GOUVERNEUR HEALTHCARE SERVICES

NEW YORK, NY

TECHNICAL REPORT 2

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Structural

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EXECUTIVE SUMMARY

In total, four floor systems were compared during the course of this technical report. They consisted of the existing floor system with composite cellular gravity beams, traditional composite steel framing, hollow core slab on steel framing, and a two-way flat plate, concrete system. In this simplified approach, the concrete floor system proved to be most cost effective, but other considerations must be taken into account.

Although concrete was the cheapest construction in terms of cost per square foot of floor system, the fact that it is heavier than steel framing will impact foundation design and other aspects of the structure. Using concrete may increase the overall building cost, possibly offsetting any benefits of the cheaper floor system. Furthermore, the need for an additional line of columns may negatively impact the function of the building. However, despite these drawbacks, two-way flat plant construction provides a completely unobstructed space for MEP systems and may interact better with the 35-year-old, existing structure.

Composite steel framing using traditional W-shapes proved to be the lightest and most cost effective floor system when investigating steel framing. Despite this, composite steel framing was not a feasible solution due to the tight floor-to-floor restrictions imposed by the existing building. There is simply not enough clearance between the bottom of the steel beams and the ceiling below.

The existing system comprised of composite steel framing using cellular beams for gravity members. This system proved to be the most cost effective steel framing system while still remaining feasible. The cellular beams allowed ample space for MEP systems, and the depth requirement of 27in allowed designers to reduce the overall number of columns, allowing for more freedom in the architectural design. One potential drawback to the floor system is the widespread use of a specialty shapes. Other issues arise in the use of moment frames for the lateral system. These frames would not allow adequate clearance for MEP systems and would restrict the design of these bays.

Hollow core plank was the most expensive of the steel framed floor systems, although it had many benefits. The plank was able to span long distances, allowing clear space between the slab and the ceiling below. Also, the use of the precast system would allow fast installation, potentially shortening the schedule and reducing cost. This floor system is also susceptible to the restrains on MEP design, imposed by the use of moment frames to resist lateral loads.

INTRODUCTION

The Gouverneur Health Services Modernization Project is an addition to an existing building and a renovation of the 35-year-old healthcare facility. The existing building is a 2-way flat plate floor construction with square and rectangular columns. An existing conditions survey revealed no shear-walls, so it can be assumed that lateral loads are resisting by the continuous frame construction of the flat plate slab. For the purpose of this technical report, and subsequent thesis project, only the addition will be investigated in further detail. Furthermore, portions of the addition that wrap around the existing building and tie into the existing structure will be neglected for this technical report.

The addition that will be the main focus of this thesis project consists of two distinct portions. The first portion is the 5-story ambulatory care facility. This facility is approximately 115'x175' in plan, and sits on the western side of the site, connected to the existing building. The second portion is an expansion to the floor plan to the existing building in floors 6 through 13. It is roughly square, 50'x60' in plan, and extends upwards from the ambulatory center on the western side of the existing building. The portions may be referred to as lower addition and upper addition, or ambulatory addition and tower addition, respectively. See Figures below.

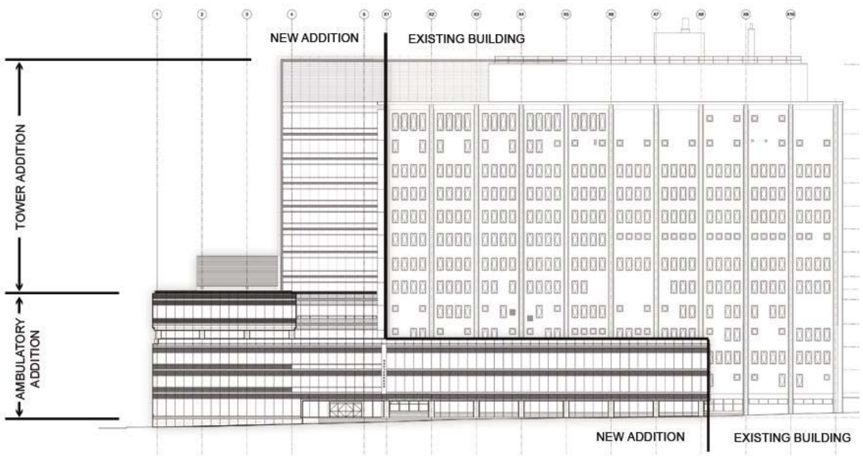

Fig 1. Gouverneur Layout Schematic

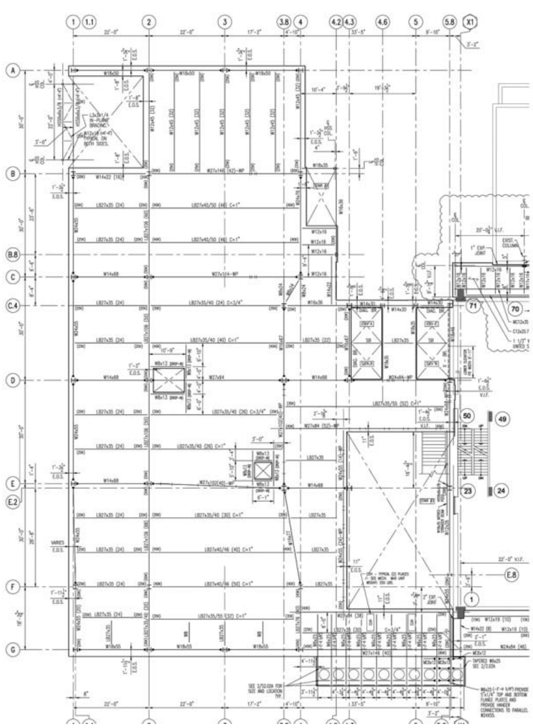

Fig 2. Typical Ambulatory Center Framing Plan

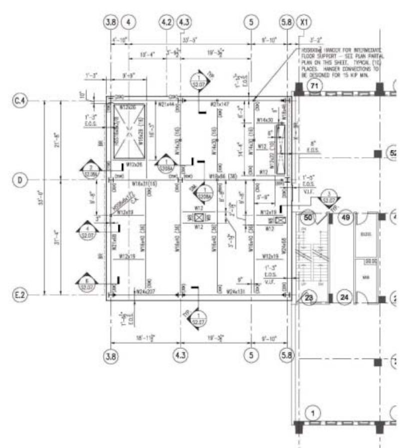

Fig 3. Typical Tower Addition Framing Plan

STRUCTURAL SYSTEM

Foundation

The Gouverneur Healthcare Facility bears on a pile foundation system, with 60-ton capacity, 12” piles. Pile caps vary from 35” to 54” thick with the number of piles ranging from 2 to 16 piles per cap. The footprint for the cellar is smaller than the extents of the overall building so the depths of the pile caps vary. The depths of the caps are either 4’-6” below datum if the columns terminate in the cellar, or 16’-9” above datum if the columns terminate on the first floor.

