistance
5. No shoring or formwork necessary
6. Erection speed
7. Pre-fabrication of structural floor system into modules w/ joist and deck joined
8. Weather and climate doesn’t significantly impact strength
9. Use of openings between joist’s bars for some MEP systems

Disadvantages

1. Deep floor system
2. Reduction of rentable space and stories, compared similar height concrete buildings
3. Resistance to overturning moments due to building weight is reduced
4. Fire protection for all structural floor members including joists and girders
5. Longer lead time for materials
6. Region doesn’t specialize or have sufficient labor pool for steel construction

Girder-Slab

The third system chosen for analysis is the girder-slab system. Girder-Slab was chosen for minimum slab depth, quick erection and extensive use of concrete. Girder-Slab system utilizes either D-sections or Δ-sections as girders, keeping hollow core planks supported and in place. All sections are chambered to achieve an acceptable code defined deflection. The sections are also used as a form for the cast-in-place concrete, since concrete is placed into the sections to create a reinforced concrete girder.

Design of the girder-slab system utilized design tables from StresCore, Girder-Slab Technologies LLC, and PEIKKO Group. Due to the lack of design tables in U.S. customary units, for 20 inch (500 mm) Δ-sections, metric tables were used instead. See hand calculations in Appendix E for more details. Design tables used can be referenced in Figure 4.6 and Figure 4.7.

Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in⁴	in	in	in³	in³	kft	in⁴	in	in	in³	in³
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Figure 4.6, D-Girder Characteristics
Source: Girder-Slab Technologies LLC

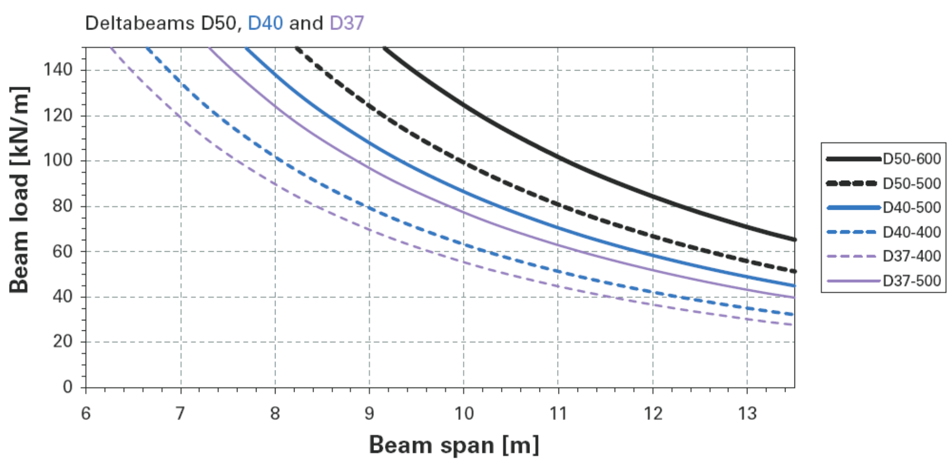

Figure 4.7, Δ-Girder Load Capacity
Source: PEIKKO Group

Assumptions concerning the section properties and component functions include:

1. All plates in Δ-section are 1 in. thick
2. Rebar traversing through the section and hollow core plank keep the planks in place
3. Rebar traversing through the section and hollow core plank transfer no significant moment
4. Use 4000 psi cast-in-place concrete

All girders span in the North-South direction and require no shoring when cast-in-place concrete has not cured. The required linear load on the sections is 7669.2 lb/ft. From the design tables, 10 in. hollow core and 20 in. deep Δ-section D50-600 were selected. The maximum depth and weight of the typical bay is 46 in. and 106.5 kips, respectively. For more details see Appendix E, Figure 4.8, and Figure 4.9.

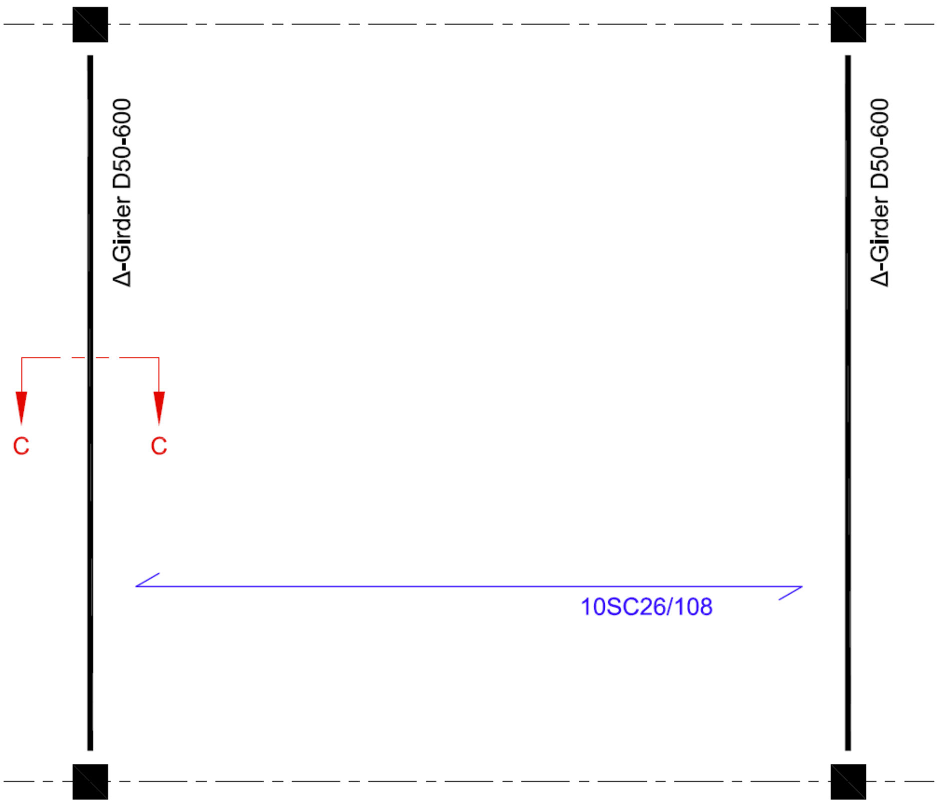

Figure 4.8, Structural Members of Typical Bay – Girder Slab

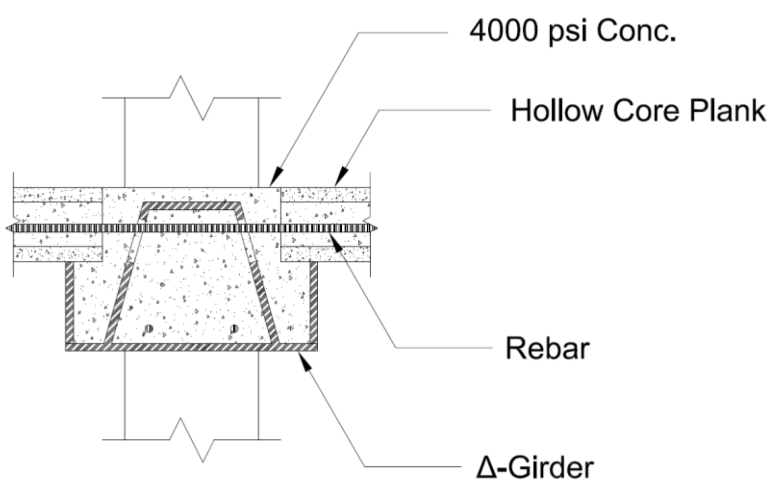

Figure 4.9, Section C-C

Advantages

1. Resistance to overturning moments due to building weight is greater than steel facility
2. No shoring or temporary formwork necessary
3. Significant pre-fabrication reduces cost and construction speed
4. Small volume of cast-in-place structural concrete
5. Shallow floor depth
6. Column material can either be concrete or steel
7. Dampen vibrations, due to floor mass

Disadvantages

1. Fire protection on exposed steel of girder section
2. Coordination between designers and fabricators
3. High weight when compared to steel facility
4. High soil bearing pressures

Two-Way Flat Slab

Two-Way flat slab was selected based upon the regional building material preference, shallow depth, and intrinsic lateral resisting characteristics. High factored loads, 152 lb/ft^2 not including self-weight, as well as large typical bay size facilitated the use of shear capitals at the column locations. Deflection was handled by using slab total depths greater than the threshold where deflection calculation is required, per ACI 318-11 Table 9.5C. In two-way slabs flexural rebar can't intersect at the same depth, as a result d is measured from the compression edge to the closest flexural rebar to the neutral axis. Hand calculations can be referenced in Appendix E.

To simplify the design process, a few assumptions were made:

1. Use 4000 psi concrete and 60 ksi reinforcing
2. Continuity of M^+ (bottom) reinforcing for redundancy against column failure
3. Flexibility of changing column spacing where column spacing deviates < 1/3 and offset < 10 percent

From the hand calculations it was determined that the maximum moment, 713.4 kip-ft, occurred at the interior columns. There was great concern for rebar congestion at the column locations. As a result the maximum number of reinforcement per strip width was determined. In the end, the (28) #8 reinforcement per 8 ft. 3 in. strip satisfied the maximum number rebar criteria [(41) #8 per 8 ft. 3 in.]. All require rebar areas were compared to maximum rebar area for yielding, maximum rebar area for Φ to equal 0.9, and minimum reinforcement to control thermal cracking. See Figure 4.10 for the middle and column strip widths.

Constructability and the possibility of construction errors facilitated the need to simplify the reinforcement design, simplifications include:

1. All mid-span reinforcement is based on the first interior mid-span reinforcement
2. All middle strips reinforcement, regardless of location in span, is based on mid-span reinforcement of the middle strip
3. All M^+ (bottom) reinforcements are continuous
4. All flexural reinforcement shall use the same bar size
5. All first stir-ups are spaced the same distance, off centered

Flexural rebar arrangement in the 12 in. concrete flat slab can be referenced in Table 4.1 and Appendix E.

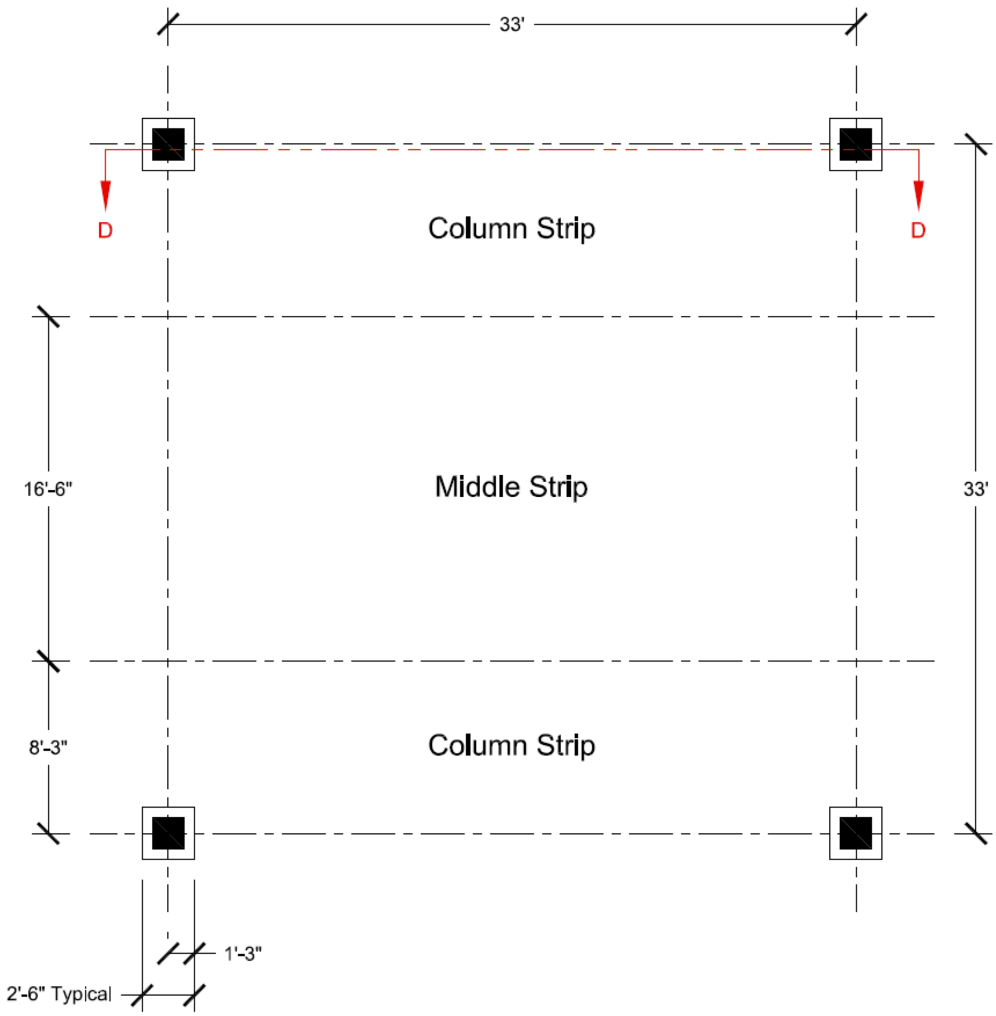

Figure 4.10, Two-Way Flat Slab Divisions

Table 4.1, Flexural Rebar in Column and Middle – Hand Calculations			
Strip	Strip Location		
	Exterior Columns	Mid-Span	Interior Columns
Column	(12) #8; $A_{s,req}$ = 9.32 in^2	(15) #8; $A_{s,req}$ = 11.45 in^2	(28) #8; $A_{s,req}$ = 21.65 in^2
Middle	(9) #8; $A_{s,min}$ = 5.86 in^2	(9) #8; $A_{s,req}$ = 7.03 in^2	(9) #8; $A_{s,req}$ = 5.88 in^2

As mentioned earlier, there was significant punching shear at the columns. To achieve the required shear strength 2 ft. 6 in. x 2 ft. 6 in. x 3 in. thick column capital was used, as well as stir-ups spaced at 4 in. off center. Each stir-up has (8) #4 legs, refer to Figure 4.11.

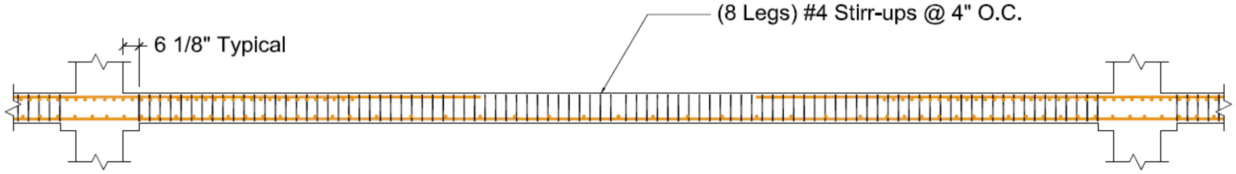

Figure 4.11, Section D-D

In addition to hand calculation, spSlab was used to design the two-way flat slab for flexure. Please see Appendix F for the computer output. As part of the input the parameters were defined and include:

1. Minimum flexural rebar size = #6
2. Minimum rebar spacing = 2.5 in.
3. Number of Bay(s) = 2
4. Shear Capital Thickness = 3 in.
5. Shear Capital Taper = 45°

As evident from the parameters, only the shear capital taper is different from the actual design. The shear capital shouldn't impact the analysis because shear reinforcement directly influence flexural design. In addition spSlab adheres to ACI 318 which defines that shear capitals only takes shear loads. The flexural reinforcement designed by spSlab can be referenced in Table 4.2 and Appendix F.

Table 4.2, Flexural Rebar in Column and Middle - spSlab			
Strip	Strip Location		
	Exterior Column	Mid-Span	Interior Column
Column	(11) #6; $A_{s,req}$ = 4.319 in^2	(29) #6; $A_{s,req}$ = 12.75 in^2	(50) #6; $A_{s,req}$ = 21.63 in^2
Middle	(19) #6; $A_{s,req}$ = 8.03 in^2	(19) #6; $A_{s,req}$ = 8.03 in^2	(19) #6; $A_{s,req}$ = 8.03 in^2

In lieu of the direct design method, used in the hand calculations, spSlab utilizes the equivalent frame method. Each design method utilizes differing moment distribution factors, resulting in slightly different required reinforcement ($A_{s,req}$). The maximum deviation between the two methods is the exterior columns, where the reinforcement in spSlab is less than 50 percent of the hand calculations. In addition, the equivalent method distributes greater moment to the middle slab and at mid-spans, evident in the higher required reinforcement.

Two-way flat slab is the heaviest of the four structural systems. Weighing at 163.6 kips per typical bay this is more than 2 times the existing structural system. Though two-way flat slab is heavy, the maximum total floor depth is 39 in. with the assumption that MEP requires 24 in. depth allowance. Thus making the system the thinnest floor system and allows for an addition of one more level to LMOB. An additional level will add greater revenue due to tenant rent and offset the construction cost.

Unlike the other three systems, the two-way cast-in-place flat slab needs shoring and re-shoring during construction. This will result in an extended construction schedule, when compared to modular steel and composite systems.

<u>Advantages</u>

1. Resistance to overturning moments due to building weight is greater than steel facility
2. Small volume of cast-in-place structural concrete
3. Small shear induced deflections
4. Shallow floor depth
5. Dampen vibrations, due to floor mass
6. No fire protection required other than adequate concrete cover

<u>Disadvantages</u>

1. Weather and climate significantly impact strength
2. Slow construction of building structure, compared to steel structural systems
3. Stringent quality control to ensure proper strength and durability
4. High weight when compared to steel facility
5. Increase inertia load when exposed seismic activity
6. High soil bearing pressures

System Comparison

Table 5.1, Structural Floor System Comparison

Criterion		Steel Beams & Girders (Existing)	Composite Joists & Girders	Girder-Slab	Two-Way Flat Slab
Cost (USD/bay)		33123.96	14332.33	36984.00	49715.87
Max. Floor Depth (in.)		53	52	46	39
Actual Weight (Kip/bay)		68.5	51.3	106.5	163.6
Structural	Lateral Resisting System	Required; either brace frames, shear walls, or moment connections	Required; either brace frames, shear walls, or moment connections	Maybe, depends on connection	Not required, intrinsically a moment frame
	Foundation Modification	No	No, but foundation can be reduced	Yes, increase foundation capacity	Yes, increase foundation capacity
Fire Protection (2-hour rating)		Yes	Yes	Yes, only underside	No
Intrinsic Vibration Dampening		Low	Low	High	High
Constructability	Schedule	Fast	Fast	Moderate	Slow, due to curing conc.
	Quality Control Level	Low	Low	Moderate	High
	Material Lead Time	Moderate	Long, due out-of-state fabrication	Short	Short
	Speed of Workforce Mobilization	Slow, due to lack of sufficient specialized labor	Slow, due to lack of sufficient specialized labor	Fast	Fast
	Regional Preference	No	No	Yes	Yes
Feasibility		Yes	Yes	Yes	No

Conclusion

Technical Report II evaluates four structural floor systems, including the existing/current steel and girder system. Total floor depth, cost, weight, and constructability are the primary factors for determining structural floor system feasibility. Only one floor system was found to be not feasible.

