40 Gold Street Residential Building

New York, New York

TECHNICAL REPORT 2

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Executive Summary

Technical Report 2: Pro-Con Structural Study of Alternate Floor Systems involves an in depth inspection of the existing slab on composite metal deck system designed by Severud & Associates. Three alternate floor systems including a two way flat plate, a hollow core precast plank on steel, and a girder slab system were also examined. Research and preliminary design procedures were conducted and used to compare the alternate floor systems to each other and to the existing floor system. The floor system designs all included shear, flexure, and deflection checks. The comparison process was structured about a set of specific comparison criteria which included slab depth, total floor depth, slab weight, vibration, thermal control, acoustic control, cost/SF, formwork, constructability, lead time, material availability, versatility, architectural effects, fire rating, and additional fire protection.

The existing floor system is steel framing with 2"-18 gauge composite metal decking with 2 ½" light weight concrete topping. ¾" headed shear studs are spaced at 1'-0" on center or less for all beams and the steel reinforcing consists of # 4 top bars at 12" spacing. A typical 16' x 11' bay size was used to conduct design calculations. By referencing the Vulcraft Metal Decking Catalogue and using proper ASCE-07 design loads, the existing metal decking design was verified. While referencing the American Institute of Steel Construction (AISC) manual, composite beam action calculations were performed. The results verified the existing design.

After inspecting the parameters and design conditions associated with 40 Gold Street, it was determined a two way flat plate system could be designed using the Direct Design Method outlined in ACI 318-08. Calculations revealed an 8" slab was required amounting to a slab weight of 100 PSF. Research revealed many significant advantages which included improved floor to ceiling height and the ability for the slab to behave as exposed floor and ceiling surfaces. Other major advantages associated with the two way flat plate includes a 2 hour fire rating and above average vibration, thermal and acoustic control. Unfortunately, the large increase in weight is a significant disadvantage that cannot be ignored.

The Hollow Core Precast Plank on Steel floor system was designed using the 6th edition PCI handbook, the AISC manual, and ASCE-07 design loads. The concrete planks were designed assuming they span the long direction of a 16' x 18' bay (compatible with a 4' wide plank). A 6" concrete plank with 2" topping and 66-S designation was selected. The 66-S represents (6) 6/16" diameter reinforcement strands in straight position. Based on a 74 PSF plank weight, the steel framing was designed. The total floor depth amounted to 1'-8" and the slab weight is 74 PSF.

The girder slab system is a steel and precast hybrid floor structure. Since the bays must be dimensionally compatible with the 4'0" concrete plank width, the bay size used for preliminary design was modified to be a 16' x 18' bay. The Girder Slab 1.4 Design Guide and ASCE-07 design loads were used together to obtain a preliminary system design. The design yielded a system comprised of 4'0" x 8" concrete planks supported by the bottom flange of an open web dissymmetric beam DB 8x35. The total system depth is 10" and the slab weight is 60 PSF.

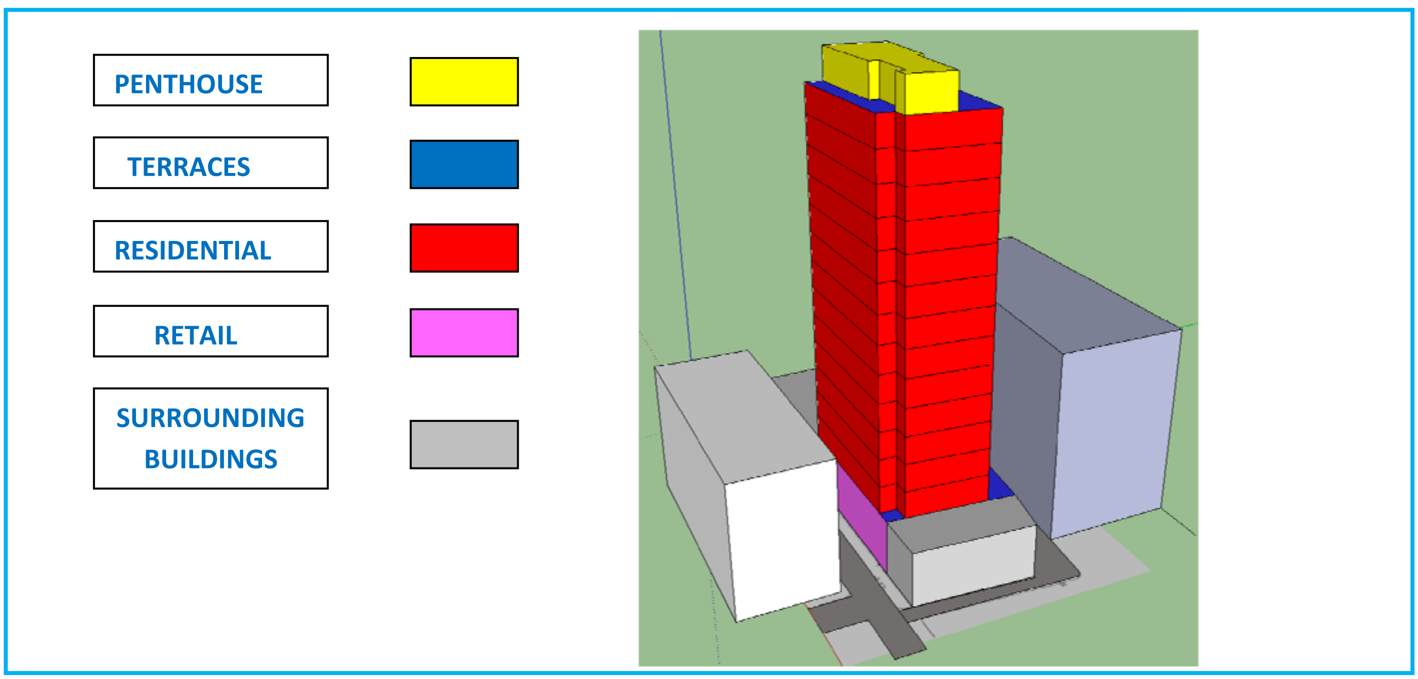
Of the three alternate floor systems, the comparison process revealed that the girder slab system is the most appropriate floor system to use for 40 Gold Street. Girder slab floor systems offer the combined advantages of structural steel and flat plate concrete. Of the three alternate systems, the girder slab system is unique in that it offers a relatively lightweight assembly that does not require a large total floor depth. With the inclusion of concrete in the floor system the vibration, thermal and acoustic control is very good and no additional fire protection is required except for any exposed steel members. No formwork is required and the construction process is not overly difficult. Overall, the girder slab system offers many advantages that greatly outweigh its disadvantages and it appears to be the most appropriate alternate floor system for 40 Gold Street.

Introduction

40 Gold Street is an impressive architectural package that offers retail and residential space in lower Manhattan, which is one of the fastest growing residential sections of New York City. The construction of 40 Gold Street began in March 2009 and will conclude in January 2010. The building replaces an old two story brick building and is nestled tightly between two existing structures, a narrow alley (Eden's Alley), and Gold Street. The constricted area presented special restrictions and challenges that greatly affected the final design and construction process.

Standing 175' above grade, the 40 Gold Street Building is a 14 story structure comprised of 5,900 square feet of retail space and 62,000 Square feet of residential space. The lowest two floors are primarily dedicated to retail space and serve as a podium on which a sleek 14 story residential tower rests. The lowest floor, referred to as the cellar, is below grade and functions as extra retail space as well as space for mechanical and electrical equipment. Retail spaces are appropriately located at the ground level and are highlighted with traditional floor to ceiling storefront windows to attract customers from the nearby streets and sidewalks. The storefront glazing is complemented very nicely with a pre fabricated assembly of dark stone cladding and a large bronze plaque that boldly recognizes the building as 40 Gold Street. In addition to retail space, there is a residential lobby and mailroom.

The residential tower is comprised of 12 residential floors. Identical in layout, floors 2-9 are comprised of 2 studio apartments and 3 2 bedroom apartments that all encompass the vertical circulation node located at the core of the tower. Two elevators and a stairwell serve as the buildings vertical circulation. Floors 10-13 are identical as well, but have 4 2-bedroom apartments and no studio apartments. At the top of the building, a level referred to as the penthouse provides the building's residents with two spacious recreational terraces sheltered by a gold painted metal trellis, a large recreational room enclosed by a window wall system, a kitchenette, a laundry room, and bathrooms.



F -1

Introduction Continued

The trapezoidal shape of the building closely reflects the shape of the site, which is to be expected when working with such a constricted space. The interior spaces are laid out in a very rectangular manner, and the exterior shell is also very rectangular. The residential tower boasts a sleek modern appearance with metal exterior cladding and gold toned trespa paneling.

Overall, the final design solution created by Architects Meltzer/Mandl and Structural Engineers Severud Associates makes the most of a small site, and is certainly playing a major role in the successful rebuilding of Lower Manhattan.

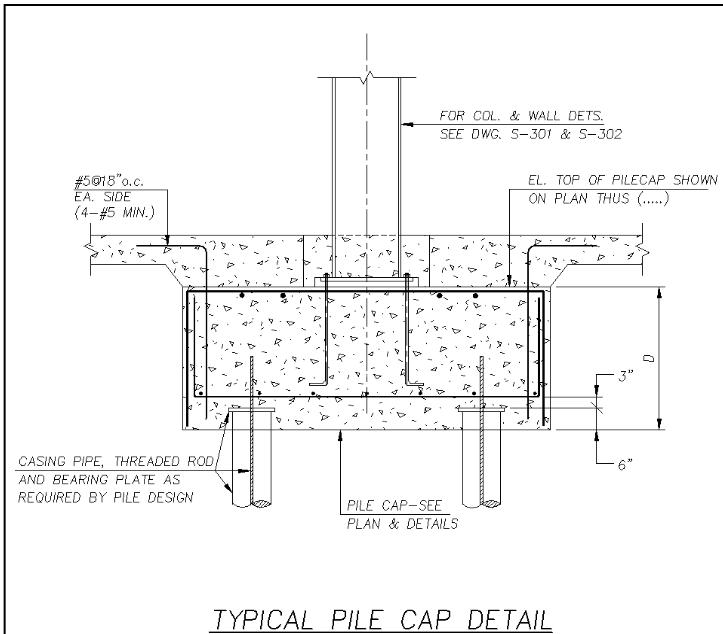
Structural System Overview

Foundations

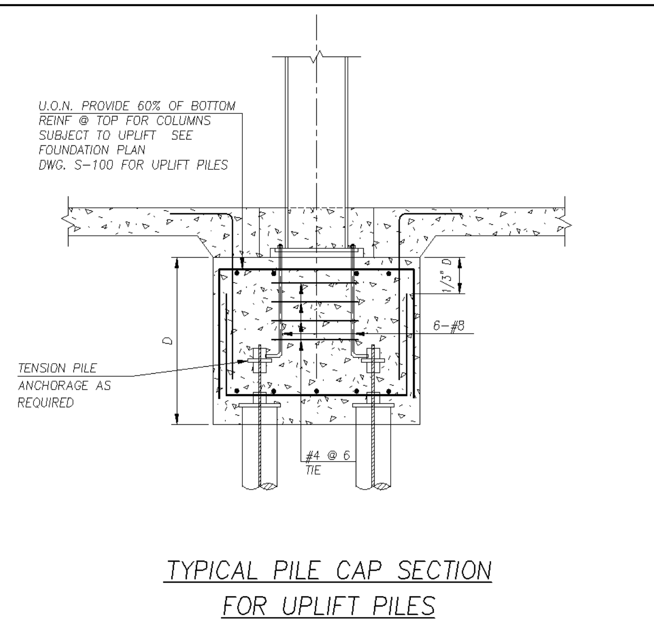
The site excavation and foundation work required a great deal of design work and creative planning compared to the average building project. As mentioned in the introduction, the site is very constricted with two existing structures against the property line, and two streets (Eden's Alley and Gold Street) are in close proximity. During excavation and foundation work, the adjacent streets required bracing and shoring for temporary and long term support. In addition, a major foundation design goal was to circumvent the need to underpin the adjacent existing structures. As a result, the depth of the various foundation components varies based on location relative to the surrounding structures and existing foundation systems.

The foundation employs a system of 101 strategically positioned micro piles. There are (88) 75 Ton compression capacity piles that are 35' long and (13) 35 Ton compression capacity piles that are 25' long. Various pile caps are used to distribute building loads to the piles: they generally range from 36"-39" in depth.

The cellar floor system is an 8" slab on grade with #5 bars @ 12" O.C. top/bottom running both directions. Resting on 6" of crushed stone, the slab on grade is attached to the pile caps via an assortment of connections. As seen in figure **S-1**, the typical pile cap is anchored to the column base plates by 6-#8 bars, and the pile caps are directly anchored to the floor slab by #5 @ 18" on each side of the column (minimum of 4 - #5 required per side). The pile caps subjected to uplift require tension pile anchorage as seen by figure **S-2**.



S-1



S-2

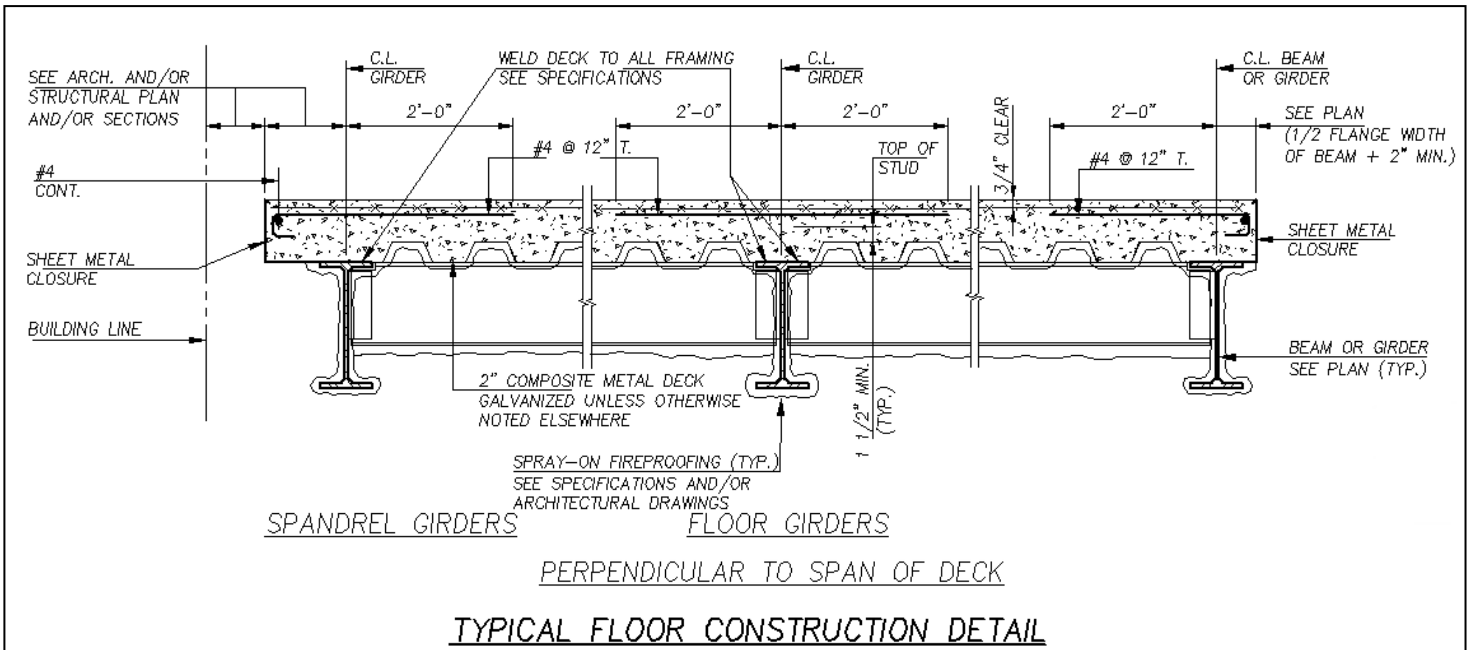
Floor System

The floor system employed in the 40 Gold Street building design is primarily slab on composite metal decking. Aside from the cellar floor system, the floor system is a 2" – 18 gage metal decks with 2 ½" light weight concrete topping as shown below in figure S-3. This one-way floor system operates to transfer gravity loads down to the supporting beams, girders, and columns.

The floor slab is reinforced with #4 @12" T., and 6x6 / W3 x W3 welded wire fabric is used with a ¾" clearance from top of slab. All concrete used has 4000 psi design strength. In several cases throughout the building, masonry partitions rest directly on the floor system. The areas where the partitions run parallel to the deck span, 2 - #6 bars are required to run on each side of the wall the full length of the wall to the first support beyond each end of the wall. Also, for the situation where the masonry partitions run perpendicular to the deck span, # 4 reinforcement bars run the full extent of the wall in each flute of the metal deck floor system.

The concrete is attached to the metal decking by equally spaced shear connectors. The shear studs extend a minimum of 1 ½" above the top of the metal decking. For the most part, the floor system throughout the building requires ¾" headed shear connectors @ 1' 0" or less.