The piles support grade beams that span between 15’ and 40’. Their sizes range from 4’-0” to 8’-3” deep with reinforcing bars from #8 to #12 bars. A structural, one-way slab-on-grade spans between grade beams to make up the cellar floor.

Floor System

The floor system for Gouverneur Healthcare Services is a composite system that utilizes cellular beams for all gravity beams in the ambulatory addition. A 4 ¼” slab rests on a 2” LOK floor composite deck, and is tied to the beam with 5” long, ¾” diameter shear studs. Typical bays are 30’-0” by 44’-0” and almost all beams are nominally 27” deep to accommodate mechanical systems. The tower addition uses traditional W-shapes in a composite floor system. Beams are W16’s in areas where clearance for mechanical equipment is not an issue, and W14’s where clearance is an issue.

Columns

Almost all columns in the Gouverneur Healthcare Services Building are W14 columns, regardless if it is a part of the lateral system or just a gravity column. Sizes range from W14x43 to W14x257, and are continuous from the foundation to the roof, with only column bearing on a transfer girder on the seventh floor. Columns are spliced on every other floor starting on the third floor. Base plates are typically 22” x 22” with bolts ranging in size from ¾” to 2”.

Lateral System

Due to the vast use of glass curtain walls and irregular plan between floors, most of the lateral system in the Gouverneur Healthcare Services Building is moment resisting frames. For the interior moment frames, sizes are either W27’s for long span beams or W14’s for the shorter spans. Most beams in exterior moment frames are W18’s and W24’s. In the tower portion of the building, lateral loads are resisted by exterior moment frames in the East-West direction, and braced frames in the North-South direction, both concentric and eccentric. Most braced frames are continuous from the roof to the column termination at the foundation. But at the interface of the upper addition and the lower addition, where one frame is discontinuous, loads transfer into columns in the floor below, and redistribute through the structure.

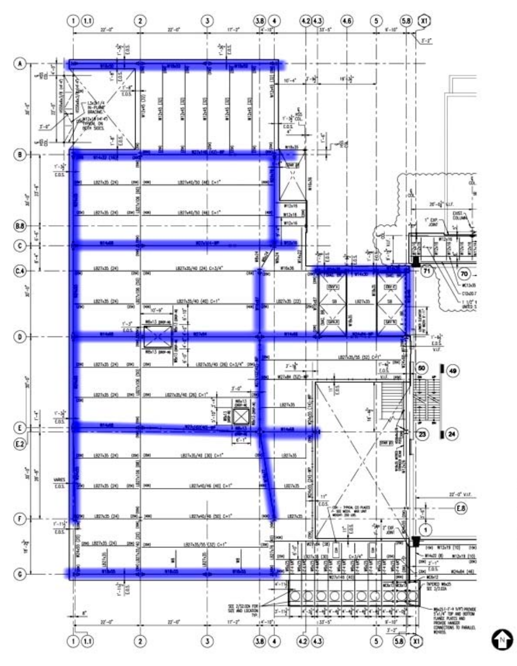

Fig 4. Typical Framing Plan Showing Moment Frames

Wind loads transfer from curtain wall system to floor diaphragm. The floor diaphragm is rigid compared to structure so loads transfer to lateral frames based off of relative stiffness. Loads then transfer to foundations in the form of shear and axial load (tension and compression) in braced frames, and transfer to the foundation through shear, axial load, and moment in moment frames

MATERIALS

Concrete	ASTM	Min Strength
Structural slab-on-grade	-	3000 psi
Pile cap	-	4000 psi
Retaining walls	-	4000 psi
Interior Slabs	-	4000 psi
Reinforcing Steel	A615	60ksi
Structural Steel		
Structural Tubing	A500	46 ksi
Steel Pipe	A53	35 ksi
Rolled Shapes	A992	50 ksi
Other Rolled Plates	A36	36 ksi
Connection Bolts	A325	90 ksi
Anchor Bolts	A307	45 ksi

APPLICABLE CODES AND DESIGN REQUIREMENTS

Codes and References

The City of New York Building and Administrative Code
New York Electrical Code
All Applicable NFPA Codes
New York State Energy Code
AIA Guidelines for Design and Construction of Hospital and Health Care Facilities

Deflection Criteria

Floor Deflection L/240 Total and L/360 Live

Lateral Deflection

Total Drift 3 ½” (due to expansion joint between addition and existing building)

Story Drift H/400

DESIGN LOADS

Dead Load (psf)	
Floor Load	
3 1/4" LW concrete fill on 3" LOK-Floor	60
Ceiling	2
Floor Finish	2
Mech/Elect	10
Partitions	12
Steel Framing	13
TOTAL	99
	(psf)

Wall assemblies	
1. Metal Panel	25
2. Glass Curtainwall	15
GFRC	40
	(psf)

Dead Load (psf)	
Penthouse Roof	
Steel	8
Deck/Insulation	8
Mechanical	10
Membrane	2
Fire Proofing	2
TOTAL	30
	(psf)

Main Roof	
3 1/4" LW concrete fill on 3" LOK-Floor	60
Ceiling	3
Mech/Elect	14
Roofing/Insulation	9
TOTAL	86
	(psf)

Live Load (psf)		
Live Load	As Designed	As per ASCE7
Dormatory Floors	40	40
Lobby	100	100
Lounge	100	100
Corridor 1st Floor	100	100
Corridor above 1st	80	80
Stairs	100	100
Mechanical Rooms	150	-
Main Roof (Mech)	150	-

Fig 5. Design Load Tables

COMPARISON CRITERIA

Structural Impact

Structural impacts are discussed in terms of how the alternative framing systems will change the existing system and affect the design of other portions of the building. The main focus in this report is the influence on foundations and analysis for lateral loads, although impacts to the structural system are not limited to those discussed in this report.