The composite joist and girder system is the lightest weight and least expensive to construct. Depth wise the composite joist and girder system is only 1" shallower than the existing system. But the composite joist's open web allows for electrical and plumbing to be run through, resulting in possible further reduction in total floor depth. Construction is similar to the existing steel beam and girder system. The reduction in cost can be attributed the use of composite joists in-lieu of solid beams. Cost is further reduced by the system's low dead weight, where member size is reduced. Composite joist and girder system is feasible but floor vibration will need to be further studied to determine serviceability. Also the degree of difficulty installing fire-protection will need to be delved more deeply.

Girder-Slab system is also feasible. Though it is heavier and slightly more expensive than the existing structural floor system, there are advantages. One of which is modularity, where the hollow core planks and Δ-sections are prefabricated. As result is shorter construction time. In addition, the girder-slab system produces the second shallowest floor system, 46" in depth. Thus, allowing greater space for future MEP additions. Vibration dampening is handled relatively well, due to the system's high mass with possibility to fill the hollow core plank's voids with attenuating material. Factors which will need to be explored in greater detail include: possibility of shallower Δ-sections, moment capacity at girder and column interface.

Only the two-way flat slab is not feasible, its weight and cost negated any advantage. Two-Way flat slab is the shallowest system, with a maximum total floor depth of 39." Though the system allowed for the possibility of an additional floor and greater revenue from rent, the high weight increases the inertial component of seismic loads. Cost will also increase with the need to seismically design the structure. Also, the 3-1/2" rebar spacing at the columns is a constructability and quality control issue. There is a potential for over congestion when column reinforcement is placed, making the concrete mix harder to fill all the voids. Due to cost, weight, and constructability issues two-way flat slab is not a feasible alternative.

Appendix A: Floor Plans & Elevation

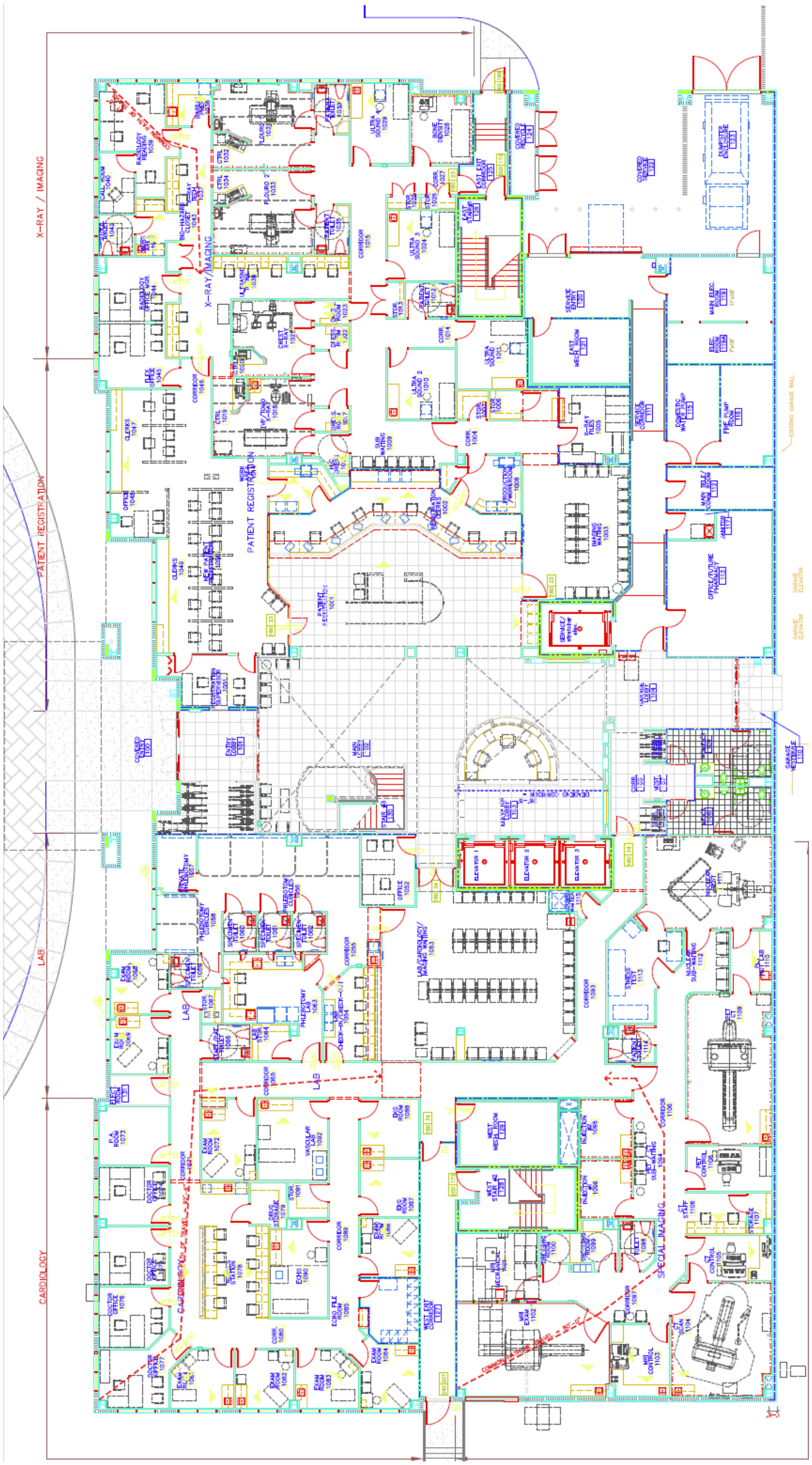

Figure AA.1, First Floor Plan w/ Tenant Build-Out
Source: Oliver, Glidden, Spina & Partners