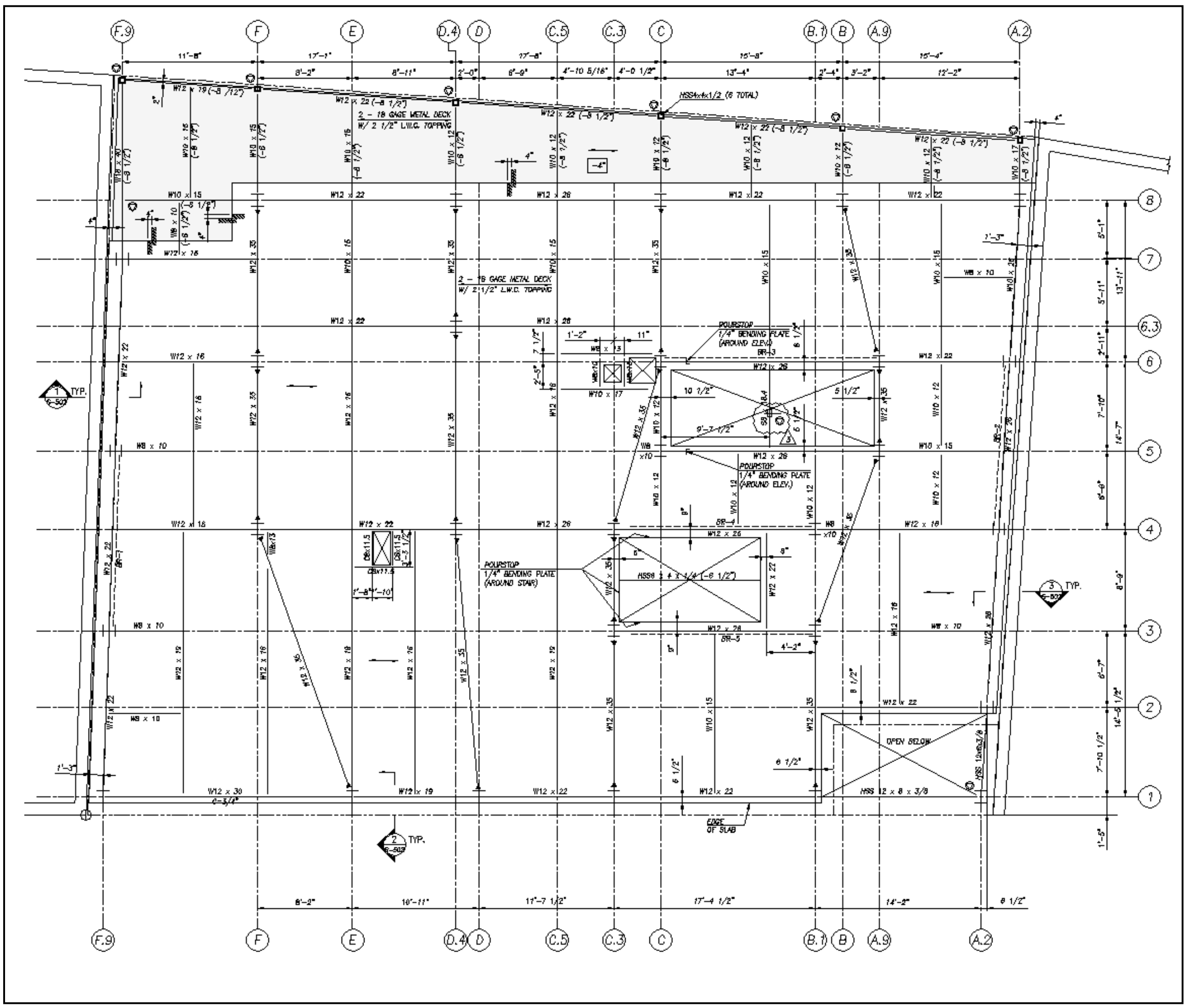
The cellar floor consists of a two-way 8" slab on grade with #5 @ 12" on center, top and bottom each way. The cellar slab rests on a 6" layer of crushed stone. More importantly, the cellar floor which is sub grade required a change in elevation as a consequence of closely surrounding structures and foundations. At the exterior sections of the cellar floor, the slab is raised up relative to the adjacent existing foundation. A slab depression of approximately 8'0" exists, allowing the center part of the cellar floor to rest much lower below grade.



S-3

Floor Framing

The floor system rests on uniform grid like layout of W-shape beams and girders. As seen below in figure S-4, there are only a few irregularities, in which beams do not run directly top to bottom across the plan. These beams are designed with moment connections, and serve as a part of lateral resisting moment frames. Figure S-4 represents the floor framing at level 2, and this same general layout is repeated throughout the rest of the building. Although the bay sizes vary, the average bay size is approximately 15’ 8” x 14’ 0”.



S-4

Level 2 Framing Layout

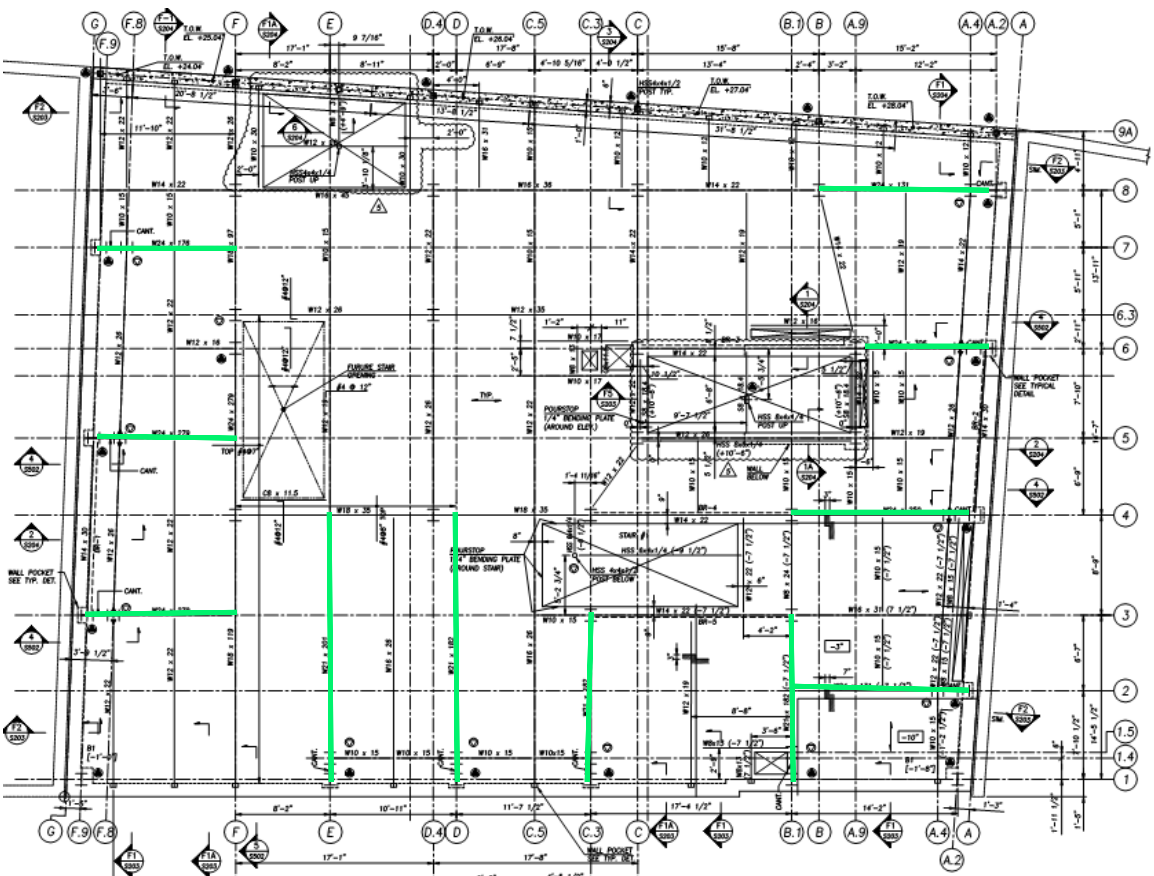
Gravity System

The gravity loads are resisted by a relatively rudimentary steel frame system. Figures **F-2** and **F-3** provide a close up look at the unfinished steel frame structure. The majority of the vertical structural elements are W-shapes aside from a few HSS4/4/3/8. The column sizes are nearly constant from level to level, but a slight reduction in size is observed near the top of the structure. The steel frame not only resists the gravity loads transferred from the floor system, but also supports the entire exterior envelope. The beams and girders are all W-shapes and are all treated with spray on fireproofing. The beams and girders range from W10's to W14's; however, at the second level several beams project 2 feet outward and behave as cantilevers to support the 13 stories above. Each cantilever is highlighted in figure S-5. These members are as large as W24x279's. The column splices are all located at 2' -6" above each finished floor. Almost all columns rise two floors.

F-2

Figures F-2 and F-3: 40 Gold Street under construction

F-3

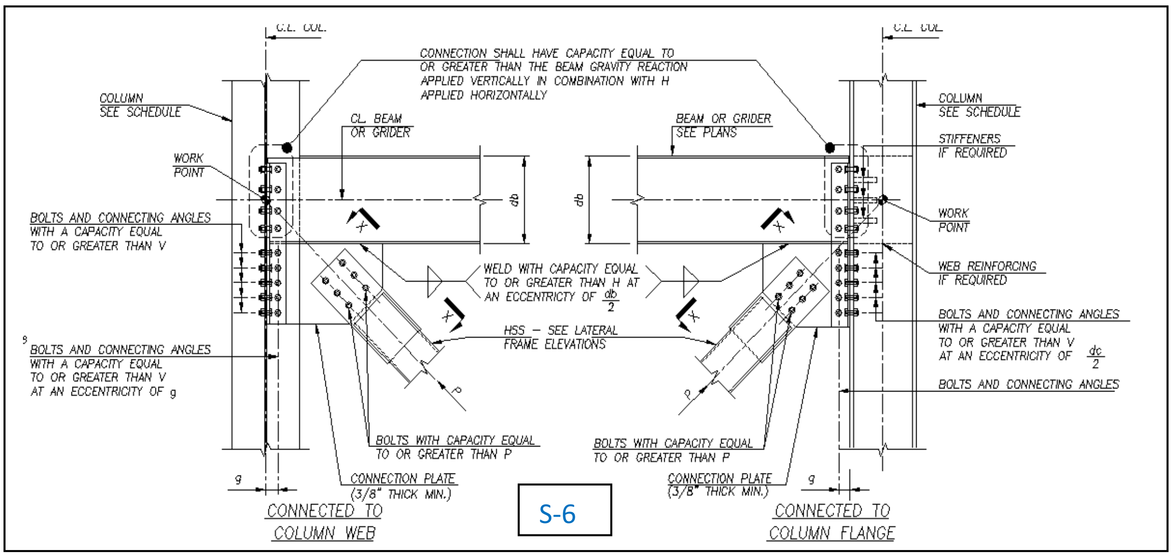


S-5

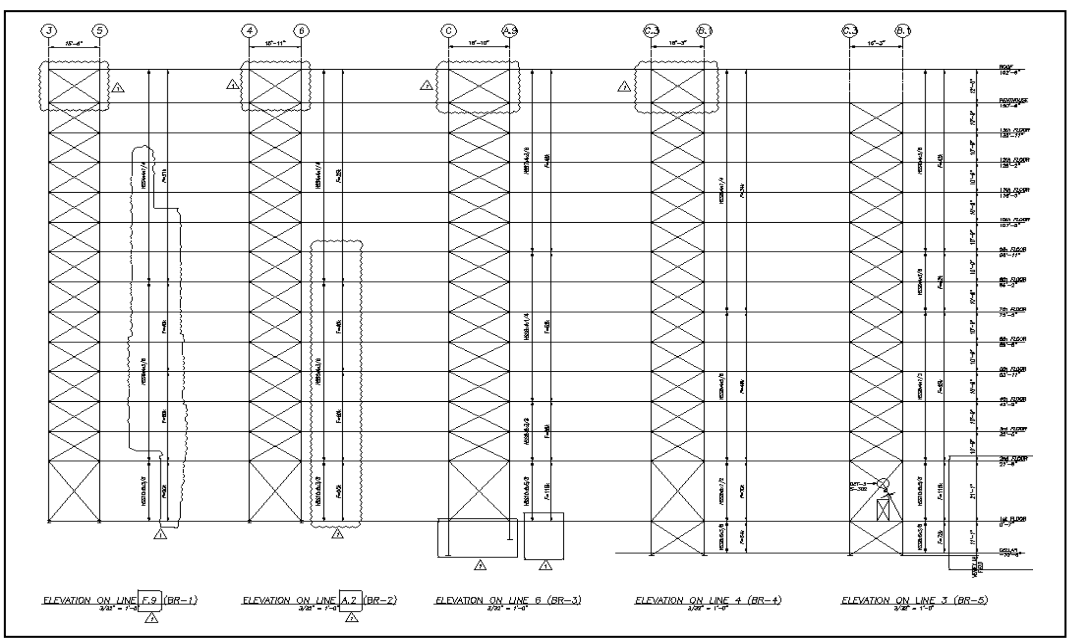
Highlighted Beams Cantilever outward 2 feet

Lateral System

The lateral system of 40 Gold Street consists of 5 braced frames and 4 moment frames. Figure S-8 shows the moment frames, which span east to west across the building, in red. The braced frames are shown in green. The moment frames are skewed since several of the building's footings are offset to avoid agitating the adjacent structures. The moment frame along column line A.9 is skewed due to architectural constraints. Figure S-6 illustrates the typical connections and structural members that form the braced frames, and figure S-7 provides an elevation view of the braced frames spanning from the foundation up to the roof level. The cross brace elements that form the braced frames are HSS shapes. The lateral system is laid out symmetrically. In addition, the building's shape and weight distribution is symmetrical. As a result, assuming the rigidity of each lateral resisting frame is not too variable; the center of rigidity is located near the center of mass. In consequence, the potential for torsion effect due to seismic load is lessened.



S-6

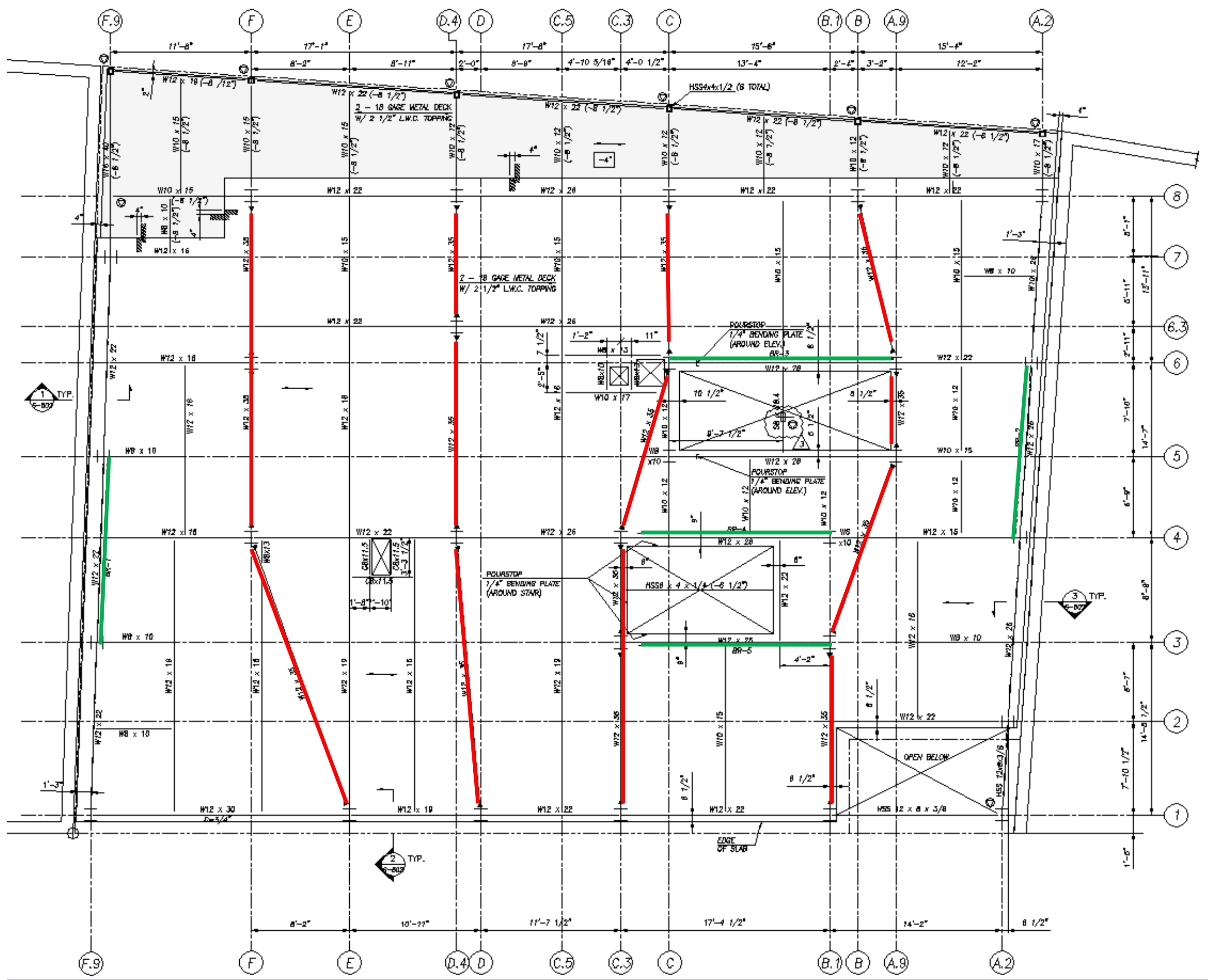


S-7

Figure S-6 details the typical brace frame.