MEP Impact

Mechanical, Electrical and Plumbing impacts are discussed simply in terms of how the floor systems limit the freedom of MEP engineers. These criteria include clearance issues and the interference of lateral systems in the layout of mechanical zones.

Architectural Impact

The architectural impacts discussed in this report focus primarily on the need for an additional line of columns along column line 3 (see figure below). Other impacts are compared including clearance issues and the impact of new columns on the floorplan.

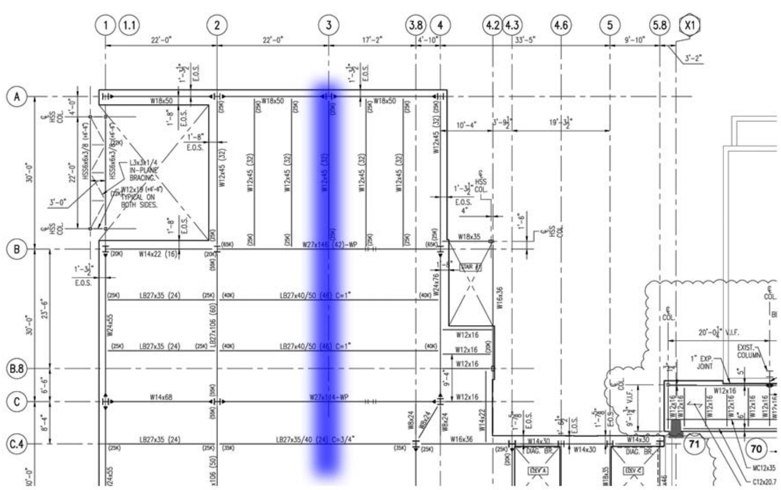

Fig 6. Partial Floorplan showing additional column line in blue

EXISTING SYSTEM – CELLULAR BEAMS

Structural Discussion

The design choice to implement deep, cellular beams for all gravity members allowed a great amount of flexibility in the structural design. The depth requirement of 27" nominally allowed the use much larger spans than originally called for in the design. Max spans were stretched to 44', reducing the number of columns and foundations, positively affecting the overall cost of the project. Because the cellular beams were unable to be used for the lateral systems, W-shapes comprise a good portion of the beams in the floor systems in the form of moment resisting frames. The use of both cellular beams and W-shapes in conjunction directly impacted the mechanical systems. Moment frames are not included in the schematic plan below, although their location can be seen in Figure 4, above.

MEP Discussion

The widespread use of cellular beams created a great deal of flexibility for the mechanical systems. Despite a tight floor-to-floor height of just 11ft, the cellular beams provided adequate web penetrations to run all MEP systems between the drop ceiling and the deck above.

However, the existence of moment frames comprised of W-shapes restricted the design of the MEP systems. Because mechanical systems cannot be run through the moment frame, the floor space between each moment frames are separate, individual zones, without the ability to provide flexibility in design. Systems access each zone through vertical shafts and branch out to service each zone.

Architectural Discussion:

With the choice to implement cellular beams, the overall floor plan became very open. The large 44ft span allowed a great deal of freedom in the architectural design. Furthermore, the ability to accommodate MEP systems between the bottom of the slab and the ceiling meant that the overall ceiling height was able to be kept at a desirable elevation on each floor.

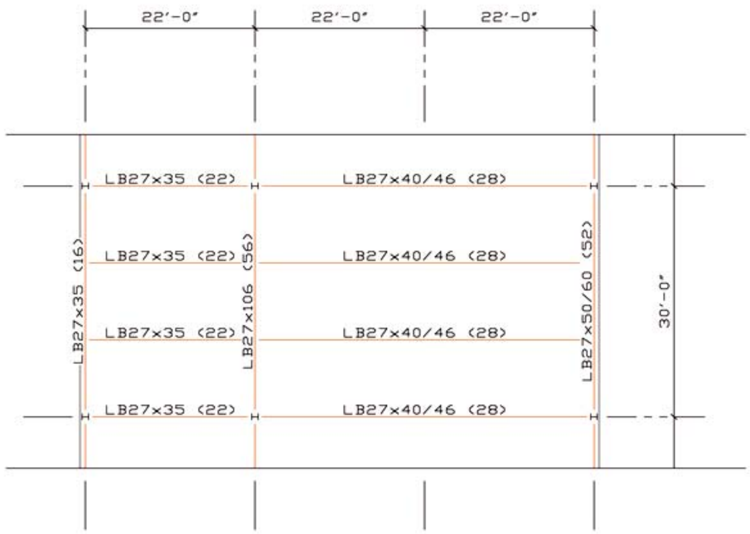

Fig 7. Representative Floorplan of existing system

Fig 8. Typical Construction of Cellular Beams

ALTERNATE SYSTEM – COMPOSITE STEEL

Description

The first, alternative design consisted of composite steel framing. Typical bays of the floor system are 22ftx 30ft, with girders spanning the 30ft length, and intermediate beams spanning the 22ft length. The floor diaphragm consists of 6 ¼"concrete slab on 2" LOK-Floor composite steel deck. A strict depth restriction of 10in maximum for intermediate beams and 12in maximum for girders was maintained in an attempt to keep adequate space between the bottom of the steel members and the drop ceiling to accommodate MEP systems.

An attempt was made to maintain the 44ft span of the existing building in the east-west direction. This was determined to be impossible if a depth restriction was put into place. For this reason, an additional column line was added, creating a total of three, 22ft spans in the east-west direction. After a preliminary assessment of the floorplans, this additional line of columns is not expected to impact the architecture significantly. Figure 6, above, shows the location of the added column line.