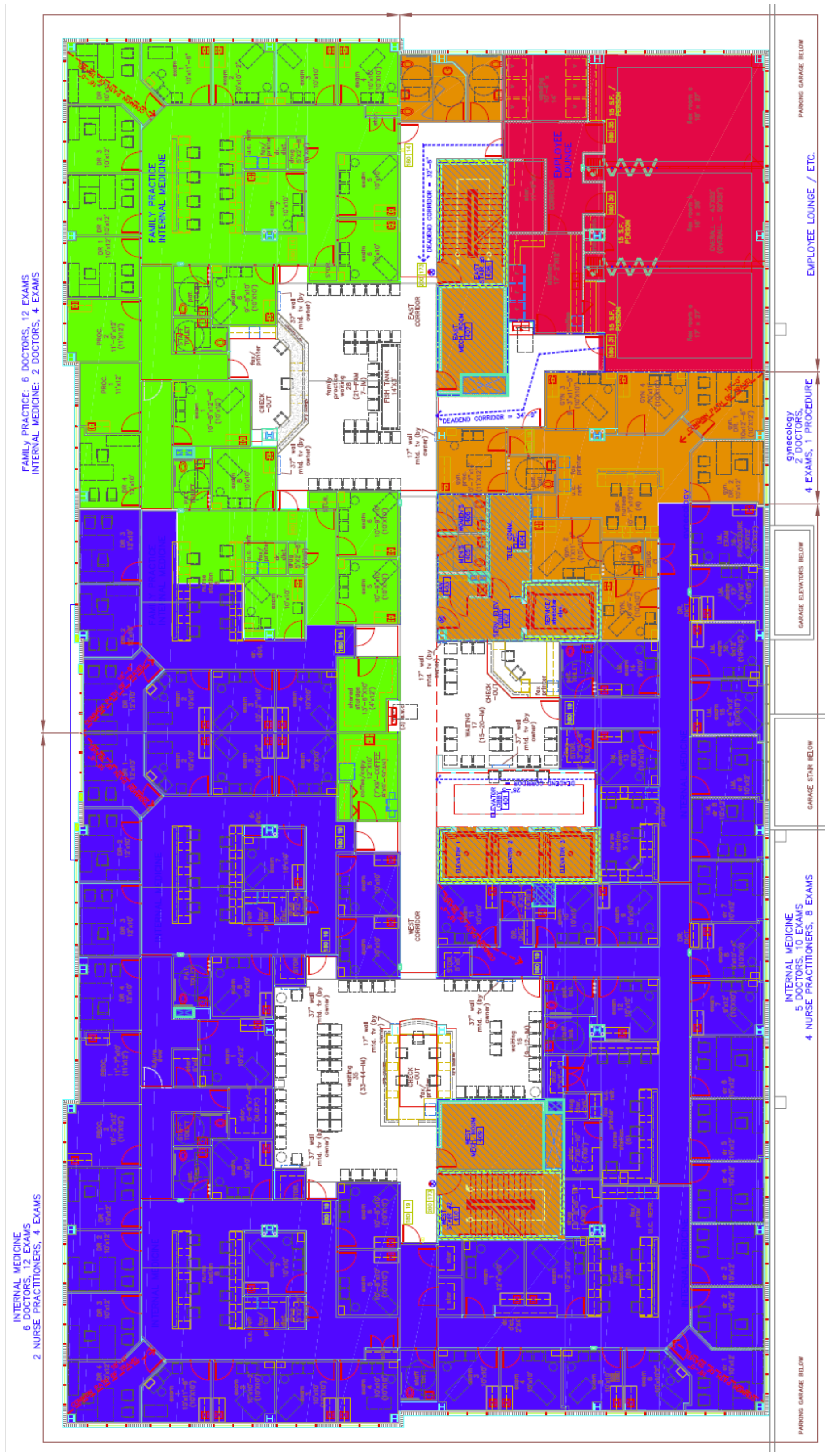

Figure AA.2, Typical Upper Floors
Source: Oliver, Glidden, Spina & Partners

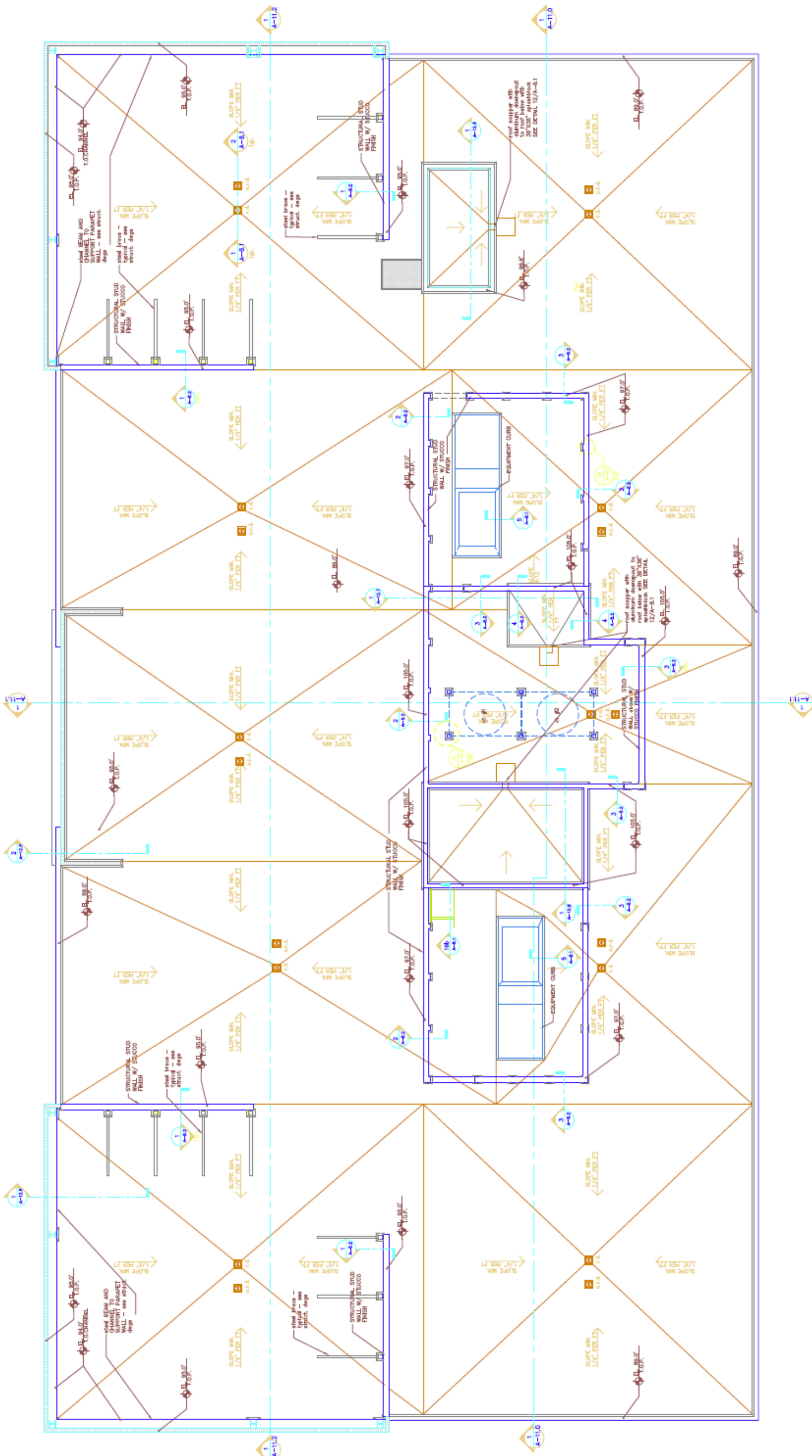

Figure AA.3, Roof Plan
Source: Oliver, Glidden, Spina & Partners

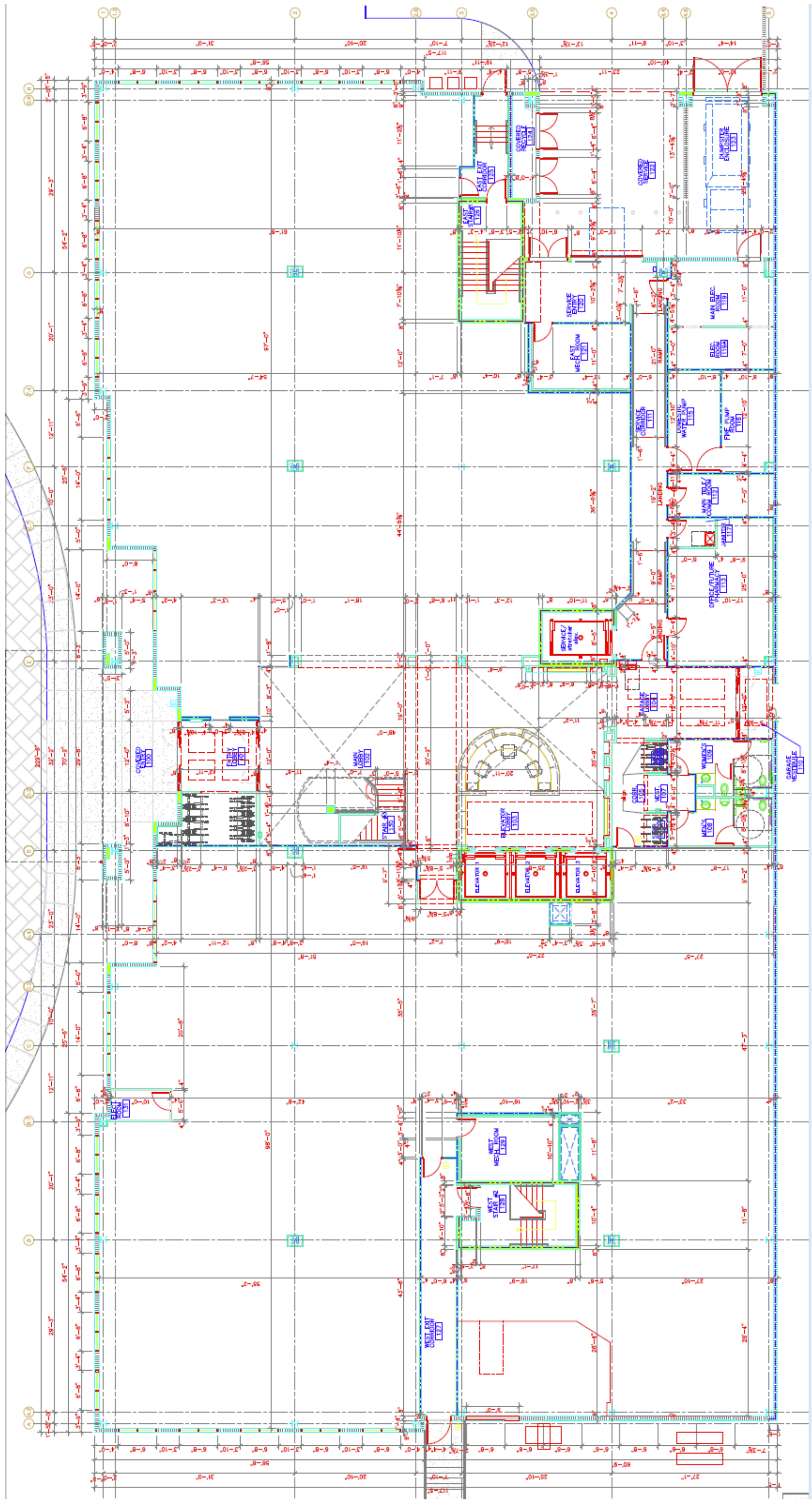

Figure AA.4, Typical Column Layout
Source: Oliver. Glidden. Spina & Partners

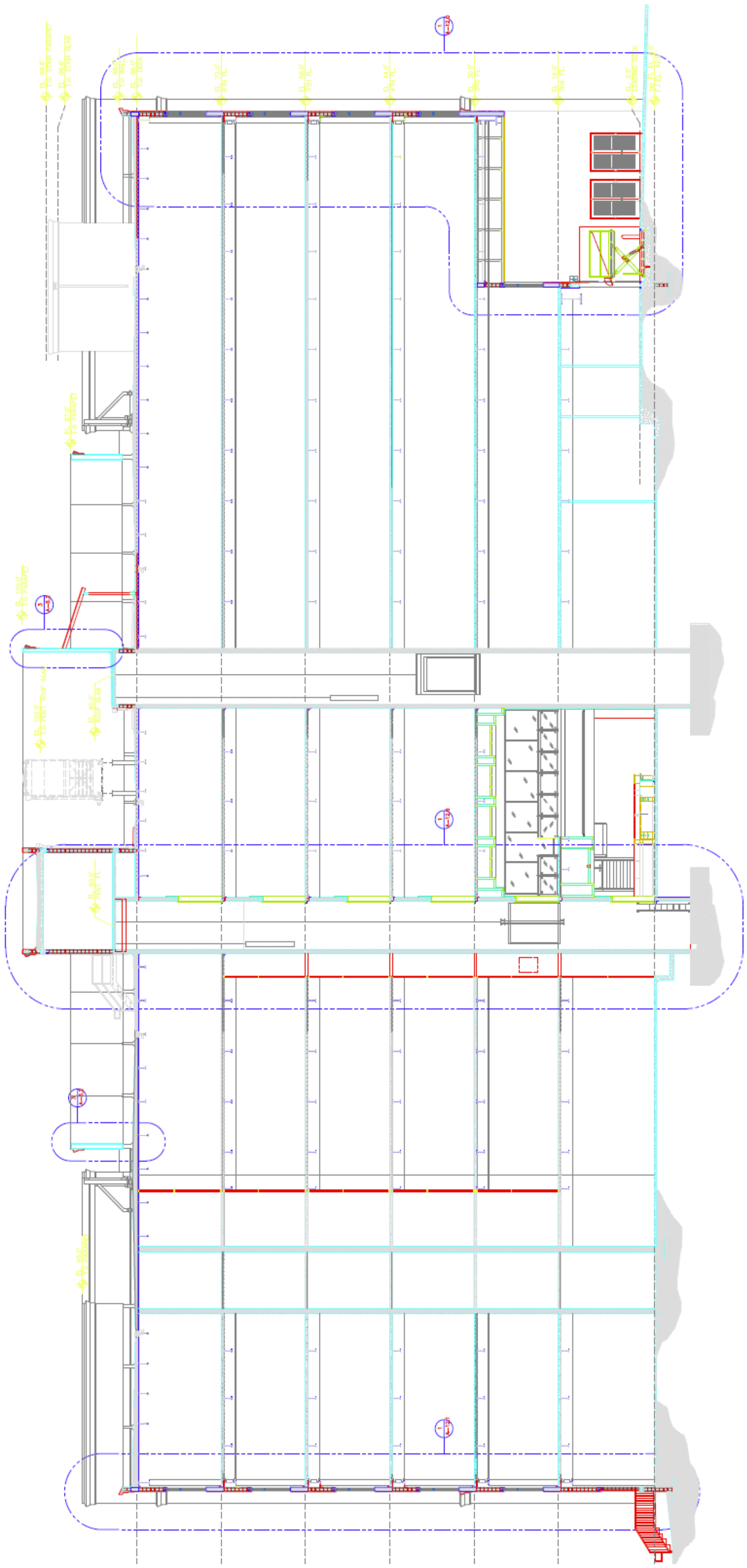

Figure AA.5, Longitudinal Building Section
Source: Oliver, Glidden, Spina & Partners

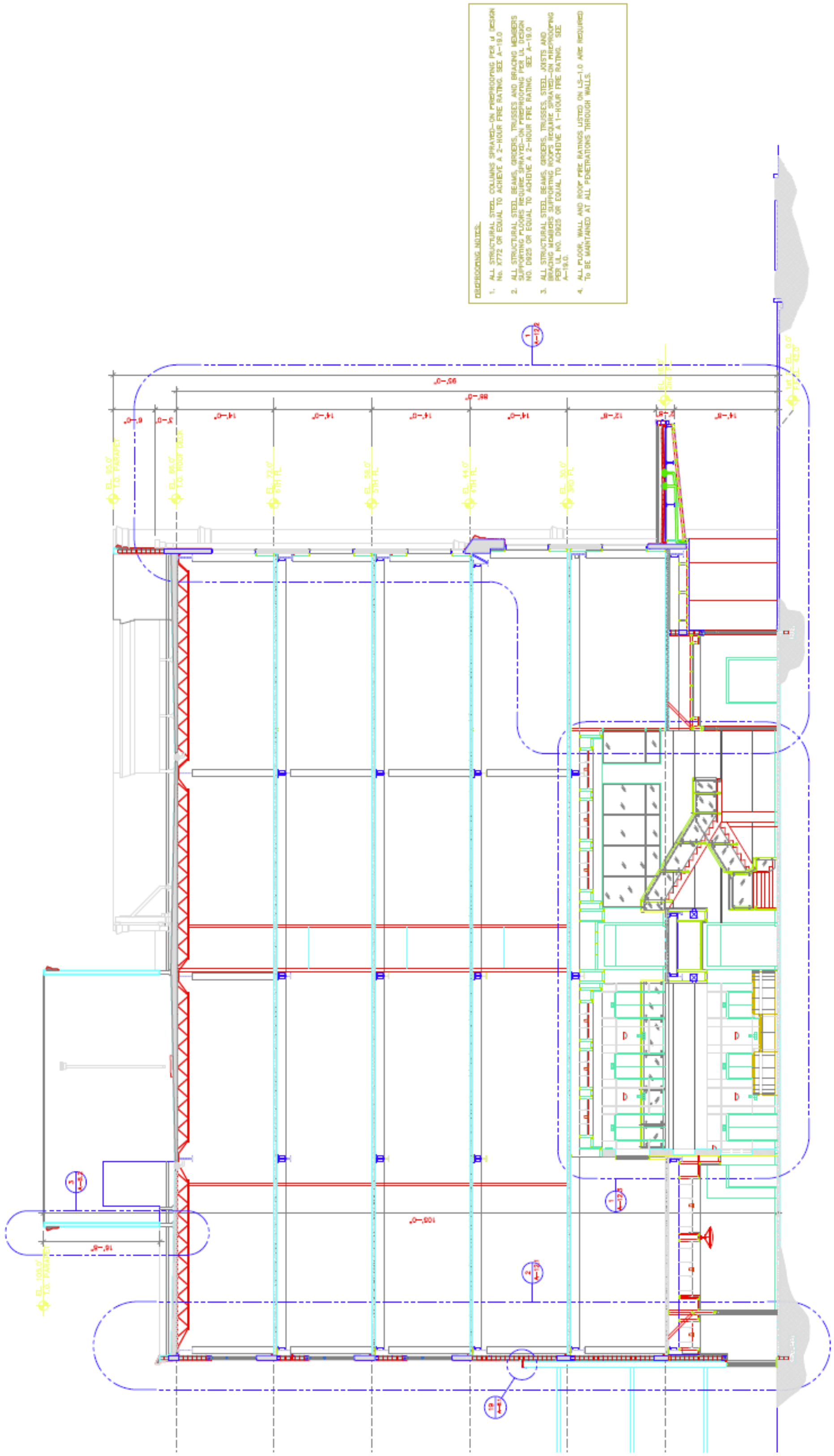

Figure AA.6, Building Section
Source: Oliver, Glidden, Spina & Partners

Appendix B: Load Determination Dead, Live, Rain

Floor Level	A_{gross} (ft^2)	$A_{fl\ opening}$[1] (ft^2)	A_{stairs} (ft^2)
0	24153.00	293.00	724.00
1	26440.00	1571.00	609.00
2	26440.00	293.00	609.00
3	26440.00	293.00	609.00
4	26440.00	293.00	609.00
5	26440.00	293.00	609.00
Roof[2]	26440.00	N/A	204.00

[1] Does not include stairwell openings
[2] Stairs extending to roof top has a roof

Story	A_{facade} (ft^2)	$A_{glazing}$ (ft^2)
1	11093.33	1588.00
2	9706.67	1920.20
3	9706.67	1846.20
4	9706.67	2681.60
5	9706.67	2780.40
6	9706.67	2780.40
Roof[3]	5079.00	N/A

[3] Roof has partitions enclosing mechanical equipment and stairwell.
** 5 lb/ft^2 dead load collateral.

Material	Weight	Notes
NW. CONC	150 lb/ft^3	AISC 14 Ed. Table 17-13
LW. CONC	113 lb/ft^3	Arch. Graphics Standards 11 Ed.
VCT	1.33 lb/ft^3	Arch Graphics Standards 11 Ed.
Ceramic / Porcelain Tile	10 lb/ft^2	AISC 14 Ed. Table 17-13
3 Ply Roofing	1 lb/ft^2	AISC 14 Ed. Table 17-13
Laminated Glass – 0.8"	8.2 lb/ft^2	
MEP	15 lb/ft^2	
Partitions	15 lb/ft^2	ASCE 7-05 4.2.2

a) Floor / Deck Thickness

1) Level: 0

$t_{floor} = 4''$, solid reinf. conc.

2) Level: 1 → 5

$d_{deck} = 2''$, assume metal deck has equal size corrugations

$t_{floor} = 5''$

$t_{floor,eq} = t_{floor} - d_{deck}/2 = 4''$, use to determine conc. weight

--->

Thaison Nguyen | Load Determination – DEAD, LIVE, RAIN | 2/5

3) Level: Roof

$d_{deck} = 1.5''$, assume metal deck has equal size corrugations

$t_{floor} = 10\,\frac{1}{8}'' \rightarrow 3\,\frac{11}{16}''$

$t_{floor,avg} = (10\frac{1}{8} + 3\frac{11}{16})/2$

$t_{floor,avg} \approx 7''$

$t_{floor,eq} = t_{floor,avg} - d_{deck}/2 \approx 6.25''$, use to determine conc. weight

b) Floor Level Dead Weight w/o structural steel, Metal Deck, Flooring, Facade

1) Level: 0

$$DL = 0.150(t_{floor})(A_{gross}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$$
$$DL = 0.150(4/12)(24153) + 0.015(24153 - 293 - 724) + 0.005(24153)$$
$$DL = 1675.5 \text{ kip}$$

2) Level: 1

$$DL = 0.150(t_{floor,eq})(A_{gross} - A_{fl\,opening}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$$
$$DL = 0.150(4/12)(26440 - 1571) + 0.015(26440 - 1571 - 609) + 0.005(26440)$$
$$DL = 1739.6 \text{ kip}$$

3) Level: 2→5

$$DL = 0.150(t_{floor,eq})(A_{gross} - A_{fl\,opening}) + 0.015(A_{gross} - A_{fl\,opening} - A_{stairs}) + 0.005(A_{gross})$$
$$DL = 0.150(4/12)(26440 - 293) + 0.015(26440 - 293 - 609) + 0.005(26440)$$
$$DL = 1822.6 \text{ kip/floor level}$$

4) Level: Roof

$$DL = 0.113(t_{floor,eq})(A_{gross}) + 0.015(A_{gross} \times 0.20) + 0.001(A_{gross}) + 0.005(A_{gross})$$
$$DL = 0.113(6.25/12)(26440) + 0.015(26440)(0.20) + 0.001(26440) + 0.005(26440)$$
$$DL = 1794.1 \text{ kip}$$

AMPAD

Thaison Nguyen | Load Determination - DEAD, LIVE RAIN | 3/5

c) Dead Weight of Flooring

Floor Level	0		1		2 or 3 or 4 or 5	
Flooring	VCT	Ceramic	VCT	Ceramic	VCT	Ceramic
Area (ft^2)	1410	2841	531	653	531	339

*Other areas have exposed conc.

1) Level: 0

$DL = 1.33(1410) + 10(2841) = 30.3$ Kip

2) Level: 1

$DL = 1.33(531) + 10(653) = 7.2$ Kip

3) Level: 2 → 5

$DL = 1.3(531) + 10(339) = 4.1$ Kip / floor level

d) Dead Weight of Facade Envelope (by story)

1) Story: 1

$DL = 0.150(A_{facade} - A_{glazing}) + 0.0082(A_{glazing})$
$DL = 0.150(11093.33 - 1588.00) + 0.0082(1588.00)$
$DL = 1438.8$ Kip

2) Story: 2

$DL = 0.150(9706.67 - 1920.20) + 0.0082(1920.20)$
$DL = 1183.7$ Kip

3) Story: 3

$DL = 0.150(9706.67 - 1846.20) + 0.0082(1846.20)$
$DL = 1194.2$ kip

4) Story: 4

$DL = 0.150(9706.67 - 2681.60) + 0.0082(2681.60)$
$DL = 1073.7$ Kip

→

AMPAD

5) Story : 5

$DL = 0.150(9706.67 - 2780.40) + 0.0082(2780.40)$
$DL = 1061.7$ kip

6) Story : 6

$DL = 0.150(9706.67 - 2783.40) + 0.0082(2783.40)$
$DL = 1061.3$ kip

7) Story : Roof

$DL = 0.150(5079.00)$
$DL = 761.85$ kip

e) Live Load w/o Live Load Reduction

Room Type	Load (lb/ft²)	Notes
Stairs	100	ASCE 7-05 Table 4-1
Lobby & First Floor Corridor	100	↓
Corridor Above First Floor	80	
Ordinary Flat Roofs	20	

* Partitions : 15 lb/ft², per ASCE 7-05 4.2.2

1) Level : 0

$LL = 0.100(A_{gross} - A_{el\ opening} - A_{stairs}) + 0.100(A_{stairs})$
$LL = 0.100(24153 - 293 - 724) + 0.100(724)$
$LL = 2313.6$ kip

2) Level : 1

$LL = 0.080(26440 - 1571.00 - 609.00) + 0.100(609.00)$
$LL = 2001.7$ kip

3) Level : 2 → 5

$LL = 0.080(26440 - 293.00 - 609.00) + 0.100(609.00)$
$LL = 2103.9$ kip

→

f) Rain Load

Rainfall Rate(I): 4.5" per hour (100 year return period) ; per International Plumbing Code 2009 Appendix B, ASCE 7-05 C8.5

$(A) = 52 * 60.17 = 3128.7$, per ASCE 7-05 C8.5
$(Q) = 0.0104(A)(I) = 146.42$, per ASCE 7-05 C8.3

$d_s = 2\frac{5}{8} + 4(\frac{1}{4}) = 3.63''$
$d_h = 1 + \left[\frac{(Q-80)}{(170-80)}\right] = 1.738''$, interpolation of ASCE 7-05 Table C 8-1

$R = 5.2(d_s + d_h)$
$R = 5.2(3.63 + 1.738)$
$R = 27.89\ lb/ft^2 > $ (Roof live load = 20 lb/ft²)

Appendix C: Gravity Load Calculations

Thaison Nguyen | | Gravity Spot Check | 1/5

Member Type	Typical Span (ft)	Typical Spacing (ft)	Location
Beam	33	8.25	B1 → B2
Girder	33	33	B2 → C2
Joist	28.67	5.5	B1 → B2

a) Roof and Floor Deck, Joists

Load Combination: $1.2D + 1.6L + 0.5(L_r \text{ or } R \text{ or } S)$

	Roof Deck[1]	Floor Deck[1]	Joist
ℓ_{span} (ft)	5.5	8.25	28.67
Spacing (ft)	N/A	N/A	5.5

[1] Assume 3 span decks

1) Roof Deck

* Assume 2 hr. fire rating.

Total Load (TL) = DL + LL + R

$TL = 79.9 + w_{DECK} + 27.98$

$TL = 107.9 \text{ lb/ft}^2 + w_{DECK}$

$DL = 0.113(6.25/12) + 0.015 + 0.001 + 0.005 + w_{DECK}$

$DL = 0.0799 \text{ kip/ft}^2 + w_{DECK}$

$DL = 79.9 \text{ lb/ft}^2 + w_{DECK}$

Check 1.5B24 (using Vulcraft 2008 Manual)

Max SDI Span = 5'-10" > 5'-6" ✓, Good.

Max Allowable Load = 128 lb/ft²

TL = 107.9 + 1.46

TL = 109.4 lb/ft² < 128 lb/ft² ✓, Good

* Since roof live load = 20 lb/ft² is smaller than Rain load (27.98 lb/ft²) and unlikelines of work performed on roof during rain → Use Rain load

Load Causing $\ell/180$ = 4/3 * 90

Load Causing $\ell/180$ = 120 lb/ft² > 109.4 lb/ft² ✓, Good.

* Serviceability Criteria
  $\Delta \le \ell/180$, Supporting Non-Plaster Ceiling

* Un-protected deck is rated up to 2 hrs ✓, Good.

May use 1.5B24

2) Floor Deck

* Assume 2hr fire rating
* Assume floor deck is composite type

LL = 100 lb/ft², areas close to stairs

Check 2VLI22 using Vulcraft 2008 Manual

Weight of deck = 1.62 lb/ft²

Max SDI span = 8'-11" > 8'-3" ✓, Good

Max Superimposed Live Load = 153 lb/ft² > 100 lb/ft² ✓, Good

→

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- Use Cementitious or sprayed fiber fire protection to achieve 2 hr rating

May use 2VLI22 w/ either cementitous or spray fiber protection.