Figure S-7 shows the five braced frames in elevation

LATERAL SYSTEM LAYOUT



S-8

Moment Frames

Braced Frames

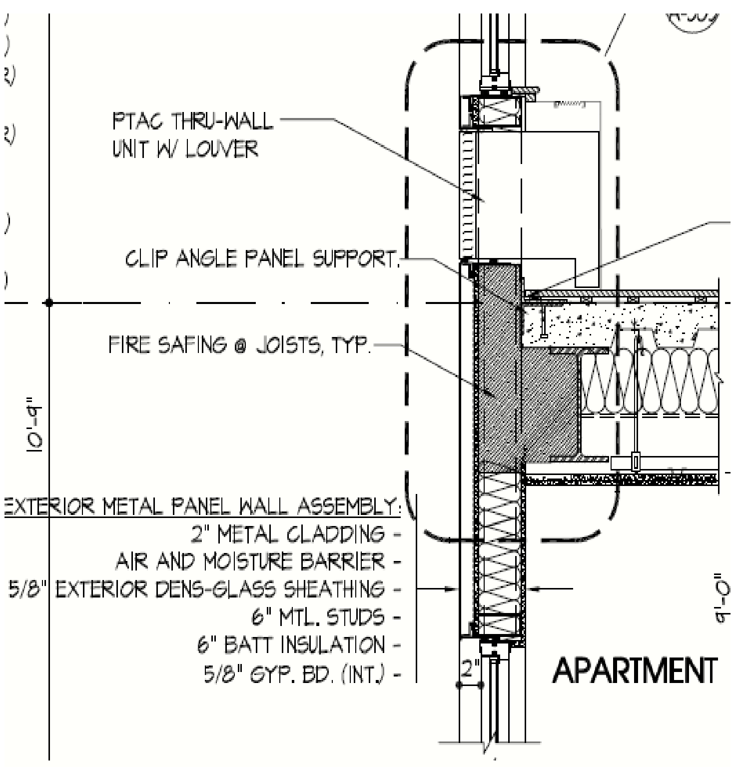
Sustainability

Although the overall design wasn't driven by sustainability, the 40 Gold Street building includes several green features throughout the design. The apartments are equipped with energy star appliances. In addition, the windows are assembled with low-emissive glass. The roofing materials are designed to prevent or minimize the heat island effect, and the building envelope is highly proficient for thermal and moisture protection. The exterior façade also has an 8" metal fin projecting out from above each of residential windows, which serves as a shade device.

Building Envelope

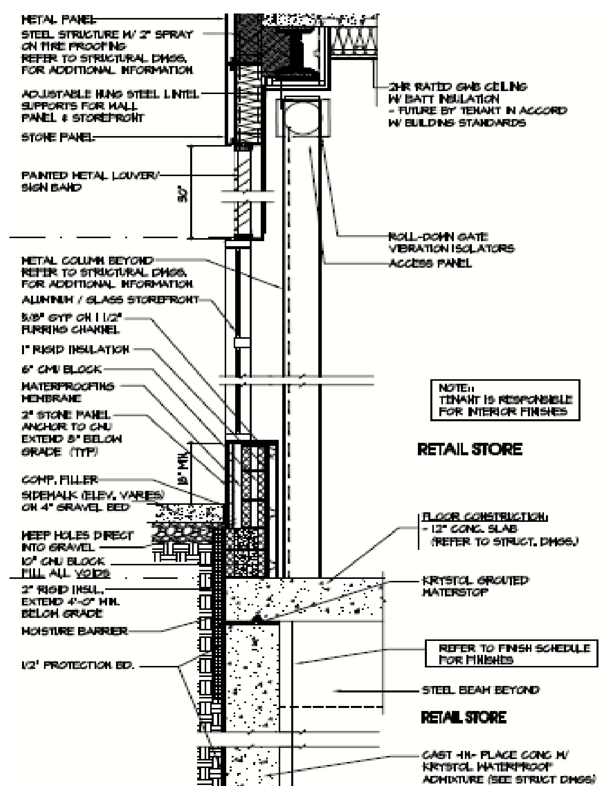
Floors 2-14 are enclosed by a basic non-bearing exterior metal panel wall assembly. The general composition of the wall shown in figure S-9 is 2" metal cladding (exterior), air and moisture barrier, 5/8" exterior dens-glass sheathing, 6" metal studs, 6" batting insulation, and 5/8" gypsum board (interior).

The sub grade spaces, also referred to as the cellar, are enclosed by a cast-in-place concrete wall with Krystol waterproof admixture. A detail of the enclosure can be seen in Figure S-10. Retail areas on the street level are enclosed by a large aluminum and glass storefront anchored to a basic CMU block wall assembly which consists of 2" stone panel (exterior), waterproofing membrane, 6" CMU block, 1" rigid insulation, 5/8" gypsum on 1 ½" furring channel (interior). The storefronts are also equipped with a roll-down gate for security purposes.



S-9

Typical Building Envelope for Residential Tower



S-10

Typical Building Envelope for Ground Floor

Roof System

40 Gold Street features an ordinary flat roof comprised primarily of W12x22 and W12x30 beams supporting the typical 2" – 18 gage metal decks with 2 ½" light weight concrete topping. Mechanical equipment is located on the roof and C channels are used for additional support. The roof terraces feature a slight different assembly. The terraces feature the Inverted Roof Membrane Assembly (IRMA) that works in conjunction with 2'x2' Concrete Pavers on pedestals. The insulation layer is an extruded polystyrene layer placed over the roofing membrane.

Codes, Design Standards:

- Original Design:

  Building Code
  New York City Building Code

  Lateral Loads
  Seismic: New York City Building Code

  Wind: American Society of Civil Engineers (ASCE), ASCE7-02

  Design Load and Standards
  New York City Building Codes

- Thesis Design:

  Building Code
  American Society of Civil Engineers (ASCE), ASCE7-05
  International Building Code (IBC) 2006

  Lateral Loads
  American Society of Civil Engineers (ASCE), ASCE7-05
  International Building Code (IBC) 2006

  Design Code References
  Steel Construction Manual 13th edition, American Institute of Steel Construction
  ACI 318-05, Building Code Requirements for Structural Concrete, American Concrete Institute

Required Loads

Building Dead Loads were provided by the Structural Engineering Firm Severud Associates.

DEAD LOADS		
Floor Level	**Building Component (Location)**	**Design Dead Load**
Ground Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip.	8 psf
	Partitions	12 psf
	Miscellaneous Dead Load (Lobby)	38 psf
	Miscellaneous Dead Load (Retail)	20 psf
2nd Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip.	3 psf
	Partitions (residential areas)	12 psf
	Miscellaneous Dead Load (Roof Terrace)	30 psf
3rd - 9th Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip.	3 psf
	Partitions (residential)	12 psf
10th - 13th Floor	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equipment	3 psf
	Partitions (residential)	12 psf
Penthouse	Slab	34 psf
	Steel	4 psf
	Ceiling / Mechanical Equip. (terrace)	3 psf
	Ceiling / Mechanical Equip. (Mechanical Area)	8 psf
	Ceiling / Mechanical Equip. (Recreational Area)	8 psf
	Miscellaneous Dead Load (Roof Terrace)	30 psf
	Miscellaneous Dead Load (Mechanical Area)	15 psf
Roof	Slab	25 psf
	Steel	4 psf
	Ceiling/Mechanical Equip.	8 psf
	Miscellaneous Dead Load (Roof Terrace)	10 psf
Bulkhead	Slab	34 psf
	Steel	4 psf
	Ceiling/Mechanical Equip.	8 psf
	Miscellaneous Dead Load (Roof)	25 psf

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Building live loads were determined by referencing ASCE 7. The actual design loads used by Severud Associates were verified.

Area	Actual Design Load	Thesis Design Load (ASCE 7-05)	Code/Table
Residential	40 psf	40 psf	ASCE7-05 Table 4-1
Retail	100 psf	100 psf	
Corridors	100 psf	100 psf	
Roof	60 psf	60 psf	
Terraces/Pedestrian	100 psf	100 psf	

T - 2

EXISTING SYSTEM

STEEL FRAMING WITH SLAB ON COMPOSITE METAL DECK

Material Properties and Important Design Dimensions:

Metal Decking
2” – 18 gauge composite deck
¾” headed shear connectors @ 1’ O/C
ASTM A611, Grade C
F_y = 40 ksi

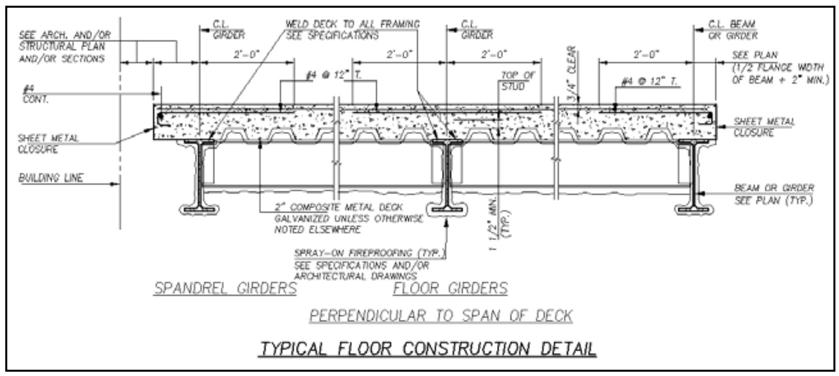
Steel Reinforcement
#4 @ 12” O.C.
ASTM A615, Grade 60
WWF ASTM A82 and A185

Steel Framing
ASTM A572, Grade 50
W-shapes

Concrete
f'_c = 4 ksi
f_y = 60 ksi
Light Weight

Description:

The existing structural floor system, illustrated in the adjacent figure S-11, consists of a 2” – 18 gauge metal deck with 2 ½ inch light weight concrete topping. The concrete is reinforced with #4 top bars @ 12”. Composite decking is used with ¾” headed shear connectors @ 1’-0” o/c or less for all beams.



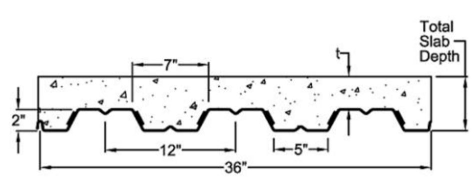
S - 11

Calculations were performed to analyze the existing design. For the analysis, the Vulcraft Metal Decking Catalogue was referenced to verify that the existing metal decking satisfies fire rating requirements and is within span limitations. Figures S-11, S-12 and S-13 represent the existing floor system and its properties. Based on the largest actual clear span of 9’-0”in the existing structure, the 199 PSF was established as the allowable superimposed live load. This verifies that 2” 18 gauge composite decking is sufficient to carry the actual building loads. Composite beam action calculations were also performed to check a typical interior beam. Supporting calculations can be found in Appendix A.

Figures S-12 and S-13 are provided by the Vulcraft Metal Deck Catalogue and show the existing floor system of 40 Gold Street.

2 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)



Interlocking side lap is not drawn to show actual detail.

STEEL SECTION PROPERTIES

Deck Type	Design Thickness in	Deck Weight psf	Section Properties I_p in⁴/ft	S_p in³/ft	I_n in⁴/ft	S_n in³/ft	V_a lbs/ft	F_y ksi
2VLI22	0.0295	1.62	0.324	0.263	0.321	0.266	1832	50
2VLI20	0.0358	1.97	0.409	0.341	0.406	0.346	2698	50
2VLI19	0.0418	2.30	0.492	0.420	0.489	0.426	3190	50
2VLI18	0.0474	2.61	0.559	0.495	0.558	0.504	3608	50
2VLI16	0.0598	3.29	0.704	0.653	0.704	0.653	3618	40

S - 12

(N=14.15) LIGHTWEIGHT CONCRETE (110 PCF)

S - 13

TOTAL SLAB DEPTH	DECK TYPE	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0
4.00 (t=2.00) 30 PSF	2VLI22	8'-1	10'-3	10'-7	238	209	186	167	152	120	108	98	90	82	75	69	64	59	55
	2VLI20	9'-6	11'-8	12'-1	268	235	209	187	169	153	140	129	101	92	84	78	72	66	61
	2VLI19	10'-10	13'-0	13'-2	297	260	230	206	185	168	153	141	130	121	93	86	79	73	68
	2VLI18	11'-7	13'-7	13'-7	324	285	253	227	205	187	171	158	146	136	127	119	92	86	80
	2VLI16	12'-3	14'-3	14'-4	377	330	292	261	235	214	195	179	165	153	143	133	118	98	91
4.50 (t=2.50) 35 PSF	2VLI22	7'-8	9'-10	10'-2	276	243	216	194	155	139	126	114	104	96	88	81	75	69	64
	2VLI20	9'-0	11'-3	11'-7	312	273	243	217	196	178	163	128	117	107	98	90	83	77	72
	2VLI19	10'-3	12'-5	12'-9	346	302	268	239	215	195	178	164	151	118	108	100	92	85	79
	2VLI18	11'-2	13'-1	13'-1	376	331	294	264	238	217	199	183	170	158	147	116	107	100	93
	2VLI16	11'-7	13'-8	13'-10	400	384	340	303	273	248	227	208	192	178	166	155	123	114	106

Advantages:
The reduction in building weight was the governing factor behind why Severud Associates and the Owner decided to construct 40 Gold Street with a slab on composite metal deck floor system. Also, the steel framing members as well as the slab on metal decking are versatile structural elements that can be used in irregular or non-simple span applications. Due to the nearby existing foundations, an irregular pattern of footings was designed to avoid disturbing the existing foundations. As a result, several non rectangular bays exist as well as non linear column lines. For the most part, the existing floor system is an economical design solution for residential mid-rise buildings. The construction process isn't too difficult and usually reduces construction time. Very little formwork is necessary. In fact, the composite decking functions both as formwork and a structural element.

Disadvantages:
Many negative aspects of the steel framing with slab on composite metal decking exist. However, almost of the following disadvantages are associated with non structural issues, which are significant but ultimately did not take precedence over the major structural design goals mentioned above. Architecturally, the existing floor system can't behave as an exposed ceiling. In fact, a drop down ceiling is often required. Additionally, the framing members hang down below the slab reducing the floor to ceiling depth and occupying valuable space normally dedicated for mechanical ductwork, piping, and electrical raceways. Also, additional fireproofing was needed in the actual design. Steel framing members received spray on fireproofing and the drop down ceiling assembly provided the necessary additional fire rating. Finally, the slab on metal decking is not known for superior acoustic and vibration control which is a significant design concern in any residential project. Finally, only a 2 ½" of concrete topping (4 ½" total) is used. This lower amount of mass often presents vibration issues.

Response:

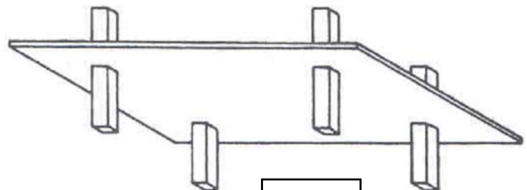
Almost all of the above disadvantages are associated with non structural issues, which are significant but ultimately did not take precedence over the major structural design goals.