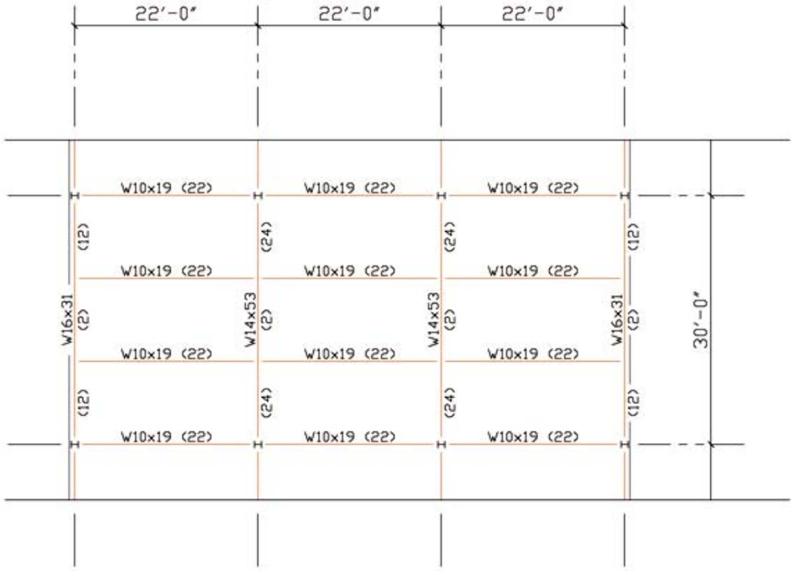

Fig 9. Representative Floorplan of Composite Framing

Fig10. Installation of Shear Studs

Structural Impact

Due to the depth restrictions to accommodate MEP systems, the use of inefficient members was required. Heavier members were used to meet strength and serviceability requirements, where deeper members would have been more efficient. In order to meet the depth requirements, an extra line of columns were added, reducing the span of girders to 22ft from 44ft. This will increase the number of foundations required, potentially driving up the total cost of construction.

MEP Impact

Despite trying to maintain a depth restriction, certain members are as deep as 14in which could cause problems for the mechanical systems. It is important to note that with this depth, MEP systems only have 13in to utilize in order to service the individual zones. This alone may exclude traditional composite steel framing from being a feasible solution in the Gouverneur Healthcare Services.

Again, the use of moment frames to resist lateral loads will impact MEP systems. The anticipated depth of the beams in the moment will require spaces between each moment frame to be split into separate zones, similar to the existing design. Although adequate design is easily possible, the lack of space between beams in the moment frame and the ceiling below will reduce the amount of freedom MEP designers will have.

Architectural Impact

The floor system did not impact the design of the floorplan significantly, although the impact on the MEP systems could make the spaces uncomfortable to occupy. This alone could make the system not feasible.

ALTERNATE SYSTEM – HOLLOW CORE SLAB

Description

The second alternative design consisted of precast hollow core plank on non-composite steel framing. 10" precast hollow core plank with 2" topping was employed in the design of this floor system. Using ½" diameter, 7-wire plank, the slab was able to span the full 30ft in the North-South direction, completely eliminating the need for intermediate beams. The floor system retains the original column grid spacing in the East-West direction, with a 22ft span and a 44ft span.

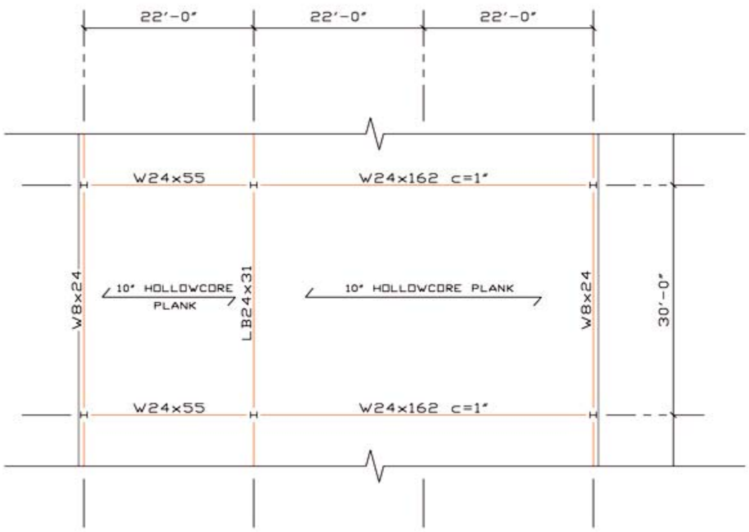

Fig 11. Representative Floorplan of Hollow core Construction

Fig12. Precast Hollow core Planks

Structural Impact

The ability for the concrete plank to span large distances allowed for a significant amount of freedom. The elimination of intermediate beams increased the load on beams between columns spanning in the East-West direction, increasing the size of these members from previous designs. These beams are typically part moment frames in the lateral load resisting system, so further analysis is needed to gauge a more accurate size of the member. Despite the inability to perform the required analysis at this current time, it is anticipated that beams in the moment frame will still be able to meet the 27in maximum depth restriction even with the hollow core plank bearing on them.

MEP Impact

The use of hollow core plank provides a completely unobstructed space within bays. This provides this floor system with some of the same benefits of flat slab concrete systems but obstructions still exist with the moment frames. MEP systems would still be constrained to individual zones between each moment frame, with access available only through vertical shafts.

Architecture Impact

The hollow core slab floor system is not anticipated to affect the architecture at all. Column locations remain the same and ceiling heights will remain the same provided the assumption is correct that lateral loads will not increase the depth of the beams in moment frames past the max depth of 27".

Alternate System – Two-Way Flat Plate

Description

Typical bays in the flat plate floor system are 22ftx30ft. In order to utilize flat plate construction, and keep a reasonable slab thickness, another column line was added, similar to the composite steel framing investigation. This allowed the use of a 10.5" slab and a completely unobstructed space between the slab and ceiling below. Typical reinforcing is #7 bars, and shear reinforcement is provided at columns to resist punching shear. Additional reinforcement is anticipated to be necessary if this design were to be implemented in order to resist lateral loads.