3) Joists

$W_u = 1.2DL + 0.5R$
$W_u = [1.2(71.5 + W_{Joist}) + 0.5(27.89)]\ 5.5$
$W_u = [99.8\ lb/ft^2 + 1.2(W_{Joist})]\ 5.5$
$W_u = 548.9\ lb/ft + 6.6\ W_{Joist}$

$DL = 0.150(4/12) + 0.015 + 0.005 + W_{deck} + W_{Joist}$
$DL = 70\ lb/ft + 1.46 + W_{Joist}$
$DL = 71.5\ lb/ft + W_{Joist}$

* Since roof live load = 20 lb/ft² is smaller than Rain load (27.89 lb/ft²) and unlikeliness of work performed on roof during rain → use Rain load

* Servicability Criteria
$\Delta \le \ell/180$, supporting Non Plaster ceiling

Check 22K6 using SJI Economy Table
* Assume 2 hr. fire rating
$W_u = 548.9 + 6.6(9.2)$, $W_{Joist} = 9.2\ lb/ft$
$W_u = 609.6\ lb/ft$

$W_{u,capacity} = (29 - 28.67)(540 - 597) + 597$
$W_{u,capacity} = 611.2\ lb/ft > 609.6\ lb/ft$ ✓, Good

$LL_{capacity} = [(29 - 28.67)(328 - 295) + 295]\ 360/180$
$LL_{capacity} = 611.8\ lb/ft > 27.89(5.5)$

$611.8\ lb/ft > 153.4\ lb/ft$ ✓, Good

* Use spray applied fire resistive materials (ex. Cementitious or fiber) to achieve 2hr. rating, per SJI

May use 22K6 w/ spray applied fire resistive materials

b) Beam, Girders

Load Combination: $1.2D + 1.6L + 0.5(L_r \text{ or } R \text{ or } S)$
* Assume beams and girders are pinned connected, A992 Gr 50

1) Beam
$W_u = [1.2(DL) + 1.6(LL)] * \text{Spacing of bm}$
$W_u = [1.2(71.6) + 1.6(80)] * 8.25 + 1.2(W_{bm})$
$W_u = 1765\ lb/ft + 1.2\ W_{bm}$

$DL = 0.150(4/12) + 0.015 + 0.005 + W_{bm} + W_{Deck}$
$DL = 71.6\ lb/ft^2 + W_{bm}$
$LL = 80\ lb/ft^2$

$M_u = W_u \ell^2/8$
$M_u = (1765 + 1.2\ W_{bm})(33^2)/8$
$M_u = 240251 + 163.4\ W_{bm}$

$V_u = W_u \ell/2$
$V_u = (1765 + 1.2\ W_{bm})(33/2)$

→

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$V_u = 29123 + 19.8\, W_{bm}$

Check W14x74 using AISC 14 Ed. Table 3-10, 3-6

$M_u = 240261 + 163.4(74)$
$M_u = 252.4$ kip·ft

$\phi M_n = 272.0$ kip·ft > 252.4 kip·ft ✓, Good.

$V_u = 29123 + 19.8(74)$
$V_u = 30.6$ kip

$\phi V_n = 192$ kip > 30.6 kip ✓, Good.
May use W14x74

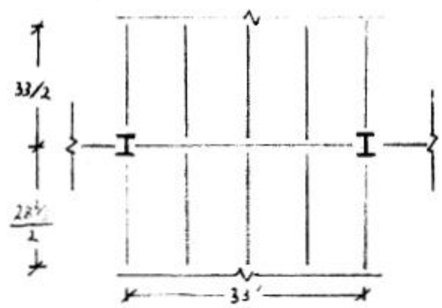
$\Delta_{TL} \le \ell/240$
$\Delta_{TL} \le 33(12)/240$
$\Delta_{TL} \le 1.65''$

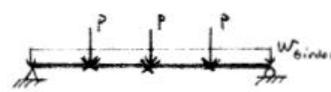
$W_T = (DL + LL)$
$W_T = 8.25(71.6 + 80) + W_{bm}$
$W_T = 1250.7 + W_{bm}$
$\Delta_{TL} = \dfrac{5(1250.7 + 74)(33^4)}{384(29\times10^6)(795)}$
$\Delta_{TL} = 1.53'' < 1.65''$ ✓, Good

2) Girder
* Assume girders use shear studs to have composite action.
* For ease in constructability assume all beams are W16x89

$\ell_{brace} = 0$

$M_u = \frac{33}{4}(P_u)(1.5) + 1.2 W_{Girder}\left(\frac{33^2}{8}\right)$
$M_u = \frac{33}{4}(98.4)(1.5) + 1.2 W_{Girder}\left(\frac{33^2}{8}\right)$
$M_u = 1217.7 + 204.2\, W_{Girder}$
in kip





$P_D = [0.150(4/12) + 0.015 + 0.005 + 1+2](8.25)(33 + 28\tfrac{2}{3})/2 + 0.089(33 + 28\tfrac{2}{3})/2$
$P_D = 52.1 + 2.7$
$P_D = 54.8$ kip, unfactored Dead Load

$P_L = 0.080(8.25)(33 + 28\tfrac{2}{3})/2$
$P_L = 20.4$ kip, unfactored Live Load

$P_u = 1.2 P_D + 1.6 P_L$
$P_u = 1.2(54.8) + 1.6(20.4)$
$P_u = 98.4$ kip

$W_u = 1.2\, W_{Girder}$

$b_{eff} = \min \begin{cases} 2 \cdot \ell/8 \\ (\text{Spacing 1} + \text{Spacing 2})\tfrac{1}{2} \end{cases}$
$b_{eff} = \min \begin{cases} 8.25' \\ 30.8' \end{cases} = 8.25'$

$A_s f_y = 0.85 f'_c b_{eff} a$, assume $f'_c = 4000$ psi
$a = \dfrac{50 A_s}{0.85(4)(8.25)(12)}$
$a = 0.149 A_s$, if neutral axis (Plastic) is in conc.

→

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Check W24x76 using AISC 14Ed. Table 3-19, Table 1-1

* Assume perfect shear transfer

$A_s = 22.4\ in^2$

$a = 0.149(22.4)$

$a = 3.34'' > 3''$ (Solid part of floor slab), PNA is in flange of STL. member

$A_s f_y - 0.85 f'_c\, b_{eff}\, T_{solid} + 2 F_y b_f x$

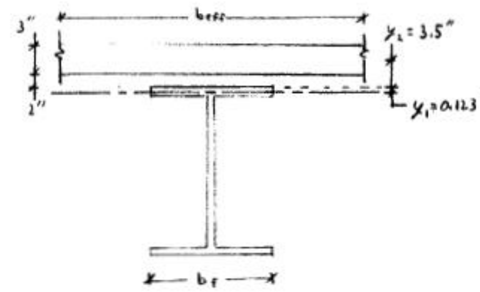
$\frac{A_s f_y - 0.85 f'_c\, b_{eff}\, T_{solid}}{2 f_y b_f} = x$

$x = \frac{22.4(50) - 0.85(4)(8.25 \cdot 12)(3)}{2(50)(8.99)}$

$x = 110.2 / 899$

$x = 0.123''$

$y_1 = x$



$M_u = 1217.7 + 204.2(76/1000)$

$M_u = 1233.2$ kip·ft

$I_{LB} = \frac{(0.17 - 0.123)(4770 - 4580)}{0.17} + 4580$

$I_{LB} = 4632.5\ in^4$

$\phi M_n = \frac{(0.17 - 0.123)}{0.17} \cdot (1300 - 1260) + 1260$, interpolation of Table 3-19

$\phi M_n = 1271.1$ kip·ft > 1233.2 kip·ft ✓, Good.

$\Delta_{LL} \leq l/360$, final live load

$\Delta_{LL} = \frac{5(P_L/33)(33^4)(1728)}{384(29000)(4632.5)}$

$\Delta_{LL} = 0.123'' < 1.1''$ ✓, Good.

May use W24x76 w/ Shear Studs (Composite Action, Partial)

$\Delta_{LLD} \leq l/360$

$\Delta_{LLD} \leq 33(12)/360$

$\Delta_{LLD} \leq 1.1''$

$P_{constr} = [0.150(4/12) + 0.005 + 1.52](8.25)(33 + 28^{1/3})/2 + 0.089(33 + 28^{1/3})/2$

$P_{constr} = 48.3 + 2.7 = 51.0$ Kip

$W_{Girder} = 0.076$ Kip/ft

$\Delta_{LLD} = \frac{5(0.076)(33^4)(1728)}{384(29000)(2100)} + \frac{5(51/33)(33^4)(1728)}{384(29000)(2100)}$

$\Delta_{LLD} = 0.033 + 0.677$

$\Delta_{LLD} = 0.71''$, during construction

$0.71'' < 1.1''$ ✓, no shoring req.

→

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C) Column

Location: B-2
*Assume pinned base

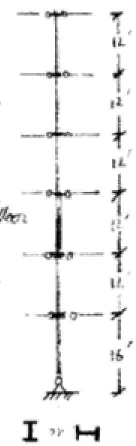
K = 1

$P_L = 0.080(A_{trib})(5 \text{ floors})$
$P_L = 0.080(990.5)(5)$
$P_L = 396.2$ Kip , live load w/o reduction

$A_{trib} = \frac{(31.25+33)}{2} * \frac{(33+28^{2/3})}{2}$

$A_{trib} = 990.5$ ft²/floor

$P_R = 0.02789(990.5)(1)$
$P_R = 27.6$ Kip , rain load



Dead Load Components [2]	Weight	Notes
LW CONC.	113 lb/ft³	Arch Graphics Standard 11 Ed.
Roof Deck	1.46 lb/ft²	Vulcraft 2008 Deck Manual, 1.5B24
Joist	9.2 lb/ft	Vulcraft 2008 Joist Manual, 22K6
3 Ply - Roofing	1 lb/ft²	AISC 14 Ed Table 17-13
NW CONC.	150 lb/ft³	AISC 14 Ed Table 17-13
Floor Deck	1.62 lb/ft²	Vulcraft 2008 Deck Manual, 2VLI 22
Beam	74 lb/ft	AISC 14 Ed , W14×74
Girder	76 lb/ft	AISC 14 Ed , W24×76
MEP	15 lb/ft²	

[2] 5 lb/ft² per floor level , dead load collateral to be included

$P_D = 113(7/12)(990.5) + 1.46(990.5) + 9.2(33+28^{2/3})(0.5)(5.5) + 1(990.5)$
$+ [150(4/12)(990.5) + 1.62(990.5) + 74(33+28^{2/3})(0.5)(3.5) + 76(33+31.25)(0.5)]5$
$+ 15(990.5)(6) + 5(990.5)(6)$
$P_D = 69.3 + 61.6(5) + 14.9(6) + 5.0(6)$
$P_D = 496.7$ Kip , dead load

$P_{TL} = 1.2P_D + 1.6P_L + 0.5P_R$
$P_{TL} = 1243.8$ Kip

$Kl_x = 1(16) = 16'$
$Kl_y = 1(16) = 16'$, weak axis bending controls.

Check W14×120 using Table 4-1 in AISC 14Ed

$\phi P_n = 1310$ kip > 1243.8 kip ✓ , Good

May use W14×120 for Column B-2

Appendix D: Current Structural System

Thaison Nguyen		Current Structural system	1/1

*** See Hand calculations in "Gravity spot Check - b) Beams, Girders" for Beam and Girder Sizes.

a) Determine number of shear studs per Girder

$Y_1 = 0.123$

$$\Sigma Q_n = \frac{0.17 - 0.123}{0.17}(1120 - 967) + 967$$

$\Sigma Q_n = 1009.3$ kip

$Q_n = 21.5$ kip, 3/4" dia shear stud

$b_f = 8.99$ in for W24x76

Studs/Girder $= 2\left(\frac{1009.3}{21.5}\right)$

Studs/Girder = 94, req. more than 1 stud per rib → use 3 studs/rib

b) Determine Wt of Typical Bay, Structural Only.

Member	Weight
W14x74	74 lb/ft
W24x76	76 lb/ft
MTL. Deck - 2VLI22	1.62 lb/ft^2
Concrete	150 lb/ft^3

*** Assume MTL. deck has equal corrugations

Equivalent Conc. Depth = 5 - 1/2

Equivalent Conc. Depth = 4"

Weight of Typical Bay = (33)(33)(4/12)(150) + (33)(33)(1.62) + 4(33)(74) + 33(76)

Weight of Typical Bay = 68490.2 lb.

Effective Weight of Typical Bay = Weight of typical Bay + 10 lb/stud * 94

Effective Weight of Typical Bay = 69430.2 lb.

Appendix E: Alternate Structural Systems

Member Type	Typical Spans (ft)	Typical Spacing (ft)	Allowable Live Load Deflection, L/360 (in)	Location
Beam	33	8.25	1.10	B1 → B2
Girder	33	33	1.10	B2 → C2
Roof Joist	28.67	5.5	0.96	B1 → B2

Typical Column Tributary Area = 990.5 ft^2 ; Column B2, per Gravity Spot Check - Column

*** Assume 24" Member Depth Limit
*** Assume no floor shoring during construction

a) System: Composite Joists and Non-Composite Joist-Girder w/ Composite MtL Deck

Composite Joist Span = 33'
Non composite Joist-Girder Span = 33'

$DL = DL_{mechanical} + DL_{collateral}$
$DL = 15 + 5$
$DL = 20$ lb/ft^2 ; doesn't include slab system, joist, or joist-girder self wt.

$LL = 80$ lb/ft^2

*** Vulcraft 2008 Steel Roof and Floor Deck Manual, for deck selection
*** Assume 3-Span Condition, Cementitous/Spray Fibre fire proofing

Spacing (ft)	Floor Deck Type	Composite Superimposed Live Load Capacity, lb/ft^2	Max Unshored Length (ft)	Deck Wt (lb/ft^2)	Total Slab Thickness (in)
5.50	1.5VL22	325	7'-5"	1.78	4.0
6.60	1.5VL22	325	7'-5"	1.78	4.0
8.25	1.5VL20	289	8'-11"	2.14	4.0

$DL_{sw} = DL_{deck} + DL_{conc}$
$DL_{sw} = 1.78 + 39 = 40.78$; Spacing = 5.5 and 6.0 (Joists)
$DL_{sw} = 2.14 + 39 = 41.14$; Spacing = 8.25 (Joist)

i) Design: Floor Composite Joist and Joist-Girder

*** Use Vulcraft 2009 Composite Joist Manual

Spacing (ft)	DL (lb/ft)	DL_{sw} (lb/ft)	LL (lb/ft)	TL_u (lb/ft)
5.50	110.0	224.3	440.0	1105.1
6.60	132.0	269.1	528.0	1326.2
8.25	165.0	339.4	660.0	1561.3

$TL_u = 1.2(DL + DL_{sw}) + 1.6(LL)$, lb/ft

Spacing (ft)	Economical Composite Joist	Shear Studs		Joist Wt.	LL causing
		Dia (in)	Quantity	lb/ft-Joist	L/360 (lb/ft)
5.50	24 CJ 1106/704/132	1/2	30	12	764
6.60	24 CJ 1327/845/159	1/2	36	14.3	948
8.25	24 CJ 1662/1056/198	5/8	34	15.1	1210

*** Use SJI Manual to select Joist-Girder
*** Use L = 35' span, due to no 33' span and conservative

Joist Spacing (ft)	TL_u [1] (lb/Panel Pt)	Economical Joist-Girder	Joist-Girder [4] Wt. (lb/ft-Joist+Girder)	LL Deflection (in) [3]	I [2] (in^4)
5.50	7389.0	48G5N7.4F	20	2.207	1265.9
6.60	11082.5	48G4N11.1F	24	1.763	1582.4
8.25	18513.1	48G3N18.5F	38	1.317	2121.5

[1] $TL_u = [1.2(DL + DL_{sw} + DL_{Joist}) + 1.6(LL)]$ / # of Panel Points
[2] I = 0.018 * Number of Panels * Load @ each panel * Span * depth (in)
[3] LL Deflection = 1.15 * Joist Spacing (ft) * Live Load (lb/ft²) * Span^4 (ft) * 5 * 1728 / (384 * E * I)
Per Vulcraft 2007 Joist & Joist-Girder Manual
[4] Joist-Girder Wt. is assumed to equal 40" deep member

*** Using Vulcraft 2009 Composite Joist Manual, determine Maximum Duct Size

Economical Joist-Girder	Max. Duct Size		
	Round	Square	Rectangular
48G5N7.4F	28" Dia	22"×22"	18"×29"
48G4N11.1F	28" Dia	22"×22"	18"×29"
48G3N18.5F	28" Dia	22"×22"	18"×29"

Due to 24" and 48" deep joist-girders not satisfying LL deflections; either use castellated girders or W-shape girders.

2) Design: W-Shape Girder (Floor)

*** Assume girders use shear studs to have composite action
$L_{br} = 0$

*** Distribute joist loads along entire length of girder

Thaison Nguyen | Alternate Structural System | 3/17

AMPAD

Load Deconstruction (lb/ft of Girder)

Load Type	Span (ft) 5.50	6.60	8.25	Notes
W_{DL} [5]	660	660	660	; doesn't include slab system, joist, or girder self wt.
$W_{DL,sw}$ [6]	1405.7	1402.9	1411.9	, Slab system and joist self wt.
W_{LL} [7]	2640	2640	2640	

[5] $W_{DL} = DL * (\text{Beam 1 Span} + \text{Beam 2 Span})/2$

[6] $W_{DL,sw} = DL_{sw} * (\text{Beam 1 Span} + \text{Beam 2 Span})/2$ + Number of Joist * Joist Wt (lb/ft) * Joist Length / Girder Length

[7] $W_{LL} = LL * (\text{Beam 1 Span} + \text{Beam 2 Span})$

*** AISC Steel Manual 14th Ed.

Joist Spacing (ft)	Total Factored Uncured Slab Load (lb/ft of Girder) [8]	Total Factored Composite Slab Load (lb/ft of Girder) [9]
5.50	2201.7	6702.9
6.60	2198.3	6699.5
8.25	2209.1	6710.3

[8] Load = $1.2(W_{DL,sw} - \text{Weight of Conc.}) + 1.6(\text{Weight of Conc})$, where Weight of Conc is $39\ lb/ft^2 * (\text{Beam 1 Span} + \text{Beam 2 Span})$

[9] Load = $1.2(W_{DL,sw} + W_{DL}) + 1.6(W_{LL})$

Joist Spacing (ft)	M_u (kip·ft) [10] Uncured Slab	Composite Slab	I_{min} for $\Delta_{LL} < L/360$, (in^4) [11] Uncured Slab	Composite Slab
5.50	299.7	912.4	1175.9	2208.3
6.60	299.2	912.0	1173.3	2208.3
8.25	300.7	913.4	1181.0	2208.3

[10] $M_u = w_u l^2/8$

[11] $I_{min} = \frac{5wl^4}{384E\Delta_{LL}}$; $E = 29000$, $\Delta_{LL} = 1.1''$, w is in kip

Check if W24x62 works, using AISC Steel Manual 14th Ed.

$$b_{eff} = \min\begin{cases} 2 * 33/8 \\ (33+33)/2 \end{cases} = 8.25\ ft$$

$b_f = 7.04''$

$I_{non\text{-}composite} = 1550\ in^4 > 1181.0\ in^4$ ✓

$A_s = 18.2\ in^2$

*** Assume $f'_c = 4000\ psi$, $f_y = 50\ ksi$

$a = \frac{f_y A_s}{0.85 f'_c b_{eff}} = \frac{50 A_s}{0.85(4)(8.25 \times 12)} = 0.149 A_s$, in

$a = 0.149(18.2)$

$a = 2.704$ in > 2.5 in , PNA is in flange of STL.