ALTERNATE OPTION # 1 – TWO WAY FLATE PLATE

Material Properties and Important Design Dimensions:

Steel Reinforcement:
- Grade 60 (f_y= 60 ksi)
- # 5 Bars (.31 in^2)
- .75" Clear Cover

Concrete:
- 8" Normal Weight Concrete Slab
- 15" x 15" Columns
- No beams / No Drop Panels
- f'c = 4 ksi

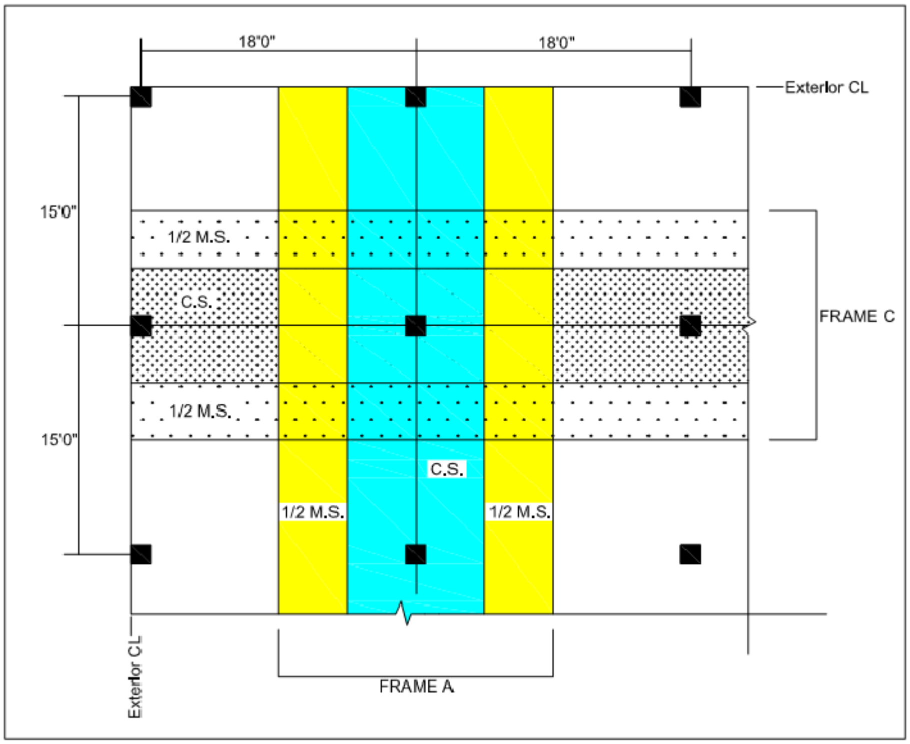


S - 14

Description:

In a two way flat plate system, no beams exist on the edge or between interior supports. It is essentially a plate of reinforced concrete that is supported only by columns. The term two way refers to the steel reinforcement that runs in both directions allowing load paths to travel in all four directions.

Design calculations were performed to obtain preliminary slab and reinforcement values for comparison purposes. By inspecting the existing structural layout and considering its possible alternate layouts, it was determined that a two way flat plate system met the ACI 13.6.1 Limitations (ACI 318-08) and could be appropriately designed using the Direct Design Method. Calculations revealed a two way flat plate floor system would require an 8" thick normal weight concrete slab with 4,000 psi compressive strength (f'_c). Supporting column dimensions were assumed to be 15" x 15". Factored load values used for design were conservative and included a $W_u = 1.2D + 1.6L$ = 224 PSF where W_D = 133 PSF (slab and superimposed loads) and W_L = 40 PSF (residential spaces). The total static moment was calculated using $M_o = 1/8(W_u)(l_2)(l_n^2)$ and then distributed longitudinally and laterally for Frames A and B shown in figure F-5. A 15' x 18' bay, representative of a typical bay size in 40 Gold Street, was used for the design. A summary of design moments and steel reinforcement requirements are recorded in tables T-3 and T-4 respectively. 4,000 psi concrete was appropriately chosen for the design because higher strength concrete will raise costs of concrete without any reduction in quantity. See Appendix A for supporting calculations and final design diagrams.



F - 5

Advantages:
The two way flat plate system is a preferred structural floor system for several significant reasons. Due to the absence of beams and drop panels, the floor depth is kept to a minimum (8" in this case). This characteristic gives owners and designers the ability to design shorter buildings to reduce wind loads and the option to add additional floors without exceeding height limits. The two way flat plate system is also attractive because it requires simple construction, simple formwork, and widely available materials. Additional advantages include flexibility for partition location, and no additional fire resistance is required. Perhaps the most significant advantage is the ability to utilize the floor system as both an architectural and structural element. The concrete slab is commonly left exposed and serves as finished floors and ceilings in residential spaces. This is an extremely important benefit since the 40 Gold Street building is a residential building. More importantly, the shape and texture of the concrete can be established using forms as opposed to being limited by the availability of standard manufactured members. Finally, due to the mass of the concrete in a two way flat plate system, the structure is more rigid reducing vibration issues.

Disadvantages:
The two way floor system is not intended for long spans or live loads the exceed 50 PSF. Fortunately, the 40 Gold Street building features small bay sizes and low live loads of 40 PSF (residential). Due to the use of an 8" Normal Weight Concrete slab, the overall weight of the building will exceed that of a steel frame building. 40 Gold Street is located on poor site conditions and is located very close to existing foundations. A major design goal of 40 Gold Street was reducing the overall building weight to eliminate settlement potential and to avoid disturbing the nearby existing foundations. Additional building weight will also increase seismic design loads. Finally, the two way flat plate requires formwork and curing time which can become costly. Specifically for the 40 Gold Street project, additional construction time is a serious problem. Due to a constricted site, construction facilities, vehicles, and cranes were located on Gold Street, and the City of New York issued a limited amount of time in which Gold Street was allowed to be closed.

Response: Currently the two way flat plate system appears to be a feasible floor system for the 40 Gold Street building. Based on initial research it appears the floor system entails far more advantages than disadvantages.

Summary of Distributed Total Static Design Moments (Mu = ft-k)

	Frame A		Frame C	
	Column Strip	Middle Strip	Column Strip	Middle Strip
EXTERIOR SPAN				
$M^-_{EXTERIOR}$	-15.3	0	-18.8	0
$M^+_{EXTERIOR}$	29.76	19.84	36.78	24.52
$M^-_{INTERIOR\ (ext\ span)}$	-50.025	-16.7	-61.88	-20.63
INTERIOR SPAN				
$M^-_{(interior\ span)}$	-46.5	-15.5	-57.45	-19.15
$M^+_{(interior\ span)}$	20.04	13.36	24.74	16.49

T - 3

Summary of Reinforcement Design (7" N.W. Concrete Slab)

	Frame A		Frame C	
	Column Strip	Middle Strip	Column Strip	Middle Strip
EXTERIOR SPAN				
$M^-_{EXTERIOR}$	(9) #5 Top Bars per 1/2 M.S.	(5) #5 Top Bars per 1/2 M.S.	(7) #5 Top Bars	(4) #5 Top Bars per 1/2 M.S.
$M^+_{EXTERIOR}$	(9) #5 Bottom Bars per 1/2 M.S.	(5) #5 Bottom Bars per 1/2 M.S.	(7) #5 Bottom Bars	(4) #5 Bottom Bars per 1/2 M.S.
$M^-_{INTERIOR\ (ext\ span)}$	(10) #5 Top Bars per 1/2 M.S.	(5) #5 Top Bars per 1/2 M.S.	(10) #5 Top Bars	(4) #5 Bottom Top Bars per 1/2 M.S.
INTERIOR SPAN				
$M^-_{(interior\ span)}$	(10) #5 Top Bars per 1/2 M.S.	(5) #5 Top Bars per 1/2 M.S.	(10) #5 Top Bars	(4) #5 Top Bars per 1/2 M.S.
$M^+_{(interior\ span)}$	(9) #5 Bottom Bars per 1/2 M.S.	(5) #5 Bottom Bars per 1/2 M.S.	(7) #5 Bottom Bars	(4) #5 Bottom Bars per 1/2 M.S.

Please Note, the final design requires a slab thickness of 8", please see Appendix B for supporting calculations

T - 4

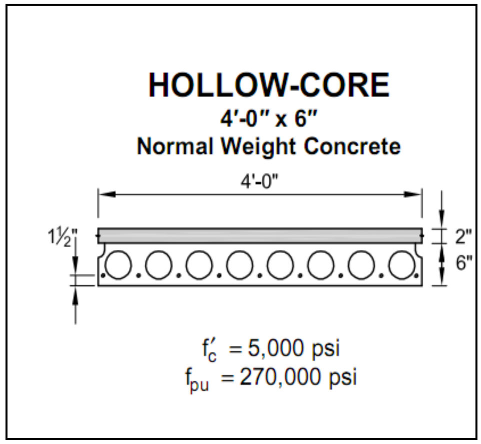
ALTERNATE OPTION # 2 – Hollow Core Planks on Steel Framing

Material Properties and Important Design Dimensions:

Hollow-Core Precast Planks
4'-0" x 6" w/ 2" topping
66-S: (6) 6/16"ø straight strands
f'c = 5 ksi

Steel Framing
W-shapes
ASTM A572, Grade 50

Description: Hollow-core precast plank structural floors are commonly used in low-midrise residential projects, and so research and preliminary design procedures were conducted to determine the suitability of this alternate floor system. Using design loads from ASCE-07 and the 6th edition PCI Handbook, a 4'-0" x 6" N.W.C. plank was selected. As shown by figures S-15 and S-16, the selected precast plank is 6" deep with 2" topping and a 66-S designation which specifies 6 strands of 6/16" diameter in straight position. The existing bay sizes are not compatible with the 4' wide plank dimension, so the bay sizes were modified to 16'x18' with planks spanning in the long direction (18'-0"). The 2" topping wasn't necessary for structural purposes; however, it was a conservative design decision and also helps to establish a level and more functional exposed floor surface. As one can see in figure S-16, the planks can safely resist a superimposed service load of 182 PSF which exceeds the conservative design load of 75 PSF (includes 40 PSF LL, 20 PSF SD, and 15 PSF for topped members). Using the AISC manual, the steel framing supporting the hollow-core planks was designed based on ASCE-07 loads and the 74 PSF precast plank self weight (from PCI). A W12x22 was determined as the least weight W-shape to resist the loads which amounts to a total floor depth of 1'-8".



S-15

4HC6 + 2

Table of safe superimposed service load (psf) and cambers (in.)

2 in. Normal Weight Topping

Strand Designation Code	Span, ft																		
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
66-S	470	396	335	285	244	210	182	158	136	113	93	75	59	46	34				
	0.2	0.2	0.2	0.2	0.2	02	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2				
	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2				
76-S		461	391	334	287	248	216	188	163	137	115	95	78	63	50	38	27		
		0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.1	0.1	–0.0	–0.1	–0.3		
		0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.5		
96-S			473	424	367	319	279	245	216	186	160	137	116	98	82	68	55	43	33
			0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.1	0.0	–0.1
			0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	–0.1	–0.3	–0.5	–0.7	–1.0	–1.4	–1.7
87-S			485	446	415	377	331	292	258	224	195	169	147	127	109	94	80	67	55
			0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.7	0.8	0.8	0.7	0.7	0.7	0.6	0.5	0.4	0.3
			0.5	0.5	0.5	0.6	0.6	0.6	0.5	0.5	0.4	0.4	0.2	0.1	–0.1	–0.3	–0.5	–0.8	–1.2
97-S			494	455	421	394	357	327	288	251	219	192	168	146	127	110	95	82	70
			0.5	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9	1.0	0.9	0.9	0.9	0.8	0.7	0.6
			0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.2	0.0	–0.2	–0.5	–0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

PCI Design Handbook/Sixth Edition
First Printing/CD-ROM Edition

2–31

Figure S-16 and S-15 were obtained from the 6th Edition PCI Handbook. The figures represent the selected design.

S-16

Advantages:
Precast hollow-core planks offers developers and owners the opportunity to significantly reduce construction time without any decline in quality. The precast planks can be shipped directly to the site for immediate erection and are considered an all weather construction material which lowers potential for construction delays. Construction speed is maximized and costs are reduced because the floor system doesn't require forming or curing of concrete. The hollow cells reduce the weight of the floor system increasing the span-to-depth ratio. More importantly, the voids help to conserve space by serving as areas for running electrical wiring, piping, heating and air conditioning ducts, and exhaust to the outside. The concrete planks are an optimal structural material for thermal control and are highly resistant to airborne and impact noises. Finally, the floor system is considered a durable, low maintenance, and fire resistant assembly.

Disadvantages:
Although there are a lot of advantages to using the precast hollow-core plank, several serious disadvantages must be addressed. The selected hollow core planks have a 74 PSF self weight which exceeds the existing 34 PSF slab (doesn't include decking). The additional weight inflates seismic design values and presents issues regarding the previously mentioned poor soil conditions. Perhaps the least attractive feature of the floor system is the overall floor depth of 1'-8" which includes a 2" topping, 6" plank, and 12" deep steel beam. Finally, the flexibility for laying out framing and bays is significantly reduced when using the precast planks. Not only must bays be dimensionally compatible with the 4'-0" plank width, but the precast planks are intended for simple span use only. Therefore, any irregular shapes or spans must be avoided during design which is not an easy task.

Response:
As one can see, despite all the above advantages, the floor system lacks practicality with the increase in both total floor depth and weight. Until further research is completed, the hollow-core planks on steel will not be ruled out as an alternate floor system; however, the competing floor systems appear much more suitable.

ALTERNATE OPTION # 3 – GIRDER SLAB

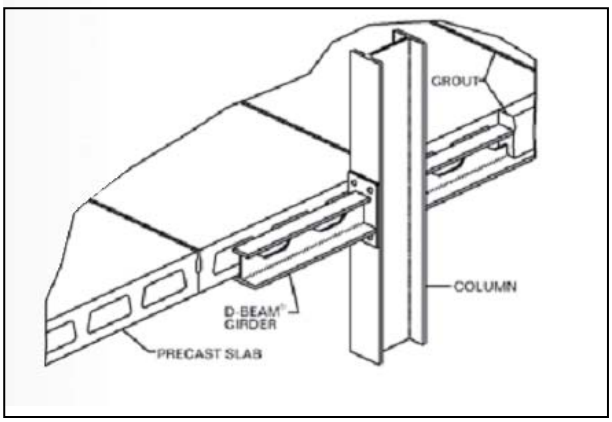
Material Properties and Important Design Dimensions:

Hollow-Core Precast Planks
- 4’-0”x 8” without topping
- 60 PSF weight
- $F_c' = 5$ ksi
- Grout = 4 ksi

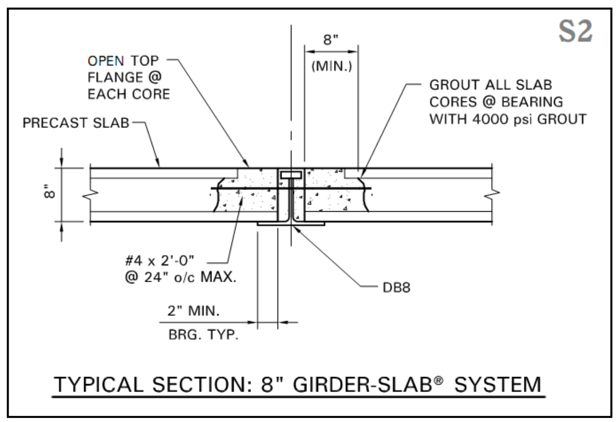
Steel Framing
- W-shapes
- Open Web Dissymmetric beam (D-beam)
- DB-8x35: $f_y = 50$ ksi and $Ix = 279$ in^4

Description:

This steel and precast hybrid floor structure is an innovative system that is intended for building projects exactly like 40 Gold Street, which has small rectangular bays and residential live loads. Girder slab floor systems seem tailor made for 40 Gold Street and will offer the combined advantages of structural steel and flat plate concrete. This girder slab system was designed for flexure, shear, and deflection based on the procedures found in the Girder Slab 1.4 Design Guide. A 16’ x 18’ bay size was used for design calculations, and the planks were designed assuming they spanned the long direction (18’-0”). The final design includes a 4’-0” by 8” hollow-core plank with no topping. The primary steel framing member is an interior girder designated as DB 8x35 and its section properties can be viewed in Appendix D. This girder, pictured in figures S-17 and S-18, is known as an open web dissymmetric beam and it is designed to support the concrete planks on its bottom flange.



S-17



S-18

Advantages:

Girder slab is a relatively new floor system, but is quickly establishing itself as one of the superior floor systems in the market today. Girder slab systems reduce construction time with the absence of formwork and the ease of precast plank erection. Subsequently, there is less on site labor and reduced onsite overhead cost. Architecturally, girder slab is a premier floor system because no steel framing hangs below the slab. As a result, the floor system improves the floor-to-ceiling height, and the slab can behave not only as a structural element but also as an exposed ceiling or floor surface. The designed slab depth is only 8”, and the total floor depth does not exceed the steel framing depth of 10”. Other positive aspects of the girder slab system are its above average performances in acoustical, thermal, and vibration control. Additionally, fire protection is provided by the concrete slab; however, spray on fire proofing is required for the steel framing members. The use of steel framing as opposed to concrete beams and columns allows the system to maintain a light weight assembly reducing overall building weight. Finally, girder slab systems are considered a superior system in seismic situations.

Disadvantages:
Girder slab systems are currently limited in product variation. To be more specific, only a few assembly depths are available which forces designers to use compatible spans and bay sizes to maximize the economy of the system. Although girder slab is considered a lightweight assembly, the 8" concrete planks still amounts to more overall weight than is observed in the existing floor system. Another disadvantage is the need to apply spray on fireproofing to the steel framing members.

Response:
Based on design calculations and thorough research, the girder slab system appears to be the best alternate floor system. The system doesn't interfere with critical MEP spaces, is architecturally friendly, has a small floor depth, and the composite action of the hybrid system magnifies its structural benefits by offering the benefits of both structural steel and flat plate concrete.