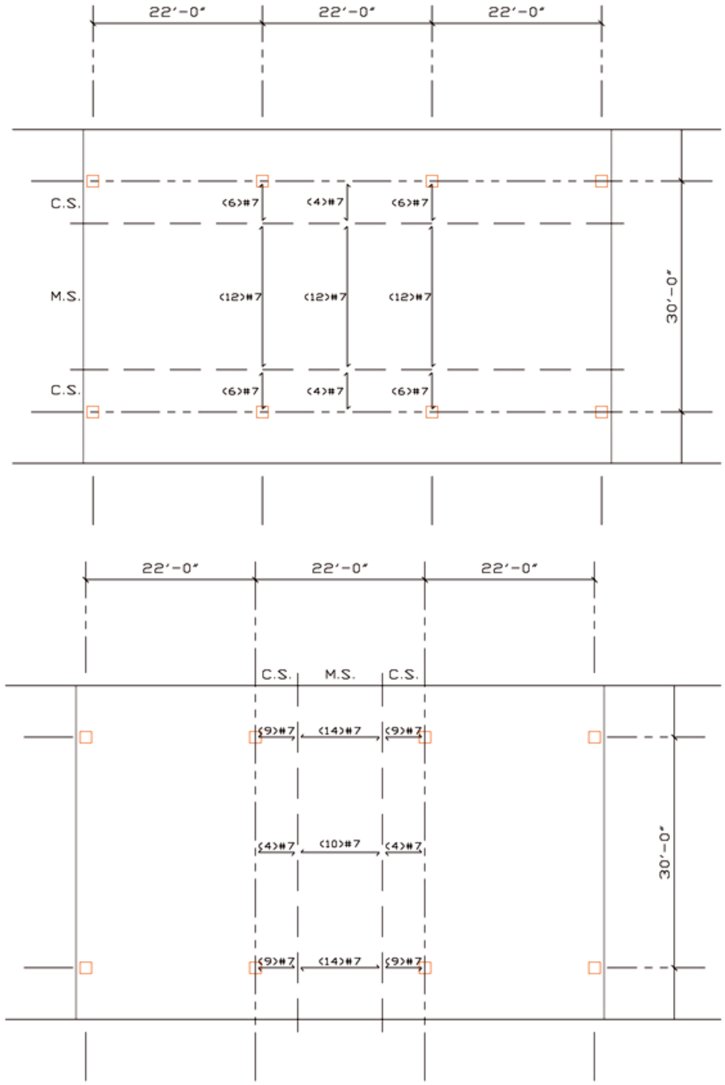

Fig 13. Reinforcing Layout for Two-Way Flat Plate System

Structural Impact

The impacts that stem from changing the material of a floor system are great and varied. Due to the inherent properties of concrete floor systems, the total weight of the building will increase significantly and enlarge the size of the overall foundation system. Furthermore, changing materials will significantly affect the results of seismic analysis, potentially increasing the base shear.

MEP Impact

Flat plate construction will provide a completely unobstructed space between the slab and ceiling enabling great freedom in design of these systems. The size and space of zones will be able to be assigned based on function, not solely due to locations of moment frames as in the steel framing systems.

Architectural Impact

As stated earlier, the addition of another column line is not anticipated to significantly affect the function of the Gouverneur Healthcare Services. However, concrete columns will be larger than steel columns, and it may be harder to incorporate these columns into walls without creating unusable space in rooms, or create significant obstructions in open areas.

POST-TENSIONED DISCUSSION

An attempt was made to take a brief look at a post-tensioned floor system in order to maintain the existing column layout with long spans and a thin slab. Issues arose almost immediately when conducting a preliminary equivalent frame analysis. Moments at the interior support of the 44ft span were determined to be two orders of magnitude greater than the moment at the supports of the shorter, 22ft span.

Another issue arose when calculating the pre-jacking stresses. Stresses created by tendons used to balance 75% of the self-weight were almost 50% higher than the allowable pre-jacking stress of 300psi. In order to reduce stresses to the allowable range, the slab would have to be thickened to over 15in, or the balancing moment would have to be significantly reduced, greatly increasing the required amount of mild steel. Both consequences eliminate two benefits to post-tensioned floor systems.

Other issues arose due to the fact that the Gouverneur Healthcare Services project is an addition to an existing building. It would normally be possible to post-tension only the 44 ft span in the East-West direction, and use mild steel reinforcing for the 22ft span. In order to implement this sort of design, tendons would have to be anchored in the slab, and jacked from the exterior. Unfortunately, this solution is not feasible because the side of the slab that would require the jacking is the interface between the addition and the existing building.

For these reasons, post-tensioned floor systems were not investigated further for the scope of this project. In order to perform an adequate analysis, a comprehensive redesign of the column layout would be necessary and a broad investigation into the impacts on the architecture would have to be carried out.

PRO/CONS AND COST SUMMARY

Existing Floor System

Pro:	Con:
-Cellular beams accommodate MEP	-Cellular beams are proprietary
-Allows the reduction in number of columns	-Moment Frames restrict MEP zones
Cost: $25/SF	

Composite Steel Framing Floor System

Pro:	Con:
-Standardized construction	-Too deep for adequate MEP space
-Low tonnage of steel	-Moment Frames restrict MEP zones
Cost: 23$/SF	

Hollow core Slab System

Pro:	Con:
-Fast construction	-Non-composite design can be inefficient
-Allows a reduction in number of columns	-Moment Frames restrict MEP zones
Cost: 27$/SF	

Two-way Flat Plate System

Pro:	Con:
-Unrestricted space between ceiling and slab	-Slower construction than steel
-Ease of construction	-Additional line of columns required
	-Heavier overall structure
Cost: $17/SF	

APPENDIX A LIST OF FIGURES

APPENDIX B COMPOSITE DESIGN RAM OUTPUT

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Gravity Beam Design

RAM Steel v11.2
DataBase: steel2 — 10/23/08 21:06:24
Building Code: IBC — Steel Code: ASD 9th Ed.

Floor Type: 2nd Floor **Beam Number = 76**

SPAN INFORMATION (ft): I-End (22.00,50.00) J-End (44.00,50.00)

Maximum Depth Limitation specified = 12.00 in
Beam Size (Optimum) = W10X19 Fy = 50.0 ksi
Total Beam Length (ft) = 22.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	4.25	4.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	USD 2" Lok-Floor	USD 2" Lok-Floor

beff (in)	=	66.00	Y bar(in)	=	12.63
Seff (in3)	=	38.64	Str (in3)	=	43.03
Ieff (in4)	=	446.40	Itr (in4)	=	523.96
Stud length (in)	=	5.00	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 8.6
\# of studs: Max = 22 Partial = 22 Actual = 22
Number of Stud Rows = 1 Percent of Full Composite Action = 67.01

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.760	0.000	1.000	3.5%	Red	0.000
	22.000	0.760	0.000	1.000			0.000
2	0.000	0.019	0.019	0.000	---	NonR	0.000
	22.000	0.019	0.019	0.000			0.000

SHEAR: Max V (DL+LL) = 19.19 kips fv = 7.52 ksi Fv = 20.00 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	1.2	11.0	0.0	1.00	0.74	33.00	0.74	33.00
	Max +	105.5	11.0	---	---				
	Mmax/Seff					32.77	33.00	---	---
	Mconst/Sx+Mpost/Seff					33.15	45.00	---	---
Controlling		105.5	11.0	---	---	32.77	33.00	---	---

fc (ksi) = 0.95 Fc = 1.80

REACTIONS (kips):

	Left	Right
Initial reaction	0.21	0.21
DL reaction	8.57	8.57
Max +LL reaction	10.62	10.62
Max +total reaction	19.19	19.19

DEFLECTIONS:

Initial load (in) at 11.00 ft = -0.036 L/D = 7315

Live load (in)	at	11.00 ft =	-0.393	L/D =	672
Post Comp load (in)	at	11.00 ft =	-0.702	L/D =	376
Net Total load (in)	at	11.00 ft =	-0.738	L/D =	357

Gravity Beam Design

RAM Steel v11.2
DataBase: steel2 — 10/23/08 21:06:24
Building Code: IBC — Steel Code: ASD 9th Ed.