$A_s f_y = 0.85 f'_c b_{eff} t_{solid} + 2 f_y b_f x$

$x = \frac{A_s f_y - 0.85 f'_c b_{eff} t_{solid}}{2 f_y b_f}$

$x = \frac{18.2(50) - 0.85(4)(8.25 \cdot 12)(2.5)}{2(50)(7.04)}$

$x = 0.097''$

$y_1 = x$

$y_2 = 2.5/2 + 1.5$

$y_2 = 2.75''$

*** Interpolate values in table 3-19 , to determine ϕM_n and ΣQ_n

Y1	Y2			ΣQ_n
	2.5	2.75	3	
0	979	994.5	1010	910
0.097		981.4		841.6
0.148	959	974.5	990	806

$\Sigma Q_n = 841.6$ kip

$\phi M_n = 981.4$ kip*ft

$w_r/h_r = 2.5/1.5 = 1.67$

$Q_n = 21.5$ kip , ribs are parallel to girder

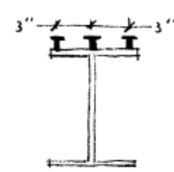
Studs/Girder $= 2\left(\Sigma Q_n / Q_n\right) \approx 80$, req more than 1 row of studs → use 3 rows of studs

*** Interpolate values in table 3-20 , to determine I_{LB}

Y1	Y2		
	2.5	2.75	3.0
0	3420	3490	3560
0.097		3414.4	
0.148	3310	3375	3440

$I_{LB} = 3414.4 \text{ in}^4 > (1181 + 2208.3)$ ✓

*** May use W24x62 , using 80 shear studs (3/4")



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Check if W21x68 works, using AISC Steel Manual 14th Ed.

$b_{eff} = \min \begin{cases} 2 \times 33/8 \\ (28\frac{1}{3}+33)/2 \end{cases} = 8.25\ ft$

$b_f = 8.27''$

$I_{non\text{-}composite} = 1480\ in^4 > 1181.0\ in^4$ ✓

$A_s = 20\ in^2$

*** Assume $f'_c = 4000\ psi$, $f_y = 50\ ksi$

$a = 0.149(20)$

$a = 2.98\ in > 2.5\ in$, PNA is in flange of steel

$x = \dfrac{20(50) - 0.85(4)(8.25) \times 12(2.5)}{2(50)(8.27)}$

$x = 0.192\ in$

$Y_1 = 0.192\ in$

$Y_2 = 1.5 + 2.5/2 = 2.75\ in$

***Interpolate Values in Table 3-19, to determine ϕM_n and ΣQ_n

Y1	Y2 2.5	Y2 2.75	Y2 3	ΣQ_n
0.171	951	967.0	983	858
0.192		963.2		841.1
0.343	922	935.5	949	717

$\Sigma Q_n = 841.1\ kip$

$\phi M_n = 963.2\ kip\cdot ft$

$w_r/h_r = 2.5/1.5 = 1.67$

$Q_n = 21.5\ kip$, ribs are parallel to girder

Studs/Girder $= 2\Sigma Q_n / Q_n \approx 80$, req more than 1 row of studs → Use 3 rows of studs.

*** Interpolate values in table 3-20, to determine I_{LB}

Y1	Y2 2.5	Y2 2.75	Y2 3
0.171	3050	3115.0	3180
0.192		3095.8	
0.343	2900	2955.0	3010

$I_{LB} = 3095.8\ in^4 < (1181 + 2208.3)$ ✓

*** Can't use W21x68 w/ 80 shear studs (3/4")
unless shoring is used

→

3) Determine Weight of typical Bay.

*** Include 10 lb/stud in effective composite joist system wt. (per

Joist Spacing (ft)	Economical Joist	Shear Studs		Composite Joist Wt (lb/Joist)[12]	Effective Composite Joist Wt (lb/Joist)[13]
		Dia(in)	QTY		
5.50	24CJ 1106/704/132	1/2	30	396	696
6.60	24CJ 1327/845/159	1/2	36	471.9	831.9
8.25	24CJ 1662/1056/198	5/8	34	597.3	937.3

[12] load = Joist Wt (lb/ft) * Length.
[13] load = Joist Wt (lb/ft) * Length + 10 lb/stud * Number of Studs.

Composite Joist w/ Composite W-Shape Girder

Joist Spacing (ft)	Total Unfactored System Wt. (lb)[14]	Effective Unfactored System Wt (lb)[15]
5.50	50837.2	53437.2
6.60	50820.7	53420.7
8.25	51254.3	53414.3

[14] Load = (Number of Joists x Joist Wt) + (Number of Girders x Girder Wt.) + (Area of Bay * Wt. of slab)
[15] Load = (Number of Joists * Joist Wt) + (Number of Girders * Girder Wt) + (Area of Bay x Wt of slab) + (10 lb/stud * Number of Studs)

→

b) System : Girder Slab System

Hollow Core Span = 33'
Girder Span = 33'

*** Use STRESCORE load tables to determine appropriate hollow core
*** Assume no topping on hollow core
*** Include 5 lb/ft^2 dead load collateral

Load Determination (lb/ft^2)

Load Type	Span (ft) 33	Notes
DL	20	; doesn't include slab system, joist, or joist girder self wt.
LL	80	

*** Non-Composite is when only dead load is applied (during construction), per Girder Slab Design Guide v1.5

1) Select Hollow Core Plank

Check if 10SC26/108 works

$DL_{sw} = 67\ lb/ft^2$
$TL = (20+67)+80$
$TL = 167 < 170$ ✓

*** May use 10SC26/108

2) Design Girder

Check if DB 9x46 works

*** Use properties provided in Girder Slab Design Guide v1.5

$DL_{sw\ D\text{-}Girder} = 45.8\ lb/ft$
$M_u = 84000\ lb/ft$
$I_{min,L/360} = \frac{5(33)^4(1728)}{29000000(1.1)} * Load\ (lb/ft^2)$
$I_{min,L/360} = 0.836 * Load\ (lb/ft^2)$

$M_{DL} = 33(20+67)(33^2)/8$
$M_{DL} = 397049\ lb\text{-}ft$

$M_{LL} = 33(80)(33^2)/8$
$M_{LL} = 359370\ lb\text{-}ft$

$M_{TL,u} = 1.2M_{DL} + 1.6M_{LL} = 1051451 > 84000$ X, can't use D-Girder

Check if Δ-Girder D50-600 works

$TL_{sw,u} = 33 * [1.2(20+67) + 1.6(80)] = 7669.2$ lb/ft , doesn't include girder self-wt

*** Use PEIKKO Capacity Curves (metric) to determine capacity.
*** PEIKKO (metric) uses C25/30 conc., equivalent to 4000 psi Conc
*** PEIKKO (metric) uses S355J2N Steel, equivalent to Gr 50 steel

Max Linearly Distrib. Load (lb/ft) = 8570.5 > 7669.2 ✓, Can use D50-600 w/o Self-Wt.

*** PEIKKO cambers all Δ-Girders to ensure adherence to deflection requirements.

3) Determine Wt. of Bay.

*** Web holes are 6" dia and 12" O.C.
*** Assume plate thickness is 1"
*** STL density is 490 lb/ft^3

Number of holes in 33' Girder = 68
Total Volume of Web Holes in Girder = $68(\pi * 3^2) = 1922.7$ in^2
Total Volume of STL membre w/o Web Holes = $84.5(33 \times 12) = 33462.0$ in^3

Area of section STL.

Total Volume of STL membre = $33462.0 - 1922.7(1) = 31539.3$ in^3

Total Volume of Conc. (in^3) = $[(20 * 33.84)(33 \times 12) - 31539.3](1.5) = 354710.2$

Height and Width of section

Girder Wt (lb/girder) = $31539.3(490/12^3) + 354710.2(150/12^3) = 39734.3$

Total Bay Wt. = $39734.3 + (33)(33 - 33.84/12) \times 67 = 106462.5$ lb/bay.

→

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C) Design: 2-Way Flat Plate

* Assume columns can be arranged regularly such that adjacent spans are no more than 1/3 different in length and columns are offset no more than 10% of span.
→ Use direct design method.

$f'_c = 4000\ lb/in^2$
$f_y = 60000\ lb/in^2$

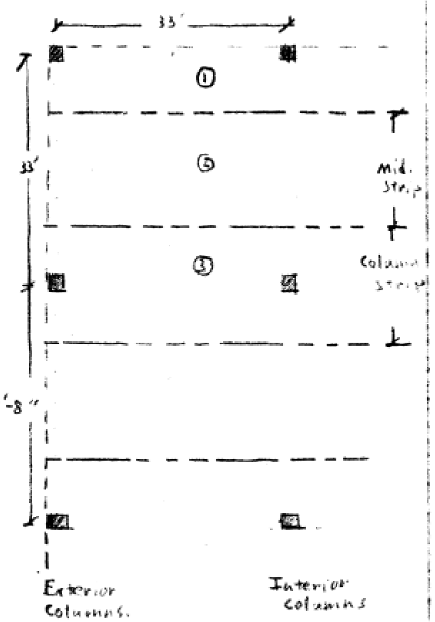
$L_1 = 33\ ft$
$L_2 = 33\ ft$

Column strip (3T) $= \frac{33}{4} = 8.25'$

Column strip (3B) $= \frac{28\ 2/3}{4} = 7.17'$

Column strip (1B) $= \frac{33}{4} = 8.25'$

Middle strip (2) $= 33 - 8.25 - 8.25$
Middle strip (2) $= 16.5'$



*** Assume columns are 18" x 18"

$T_{min} = \frac{(33-1.5)}{33} * 12$, flat plate per ACI 318-11 Table 9.5(c)

$T_{min} = 11.45" \approx 12"$

DL = 20 lb/ft^2 ; includes MEP and 5 lb/ft collateral
LL = 80 lb/ft^2

$q_{DL,u} = 1.2(20) = 24\ lb/ft^2$

$q_{LL,u} = 1.6(80) = 128\ lb/ft^2$

$M_o = q_u \ell_2 \ell_n^2 / 8$, lb-ft
$M_o = (24+128)(33)(33-1.5)^2/8 + 150\left(\frac{1.2}{12}\right)(33)(33-1.5)^2/8 * \text{Slab Thickness (in)}$
$M_o = 622140.8 + 61395.5 * \text{Slab Thickness (in)}$

$M_{o,ext}^- = 0.26 M_o$

$M_{o,int}^- = 0.52 M_o$

$M_o^+ = 0.7 M_o$

→

1) Distribute moments between middle and column strip.

Interior Column Distribution

Column Strip Share = 75%

$M^{-}_{int, col strip}$ (lb-ft) = 326623.9 + 32232.6 * Slab Thickness

$M^{+}_{int, mid strip}$ (lb-ft) = 108874.6 + 10744.2 * Slab Thickness

Exterior Column Distribution

Column Strip Share = 100%

$M^{-}_{ext, col strip}$ (lb-ft) = 161756.6 + 15962.8 * Slab Thickness

$M^{+}_{ext, mid strip}$ (lb-ft) = 0

Mid-Span Distribution

Column Strip Share = 60%

$M^{+}_{mid, column strip}$ (lb-ft) = 194107.9 + 19155.4 * Slab Thickness

$M^{+}_{mid, mid strip}$ (lb-ft) : 129405.3 + 12770.3 * Slab Thickness

2) Design Slab.

*** 0.75" cover, per ACI 318-11 Section 7.7

Check if 12" Slab Works for Flexure

Design Moment (kip-ft)			
Strip	Location		
	Exterior Column Face	Mid-Span	Interior Column Face
Column	-353.3	424.0	-713.4
Middle	0	282.6	237.8

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

$\frac{M_u}{\phi} = A_s f_y [d - 17.65 A_s/(2b)]$

$\frac{M_u}{\phi f_y} = d A_s - 8.83 A_s^2 / b$

$d = 12 - 0.75 - 0.5 = 10.75$

$a = \frac{A_s f_y}{0.85 f'_c b}$

$a = \frac{60}{0.85(4)} * \frac{A_s}{b}$

$a = 17.65 A_s / b$

$d = h_{tot} - 0.75 - 1.5 d_{bar} - d_{stirrup}$

→

*** Assume one size of rebar for flexure in slab.

*** Assume #4 stirrups

Strip Width (ft)	$A_{s,min}$ crack control (in^2)
8.25	2.14
16.5	4.28

*** Flexural reinforcement (required) is on following page

• Determine maximum number of #8 rebar w/ 1" Aggregate per strip width

$$S_c = \max\begin{cases} 1'' \\ d_{bar} \\ 4/3 \cdot \text{Aggregate size} \end{cases}$$

$$S_c = 4/3''$$

$$b - 2C_c - 2d_{tr} > n d_b + (n-1) S_c$$

$$b - 2C_c - 2d_{tr} + S_c > n(d_b + S_c)$$

$$n < \frac{b - 2C_c - 2d_{tr} + S_c}{d_b + S_c}$$

$$n_{b=8.25'} \le \frac{8.25(12) - 2(0.75) - 2(0.5) + 4/3}{1 + 4/3}$$

$$n_{b=8.25'} \le 97.83 / 2\tfrac{1}{3} = 41 \text{ bars}$$

$$n_{b=16.5'} \le 84 \text{ bars}$$

• Determine $A_{s,max\,\phi=0.9}$, $A_{s,min}$ when using #8 rebar

$A_{s,min} = 0.0032(8.25)(12)(9.25)$, when strip width is 8.25'

$A_{s,min} = 2.93\ in^2$

$A_{s,min} = 0.0032(16.5)(12)(9.25)$, when strip width is 16.5'

$A_{s,min} = 5.86\ in^2$

$A_{s,max\,\phi=0.9} = 0.0719(8.25 \cdot 12)(9.25)$, when strip width is 8.25'

$A_{s,max\,\phi=0.9} = 65.84\ in^2$

$A_{s,max\,\phi=0.9} = 131.68\ in^2$, when strip width is 16.5'

$\varepsilon_s \ge 0.005$ for $\phi = 0.9$

$$c = a/\beta_1$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d - c)$$

$$\frac{c\,\varepsilon_s}{\varepsilon_u} = (d - c)$$

$$d = c\left(\frac{\varepsilon_s}{\varepsilon_u} - 1\right)$$

$$c = \frac{h_{tot} - 0.75 - 1.5 d_{bar} - d_{stirrup}}{\frac{0.005}{0.003} - 1}$$

$$A_{s,max\,\phi=0.9} = \frac{0.85 f'_c b (h_{tot} - 0.75 - 1.5 d_{bar} - d_{stirr})}{0.67 f_y} \cdot \beta_1$$

$$A_{s,max\,\phi=0.9} = 0.0719 b (h_{tot} - 0.75 - 1.5 d_{bar} - 1/2)$$

$$A_{s,max\,\phi=0.9} = 0.0719 b (h_{tot} - 1.25 - 1.5 d_{bar})$$

$A_{s,min\ crack\ control} = 0.0018 A_g$, ACI 318-11 Section 7.12.2.1

$$A_{s,min} = \min\begin{cases} 3\sqrt{f'_c} \\ 200 \end{cases} \cdot \frac{bd}{f_y}$$

$$A_{s,min} = 0.0032 b (h_{tot} - 1.25 - 1.5 d_{bar})$$

$$A_g = \begin{cases} 8.25 \cdot 12 \cdot 12, & \text{strip width} = 8.25' \\ 16.5 \cdot 12 \cdot 12, & \text{strip width} = 16.5' \end{cases}$$

$$A_g = \begin{cases} 1188\ in^2 \\ 2376\ in^2 \end{cases}$$

AMM

$ax^2 + bx + c = 0$

a					
b (ft)	Design Moment (Kip-ft)				
	237.8	282.6	353.3	424.0	713.4
8.25			-0.089		
16.5	-0.045				

b = depth to rebar, assume #8 rebar					
b (ft)	Design Moment (Kip-ft)				
	237.8	282.6	353.3	424.0	713.4
8.25	9.25				
16.5	9.25				

c					
b (ft)	Design Moment (Kip-ft)				
	237.8	282.6	353.3	424.0	713.4
8.25			-78.51	-94.22	-158.54
16.5	-52.85	-62.81			

A_s (in^2), flexure					
b (ft)	Design Moment (Kip-ft)				
	237.8	282.6	353.3	424.0	713.4
8.25			9.32	11.45	21.65
16.5	5.88	7.03			

*** Use #8 rebar, 1" aggregate

d_{actual} = 9.25 in

Quantity of #8 Rebar in Strip, flexure					
b (ft)	Design Moment (Kip-ft)				
	237.8	282.6	353.3	424.0	713.4
8.25			12	15	28
16.5	8	9			

Actual Rebar Area (in^2)					
b (ft)	Design Moment (Kip-ft)				
	237.8	282.6	353.3	424.0	713.4
8.25			9.48	11.85	22.12
16.5	6.32	7.11			

• Check if rebar required for flexure satisfies minimum and maximum A_s

$6.32\,in^2 > 2.93\,in^2$ ✓
$9.48\,in^2 > 5.86\,in^2$ ✓

$7.11\,in^2 < 65.84\,in^2$ ✓
$22.12\,in^2 < 131.68\,in^2$ ✓

*** See rebar arrangement in following page, flexure

Check if 12" slab works for Shear

$\ell_1 = \ell_2$, due to sym. of bay.
$\ell_1 = 33'$
$h = 12''$
$d = 9.25''$

$d/2 = 4.625''$
$b_o = 2[18+18+2(9.25)]$
$b_o = 109''$

$\alpha_s = 40$, interior columns

$q_u = q_{DL,u} + q_{LL,u} + 150(1)(1.2)$
$q_u = 24 + 128 + 180$
$q_u = 332\ lb/ft^2$
$q_u = 0.332\ kip/ft^2$

$V_u = q_u[33^2 - (\frac{18+9.25}{12})^2]$
$V_u = 360$ kip

$$V_c = \begin{cases} 2 + 4/\beta \\ \frac{\alpha_s d}{b_o} + 2 \\ 4 \end{cases} * \lambda\sqrt{f'_c}\ b_o d$$

$$V_c = \begin{cases} 6 \\ 3.4+2 \\ 4 \end{cases} * \sqrt{4000}\ (109)(9.25)$$

$V_c = 4 * 63767$
$V_c = 255.1$ kip
$\phi V_c = 0.75(255.1)$
$\phi V_c = 191.3$ kip < 360 kip, need shear reinf.