FLOOR SYSTEM COMPARISON

Floor System Comparison				
	Existing Floor System	Alternate Floor Systems		
Category of Comparison	Slab on Composite metal deck on Steel Framing	Two way flat plate	Hollow Core Slab on steel framing	Girder Slab
Slab Depth	4-1/2"	8"	8" (includes 2" topping)	8"
Total Floor Depth	1' 7-1/2" (includes steel framing)	8"	1'-8" (includes steel framing)	10"
Slab Weight	34 PSF	100 PSF	74 PSF	60 PSF
Vibration Issues	Yes	No	No	No
Cost / SF	$25.28	$18.47	$26.92	N/A
Formwork	No	Yes	No	No
Constructability	Medium	Medium	Easy	Easy-Medium
Lead Time	Long	Short	Long	Long
Material Availability	Good	Good	Good	Good
Versatility	Very Good	Okay	Bad	Bad
Architecture Effect	Negative Effects - less floor to ceiling height and limited to the standard manufactured shapes	Positive Effects - improved floor to ceiling height. Slab can behave as exposed floor and ceiling surface. No elements project down below slab	Neutral - Slab can behave as exposed floor and ceiling surfaces. However total floor depth is rather large with steel framing projecting down below slab. Voids provide spaces for ductwork, wiring, and piping.	Positive effects - improved floor to ceiling height. Slab can behave as exposed floor and ceiling surfaces.
Fire Rating	1.5 - 2 hour	2 hour	2 hour	2 hour
Additional Fire Protection	Spray On	None	Spray On	Spray On
Acoustical Control	Okay	Good	Great	Great
Thermal Control	Good	Good	Good	Good
Overall Suitability (scale: 1-10)	10	7	5	9

*Suitability Scale: 10 = Most Suitable and 1= Least Suitable

T-5

COST ANALYSIS

Cost per square foot values were estimated by referencing the 2009 RS Means manual. Each floor system was estimated using the preliminary design values obtained in the report. Since RS Means cost values are national averages, a location of 1.31 was used to represent 40 Gold Street, a commercial building located in New York City.

Existing Floor System – W-shape, composite deck, and slab

Material = $13.4 / SF
Installation = $5.9 / SF
Total = Location factor * (13.4 + 5.9) = 1.31 * (19.3) = **$25.28 / SF**

Alternate Floor System # 1 – Flat Plate

Material = $6.3 / SF
Installation = $7.8 / SF
Total = Location factor * (6.30 + 7.80) = 1.31*(14.10) = **$18.47 / SF**

Alternate Floor System # 2 – Hollow Core Precast Plank on Steel

Plank Material = $8.35 / SF
Plank Installation = $4.74 / SF
W-shape Material =$ 5.7 / SF
W-shape Installation = $1.76 / SF
Total = Location factor * (8.35 + 4.74 + 5.7 + 1.76) = 1.31 * (20.55) = **$26.92 / SF**

Alternate Floor System # 3 – Girder – Slab

No cost Analysis information is available. However based on engineering judgment, the girder-slab system should cost slightly more than the Hollow Core Precast plank on steel due to more intensive labor. However, no value will be used for comparison purposes since there are no supporting calculations.

CONCLUSION – Floor System Comparison

After inspecting 40 Gold Street's design parameters and limitations, the most suitable alternate floor systems were determined to be a two way flat plate, hollow core precast plank on steel, and a girder slab system. Research and preliminary design procedures were conducted and used to compare the alternate floor systems to each other and to the existing slab on composite metal decking. Based on the comparisons summarized in the above table **T-5**, girder slab is the most appropriate alternate floor system, and the two way flat plate is the next best floor system.

The hollow-core precast plank on steel framing presents several major issues that render the system unfit for use in the 40 Gold Street structure. The floor slab weight (74 PSF) is approximately 2.17 times heavier than the existing floor slab weight (34 PSF) and the total floor depth of 1'-8" is a major concern.

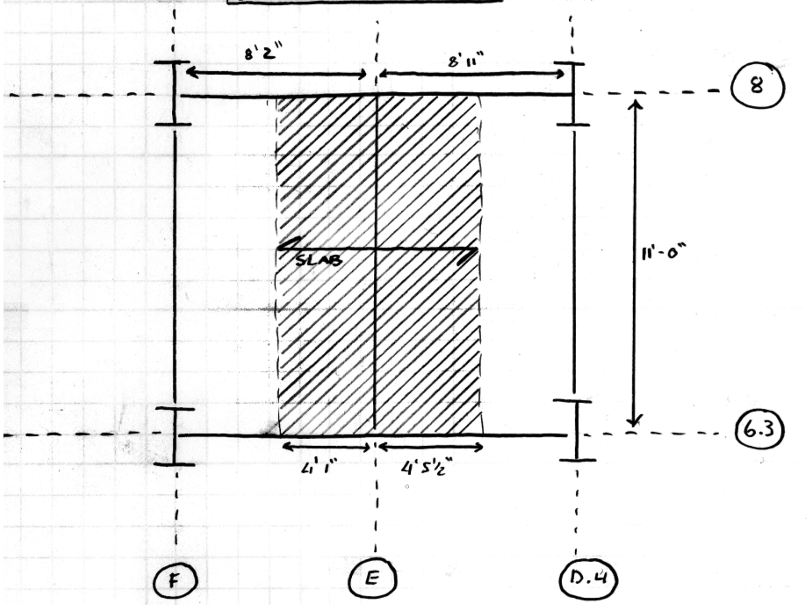
With a residential live load of only 40 PSF and small spans and rectangular bays, a two way flat plate would be a very economical and practical floor system. Not only is the total floor depth 8", but no structural elements hang down below the slab. As a result, critical MEP space is not occupied and the slab can remain exposed to serve as a ceiling or floor surface. In addition, the improved floor-to-ceiling height provides the designer with the option to either increase ceiling height without increasing building height, maintain the current story height but add extra floors without increasing building height, or maintain the same ceiling height and reduce the total building height. Several other advantages include good vibration, thermal, and acoustic control. Unfortunately, the 8" solid concrete slab weighs 100 PSF which is nearly 3 times greater than the existing slab weight. 40 Gold Street is situated on poor site conditions and near existing foundation systems that must not be disturbed. These conditions magnify the negative effects of additional building weight. Finally, the use of cast-in-place concrete requires formwork and increases construction time and difficulty.

Essentially all of the advantages associated with the two way flat plate system are also offered by the girder slab system. The 8" concrete planks provide extra floor mass enabling above average vibration, acoustic, and thermal control. Also, the girder slab system is considered a superior system in seismic situations. The concrete precast planks allow for easy all weather erection eliminating potential construction delays and costs. The total floor depth is only 10" and the slab weight is only 60 PSF. Out of the three alternate floor systems, the girder slab system is the only floor system to offer a relatively lightweight assembly that does not require a large floor depth. Unlike the two way flat plate, no formwork is necessary but spray on fireproofing is required for any exposed steel supporting members. The only major concern regards the girder slab system's lack of versatility. Although there is no supporting data, engineering judgment suggests the girder slab system is a relatively expensive floor system. The Structural Engineering firm Severud Associates emphasized that the final design required a few nonlinear column lines and irregular bays in order to avoid disturbing nearby existing foundations. As a result, it was determined slab on composite metal decking with steel framing offered the most versatility to appropriately accommodate for these irregularities.

Overall, the girder slab system offers the combined advantages of structural steel and flat plate construction which greatly outweigh the few disadvantages mentioned above. In conclusion, the girder slab system will be subject to further research and analysis to determine its suitability as a possible area of focus for the AE Senior Thesis Proposal.

APPENDIX A – Slab on Composite Metal Deck – Design Calculations (Please see pg. 18 for more details)

BEAM SPOT CHECK – TYPICAL INTERIOR BEAM
→ 2ND FLOOR FRAMING PLAN S-102
BEAM E8 – E6.3



STEP ONE: DEFINE APPLICABLE DESIGN LOADS
- 2ND FLOOR RESIDENTIAL : LIVE LOAD = 40 PSF
- DEAD LOAD - SLAB = 34 PSF (VULCRAFT CATALOGUE AND ACTUAL VALUE ON STRUCTURAL DWGS.)
- STEEL = 4 PSF
- CEILING / MECH. = 3 PSF
- PARTITIONS = 12 PSF
- FLOOR FINISHES

$\sum$ DEAD LOAD = 53 PSF

STEP TWO: $W_u = 1.2D + 1.6L = 1.2(53) + 1.6(40) = \underline{127.6 \text{ PSF}}$

$127.6 \text{ PSF} \times (4'\text{-}1'' + 4'5\tfrac{1}{2}'') = 1089.9 \text{ lb/ft}$

$M_u = \frac{W_u L^2}{8} = \frac{(1089.9/1000)(11')^2}{8} = \underline{16.485^{\text{k-1}}}$

STEP THREE: Assumption (MUST CHECK LATER)
~ Assume stress block is 1" THICK ($a = 1''$)

IF $a = 1'' \longrightarrow Y_2 = 4\tfrac{1}{2} - 1 = 3\tfrac{1}{2}$

*NOTE: a = DISTANCE FROM TOP OF SLAB (T.O.S.) TO CONCRETE FLANGE FORCE
Y_2 = DISTANCE FROM TOP OF STEEL BEAM TO FLANGE FORCE

SLAB = 4½" TOTAL DEPTH → 2½" TOPPING

TABLE 3-19 PNA 7 @ $Y_2 = 3.5"$ → TO CARRY 16.48 ft-k

W10 X 15 $\phi M_p = 90.3$ OKAY Page 3-188 AISC MANUAL "AVAILABLE STRENGTH IN FLEXURE"

$\Sigma Q_n = 55.1$

STEP FOUR:

$$b_{eff} = \text{minimum}\begin{bmatrix} \text{SPACING} = 8' \times 12" = 96" \\ \tfrac{1}{4}\text{ SPAN} = \tfrac{1}{4}(11' \times 12") = 33" \end{bmatrix} = \underline{33"}$$

STEP FIVE:

$$a = \frac{\text{Area}}{b_{eff}} \Rightarrow a = 16.21/33 = .491 < a = 1"$$

OKAY ✓

Original assumption a = 1" conservative

STEP FIVE

$$\text{Area} = \frac{\Sigma Q_n}{.85} = \frac{55.1}{.85(4)} = 16.21 \text{ in}^2$$

STEP SIX: POSSIBLE BEAM CHOICES - ECONOMICAL ANALYSIS

BEAM	ϕM_p	ϕM_n	Y_1	ΣQ_n	# STUDS
W10 X 15	60.0	90.3	7	55.1	26
W10 X 17	70.1	104	7	104	48
W12 X 14	65.3	97.2	7	51.9	24.
W12 X 16	75.4	111	7	58.9	27

$$\text{\# OF STUDS} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{55.1}{4.31} \times 2 = 12.78 \times 2 = 13 \times 2 = 26$$

TABLE 3-21 AISC → ¾" ø HEADED SHEAR CONNECTORS
L.W. CONCRETE
DECK ⊥ TO SPAN
$f'_c = 4$ ksi

} $Q_N = 4.31$

STEP SEVEN:

11' SPANS	EQUIVALENT WEIGHT OF STEEL
W10 X 15	(11' span)(15 lb/ft) + (10 lb/stud)(26) = 425
W10 X 17	(11 X 17) + (10 X 48) = 667
* W12 X 14	(11 X 14) + (10 X 24) = 394
W12 X 16	(11 X 16) + (10 X 27) = 446

W10X15 OR W12X14 WOULD BE LEGITIMATE

*NOTE THIS DESIGN DOES NOT TAKE INTO ACCOUNT LATERAL LOADS.

STEP EIGHT: MUST CONSIDER THE CONSTRUCTION LOADS (W12 X 14)

~ CARRIED JUST BY THE BARE STEEL BEAM

WEIGHT OF CONCRETE = 35 PSF (VULCRAFT CATALOGUE)
WEIGHT OF DECK = 2.61 PSF (vulcraft catalogue)

37.61 PSF × (8' TRIB width) = 300.9 lb/ft
+ 14 lb/ft (beam s.w.)
= 315 lb/ft

W_{LL} = 20 PSF

$W_u = 1.2(315) + 1.6(20 \times 8')$

$W_u = 634 = .634\ ^{k}/ft$

$M_u = \frac{w_u l^2}{8} = \frac{(.634)(11)^2}{8} = 9.589^{\text{ 1-k}} < \phi M_p = 65.3^{\text{ k-1}}$ OKAY ✓

STEP NINE: CHECK BEAM DEFLECTION DUE TO WEIGHT OF CONCRETE + S.W.

W12x14 $I_x = 88.6$

$\Delta = \frac{(5/384)(.315\ ^{k}/ft)(11^4)(12^3)}{29000(88.6)}$

"constant load deflection"

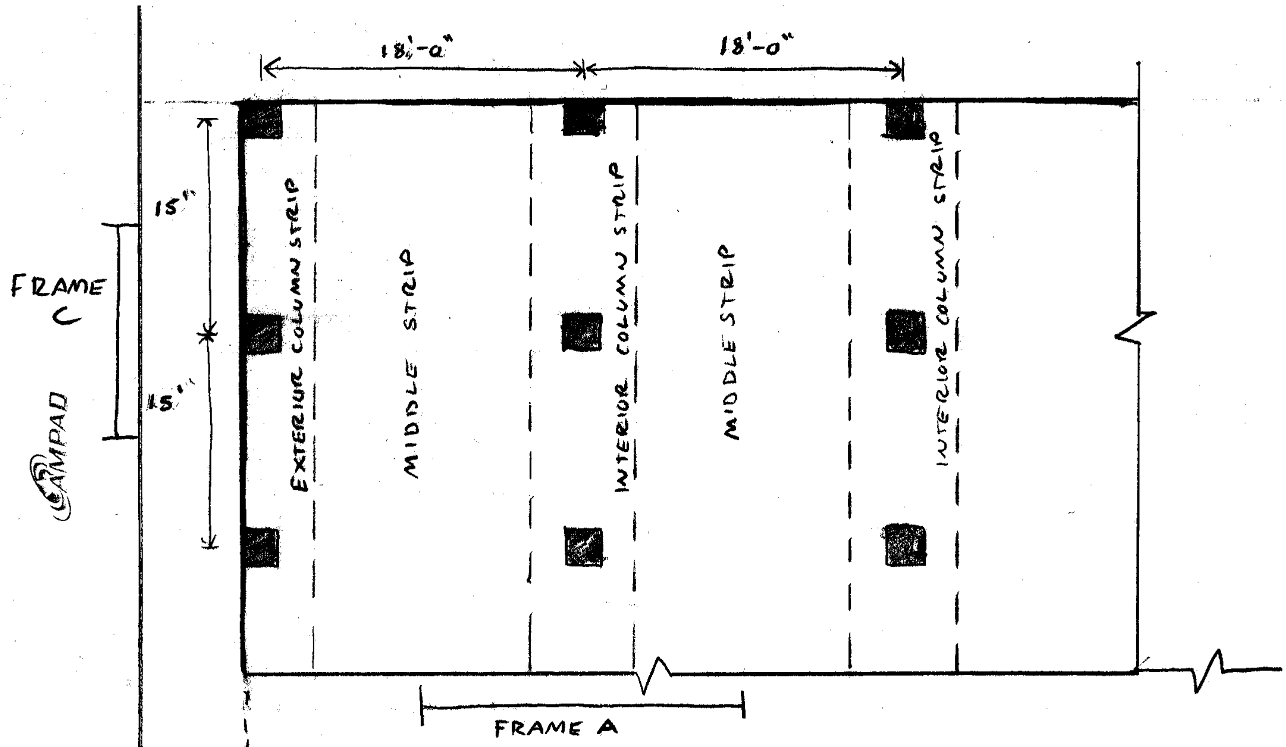
$\Delta = .040386" < l/360$ or 1"

$l/440$

< 1" OKAY ✓

W12 X 14

Appendix B - Two Way Flat Plate – Design Calculations (please see pg. 20 for more details)



DIRECT DESIGN METHOD

– CONDITIONS TO BE MET, TO USE DDM

1. AT LEAST 3 CONTINUOUS SPANS IN EACH DIRECTION <u>YES</u>

2. $\ell_2 / \ell_1 \leq 2$ $\quad 18'0'' / 15'0'' = 1.2 \leq 2$ <u>YES</u>

3. SUCCESSIVE SPAN LENGTHS C-C OF SUPPORTS IN EACH DIRECTION SHALL NOT DIFFER BY MORE THAN ONE THIRD THE LONGER SPAN.
$\ell_2 > 1 \rightarrow \ell_2 - \ell_1 < \frac{1}{3}\ell_2$
$18 - 15 = 3 < \frac{1}{3}(18) = 6'-0''$ <u>YES</u>