Floor Type: 2nd Floor **Beam Number = 50**

SPAN INFORMATION (ft): I-End (44.00,30.00) J-End (44.00,60.00)

Maximum Depth Limitation specified = 14.00 in
Beam Size (Optimum) = W14X53 Fy = 50.0 ksi
Total Beam Length (ft) = 30.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	4.25	4.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	4.00	4.00
Decking Orientation	parallel	parallel
Decking type	USD 2" Lok-Floor	USD 2" Lok-Floor

beff (in)	=	90.00	Y bar(in)	=	14.60
Seff (in3)	=	126.69	Str (in3)	=	134.96
Ieff (in4)	=	1707.10	Itr (in4)	=	1904.30
Stud length (in)	=	5.00	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 11.4

# of studs per stud segment:	Full	=	35,1,35
	Partial	=	24,2,24
	Actual	=	24,2,24

Number of Stud Rows = 1 Percent of Full Composite Action = 70.56

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
10.000	8.57	0.21	11.00	24.4	0.00	0.00	0.0	0.00	Snow	0.00
10.000	8.57	0.21	11.00	24.4	0.00	0.00	0.0	0.00	Snow	0.00
20.000	8.57	0.21	11.00	24.4	0.00	0.00	0.0	0.00	Snow	0.00
20.000	8.57	0.21	11.00	24.4	0.00	0.00	0.0	0.00	Snow	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.053	0.053	0.000	---	NonR	0.000
	30.000	0.053	0.053	0.000			0.000

SHEAR: Max V (DL+LL) = 34.56 kips fv = 6.72 ksi Fv = 20.00 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp+	10.2	15.0	10.0	1.00	1.57	30.00	1.57	30.00
	Max +	343.6	15.0	---	---				
	Mmax/Seff					32.55	33.00	---	---
	Mconst/Sx+Mpost/Seff					33.15	45.00	---	---
Controlling		343.6	15.0	---	---	32.55	33.00	---	---

fc (ksi) = 1.19 Fc = 1.80

REACTIONS (kips):

	Left	Right
Initial reaction	1.22	1.22
DL reaction	17.94	17.94
Max +LL reaction	16.62	16.62
Max +total reaction	34.56	34.56

DEFLECTIONS:

Initial load (in)	at	15.00 ft	=	-0.106	L/D	=	3394
Live load (in)	at	15.00 ft	=	-0.556	L/D	=	647
Post Comp load (in)	at	15.00 ft	=	-1.115	L/D	=	323
Net Total load (in)	at	15.00 ft	=	-1.221	L/D	=	295

APPENDIX C HOLLOW CORE SLAB CALCULATIONS

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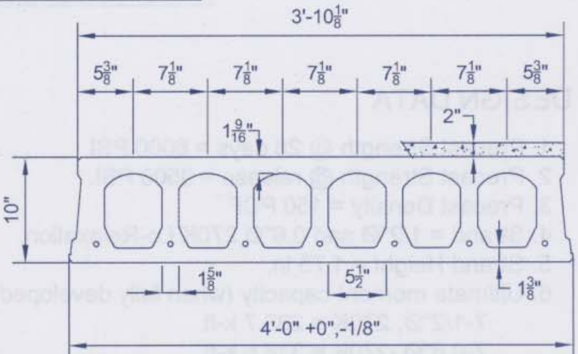
Prestressed Concrete
10"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast S_{bc} = 824 in.^{3}
I_c = 5102 in.^{4}	Topping S_{tc} = 1242 in.^{3}
Y_{bc} = 6.19 in.	Precast S_{tc} = 1340 in.^{3}
Y_{tc} = 3.81 in.	Wt.= 272 PLF
	Wt.= 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI or 4000 PSI.
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   7-1/2"Ø, 270K = 192.2 k-ft
   7-0.6"Ø, 270K = 256.4 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS												IBC 2003 & ACI 318-02 (1.2 D + 1.6 L)								
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
7 - 1/2"ø	LOAD (PSF)	234	210	189	[illegible]	153	[illegible]	123	110	98	87	77	68	60	52					
7 - 0.6"ø	LOAD (PSF)			256	[illegible]	[illegible]	[illegible]	202	185	168	154	140	128	116	106	96	87	78	70	63

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

05/14/07

10F2.0T

Smartbeam Design

RAM Steel v11.2
DataBase: hollowcore
Building Code: IBC

10/23/08 22:04:33
Smartbeam Code: ASD 9th Ed.

Floor Type: 2nd Floor **Beam Number = 123**

SPAN INFORMATION (ft): I-End (22.00,30.00) J-End (22.00,60.00)

Cellular
Maximum Depth Limitation specified = 27.00 in
Minimum Depth specified = 26.00 in
Beam Size (Optimum) = LB24x31 Fy = 50.0 ksi
Top: W16x31 Bottom: W16x31
Do = 20.750 in Smin = 23.750 in Smax = 25.375 in
Depth = 26.17 in
Connection Type Left: Web Right: Web
Total Beam Length (ft) = 30.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.031	0.000	---	NonR
	30.000	0.031	0.000		

SHEAR:

Gross:	Max V = 0.47 kips	fv = 0.07 ksi	Fv = 9.95 ksi	fv/Fv = 0.007
Net:	Max V = 0.43 kips at 1.25 ft			
		Top: fv = 0.30 ksi	Fv = 20.00 ksi	fv/Fv = 0.015
		Bot: fv = 0.30 ksi	Fv = 20.00 ksi	fv/Fv = 0.015
Horiz:	Max Vh = 0.39 kips at 2.10 ft			
		fv = 0.47 ksi	Fv = 20.00 ksi	fv/Fv = 0.023