$\phi V_n \le 6\sqrt{4000}\ (109)(9.25)(0.75)$
$\phi V_n \le 287$ kip < 360 kip, req. shear capital

Check if 12" slab w/ 15" thk capital works

Capital Width = 2.5'

$h_1 = 15''$
$d_1 = 15 - 1.5(1) - 0.5 - 0.75$
$d_1 = 12.25''$

$h_2 = 12''$
$d_2 = 9.25''$

$b_{o,1} = 2(18+18+2*12.25)$
$b_{o,1} = 121''$

$b_{o,2} = 2(30+30+2*9.25)$
$b_{o,2} = 157$

$\rightarrow$

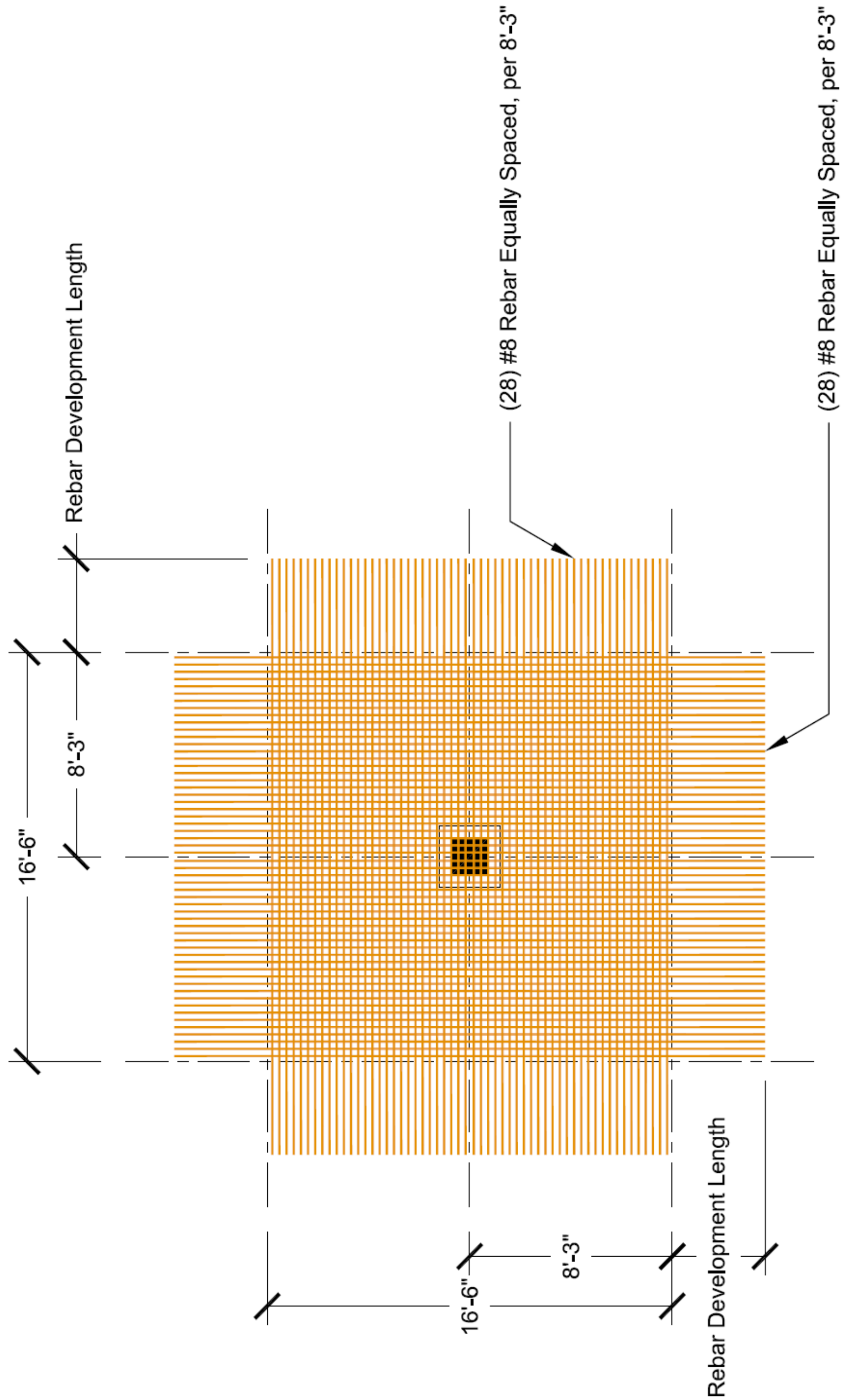


Figure AE.1, Typical Reinforcement at M^- (Column)

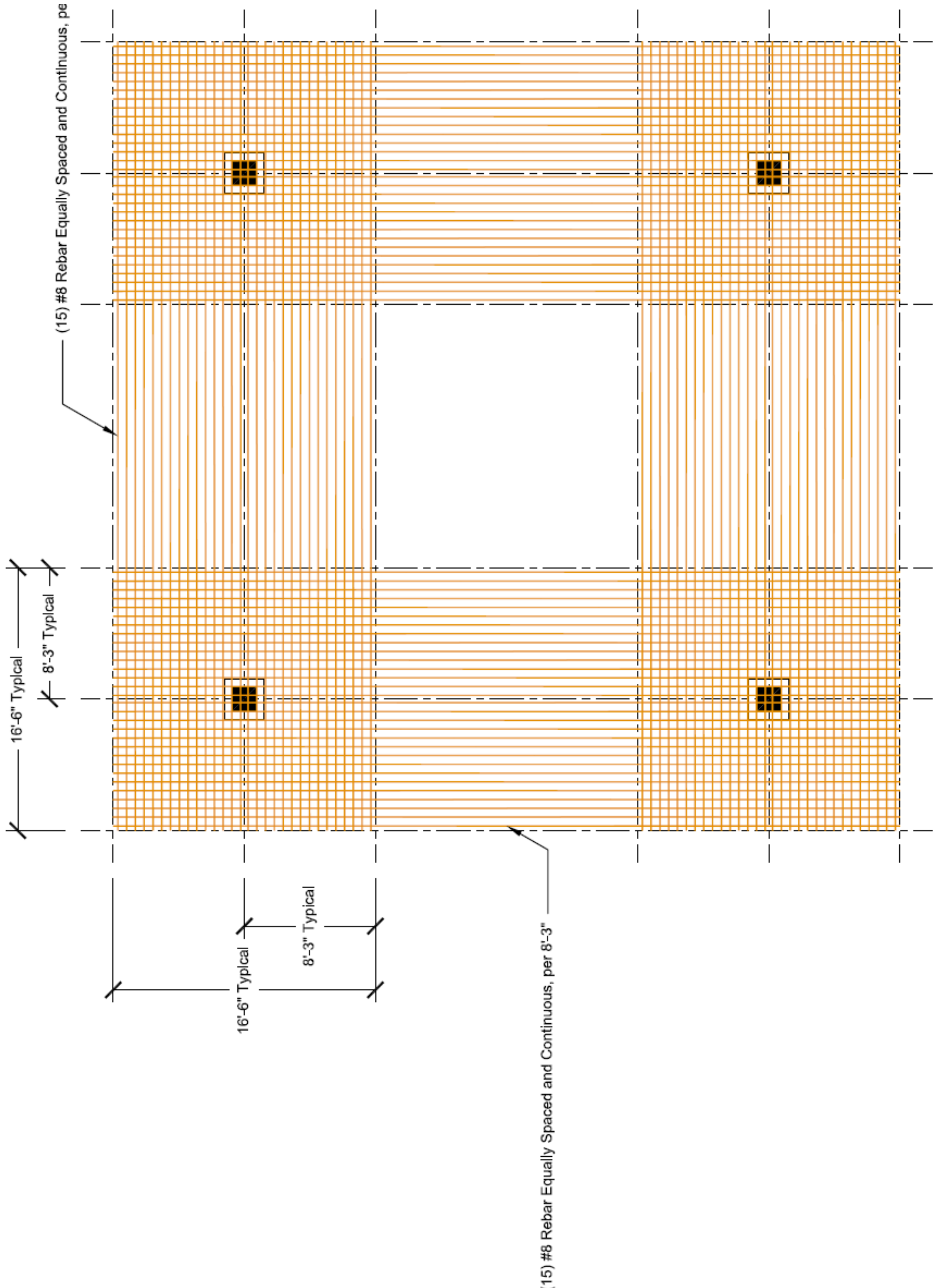


Figure AE.2, Typical Reinforcement at M^+ (Column Strip)

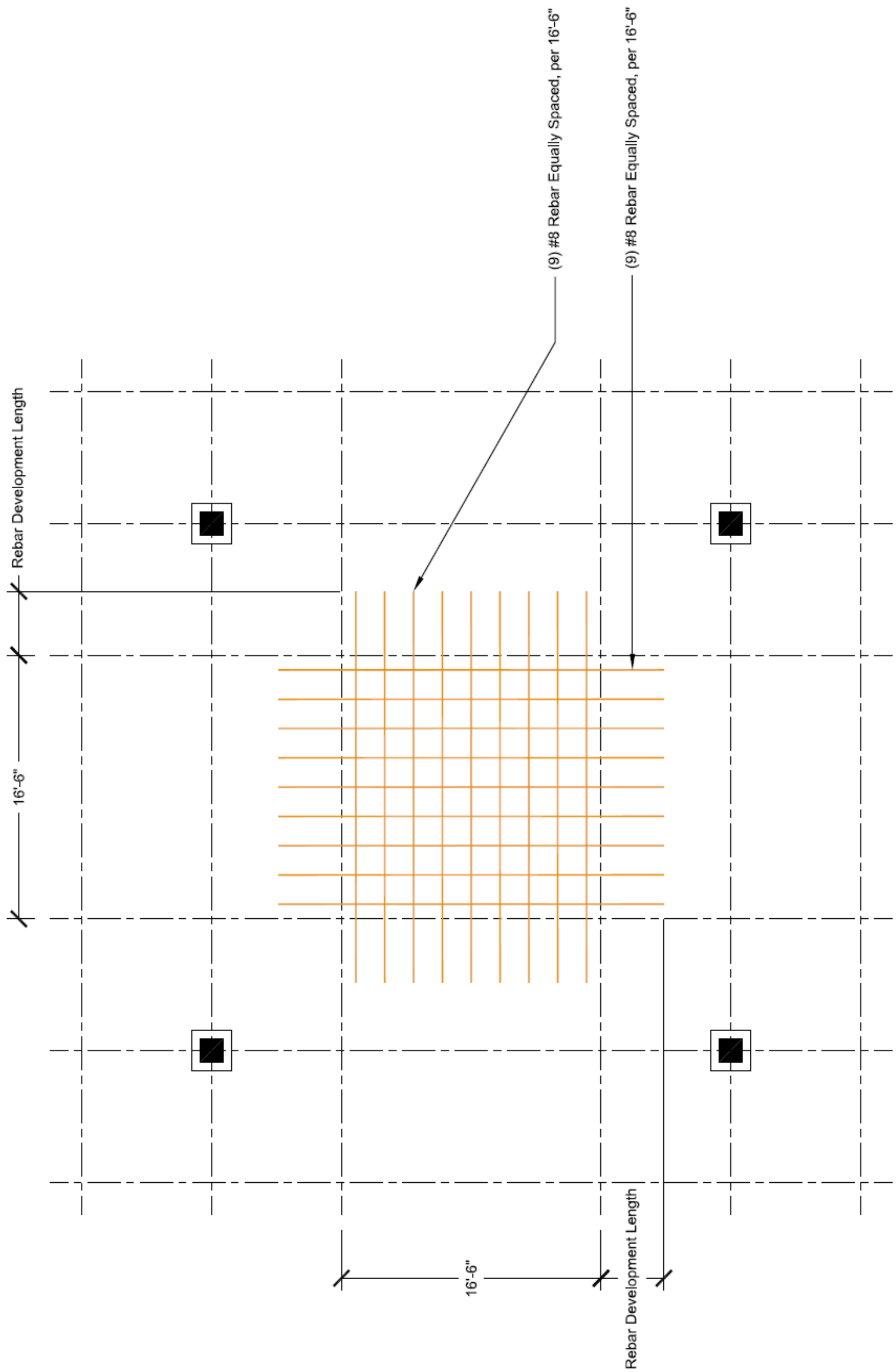


Figure AE.3, Typical Reinforcement at M^+ (Middle Strip)

$V_{u,1} = q_{u,slab}\left[33^2 - \left(\frac{121}{48}\right)^2\right] + q_{u,capital}\left[2.5^2 - \left(\frac{121}{48}\right)^2\right]$

$V_{u,1} = 360$ kip

$q_{u,slab} = 24 + 128 + 150(1.2) = 332$ lb/ft^2

$q_{u,capital} = 1.2(0.25 \times 150) = 45$ lb/ft^2

$V_{u,2} = q_{u,slab}\left[33^2 - \left(\frac{157}{48}\right)^2\right]$

$V_{u,2} = 358$ kip

$\varnothing V_{n1} = 0.75(6)\sqrt{4000}\,(121)(12.25)$

$\varnothing V_{n1} = 421.8$ kip > 360 kip ✓

$\varnothing V_{n2} = 413.3$ kip > 360 kip ✓

$$V_{c,1} = \begin{cases} 2 + 4/\beta \\ \frac{\alpha_s d}{b_o} + 2 \\ 4 \end{cases} * \sqrt{4000}\,(121)(12.25)$$

$$V_{c,1} = \begin{cases} 6 \\ 6.05 \\ 4 \end{cases} * 93745.7$$

$V_{c1} = 375$ kip

$\varnothing V_{c1} = 281.2$ kip < 360 kip, shear reinf. req.

$V_{c,2} = 4 * 91848.4$

$\varnothing V_{c,2} = 275.5$ kip < 358 kip, shear reinf. req.

*** Using stirrups require $V_c \le 2\sqrt{f'_c}\, b_o d$ and $V_s \le 4\sqrt{f'_c}\, b_o d$

$\varnothing V_{c,1a} = 140.6$ kip

$\varnothing V_{c,2a} = 137.8$ kip

$V_s = A_v f_y d/s = V_u - \varnothing V_{c,actual}$

$A_v f_y d/s = 4\sqrt{f'_c}\, b_o d$

$s \le \frac{A_v f_y}{4\sqrt{f'_c}\, b_o}$, for full capacity (allowed in code)

***(8) stirr-up legs will be arresting shear cracks

$S_1 \le \frac{8(0.2)(60000)(12.25)}{360 - 140.6}$

$S_1 \le 5.36''$

$S_{max} \le d/2$, ACI318-11 Fig. 4.8

$S_{max,1} \le 6.125''$

$S_{max,2} \le 4.625''$

$S_2 \le \frac{8(0.2)(60000)(9.25)}{358 - 137.8}$

$S_2 \le 4.03''$

*** See rebar arrangement in following page, shear.

→

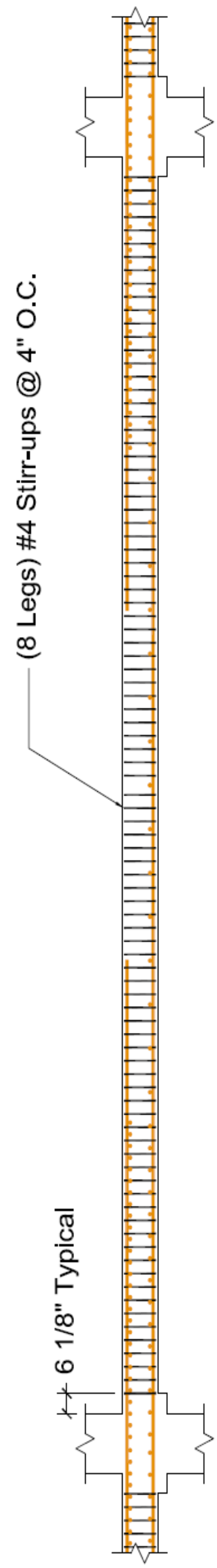


Figure AE.4, Typical Shear Reinforcement

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3) Weight of typical bay

Weight of Bay = 33(33)(1)(150) + 2.5(2.5)(0.25)(150)
Weight of Bay = 163584.4 lb.

Appendix F: Structural Computer Modeling

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	16.50	0.19	0.217	4.277	31.740	18.000	0.005	11-#6
		Middle	16.50	0.61	0.402	4.277	31.740	18.000	0.015	11-#6
		Right	16.50	1.38	0.619	4.277	31.740	18.000	0.035	11-#6
	Middle	Left	16.50	0.00	0.000	4.277	31.740	18.000	0.000	11-#6
		Middle	16.50	0.00	0.309	4.277	31.740	18.000	0.000	11-#6
		Right	16.50	0.00	0.619	4.277	31.740	18.000	0.000	11-#6
2	Column	Left	16.50	168.75	0.753	4.277	31.740	18.000	4.319	11-#6
		Middle	16.50	0.00	16.499	0.000	31.740	0.000	0.000	---
		Right	16.50	769.93	32.244	4.277	31.740	3.960	21.627	50-#6
	Middle	Left	16.50	0.40	1.385	4.277	31.740	18.000	0.010	11-#6
		Middle	16.50	0.00	16.499	0.000	31.740	0.000	0.000	---
		Right	16.50	256.65	32.244	4.277	31.740	12.375	6.648	16-#6

Table AF.1, *spSlab* Model – Two Way Flat Slab Design, M^-

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	16.50	0.00	0.309	0.000	31.740	0.000	0.000	---
	Middle	16.50	0.00	0.309	0.000	31.740	0.000	0.000	---
2	Column	16.50	476.61	13.981	4.277	31.740	6.828	12.750	29-#6
	Middle	16.50	317.74	13.981	4.277	31.740	10.421	8.302	19-#6

Table AF.2, *spSlab* Model – Two Way Flat Slab Design, M^+

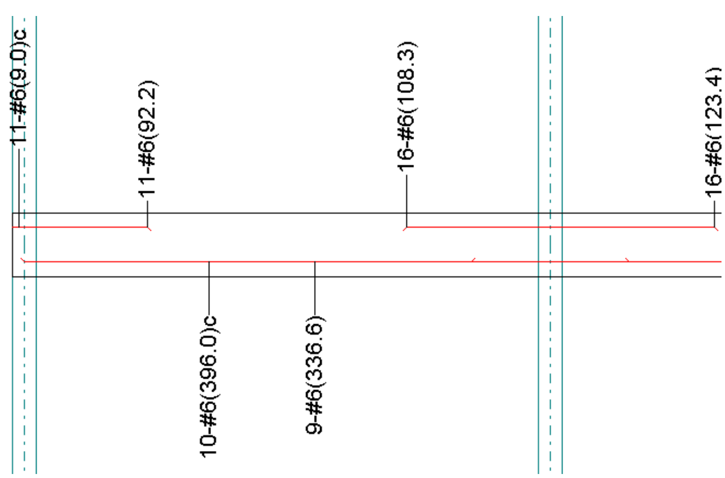

Figure AF.1, *spSlab* Model – Illustration Flexural Reinforcement for Middle Strip

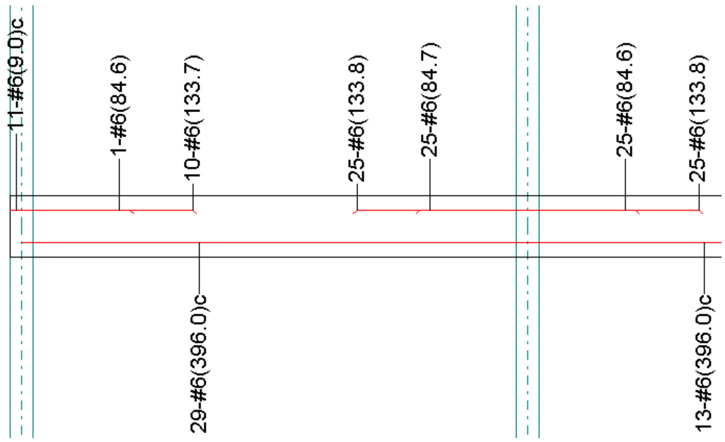

Figure AF.2, *spSlab* Model – Illustration of Flexural Reinforcement for Column Strip

Appendix G: Cost Analysis

Cost associated with the material and construction of the four structural systems was estimated with the use of RS. Means 2012. The electronic version of RS. Means incorporates the location factor into all unit costs. Since Largo, FL. is not in the RS. Means database; the closest city was used (Tampa, FL.).