4. OFFSET OF COLUMN BY MAXIMUM OF 10% OF SPAN <u>YES</u>
OFFSET $< 0.1\ell_1$

5. DESIGN ONLY FOR GRAVITY LOADS
$W_L \leq 2W_D$ <u>YES</u>

6. NO BEAMS ✓ NO REQUIREMENT TO MEET <u>OKAY</u>

INTERIOR PANEL - FRAME A

STEP ONE: COMPUTE THE FACTORED LOADS,

W_L = 40 PSF (RESIDENTIAL)

9" N.W. CONCRETE → $(9/12 \cdot 150 \text{ PCF})$ = 113 PSF + 20 PSF = 133 (Partition, mech, E/P + Superimposed loads)

$W_U = 1.2D + 1.6L$

$W_U = 1.2(133) + 1.6(40) = 223.6 \text{ PSF} \approx 224 \text{ PSF}$

STEP TWO: TOTAL FACTORED STATIC MOMENT, Assume all columns 15 in × 15 in

$M_o = \frac{1}{8} W_u \ell_2 \ell_n^2$ where ℓ_n = clear span = 15'0" - 15/12 = 13.75'

ℓ_2 = 18'0"

$M_o = \frac{1}{8}(224/1000)(18)(13.75)^2 = 95.3^{\text{1-K}}$

STEP THREE: DISTRIBUTION OF M_o

FRAME A - EXTERIOR SPAN

- SLAB WITHOUT BEAMS BETWEEN INTERIOR SUPPORTS
- w/o edge beams
- TABLE → 13.6.3.3 ACI 318

$M_{ext}^- = 0.16\, M_o = -15.3^{\text{1-K}}$

$M_{ext}^+ = 0.52\, M_o = 49.6^{\text{1-K}}$

$M_{int}^- = 0.7\, M_o = -66.71^{\text{1-K}}$

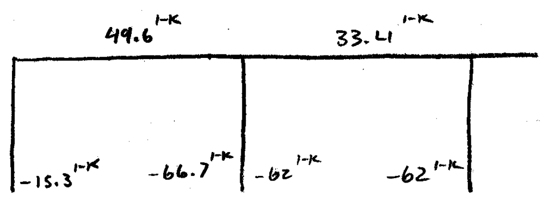
FRAME A - INTERIOR SPAN

$M^- = 0.65 M_o = -62^{\text{1-K}}$

$M^+ = 0.35 M_o = 33.41^{\text{1-K}}$

SUMMARY

FRAME A



STEP FOUR: LATERAL DISTRIBUTION OF MOMENTS

* NOTE → NO BEAMS BETWEEN COLUMNS

$\therefore I_b = 0 \rightarrow \alpha_f = 0 \rightarrow \alpha_f \ell_2/\ell_1 = 0$

$\alpha_m = \dfrac{\alpha_1 + \alpha_2 + \alpha_3 + \alpha_4}{4} = 0 \leq .2$ ✓

$I_b = k \dfrac{b_w h^3}{12} = 0$ (NO BEAMS)

$I_s = \dfrac{\ell_2 t^3}{12} \quad \dfrac{17(9)^3 \times 12}{12} = 12{,}393 \text{ in}^4$

$\therefore \alpha = \dfrac{(EI)_b}{(EI)_s} = 0$

C = TORSIONAL CONSTANT - BUT NO BEAMS ∴ C = 0

$C = \sum \left(1 - 0.63 \frac{x}{y}\right) \frac{x^3 y}{3}$

IF C = 0 → $\beta_t = \dfrac{C}{2 I_s} = 0$

CHECK MINIMUM SLAB THICKNESS

TABLE:
TWO WAY SLAB SYSTEM
FLAT PLATE.
MINIMUM THICKNESS = $h = \ell_n / 30$

$(15' \times 12'' - 15'') = 165''$

$165/30 = 5.5 = 8''$ THICK SLAB

% OF MOMENTS GOING TO COLUMN STRIP

$M^-_{EXT} = -15.3$

18/15

ℓ_2/ℓ_1 →	0.5	1.0	1.2	2.0
$\alpha_{f1}\ \ell_2/\ell_1 = 0 \rightarrow \beta_t = 0$ →	100	100	100	100

M^- @ ext. face of 1st interior support = -66.7

ℓ_2/ℓ_1 →	0.5	1.0	1.2	2.0
$\alpha_{f1}\ \ell_2/\ell_1 = 0$ →	75	75	75	75

M^+

ℓ_2/ℓ_1 →	0.5	1.0	1.2	2.0
$\alpha_{f1}\ \ell_2/\ell_1 = 0$	60	60	60	60

FACTORED MOMENTS IN MIDDLE STRIPS

↳ the proportion of negative and positive factored moments not resisted by column strips shall be proportionately assigned to CORRESPONDING HALF MIDDLE STRIPS

NOTE: BEAMS BETWEEN SUPPORTS ARE TO RESIST 85% OF COLUMN STRIP MOMENT... HOWEVER IN FLAT PLATE SYSTEMS, THERE ARE NO BEAMS

LATERAL DISTRIBUTION

M^{-}_{EXT}

−15.3^1-k
→ 100% TO colum strip = −15.3^1-k
→ NO BEAMS IN C.S. = 0^1-k
→ 100% TO COLUMN STRIP SLAB = −15.3^1-k
→ 0% TO MIDDLE STRIP = 0^1-k

M^{-} @ EXTERIOR FACE OF 1ST INTERIOR SUPPORT

−66.7^1-k
→ 75% TO C.S. = −50.025^1-k
→ NO BEAMS IN C.S. = 0^1-k
→ 100% TO C.S. SLAB = −50.025^1-k
→ 25% TO M.S. slab = −16.675^1-k

M^{+} EXTERIOR SPAN

49.6^1-k
→ 60% TO C.S. = 29.76^1-k
→ NO BEAMS IN C.S. = 0^1-k
→ 100% TO C.S. SLAB = 29.76^1-k
→ 40% TO M.S. slab = 19.84^1-k

M^{+} INTERIOR SPAN

33.4^1-k
→ 60% TO C.S. = 20.04^1-k
→ NO BEAMS IN C.S. = 0^1-k
→ 100% TO C.S. slab = 20.04^1-k
→ 40% TO M.S. SLAB = 13.36^1-k

M^{-} @ INT SUPPORTS

−62^1-k
→ 75% TO C.S = −46.5^1-k
→ NO BEAMS IN C.S = 0^1-k
→ 100% TO C.S. slab = −46.5^1-k
→ 25% TO M.S. slab = −15.5

Reinforcement Design – Column Strip Slab

Item Description	Exterior Span M^-_{EXT}	Exterior Span M^+	Exterior Span M^-_{INT}	Interior Span M^-	Interior Span M^+
① Mu (ft-k)	-15.3	29.76	-50.025	-46.5	20.04
② CS width b (slab only)	108"	108"	108"	108"	108"
③ Effective depth, d	4.31"	4.31"	4.31"	4.31"	4.31"
④ Mu·12/b	-1.7	+3.31	-5.56	-5.17	2.23
⑤ Mn = Mu/ø=.9	-17	33.1	-55.58	-51.67	+22.3
⑥ R = Mn / b·d²	101.7	198	332	309	133.4
⑦ ρrequired	.0033	.0035	.006	.006	.0033
*Note: Table A-3 from textbook (see below)					
⑧ As,req'd = ρbd	1.54	1.63	2.79	2.79	1.54
⑨ As,min = .0026t	1.296	1.296	1.296	1.296	1.296
⑩ Larger of 8,9 / (Bar Area) *Note: bar area for a #5 bar = .31 in²	4.97 ↳ 5 bars	5.26 ↳ 6 bars	9.01 ↳ 10 bars	9.01 ↳ 10 bars	4.97 ↳ 5 bars
⑪ Nmin = width of strip / 2t = 108/2(6) = 9	9	9	9	9	9
	Use 9 #5 top bars	Use 9 #5 bot bars	Use 10 #5 top bars	Use 10 #5 top bars	Use 9 #5 bot bars

CS width $b = 18'/2 = 9' - \text{beam} = 9' - 0 = 9' \times 12'' = 108$

Effective depth, D
* clear cover = .75"
* bar diameter: #5 → 5/8 inches = .625"

$d_{short} = t_{slab} - \text{Clear Cover} - \frac{1}{2}\text{ bar diameter} = 6 - .75 - (.625/2) = \underline{4.94''}$

$d_{long} = t_{slab} - \text{Clear Cover} - \frac{1}{2}\text{ bar diameter} - \text{bar diameter} = \underline{4.31''}$

Sample R calculation: $R = \frac{(-17^{1\text{-}k})}{(108'')(4.31^2)} \times 1000 \times 12$

$R = \underline{101.7}$

ρ determination – Reinf. Conc. Design Mechanics + Design 5th Edition James K Wight
Table – A-3 = Grade 60 Reinf ⟶ f'c = 4000 = conc. comp. strength

DESIGN OF FRAME A - MIDDLE STRIP SLAB REINFORCEMENT

ITEM	DESCRIPTION	EXTERIOR SPAN M^-_{EXT}	M^+	M^-_{INT}	INTERIOR SPAN M^-	M^+
1	Mu (ft-k)	0	19.84	-16.7	-15.5	13.36
2	M.S. width b	108"	108"	108"	108"	108"
3	EFFECTIVE DEPTH, d	4.31"	4.31"	4.31"	4.31"	4.31"
4	Mu·12/b	0	2.2	-1.85	-1.72	1.48
5	Mn = Mu/ø = .9	0	2.44	2.1	-1.91	1.64
6	R = Mn/bd²	0	141.6	12.56	11.4	79.6
7	ρrequired *NOTE FROM REINFORCED CONCRETE DESIGN MECHANICS AND DESIGN TEXTBOOK: TABLE A-3 f'c = 4000, grade 60 steel	0	.0033	.0033	.0033	.0033
8	As, req'd = ρbd	0	1.54	1.54	1.54	1.54
9	As, min = .002 bt	1.296	1.296	1.296	1.296	1.296
10	Larger of 8, 9 / BAR Area *NOTE: #5 bar = .31 in²	4.18	4.97	4.97	4.97	4.97
11	$N_{min} = \frac{\text{Strip width}}{2t}$ =	9	9	9	9	9
12	N = larger 10, 11	9 #5 TOP BARS	9 #5 BOT BARS	9 #5 TOP BARS	9 #5 TOP BARS	9 #5 BOT BARS

EACH ½ Middle strip => 9 BARS/2 #5
GET ½ OF MOMENT = 4.5 → 5 BARS →
FOR EACH ½ M.S.

FRAME C – FLAT PLATE DESIGN

$M_o = \frac{1}{8} W_u l_2 l_n^2$ where $l_n = 18'0'' - 15/12 = 16.75$

$M_o = 1/8 (224/1000)(15)(18 - 15/12)^2 = 117.8^{\text{1-K}}$

FRAME C – EXTERIOR SPAN

- SLAB W/O BEAMS BETWEEN INT. SUPPORTS
- W/O EDGE BEAMS
- TABLE → 13.6.3.3 ACI 318

$M_{EXT}^- = 0.16 M_o = -18.8$

$M_{EXT}^+ = 0.52 M_o = 61.3$

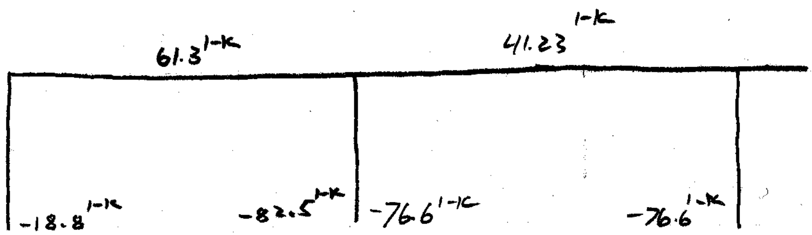
$M_{INT}^- = 0.7 M_o = -82.5$

FRAME C – INTERIOR SPAN

$M^- = 0.65 M_o = -76.6$

$M^+ = 0.35 M_o = 41.23$

SUMMARY:



LATERAL DISTRIBUTION OF MOMENTS

NO BEAMS → $I_b = 0$, $\alpha_f = 0$ → $\alpha_f l_2 / l_1 = 0$

$\therefore \alpha_m = \frac{\alpha_1 + \alpha_2 + \alpha_3 + \alpha_4}{4} = \frac{0}{4} = 0$

$I_s = \frac{l_2 t^3}{12} = \frac{15(7)^3(12)}{12} = 5415$

C = TORSIONAL CONSTANT – BUT NO BEAMS ∴ C = 0

$$C = \sum\left(1 - 0.63\frac{x}{y}\right)\left(\frac{x^3 y}{3}\right) = 0$$

$\beta_t = \frac{C}{2 I_s} = \frac{0}{2(5415)} = 0$

NOW GO TO TABLES TO ACQUIRE % TO DISTRIBUTE LONGITUDINAL MOMENTS LATERALLY!

CHECK MINIMUM SLAB THICKNESS

FOR TWO WAY FLAT PLATE – $h = \ell_n / 30$ (15x15 column)

$\ell_n = (18' \times 12") - 15" = 201"$

$h = 201"/30 = 6.7" \rightarrow$ SLAB MUST BE 7"

ORIGINALLY DESIGNED AS A 6" slab, **7" Slab** IS REQUIRED BASED ON THE $\ell_n/30$ DEFLECTION LIMITATION.

% of MOMENTS TO Column Strip

M^-_{EXT}

			(15/18)	
$\ell_2/\ell_1 =$	$\longrightarrow$	0.5	.833	1.0
$\alpha_{f1}\,\ell_2/\ell_1 = 0 \rightarrow \beta_t = 0$	$\rightarrow$	100	100	100

M^- @ EXT FACE OF FIRST INTERIOR SUPPORT

ℓ_2/ℓ_1	$\longrightarrow$	0.5	.833	1.0
$\alpha_{f1}\,\ell_2/\ell_1 = 0 \rightarrow \beta_t = 0$	$\rightarrow$	75	75	75

M^+

ℓ_2/ℓ_1	$\longrightarrow$	0.5	.833	1.0
$\alpha_{f1}\,\ell_2/\ell_1 = 0 \rightarrow \beta_t = 0$		60	60	60

M^-_{EXT}

−18.8 → 100 % TO C.S. = $-18.8^{\prime\text{-}k}$
- → NO BEAMS = $0^{\prime\text{-}k}$
- → 100 % TO C.S. slab = $-18.8^{\prime\text{-}k}$

−18.8 → 0 % TO M.S. slab

M^+_{EXT}

61.3 → 60 % TO C.S. = 36.78
- → NO BEAMS = $0^{\prime\text{-}k}$
- → 100 % TO C.S. Slab = $36.78^{\prime\text{-}k}$

61.3 → 40 % TO M.S. = $24.52^{\prime\text{-}k}$

M^+ INTERIOR SUPPORTS

41.23^{1-k}
- → 60 % TO C.S. = 24.74^{1-k}
  - → NO BEAMS = $\underline{0^{1-k}}$
  - → 100 % TO C.S. slab = $\underline{24.74^{1-k}}$
- → 40 % TO M.S. slab = $\underline{16.49^{1-k}}$

M^- @ EXTERIOR FACE OF 1ST INT. SUPPORT

-82.5^{1-k}
- → 75 % TO C.S
  - → NO BEAMS = $\underline{0^{1-k}}$
  - → 100 % TO C.S. slab = $\underline{-61.88^{1-k}}$
- → 25 % TO M.S. slab = $\underline{-20.63^{1-k}}$

M^- INTERIOR SUPPORT

-76.6^{1-k}
- → 75 % TO C.S. = -57.45^{1-k}
  - → NO BEAMS = $\underline{0^{1-k}}$
  - → 100 % TO C.S. slab = $\underline{-57.45^{1-k}}$
- → 25 % TO M.S. slab = $\underline{-19.15^{1-k}}$

Column strip Reinforcement Design

ITEM	DESCRIPTION	EXTERIOR SPAN M^-_{EXT}	EXTERIOR SPAN M^+	EXTERIOR SPAN M^-_{INT}	INTERIOR SPAN M^-	INTERIOR SPAN M^+
1.	M_u (ft-k)	-18.8	36.78	-61.88	-57.45	24.74
2.	C.S. width b (slab only)	90"	90"	90"	90"	90"
3.	EFFECTIVE DEPTH, d	5.3"	5.3"	5.3"	5.3"	5.3"
4.	$M_u \cdot 12 / b$	2.51	4.9	8.25	7.66	3.3
5.	$M_n = M_u / \phi = .9$	-20.9	40.9	-68.76	-63.8	27.49
6.	$R = M_n / bd^2$	99.2	194	326.4	302.8	130.5
7.	ρ required *NOTE: TABLE A3 FROM TEXT BOOK	.0033	.0035	.006	.006	.0033
8.	A_s, req'd = $\rho b d$	1.57	1.67	2.86	2.86	1.57
9.	$A_{s,min}$ = .002bt	1.26	1.26	1.26	1.26	1.26
10.	Larger of 8, 9 / Bar Area # 5 BAR = .31 in²	5.1 ↳ 6 Bars	5.39 ↳ 6 bars	9.23 ↳ 10 bars	9.23 ↳ 10 bars	5.1 ↳ 6 bars
11.	$N_{min} = \frac{\text{width of strip}}{2t}$ 90/2.7 = 6.4 = 7	7	7	7	7	7
12.	Larger of 10, 11 = N	7 #5 TOP bars	7 #5 BOT Bars	10 #5 TOP Bars	10 #5 TOP BARS	7 #5 BOT BARS