WEB POST BUCKLING:

Max Vh = 0.39 kips at 2.10 ft

	Mmax kip-ft	Mallow kip-ft	Mmax/Mallow
Top:	0.30	6.78	0.044
Bot:	0.30	6.78	0.044

VIERENDEEL

Beam: V = 0.43 kips M = 0.56 kip-ft at 1.250 ft
Top Tee:

fa = 0.08 ksi	fb = 1.05 ksi	H1-1: 0.003 + 0.030 = 0.033
Fa = 29.45 ksi	Fb = 30.00 ksi	H1-2: 0.003 + 0.035 = 0.038

Beam: V = 0.43 kips M = 0.56 kip-ft at 1.250 ft
Bot Tee:

fa = 0.08 ksi	fb = 1.05 ksi	H1-1: 0.003 + 0.030 = 0.033
Fa = 29.45 ksi	Fb = 30.00 ksi	H1-2: 0.003 + 0.035 = 0.038

Smartbeam Design

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	Max +	3.5	15.0	30.0	1.00	0.57	30.00	0.57	3.08
Controlling		3.5	6.1	30.0	1.00	---	---	0.57	3.09

REACTIONS (kips):

	Left	**Right**
DL reaction	0.47	0.47
Max +total reaction	0.47	0.47

DEFLECTIONS:

Dead load (in)	at	15.00 ft	=	-0.029	L/D	=	12344
Live load (in)	at	15.00 ft	=	0.000			
Net Total load (in)	at	15.00 ft	=	-0.029	L/D	=	12344

Gravity Beam Design

RAM Steel v11.2
DataBase: hollowcore
Building Code: IBC

10/23/08 22:04:33
Steel Code: ASD 9th Ed.

Floor Type: 2nd Floor **Beam Number = 119**

SPAN INFORMATION (ft): I-End (22.00,60.00) J-End (66.00,60.00)

Maximum Depth Limitation specified = 27.00 in
Beam Size (Optimum) = W24X162 Fy = 50.0 ksi
Total Beam Length (ft) = 44.00

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	1.700	0.000	---	NonR
	44.000	1.700	0.000		
2	0.000	0.780	3.000	45.8%	Red
	44.000	0.780	3.000		
3	0.000	0.162	0.000	---	NonR
	44.000	0.162	0.000		

SHEAR: Max V (DL+LL) = 93.90 kips fv = 5.33 ksi Fv = 20.00 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	Max +	1032.9	22.0	0.0	1.00	29.94	33.00	29.94	33.00
Controlling		1032.9	22.0	0.0	1.00	29.94	33.00	---	---

REACTIONS (kips):

	Left	**Right**
DL reaction	58.13	58.13
Max +LL reaction	35.77	35.77
Max +total reaction	93.90	93.90

DEFLECTIONS: (Camber = 1)

Dead load (in)	at	22.00 ft	=	-1.486	L/D	=	355
Live load (in)	at	22.00 ft	=	-0.914	L/D	=	577
Net Total load (in)	at	22.00 ft	=	-1.401	L/D	=	377

APPENDIX D FLAT PLATE CALCULATIONS

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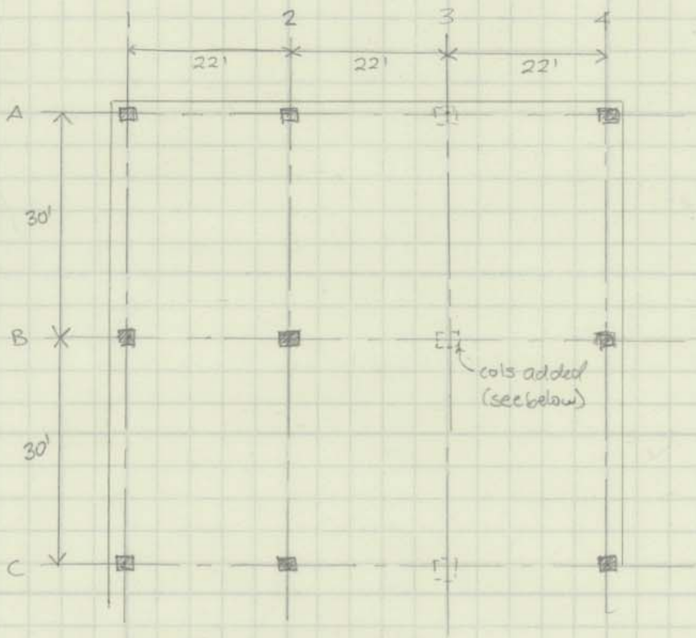
2-way Flat Plate floor system



$f'_c = 4000$ psi ?

$f_y = 60\,000$ psi

Representative Frames

- showing existing column locations
- extra columns anticipated in col line 3

- check slab thickness to determine feasability of 44' span
- Table 9.5(c) ACI 318 08

$t_s \geq \ell_n/33$ — need ℓ_n - use 44'

$\geq 44 \times 12/33 = 16''$ ← too thick, add cols. along col line 3

$\therefore$ typ bay = $22' \times 30'$

$t_s \geq \ell_n/33$

$\geq 30 \times 12/33 = 10.9''$ ← more reasonable, estimate col size to get more accurate ℓ_n

-Estimate col. size

superimposed DL

$$
\begin{aligned}
w_D = \; & 2 \quad \text{ceiling} \\
& 2 \quad \text{floor finish} \\
& 10 \quad \text{mech/elec.} \\
& \underline{12} \quad \text{partitions} \\
& 26 \text{ psf}
\end{aligned}
$$

slab weight assume 11"

$$w_D = (150\text{pcf})(\tfrac{11}{12})(1) = 138 \text{ psf}$$

$$A_{T_{floor}} = 22 \times 30 = 660 \text{ ft}^2$$

Live load

$$w_L = 100 \text{psf}$$

load combinations

$$1.4D = 1.4(26+138) = 230 \text{ psf}$$

$$1.2D + 1.6L = 1.2(26+138) + 1.6(100) = 357 \text{psf} \leftarrow \text{controls}$$

$$P_u = 5(357)(660)/_{1000} = 1178 \text{ k} \quad \Rightarrow \text{use} \quad f'_c = 5{,}950 \text{ psi} \Leftarrow$$

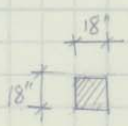
$$\phi P_n = 0.8\phi\,[0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$$

assume (10) #8 $\quad A_{st} = 10 \text{ in}^2$

$$\phi P_n \geq P_u$$

$$1178 = 0.8(0.65)[0.85(5.95)(A_g - 10) + 60(10)]$$

$A_g = 336 \text{ in}^2 \quad \therefore$ use $\underline{\underline{18" \times 18"}}$ col.