Assumptions and simplifications were used to expedite the cost analysis, which include:

1. Open-Shop labor
2. Only two types formwork panels are used, one type is for establishing edges
3. Formwork is bought for project and can be used multiple times
4. Each shoring component has a 10 kip load capacity
5. Use of chemical additives to improve concrete workability and prevent premature water evaporation
6. All composite joists are a combination of K-joists and welded shear studs
7. Use 5/8" shear studs, since 3/4" shear studs aren't present in RS. Means
8. All rebar are galvanized to increase corrosion resistance
9. All rebar development length is 72 bar diameters

An excel spreadsheet was used to calculate the cost (USD/bay) of each structural system, see Table AG. 4 for details. Also located below are the RS. Means 2012 tables used for the unit estimate.

Table AG. 1, General Conditions – Construction Equipment

Source: RS. Means 2012: Commercial Cost Data

Line Number	Description	Unit	Crew	Daily Output	Labor Hours	Bare Material	Bare Labor	Bare Equipment	Bare Total	Total O&P
015419600010	**Crane crews, tower crane, monthly use, excludes…**									
015419600100	Crane crew, tower crane, static, 130' high, 106' jib, 6…	Month	A3N	0.05	176.000		5341.88	22144.20	27486.08	33176.10
015433602800	**Rent crane self-propelled, 4x4 telescoping b…**	**Ea.**				**15.73**	**232.46**	**696.39**	**2079.15**	**265.13**
015433602900	Rent crane self-propelled, 4x4 telescoping boo…	Ea.				28.21	370.74	1117.23	3356.70	449.10
015433603000	Rent crane self-propelled, 4x4 telescoping boo…	Ea.				28.91	390.78	1177.35	3532.05	466.73
015433603050	Rent crane, self-propelled, 4x4 telescoping boo…	Ea.				31.11	450.90	1357.71	4083.15	520.44
015433603100	Rent crane self-propelled, 4x4 telescoping boo…	Ea.				32.72	506.01	1523.04	4559.10	566.33
015433603150	Rent crane, self-propelled, 4x4, telescoping boo…	Ea.				40.33	571.14	1713.42	5135.25	665.33

Table AG. 2, Concrete – Formwork, Reinforcement, Finish, Labor & Materials

Source: RS. Means 2012: Commercial Cost Data

Line Number	Description	Unit	Crew	Daily Output	Labor Hours	Bare Material	Bare Labor	Bare Equipment	Bare Total	Total O&P
031113050010	**FORMS, BUY OR RENT**									
031113050015	Aluminum, smooth face, 3' x 8', buy	SFCA				14.45			14.45	15.91
031113050020	2' x 8'	SFCA				18.93			18.93	20.81
031113050050	**12" x 8'**	**SFCA**				**23.99**			**23.99**	**26.60**

031113050100	6" x 8'	SFCA				32.85			32.85	35.98
031113050150	3' x 4'	SFCA				16.90			16.90	18.57
031113050200	2' x 4'	SFCA				21.38			21.38	23.47
031113050250	12" x 4'	SFCA				28.16			28.16	31.29
031113050300	6" x 4'	SFCA				38.59			38.59	42.76
031113050500	Textured brick face, 3' x 8', buy	SFCA				16.32			16.32	17.99
031113050550	2' x 8'	SFCA				22.95			22.95	25.55
031113050600	12" x 8'	SFCA				32.85			32.85	35.98
031113050650	6" x 8'	SFCA				51.11			51.11	56.32
031113050700	3' x 4'	SFCA				20.29			20.29	22.42
031113050750	2' x 4'	SFCA				27.64			27.64	30.77
031113050800	12" x 4'	SFCA				39.63			39.63	43.28
031113050850	6" x 4'	SFCA				61.54			61.54	67.80
031113051000	Average cost incl. accessories but not incl. ties, buy	SFCA				25.55			25.55	28.16
031113051100	Rent per month	SFCA				1.27			1.27	1.41
031505700010	SHORES									
031505700020	Erect and strip, by hand, horizontal members									
031505700500	Aluminum joists and stringers	Ea.	2 Carp	60.00	0.267		7.34		7.34	12.35
031505700600	Steel, adjustable beams	Ea.	2 Carp	45.00	0.356		9.82		9.82	16.44
031505700700	Wood joists	Ea.	2 Carp	50.00	0.320		8.82		8.82	14.84
031505700800	Wood stringers	Ea.	2 Carp	30.00	0.533		14.72		14.72	24.86
031505701000	Vertical members to 10' high	Ea.	2 Carp	55.00	0.291		8.02		8.02	13.47
031505701050	To 13' high	Ea.	2 Carp	50.00	0.320		8.82		8.82	14.84
031505701100	To 16' high	Ea.	2 Carp	45.00	0.356		9.82		9.82	16.44
031505701500	Reshoring	S.F.	2 Carp	1400.00	0.011	0.52	0.31		0.83	1.10
031505701600	Flying truss system	SFCA	C17D	9600.00	0.009		0.25	0.08	0.33	0.51
032110000000	Uncoated Reinforcing Steel									
032110500010	REINFORCING STEEL, MILL BASE PLUS EXTRAS									
032110500150	Reinforcing, A615 grade 40, mill base	Ton				706.42			706.42	775.58
032110500200	Detailed, cut, bent, and delivered, average	Ton				968.24			968.24	1062.10
032110500650	Reinforcing steel, A615 grade 60, mill base	Ton				706.42			706.42	775.58
032110500700	Detailed, cut, bent, and delivered, average	Ton				968.24			968.24	1062.10
032110600015	REINFORCING IN PLACE, 50-60 ton lots, A615 Gra...									
032110600020	Includes labor, but not material cost, to install...									
032110600030	Made from recycled materials									
032110600100	Beams & Girders, #3 to #7	Ton	4 Ro...	1.60	20.000	968.24	731.33		1699.57	2330.73
032110600150	#8 to #18	Ton	4 Ro...	2.70	11.852	968.24	432.83		1401.07	1818.30
032110600200	Columns, #3 to #7	Ton	4 Rodm	1.50	21.333	968.24	781.08		1749.32	2430.23
032110600210	#3 to #7, alternate method	Lb.	4 Rodm	3000.00	0.011	0.50	0.39		0.89	1.23
032110600250	#8 to #18	Ton	4 Rodm	2.30	13.913	968.24	512.43		1480.67	1947.65
032110600260	#8 to #18, alternate method	Lb.	4 Rodm	4600.00	0.007	0.50	0.26		0.76	1.00
032110600300	Spirals, hot rolled, 8" to 15" diameter	Ton	4 Rodm	2.20	14.545	1506.70	532.33		2039.03	2604.95
032110600320	15" to 24" diameter	Ton	4 Rodm	2.20	14.545	1457.30	532.33		1989.63	2506.15
032110600330	24" to 36" diameter	Ton	4 Rodm	2.30	13.913	1383.20	512.43		1895.63	2392.25
032110600340	36" to 48" diameter	Ton	4 Rodm	2.40	13.333	1309.10	487.55		1796.65	2283.33
032110600360	48" to 64" diameter	Ton	4 Rodm	2.50	12.800	1457.30	467.65		1924.95	2396.70
032110600380	64" to 84" diameter	Ton	4 Rodm	2.60	12.308	1506.70	452.73		1959.43	2465.65
032110600390	84" to 96" diameter	Ton	4 Rodm	2.70	11.852	1580.80	432.83		2013.63	2509.90
032113100010	GALVANIZED REINFORCING									
032113100150	Add to uncoated reinforcing price for galvanizi...	Ton				444.60			444.60	489.06
033105350010	NORMAL WEIGHT CONCRETE, READY MIX, delivered									
033105350012	Includes local aggregate, sand, Portland ceme...									
033105350015	Excludes all additives and treatments									
033105350020	2000 psi	C.Y.				89.85			89.85	99.18
033105350100	2500 psi	C.Y.				92.31			92.31	101.15
033105350150	3000 psi	C.Y.				100.16			100.16	109.98
033105350200	3500 psi	C.Y.				97.71			97.71	108.02
033105350300	4000 psi	C.Y.				101.15			101.15	110.97
033105350350	4500 psi	C.Y.				104.09			104.09	113.91
033105350400	5000 psi	C.Y.				107.04			107.04	117.84
033105350411	6000 psi	C.Y.				121.77			121.77	133.55
033105350412	8000 psi	C.Y.				198.36			198.36	218.99
033105350413	10,000 psi	C.Y.				281.83			281.83	309.33
033105350414	12,000 psi	C.Y.				338.79			338.79	373.16
033105351000	For high early strength cement, add	C.Y.				10.00%				
033105351010	For structural lightweight with regular sand, add	C.Y.				25.00%				
033105351300	For winter concrete (hot water), add	C.Y.				4.17			4.17	4.60
033105351400	For hot weather concrete (ice), add	C.Y.				9.18			9.18	10.07

033105351410	For mid-range water reducer, add	C.Y.				4.06			4.06	4.46
033105351420	**For high-range water reducer/superplasticize...**	**C.Y.**				**6.24**			**6.24**	**6.82**
033105351430	**For retarder, add**	**C.Y.**				**2.66**			**2.66**	**2.93**
033105351440	For non-Chloride accelerator, add	C.Y.				4.74			4.74	5.20
033105700010	**PLACING CONCRETE**									
033105700020	**Includes labor and equipment to place, strike...**									
033105700050	Beams, elevated, small beams, pumped	C.Y.	C20	60.00	1.067		22.69	12.88	35.57	51.83
033105700100	With crane and bucket	C.Y.	C7	45.00	1.600		34.40	26.05	60.45	86.52
033105700200	Large beams, pumped	C.Y.	C20	90.00	0.711		15.01	8.57	23.58	34.67
033105700250	**With crane and bucket**	**C.Y.**	**C7**	**65.00**	**1.108**		**23.79**	**18.14**	**41.93**	**59.52**
033105700400	Columns, square or round, 12" thick, pumped	C.Y.	C20	60.00	1.067		22.69	12.88	35.57	51.83
033105700450	With crane and bucket	C.Y.	C7	40.00	1.800		38.80	29.56	68.36	96.99
033105700600	18" thick, pumped	C.Y.	C20	90.00	0.711		15.01	8.57	23.58	34.67
033105700650	With crane and bucket	C.Y.	C7	55.00	1.309		28.18	21.54	49.72	70.40
033105700800	24" thick, pumped	C.Y.	C20	92.00	0.696		15.01	8.42	23.43	34.11
033105700850	With crane and bucket	C.Y.	C7	70.00	1.029		22.33	16.83	39.16	55.51
033105701000	36" thick, pumped	C.Y.	C20	140.00	0.457		9.77	5.51	15.28	22.16
033105701050	With crane and bucket	C.Y.	C7	100.00	0.720		15.37	11.77	27.14	38.60
033105701400	Elevated slabs, less than 6" thick, pumped	C.Y.	C20	140.00	0.457		9.77	5.51	15.28	22.16
033105701450	**With crane and bucket**	**C.Y.**	**C7**	**95.00**	**0.758**		**16.47**	**12.42**	**28.89**	**40.76**
033105701500	6" to 10" thick, pumped	C.Y.	C20	160.00	0.400		8.53	4.83	13.36	19.51
033105701550	With crane and bucket	C.Y.	C7	110.00	0.655		14.09	10.72	24.81	35.24
033105701600	Slabs over 10" thick, pumped	C.Y.	C20	180.00	0.356		7.58	4.29	11.87	17.35
033105701650	**With crane and bucket**	**C.Y.**	**C7**	**130.00**	**0.554**		**11.93**	**9.07**	**21.00**	**29.73**
033105701900	Footings, continuous, shallow, direct chute	C.Y.	C6	120.00	0.400		8.31	0.46	8.77	14.33
033105701950	Pumped	C.Y.	C20	150.00	0.427		9.11	5.16	14.27	20.67
033105702000	With crane and bucket	C.Y.	C7	90.00	0.800		17.20	13.13	30.33	42.98
033500000000	**Concrete Finishing**									
033529000000	**Tooled Concrete Finishing**									
033529300010	**FINISHING FLOORS**									
033529300012	**Finishing requires that concrete first be placed...**									
033529300015	**Basic finishing for various unspecified flatwork**									
033529300100	Bull float only	S.F.	C10	4000.00	0.006		0.14		0.14	0.22
033529300125	Bull float & manual float	S.F.	C10	2000.00	0.012		0.27		0.27	0.45
033529300150	**Bull float, manual float, & broom finish, w/e...**	**S.F.**	**C10**	**1850.00**	**0.013**		**0.29**		**0.29**	**0.48**
033529300200	Bull float, manual float & manual steel trowel	S.F.	C10	1265.00	0.019		0.43		0.43	0.70
033923000000	**Membrane Concrete Curing**									
033923130010	**CHEMICAL COMPOUND MEMBRANE CONCRETE CURI...**									
033923130300	Sprayed membrane curing compound	C.S.F.	2 Clab	95.00	0.168	7.32	3.34		10.66	13.65
033923130700	Curing compound, solvent based, 400 S.F./gal., 55 g...	Gal.				19.64			19.64	21.60
033923130720	5 gallon lots	Gal.				26.51			26.51	28.97
033923130800	Curing compound, water based, 250 S.F./gal., 55 gall...	Gal.				19.44			19.44	21.60
033923130820	5 gallon lots	Gal.				22.59			22.59	25.04
033923230010	**SHEET MEMBRANE CONCRETE CURING**									
033923230200	**Curing blanket, burlap/poly, 2-ply**	**C.S.F.**	**2 Clab**	**70.00**	**0.229**	**16.79**	**4.54**		**21.33**	**26.07**
034113000000	**Precast concrete planks, hollow core**									
034113500010	**Precast slab planks**									
034113500020	Precast slab, roof/floor members, grouted, solid, 4" t...	S.F.	C11	2400.00	0.023	5.22	0.89	0.76	6.87	8.07
034113500050	Precast slab, roof/floor members, grouted, solid...	S.F.	C11	2800.00	0.020	5.60	0.76	0.65	7.01	8.16
034113500100	Precast slab, roof/floor members, grouted, hollow,...	S.F.	C11	3200.00	0.018	6.16	0.67	0.57	7.40	8.52
034113500150	**Precast slab, roof/floor members, groute...**	**S.F.**	**C11**	**3600.00**	**0.016**	**6.38**	**0.59**	**0.51**	**7.48**	**8.59**
034113500200	Precast slab, roof/floor members, grouted, hollo...	S.F.	C11	4000.00	0.014	6.81	0.54	0.46	7.81	8.89

Table AG. 3, Steel – Shear Studs, Joists, Metal Decking, Labor & Material
Source: RS. Means 2012: Commercial Cost Data