SAMPLE CALCS FOR ADJACENT TABLE

C.S. = 15/2 = 7.5 × 12" = 90"

$d_L = 7" - .75" - \frac{1}{2}(.625")$ SHORT

$d_S = d_L - .625 = 5.3125"$

cover = .75"

ϕ #5 bar = .625"

$R = \frac{20.9 \times 12 \times 1000}{(90")(5.3"^2)}$

R = 99.2

FOR ρ USE

$f'_c = 4000$ psi concrete

grade 60 steel

t = 7" slab

FRAME / C – M.S. REINFORCEMENT DESIGN – 7" slab

ITEM DESCRIPTION	EXTERIOR SPAN M^-_{EXT}	EXTERIOR SPAN M^+	EXTERIOR SPAN M^-_{INT}	INTERIOR SPAN M^-	INTERIOR SPAN M^+
(1) M_u (ft-k)	0	24.52	-20.63	-19.15	16.49
(2) M.S. width b	90"	90"	90"	90"	90"
(3) EFF. DEPTH, d	5.3"	5.3"	5.3"	5.3"	5.3"
(4) $M_u \cdot 12 / b$	0	3.27	-2.75	2.55	2.20
(5) $M_n = M_u / \phi = .9$	0	27.24	-22.92	-21.3	18.3
(6) $R = M_n / bd^2$	0	129.3	108.79	101.1	86.86
(7) $\rho_{req'd}$	0	.0033	.0033	.0033	.0033
(8) $A_{s,req'd} = \rho b d$	0	(1.57)	(1.57)	(1.57)	(1.57)
(9) $A_{s,min} = .002bt$	(1.26)	1.26	1.26	1.26	1.26
(10) Larger of 8, 9; #5 BAR AREA = .31	1.26 → 2 bars	1.57 → 2 bars	1.57 → 2 bars	1.57 → 2 bars	1.57 → 2 bars
(11) $N_{min} = \frac{\text{Strip width}}{2t}$	6.43 → (7)	6.43 → (7)	6.43 → (7)	6.43 → (7)	6.43 → (7)
(12) Larger of 10, 11 → ½ to EACH ½ Middle STRIP ⇒ 7/2 = 3.5 bars → (4)	4 #5 TOP BARS IN EACH ½ M.S.	4#5 BOT BARS IN EACH ½ M.S.	4 #5 TOP BARS IN EACH ½ M.S.	4 #5 TOP BARS IN EACH ½ M.S.	4 #5 BOT BARS IN EACH ½ M.S.

CHECK SLAB DEPTH AND THICKNESS PER STRENGTH REQUIREMENTS

$$M_u = \phi M_n = \phi \rho f_y b d^2 (1 - 0.59 \rho f_y / f'_c)$$

$$\frac{M_u}{\phi b d^2} = R = \rho f_y (1 - .59 \rho f_y / f'_c)$$

USE LARGEST DESIGN MOMENT
↳ FROM FRAME C Column strip design table
$M_u \cdot 12 / b$ → WORST CASE MOMENT = 8.25

$$M_n / \phi b d^2 = -68.76$$ (ALSO from earlier calculations)

DESIGN AIDS: $R = -68.76$, $f'_c = 4000$ psi } $\rho = .0033$ (linear interpolation)

Limiting steel Reinforcement ratios for tension controlled members

$f_y = 60$ ksi, $f'_c = 4$ ksi } $\rho_{max} = .0206 < .0033$ OKAY

$$d_{required} = \sqrt{\frac{M_u}{\phi b \rho f_y (1 - .59 \rho f_y / f'_c)}} = \sqrt{\frac{5.56 \times 12 \times 1000}{.9(12'')(.0033)(60{,}000)(1 - .59(.0033)(\frac{60}{4}))}}$$

$$d_{required} = \sqrt{32.139} = 5.67''$$

$d_{used} = 5.3''$ $\qquad d_{req} = 5.67'' > 5.3'' = d_{used}$

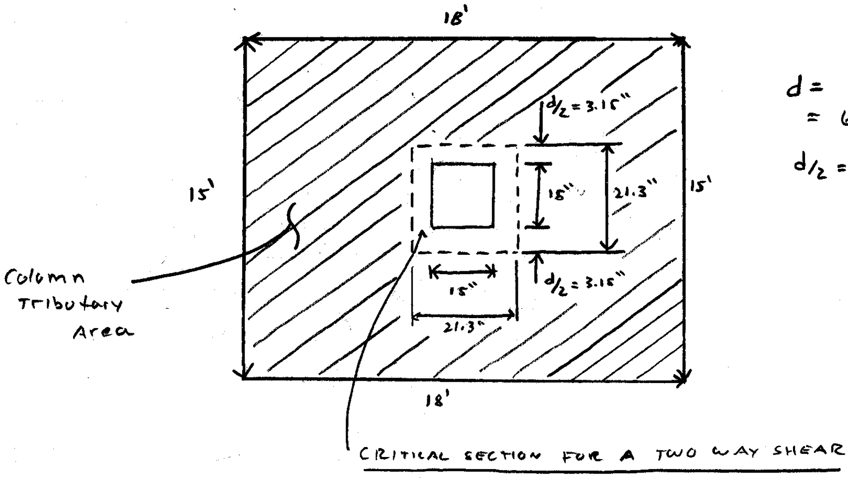
∴ 8" slab instead of 7" slab would be appropriate
↳ new effective depth, $d = 8'' - .75''\text{cover} - \frac{1}{2}(.625) - .625$

$d = 6.3125'' > d_{required}$ w/ new slab $t = 8''$

* NOTE: THE MOMENTS ARE LARGER AT FACES OF SUPPORT. THEREFORE, THE LOADING MAY ACTUALLY WARRANT THE NEED FOR DROP PANELS TO MEET THE depth REQUIREMENTS

MUST CHECK THE FLOOR SYSTEM FOR TWO-WAY PUNCHING SHEAR

A 15" × 15" COLUMN SIZE IS USED FOR DESIGN



$d = 6.3125"$
$= 6.3"$
$d/2 = 3.15"$

b_o = critical section perimeter

$b_o = (15 + 3.15 + 3.15) \times 4 = 4 \times 21.3$

$b_o = 42.6"$

$V_c = 4\sqrt{f'_c}\, b_o\, d$ where $\lambda = 1$

$V_c = 4\sqrt{4000}\,(42.6")(6.3125")(1\text{ kip}/1000\text{ lbs}) = \underline{68.03 \text{ kips}}^k$

$V_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}\, b_o\, d$ where $\beta = \frac{15}{15} = 1$

$V_c = \left(2 + \frac{4}{1}\right)(1)\sqrt{4000}\,(42.6)(6.3125)(1\text{ kip}/1000\text{ lb})$

$V_c = \underline{102.045}^k$

$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o\, d$ $\alpha_s = 40 \rightarrow$ INTERIOR COLUMN

$V_c = \left(\frac{40(6.3125)}{42.6} + 2\right)\sqrt{4000}\,(42.6)(6.3125)(1\text{ kip}/1000\text{ lb})$

$V_c = \underline{134.8}^k$

lowest value governs: $V_c = 68.03^k$

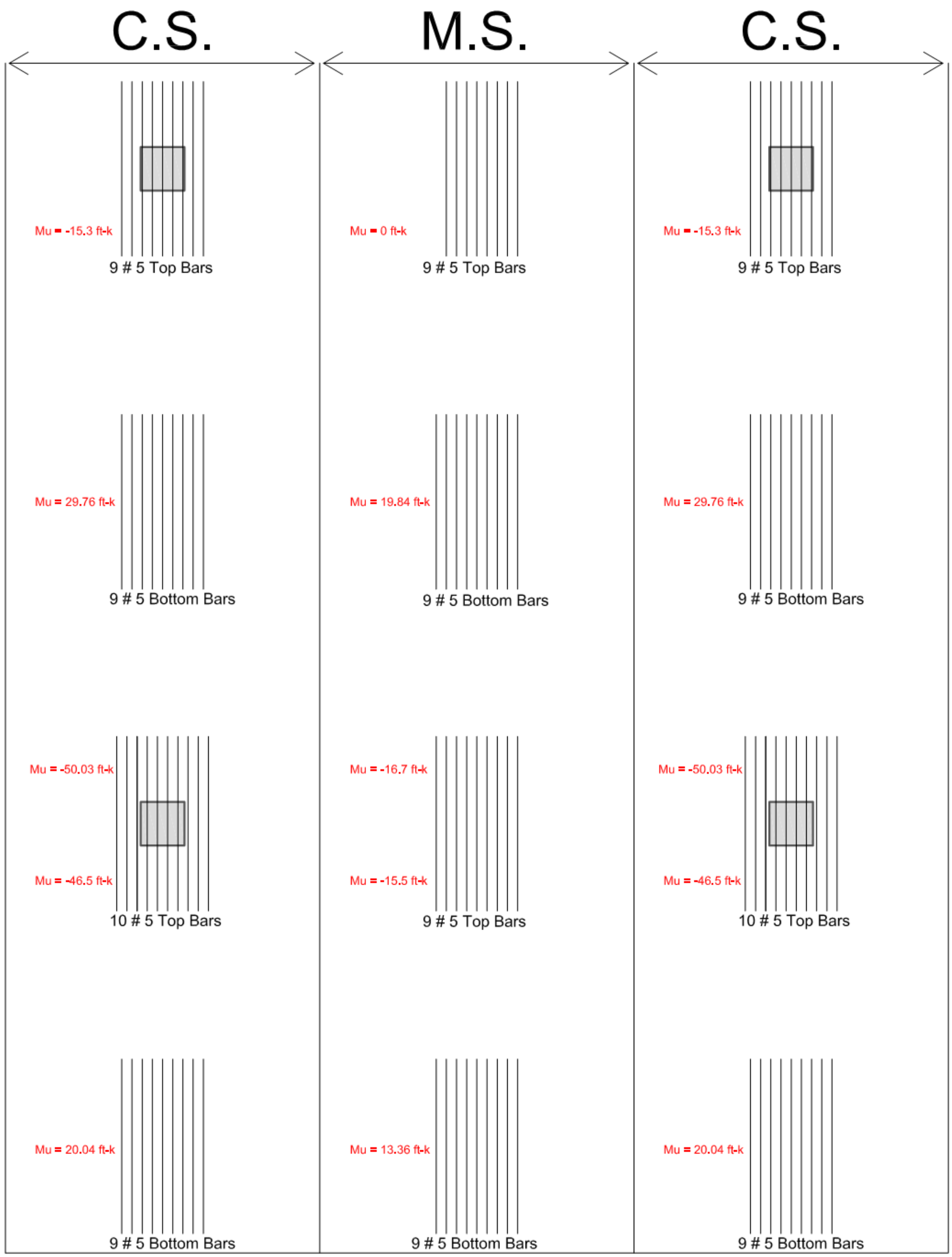
$V_u = \left[(18 \times 15) - \left(\frac{21.3}{12} \times \frac{21.3}{12}\right)\right](.224) = 59.77$

$w_u = (.224\ ^k/ft^2)$

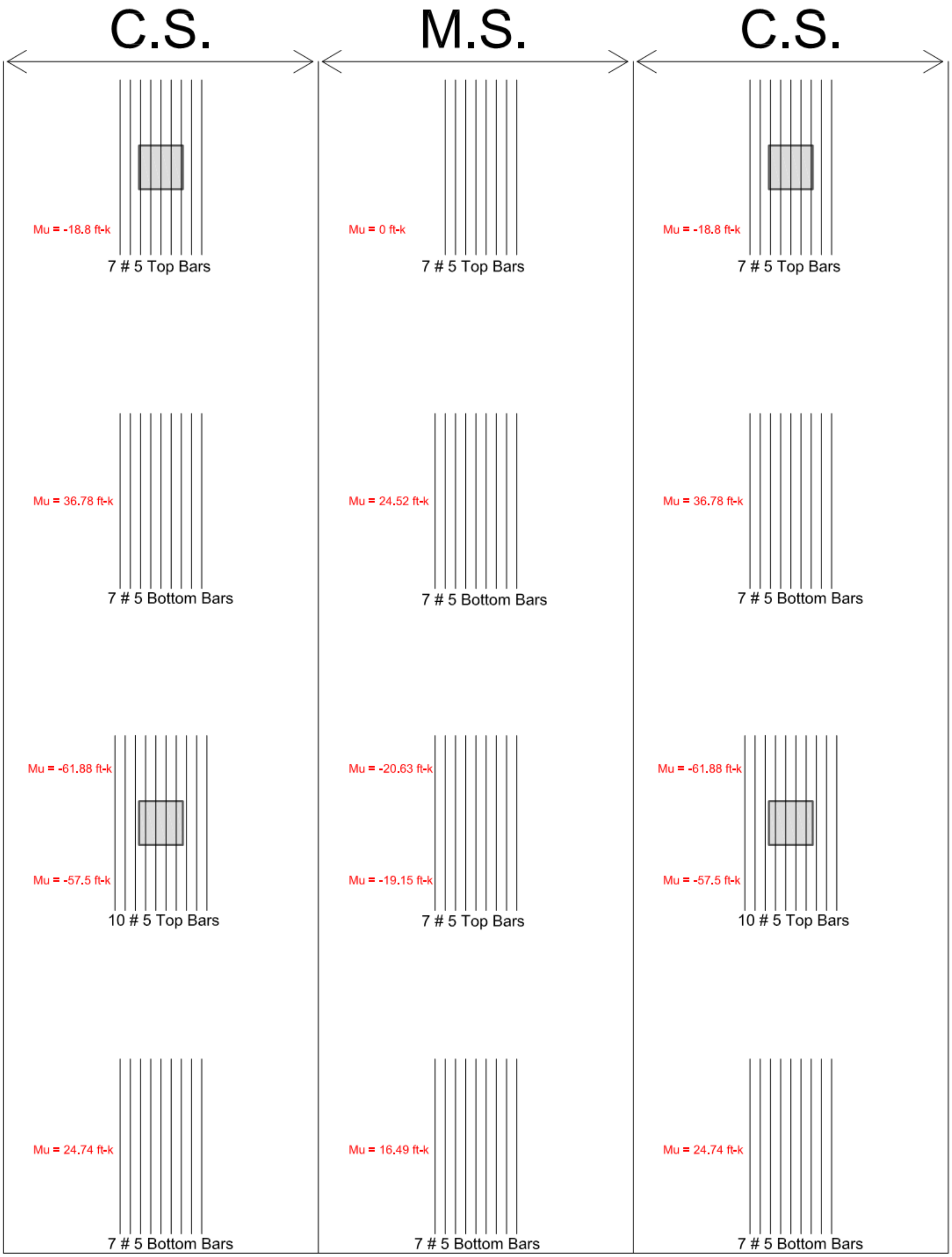
$\phi V_c = .9(68.03) = 61.23 > V_u = 59.77$