Line Number	Description	Unit	Crew	Daily Output	Labor Hours	Bare Material	Bare Labor	Bare Equipment	Bare Total	Total O&P
050523870010	**WELD STUDS**									
050523870020	1/4" diameter, 2-11/16" long	Ea.	E10	1120.00	0.021	0.35	0.83	0.34	1.52	2.35
050523870100	4-1/8" long	Ea.	E10	1080.00	0.022	0.33	0.86	0.36	1.55	2.39
050523870200	3/8" diameter, 4-1/8" long	Ea.	E10	1080.00	0.022	0.38	0.86	0.36	1.60	2.46
050523870300	6-1/8" long	Ea.	E10	1040.00	0.023	0.50	0.89	0.37	1.76	2.66
050523870400	1/2" diameter, 2-1/8" long	Ea.	E10	1040.00	0.023	0.36	0.89	0.37	1.62	2.52
050523870500	3-1/8" long	Ea.	E10	1025.00	0.023	0.44	0.90	0.38	1.72	2.62
050523870600	4-1/8" long	Ea.	E10	1010.00	0.024	0.51	0.92	0.38	1.81	2.75
050523870700	5-5/16" long	Ea.	E10	990.00	0.024	0.63	0.93	0.39	1.95	2.92
050523870800	6-1/8" long	Ea.	E10	975.00	0.025	0.68	0.95	0.40	2.03	3.02
050523870900	8-1/8" long	Ea.	E10	960.00	0.025	0.96	0.96	0.40	2.32	3.36
050523871000	**5/8" diameter, 2-11/16" long**	**Ea.**	**E10**	**1000.00**	**0.024**	**0.62**	**0.92**	**0.39**	**1.93**	**2.87**
050523871010	4-3/16" long	Ea.	E10	990.00	0.024	0.77	0.93	0.39	2.09	3.07
050523871100	6-9/16" long	Ea.	E10	975.00	0.025	1.00	0.95	0.40	2.35	3.37
050523871200	8-3/16" long	Ea.	E10	960.00	0.025	1.35	0.96	0.40	2.71	3.78
051223770010	**STRUCTURAL STEEL PROJECTS**									
051223770015	**Made from recycled materials**									
051223770020	**Shop fab'd for 100-ton, 1-2 story project, bolte...**									
051223770200	Apartments, nursing homes, etc., 1 to 2 stories	Ton	E5	10.30	6.990	2350.00	264.42	157.31	2771.73	3261.77
051223770300	3 to 6 stories	Ton	E5	10.10	7.129	2397.00	269.51	160.32	2826.83	3321.94
051223770400	7 to 15 stories	Ton	E6	14.20	8.451	2444.00	315.27	126.25	2885.52	3418.31
051223770500	Over 15 stories	Ton	E6	13.90	8.633	2538.00	325.44	129.26	2992.70	3554.07
051223770700	Offices, hospitals, etc., steel bearing, 1 to 2 stories	Ton	E5	10.30	6.990	2350.00	264.42	157.31	2771.73	3261.77
051223770800	**3 to 6 stories**	**Ton**	**E6**	**14.40**	**8.333**	**2397.00**	**315.27**	**124.25**	**2836.52**	**3364.22**
051223770900	7 to 15 stories	Ton	E6	14.20	8.451	2444.00	315.27	126.25	2885.52	3418.31
051223771000	Over 15 stories	Ton	E6	13.90	8.633	2538.00	325.44	129.26	2992.70	3554.07
051223771100	For multi-story masonry wall bearing construction,...	Ton					30.00%			
051223771300	Industrial bldgs., 1 story, beams & girders, steel bear...	Ton	E5	12.90	5.581	2350.00	211.54	125.25	2686.79	3125.00
052119100020	K series, 40-ton lots, horiz. bridging, spans to 30', sh...	Ton	E7	15.00	4.800	1525.50	184.37	116.23	1826.10	2173.46
052119100050	Average	Ton	E7	12.00	6.000	1723.25	229.69	145.29	2098.23	2490.82
052119100080	Maximum	Ton	E7	9.00	8.000	2062.25	306.94	193.39	2562.58	3083.63
052119100130	8K1, 5.1 lb./L.F.	L.F.	E7	1200.00	0.060	4.41	2.30	1.45	8.16	10.82
052119100140	10K1, 5.0 lb./L.F.	L.F.	E7	1200.00	0.060	4.33	2.30	1.45	8.08	10.73
052119100160	12K3, 5.7 lb./L.F.	L.F.	E7	1500.00	0.048	4.93	1.84	1.16	7.93	10.19
052119100180	14K3, 6.0 lb./L.F.	L.F.	E7	1500.00	0.048	5.19	1.84	1.16	8.19	10.48
052119100200	16K3, 6.3 lb./L.F.	L.F.	E7	1800.00	0.040	5.45	1.53	0.97	7.95	9.95
052119100220	16K6, 8.1 lb./L.F.	L.F.	E7	1800.00	0.040	7.01	1.53	0.97	9.51	11.64
052119100240	18K5, 7.7 lb./L.F.	L.F.	E7	2000.00	0.036	6.67	1.38	0.87	8.92	10.93
052119100260	18K9, 10.2 lb./L.F.	L.F.	E7	2000.00	0.036	8.81	1.38	0.87	11.06	13.30
052119100410	Span 30' to 50', minimum	Ton	E7	17.00	4.235	1497.25	162.74	102.20	1762.19	2087.95
052119100440	Average	Ton	E7	17.00	4.235	1695.00	162.74	102.20	1959.94	2285.70
052119100460	Maximum	Ton	E7	10.00	7.200	1808.00	276.04	174.35	2258.39	2694.18
052119100500	20K5, 8.2 lb./L.F.	L.F.	E7	2000.00	0.036	6.95	1.38	0.87	9.20	11.21
052119100520	20K9, 10.8 lb./L.F.	L.F.	E7	2000.00	0.036	9.15	1.38	0.87	11.40	13.64
052119100540	22K5, 8.8 lb./L.F.	L.F.	E7	2000.00	0.036	7.46	1.38	0.87	9.71	11.77
052119100560	22K9, 11.3 lb./L.F.	L.F.	E7	2000.00	0.036	9.61	1.38	0.87	11.86	14.09
052119100580	24K6, 9.7 lb./L.F.	L.F.	E7	2200.00	0.033	8.25	1.26	0.79	10.30	12.29
052119100600	**24K10, 13.1 lb./L.F.**	**L.F.**	**E7**	**2200.00**	**0.033**	**11.13**	**1.26**	**0.79**	**13.18**	**15.45**
052119100620	26K6, 10.6 lb./L.F.	L.F.	E7	2200.00	0.033	8.98	1.26	0.79	11.03	13.14
052119100640	26K10, 13.8 lb./L.F.	L.F.	E7	2200.00	0.033	11.70	1.26	0.79	13.75	16.13
053113500010	**FLOOR DECKING**									
053113500015	**Made from recycled materials**									
053113505100	Non-cellular composite decking, galvanized, 1-1/2" d...	S.F.	E4	3500.00	0.009	3.33	0.35	0.03	3.71	4.38
053113505120	18 gauge	S.F.	E4	3650.00	0.009	2.70	0.34	0.03	3.07	3.66
053113505140	**20 gauge**	**S.F.**	**E4**	**3800.00**	**0.008**	**2.15**	**0.33**	**0.03**	**2.51**	**3.03**
053113505200	**2" deep, 22 gauge**	**S.F.**	**E4**	**3860.00**	**0.008**	**1.86**	**0.32**	**0.03**	**2.21**	**2.71**
053113505300	20 gauge	S.F.	E4	3600.00	0.009	2.07	0.34	0.03	2.44	2.97
053113505400	18 gauge	S.F.	E4	3380.00	0.009	2.64	0.37	0.04	3.05	3.64
053113505500	16 gauge	S.F.	E4	3200.00	0.010	3.29	0.39	0.04	3.72	4.40
053113505700	3" deep, 22 gauge	S.F.	E4	3200.00	0.010	2.03	0.39	0.04	2.46	3.02

Table AG. 4a, Steel Beam and Girder

Cost Code	Item	Units	Quantity	Material Unit Cost [1][2]	Material Cost	Labor Unit Cost [1][2]	Daily Output per Crew [1][2]	Crew Size	Labor Cost	Equipment Unit Cost [1][2]	Equipment Cost	Total w/ Waste Factor [3]	Notes
	General Conditions												
015433602800	Rent Self Propelled 5 Ton Crane with Telescopic Boom	Each/Month	1	$15.73	$15.73	$232.46			$232.46	$696.39	$696.39	$944.58	
	Concrete												
031113050050	Concrete Forms 12" x 8'	ft^2	55	$23.99	$1,319.45							$1,385.42	
033105350150	Cast-In-Place 3000 psi	Yd^3	13.4	$100.16	$1,346.60							$1,413.93	
033105351400	Hot Weather Concrete (Ice Water)	Yd^3	13.4	$9.18	$123.42								
033105351420	Super-Plasticizer	Yd^3	13.4	$6.24	$83.89								
033105351430	Water Retarder	Yd^3	13.4	$2.66	$35.76								
033105701450	Crane and Bucket for Elevated Slabs ≤ 6" Thick	Yd^3	13.4			$16.47	95.00	7	$221.43	$12.42	$166.98	$388.41	
033529300150	Finish with Bull Float, Manual Float, and Broom Finish	ft^2	1097.8			$0.29	1850.00	10	$318.36			$318.36	
033923230200	Sheet Membrane for Concrete Curing	100 ft^2	11.0	$16.79	$184.32	$4.54	70.00	2	$49.84			$243.37	
	Metal												
050523871000	Welded Shear Studs 5/8" Diameter 2-11/16" Long	Each	94	$0.62	$58.28	$0.92	1000.00	10	$86.48	$0.39	$36.66	$184.33	
051223770800	Structural Steel [4]	Ton	6.1	$2,397.00	$14,712.79	$315.27	14.40	6	$1,935.13	$124.25	$762.65	$18,146.20	
053113505200	Composite Metal Decking 2" Deep 22 Gauage	ft^2	1097.8	$1.86	$2,041.87	$0.32	3860.00	4	$351.29	$0.03	$32.93	$2,528.18	
	Subtotals				$19,922.10				$3,194.98		$1,695.61	$25,552.78	
	Sales Tax (6%) [5]				$1,195.33								
	Overhead & Profit (10%) [6]				$2,111.74				$319.50		$169.56		
	Subtotal				$23,229.17				$3,514.48		$1,865.17		
	Contingency (5%)				$1,161.46				$175.72		$1,958.43		
	Adjustments				$0.00				$0.00		$0.00		
	Total				$24,390.63				$3,690.20		$3,823.60	$33,123.96	

[1] Values Referenced from *R.S. Means 2011*

[2] Open shop labor

[3] Waste Factor is assumed to be 5%, unless noted

[4] Wt. of all steel is based on stl. density of 490 lb/ft^3

[5] Sales tax is assumed to be 6%

[6] O+P is assumed to be 10%

Table AG. 4b, Composite Joist and Girder

Cost Code	Item	Units	Quantity	Material Unit Cost [1][2]	Material Cost	Labor Unit Cost [1][2]	Daily Output per Crew [1][2]	Crew Size	Labor Cost	Equipment Unit Cost [1][2]	Equipment Cost	Total w/ Waste Factor [3]	Notes
	General Conditions												
015433602800	Rent Self Propelled 5 Ton Crane with Telescopic Boom	Each/Month	1	$15.73	$15.73	$232.46			$232.46	$696.39	$696.39	$944.58	
	Concrete												
031113050050	Concrete Forms 12" x 8'	ft²	44.0	$23.99	$1,055.56							$1,108.34	
033105350150	Cast-In-Place 3000 psi	Yd³	9.2	$100.16	$925.78							$972.07	
033105351400	Hot Weather Concrete (Ice Water)	Yd³	9.2	$9.18	$84.85							$89.09	
033105351420	Super-Plasticizer	Yd³	9.2	$6.24	$57.68							$60.56	
033105351430	Water Retarder	Yd³	9.2	$2.66	$24.59							$25.82	
033105701450	Crane and Bucket for Elevated Slabs ≤ 6" Thick	Yd³	9.2			$16.47	95.00	7	$152.23	$12.42	$114.80	$267.03	
033529300150	Finish with Bull Float, Manual Float, and Broom Finish	ft²	1097.8			$0.29	1850.00	10	$318.36			$318.36	
033923230200	Sheet Membrane for Concrete Curing	100 ft²	11.0	$16.79	$184.32	$4.54	70.00	2	$49.84			$243.37	
	Metal												
050523871000	Welded Shear Studs 5/8" Diameter 2-11/16" Long	Each	216	$0.62	$133.92	$0.92	1000.00	10	$198.72	$0.39	$84.24	$423.58	
051223770800	Structural Steel [4]	Ton	1.02	$2,397.00	$2,452.13	$315.27	14.40	6	$14.73	$124.25	$127.11	$2,716.58	
052119100600	Joists 24K10	ft	132.0	$11.13	$1,469.16	$1.26	2200.00	7	$166.32	$0.79	$104.28	$1,813.22	
053113505140	Composite Metal Decking 1.5" Deep 20 Gauage	ft²	1097.8	$2.15	$2,360.22	$0.33	3800.00	4	$362.27	$0.03	$32.93	$2,873.43	
	Subtotals				$8,763.94				$1,494.93		$1,159.75	$11,856.02	
	Sales Tax (6%) [5]				$525.84								
	Overhead & Profit (10%) [6]				$928.98				$149.49		$115.97		
	Subtotal				$10,218.75				$1,644.42		$1,275.72		
	Contingency (5%)				$510.94				$82.22		$63.79		
	Adjustments				$0.00				$0.00		$0.00		
	Total				$10,729.69				$1,726.64		$1,339.51	$14,332.33	

[1] Values Referenced from *R.S. Means 2011*

[2] Open shop labor

[3] Waste Factor is assumed to be 5%, unless noted

[4] Wt. of all steel is based on stl. density of 490 lb/ft³

[5] Sales tax is assumed to be 6%

[6] O+P is assumed to be 10%

Table AG. 4c, Girder-Slab

Cost Code	Item	Units	Quantity	Material Unit Cost [1][2]	Material Cost	Labor Unit Cost [1][2]	Daily Output per Crew [1][2]	Crew Size	Labor Cost	Equipment Unit Cost [1][2]	Equipment Cost	Total w/ Waste Factor [3]	Notes
	General Conditions												
015433602800	Rent Self Propelled 5 Ton Crane with Telescopic Boom	Each/Month	1	$15.73	$15.73	$232.46			$232.46	$696.39	$696.39	$944.58	
	Concrete												
032110600150	Uncoated Reinforcement #7 → #8 [4]	Ton	0.27	$968.24	$257.68	$432.83	2.70	4	$115.19			$385.75	
032113100010	Galvanize Reinforcement Coating	Ton	0.27	$444.60	$118.32							$124.24	
033105350300	Cast-In-Place 4000 psi	Yd^3	7.6	$101.15	$764.94							$803.19	
033105351400	Hot Weather Concrete (Ice Water)	Yd^3	7.6	$9.18	$69.42							$72.89	
033105351420	Super-Plasticizer	Yd^3	7.6	$6.24	$47.19							$49.55	
033105351430	Water Retarder	Yd^3	7.6	$2.66	$20.12							$21.12	
033105700250	Crane and Bucket for Large Beams	Yd^3	7.6			$23.79	65.00	7	$179.91	$18.14	$137.18	$317.09	
033529300150	Finish with Bull Float, Manual Float, and Broom Finish	ft^2	1097.8			$0.29	1850.00	10	$318.36			$318.36	
033923230200	Sheet Membrane for Concrete Curing	100 ft^2	11.0	$16.79	$184.32	$4.54	70.00	2	$49.84			$243.37	
034113500150	Hollow Core Planks 10"	ft^2	1097.8	$6.38	$7,003.82	$0.59	3600.00	11	$647.69	$0.51	$559.87	$8,561.57	
	Metal												
051223770800	Structural Steel [4]	Ton	4.5	$2,397.00	$10,718.71	$315.27	14.40	6	$1,409.80	$124.25	$6,304.22	$18,968.66	
	Subtotals				$19,200.25				$2,953.24		$7,697.66	$30,810.38	
	Sales Tax (6%)				$1,152.01								
	Overhead & Profit (10%)				$2,035.23				$295.32		$769.77		
	Subtotal				$22,387.49				$3,248.57		$8,467.43		
	Contingency (5%)				$1,119.37				$162.43		$423.37		
	Adjustments				$0.00				$0.00		$0.00		
	Total				$23,506.86				$3,410.99		$8,890.80	$36,964.00	

[1] Values Referenced from *R.S. Means 2011*

[2] Open shop labor

[3] Waste Factor is assumed to be 5%, unless noted

[4] Wt. of all steel is based on stl. density of 490 lb/ft^3

[5] Sales tax is assumed to be 6%

[6] O+P is assumed to be 10%

Table AG. 4d, Two-Way Flat Slab

Cost Code	Item	Units	Quantity	Material Unit Cost [1][2]	Material Cost	Labor Unit Cost [1][2]	Daily Output per Crew [1][2]	Crew Size	Labor Cost	Equipment Unit Cost [1][2]	Equipment Cost	Total w/ Waste Factor [3]	Notes
	General Conditions												
015433602800	Rent Self Propelled 5 Ton Crane with Telescopic Boom	Each/Month	1	$15.73	$15.73	$232.46			$232.46	$696.39	$696.39	$944.58	
	Concrete												
031113050050	Concrete Forms 12" x 8'	ft^2	126	$23.99	$3,022.74							$3,173.88	
031113050150	Concrete Forms 3' x 4'	ft^2	1097.8	$16.90	$18,552.44							$19,480.07	
031505701100	Shoring ≤ 16' high	Each	18			$9.82	45.00	2	$176.76			$176.76	
031505701500	Reshoring	ft^2	1097.8	$0.52	$570.84	$0.31	1400.00	2	$340.31			$939.70	
032110600100	Uncoated Reinforcement #3 → #7 [4]	Ton	0.024	$968.24	$22.99	$731.33	1.60	4	$17.37			$41.51	
032110600150	Uncoated Reinforcement #7 → #8 [4]	Ton	6.4	$968.24	$6,215.53	$432.83	2.70	4	$17.33			$6,543.64	
032113100010	Galvanize Reinforcement Coating	Ton	6.443	$444.60	$2,864.63							$3,007.86	
033105350300	Cast-In-Place 4000 psi	Yd^3	40.3	$101.15	$4,075.03							$4,278.79	
033105351400	Hot Weather Concrete (Ice Water)	Yd^3	40.3	$9.18	$369.84							$388.33	
033105351420	Super-Plasticizer	Yd^3	40.3	$6.24	$251.39							$263.96	
033105351430	Water Retarder	Yd^3	40.3	$2.66	$107.16							$112.52	
033105701650	Crane and Bucket for Elevated Slabs ≥ 10" Thick	Yd^3	40.3			$11.93	130.00	7	$480.62	$9.07	$365.40	$846.03	
033529300150	Finish with Bull Float, Manual Float, and Broom Finish	ft^2	1097.8			$0.29	1850.00	10	$318.36			$318.36	
033923230200	Sheet Membrane for Concrete Curing	100 ft^2	11.0	$16.79	$184.32	$4.54	70.00	2	$49.84			$243.37	
	Metal												
	Subtotals				$36,252.65				$1,633.05		$1,061.79	$40,759.34	
	Sales Tax (6%) [5]				$2,175.16								
	Overhead & Profit (10%) [6]				$3,842.78				$163.31		$106.18		
	Subtotal				$42,270.59				$1,796.36		$1,167.97		
	Contingency (5%)				$2,113.53				$89.82		$58.40		
	Adjustments				$0.00				$0.00		$0.00		
	Total				$44,384.12				$1,886.17		$1,226.37	$49,715.87	

[1] Values Referenced from *R.S. Means 2011*

[2] Open shop labor

[3] Waste Factor is assumed to be 5%, unless noted

[4] Wt. of all steel is based on stl. density of 490 lb/ft^3

[5] Sales tax is assumed to be 6%

[6] O+P is assumed to be 10%