OKAY ✓

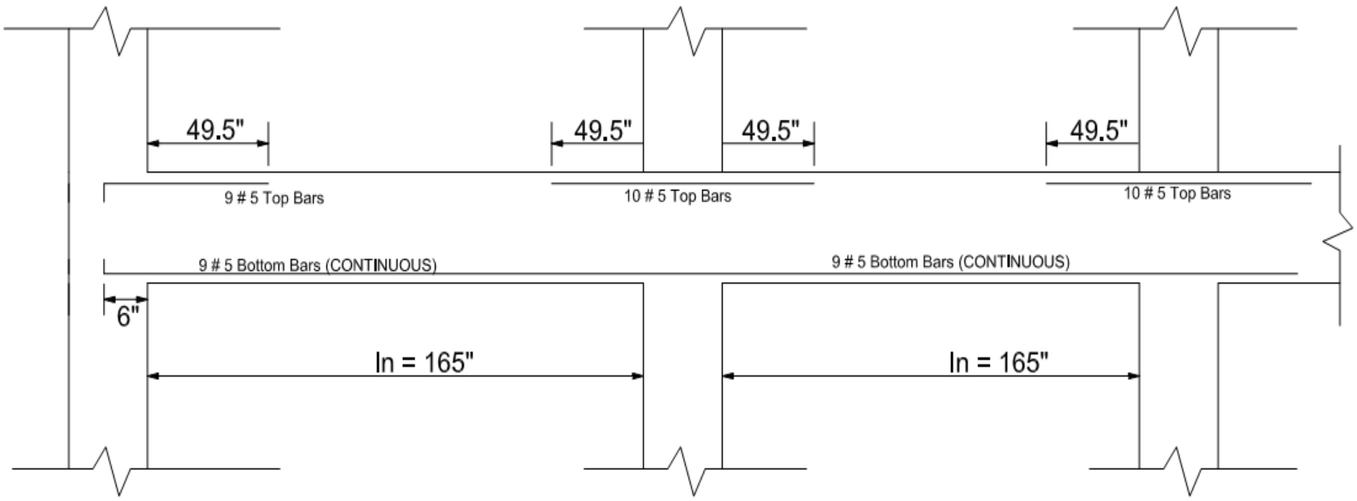
Frame A – Shear Reinforcement



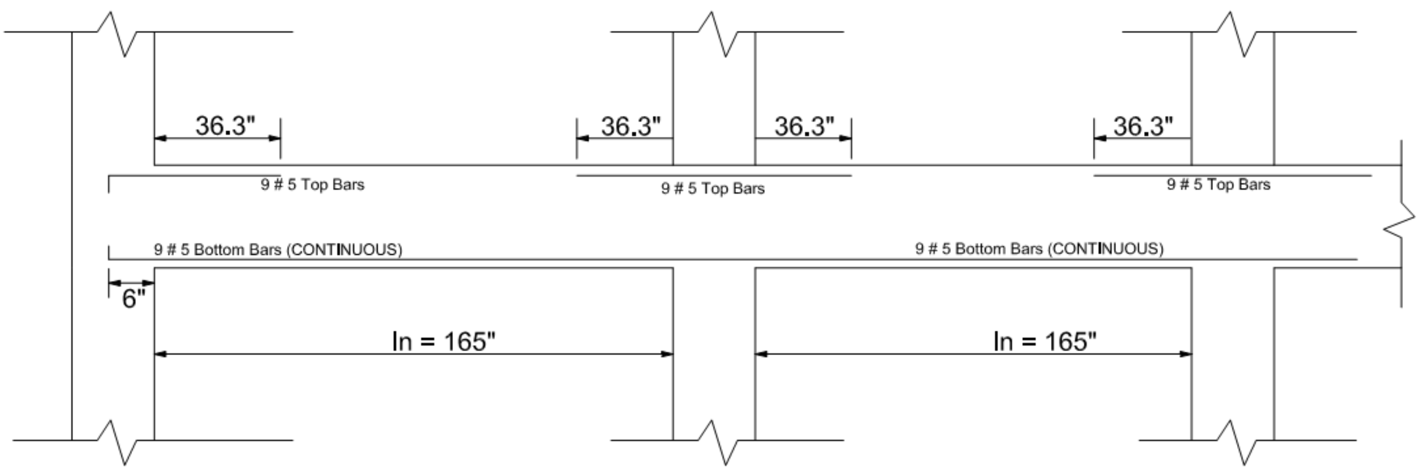
Frame B – Shear Reinforcement



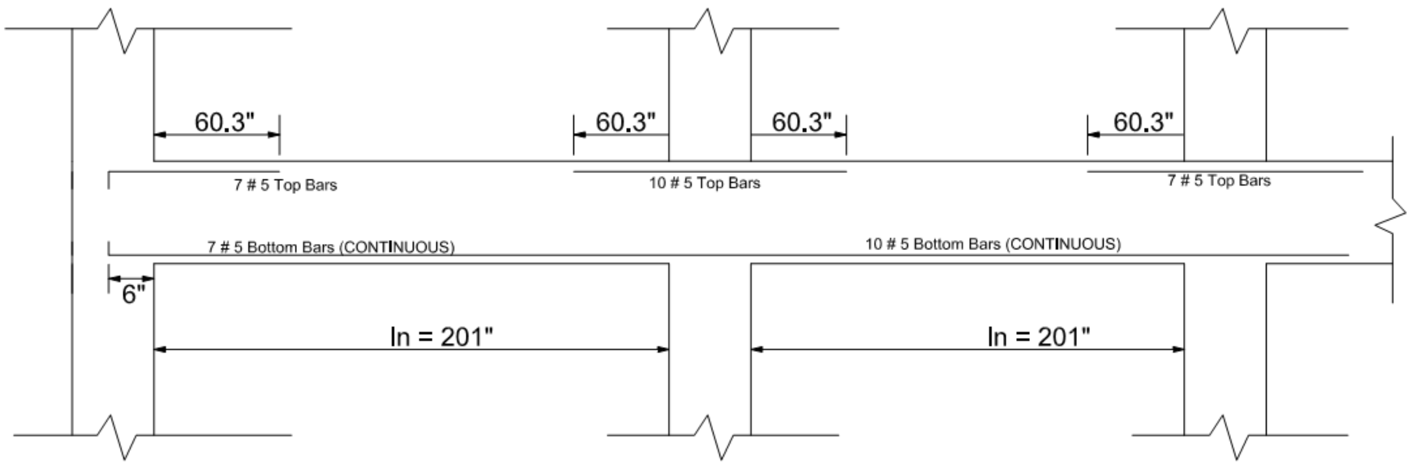
Frame A - Column Strip Reinforcement



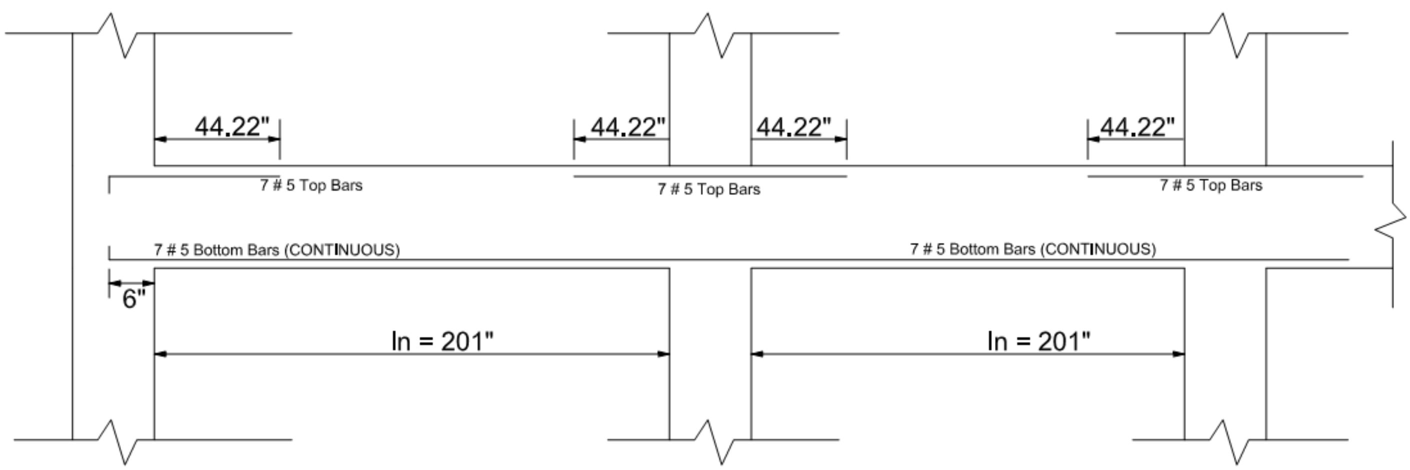
Frame A - Middle Strip Reinforcement



Frame C - Column Strip Reinforcement



Frame C - Middle Strip Reinforcement



APPENDIX C – Hollow Core Precast Planks - Design Calculations (Please See pg. 22 for more details)

HOLLOW - CORE PRECAST PLANK ON STEEL FRAMING

DESIGN CALCULATIONS

* LOADS - ASCE-07
* PRECAST PLANKS - 6TH EDITION PCI HANDBOOK
* STEEL FRAMING - AMERICAN STEEL INSTITUTE OF CONSTRUCTION MANUAL (AISC)

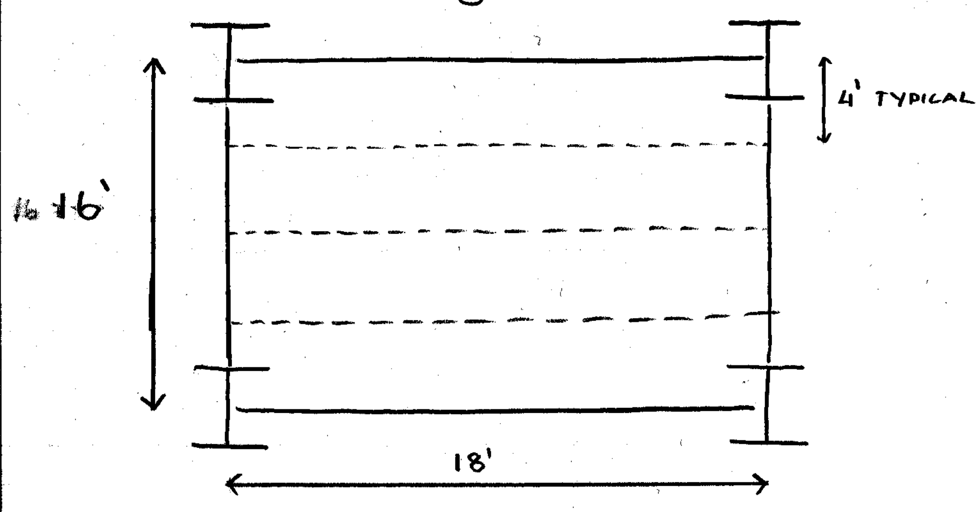
LIVE LOAD = 40 PSF
SUPERIMPOSED DEAD LOAD = 20 PSF
2" TOPPED MEMBERS = PCI → INCLUDE 15 PSF

CRITERIA FOR CHOOSING PLANK
SPAN = 18'
LOAD = 75 PSF

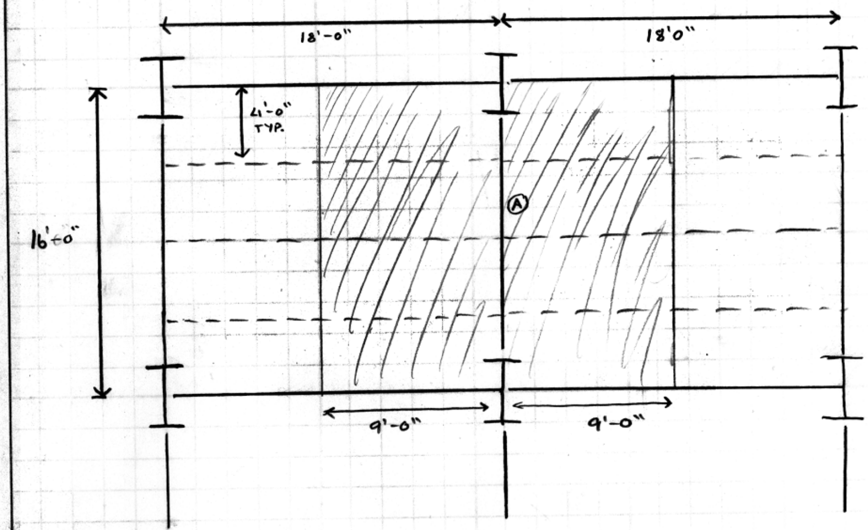
SELECTED PRODUCT: 4'-0" x 6" 66-S, 2" TOPPING

* 6 STRANDS
* ∅ STRANDS = 6/16 IN
* Straight

Allowable = 182 PSF
OKAY ✓



DESIGN SUPPORTING STEEL MEMBERS (PRE CAST PLANK)



DESIGN BEAM A

$W_u = 1.2D + 1.6L$
$= 1.2$ (s.w of Plank + SIDL) + 1.6 (LL RESIDENTIAL)

$W_u = 1.2$ (74 PSF + 25 PSF) + 1.6 (40 PSF)
$W_u = 118.8 + 64$
$W_u = 182.8$ PSF

$W_u =$ 182.8 PSF (TRIB WIDTH) = 182.8 PSF (9'0" × 2)
$W_u = 3.29$ k/ft
DO NOT KNOW SIZE → ESTIMATE s.w. → 20 lb/ft = .02 k/ft

$W_u = 3.29$ k/ft + .02 k/ft = 3.31 k/ft includes estimated s.w.

FLEXURE $M_u = \frac{(3.31)(16^2)}{8} = 105.92^{k\text{-}ft}$

AISC TABLE 3-2 : LEAST WEIGHT COLUMNS
W12×22
$\phi M_p = 110 > 105.92^{k\text{-}ft}$ ∴ OKAY ✓

SHEAR: $\frac{wL}{2} = \frac{(3.31)(16)}{2} = 26.48^k < \phi V_n = 96^k$ OKAY ✓

LL DEFLECTION: $\Delta = l/360 = 16' \times 12''/360 = .533 = \frac{5(40)(18)(16)^4(12^3)}{384(29000)(1000)(I_{required})}$ (LL unfactored)

$I_{required} = 68.69$ in^4

W12×22 → $I_x = 156$ in^4 > 68.69 in^4 OKAY ✓
OR W10×26 IF WANT TO MINIMIZE DEPTH

APPENDIX D – Girder Slab – Design Calculations (Please see pg. 24 for more details)

Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	w x t
	lb/ft	in²	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5

These two tables are provided by the Online Girder Slab 1.4 Design Guide. The highlighted portions of the tables show the properties and dimensions of the interior girder used for the following design checks.

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

DESIGN – UNTOPPED

PLANK DL = 60 PSF, PARTITION LOAD = 12 PSF, LIVE LOAD = 40 PSF
PLANK f'_c = 5 ksi, GROUT = 4 ksi
8" Hollow-Core plank span = 18'-0"
DB SPAN = 16'-0"

Allowable $\Delta_{LL} = L/360 = 16 \times 12/360 = .5333$ in

INITIAL LOAD – PRECOMPOSITE (w/o grout – before composite established)

$$M_{DL} = \frac{(18')(.06)(16^2)}{8} = 34.56 < 49 \text{ k-ft} \quad \text{OKAY} \checkmark$$

$$\Delta_{DL} = \frac{5(18)(.06)(16)^4(1728 \text{ in}^3/\text{ft}^3)}{384(102 \text{ in}^4)(29{,}000)} = .53837 \text{ in}$$

TOTAL LOAD – COMPOSITE

"THE TRANSFORMED SECTION CARRIES THE SUPERIMPOSED LOADS AND IS USED TO CALCULATE DEFLECTION"

$$M_{SUP} = (18)(.012 + .04)(16)^2/8 =$$

M_{SUP} = 29.952 KFT

M_{TL} = 29.952 + 29.952 = 59.904 k-ft

$S_{REQ} = (59.904)(12 \text{ in/ft}) / (.62)(50 \text{ ksi})$

$S_{REQ} = 23.96 \text{ in}^3 < S_{ACTUAL} = 63.5$ OKAY ✓
TRANSFORMED

$$\Delta_{SUP} = \frac{(5)(18)(.012 + .04)(16)^4(1728)}{(384)(279)(29000)} = .1706'' < \Delta_{LL} \text{ allowable} = .533$$

OKAY ✓

CHECK SUPERIMPOSED Compressive Stress on Concrete

$N_{VALUE} = E_s / E_c = 29000 / 57000\sqrt{4000} = \frac{29000}{3605}$

$N = 8.044 \therefore S_{tc} = 8.044$ (S_{tc} transformed)

$8.044 (63.5) = 510.82 \ in^3$

$f_c = \frac{(29.952 \ k\text{-}l)(12 \ in/ft)}{510.82} = .7036 \ ksi$

$F_c = .45 (4 ksi) = 1.8 \ ksi > .704 \ ksi$ OKAY ✓

CHECK BOTTOM Flange tension stress (TOTAL LOAD)

$f_b = \frac{(29.952)(12)}{36.5 \ in^3} + \frac{(29.952)(12)}{67.1 \ in^3} = 15.2 \ ksi$

$F_b = .9 (50 \ ksi) = 45 \ ksi > 15.2 \ ksi$ OKAY ✓

CHECK SHEAR CAPACITY

TOTAL LOAD = 112 PSF

$w = (.112)(18') = 2.016 \ k/ft$

$V = wL/2 = 2.016 (16)/2 = 16.128 \ kips$

$f_v = \frac{16.128}{t_w \times b} = \frac{16.128}{3 \times .34} = 17.12$

$F_v = .4 (50 \ ksi) = 20 \ ksi > 17.12 \ ksi$ OKAY ✓

REFERNCES - various manuals, design guides, and handbooks were referenced to complete this report.

1. ACI 318 – 08: Building Code Requirements for Structural Concrete and Commentary

2. Steel Construction Manual, American Institute of Steel Construction, 13th edition

3. RS Means: Assembly Cost Data, Ferguson, John H, 2009

4. Vulcraft Steel Roof and Floor Deck Catalogue, Nucor Corporation

5. Girder-Slab: Composite Steel and Precast System, Design Guide 1.4, http://www.girder-slab.com/system.asp.

6. PCI Design Handbook: Precast and Prestressed Concrete, 6th edition. Seeber, Kim P.E.

7. 40 Gold Street Structural Drawings, Severud Associates

8. 40 Gold Street Architectural Drawings, Meltzer/Mandl Architects, P.C.