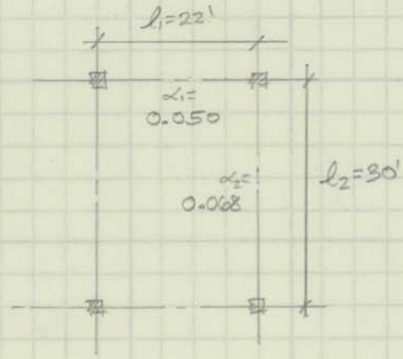


$$t_{slab} \leq \frac{l_n}{33}$$

$$\leq \frac{(30 \times 12) - 18}{33} = 10.36$$

$$\boxed{\therefore \text{ use } t_{slab} = 10.5"}$$

— Req'ts for Direct Design



1. – atleast three spans   ok ✓

2. $l_2/l_1 \leq 2$ $30/22 <$ ok ✓

3. $l_2 - l_1 < \frac{1}{3} l_2$

   $30 - 22 < \frac{1}{3}(30)$

   $8 < 10$ ok ✓

4. col. offset – none   ok ✓

5. $w_{LL} \leq 2 w_{DL}$ $100 < 2(138 + 26)$ ok ✓

6. Relative stiffness:

$$\alpha_1 = \frac{\Sigma I_{beam}}{\Sigma I_{slab}} = \frac{1736}{34729} = 0.050$$

α_1

$$I_{beam} = \frac{(18)(10.5)^3}{12} = 1736 \text{ in}^4$$

$$I_{slab} = \frac{(30 \times 12)(10.5)^3}{12} = 34729 \text{ in}^4$$

$$\alpha_2 = \frac{I_{beam}}{I_{slab}} = \frac{1736}{25468} = 0.068$$

α_2

$$I_{beam} = 1736 \text{ in}^4$$

$$I_{slab} = \frac{(22 \times 12)(10.5)^3}{12} = 25468 \text{ in}^4$$

$$= \quad 0.2 \leq \frac{l_1^2/\alpha_1}{l_2^2/\alpha_2} \leq 5$$

$$\frac{22^2/0.05}{30^2/0.068}$$

$$0.2 < 0.73 < 5 \quad \text{ok} ✓$$

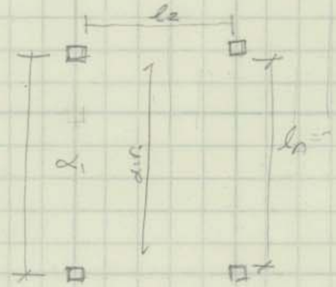
∴ use direct design method

Design interior bay using direct design method

long direction

ℓ_2 -perpendicular to ℓ_1

ℓ_1 - length in direction of moment



$$M_o = \frac{w_u \ell_2 \ell_n^2}{8}$$

$$= (.357)(22')(30 - \tfrac{18}{12})^2/8$$

$$= 797 \text{ ft-k}$$

From §13.6.3.2

$$M_u^- = 0.65 M_o$$

$$= 0.65(797) = \boxed{-518 \text{ ft-k}}$$

$$M_u^+ = 0.35 M_o$$

$$= 0.35(797) = \boxed{+279 \text{ ft-k}}$$

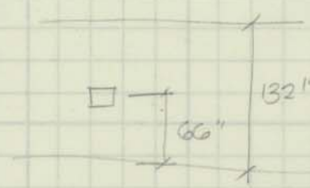
§ 13.6.4.1 % col. strip of interior negative moment

$\frac{\ell_2}{\ell_1} = \frac{22}{30} = 0.73 \leftarrow$ — or — assume $\alpha \approx 0$

$\alpha_{f1} \frac{\ell_2}{\ell_1} = 0.068(0.73)$
$= 0.05$

$\therefore$ % C.S $= \boxed{\begin{array}{ll} 0.75 \leftarrow & @\text{ negative} \\ 0.60 & @\text{ positive} \end{array}}$

		M_u^-	M_u^+
C.S.	M_{cs}	-389	+167
½ M.S.	M_{MS}	-129	+112



col stripwidth $a = \min \left| \begin{array}{l} \ell_1/4 \\ \ell_2/4 = \frac{22}{4} = 66'' \leftarrow \end{array} \right.$

(total = 132")

½ M.S. width = 66" $\leftarrow$

— Steel Reinforcing in long direction ACI §13.3.2

Max spacing $= 2t = 21'' \leftarrow$

$A_{s_{min}} = 0.0018\,bt$

Col Strip $A_{s_{min}} = 0.0018\,(132)(10.5)$
$= 2.49\ in^2$

½ M.S. $A_{s_{min}} = 0.0018\,(66)(10.5)$
$= 1.25\ in^2$

Assume #7 bars

$d_{long} = 10.5 - \tfrac{3}{4}'' - \tfrac{1}{2}(0.875)$ $b_{cs} = 132''$ $b_{ms} = 66$
$= 9.3125''$

	M^-_{cs}	M^-_{MS}	M^+_{cs}	M^+_{MS}	
M_n	-389	-129	+167	+112	
$M_u = \frac{M_n}{\phi}$	-432	-143	+186	+124	
$R = \frac{M_u}{bd^2}$	-453	-300	+195	+260	
ρ TABLE A5.a	0.008	.0052	.0035	.0045	
$A_s = \rho bd$	9.83 ←	3.20 ←	4.30 ←	2.76 ←	
$A_{min} = .002bt$	2.77	1.37	2.77	1.37	
$N = \frac{A_s}{A_b}$	(17) #7 ←	(7) #7 ←	(8) #7 ←	(5) #7 ←	
$N_{min} = \frac{width}{2t}$	6.3	3.14	6.3	3.14	
spacing	7.75''	9.43	16.5''	13.2''	ok ←

Short Direction

$$M_o = \frac{w_u l_2 l_n^2}{8} = \frac{(0.357)(30)(22-\frac{18}{12})^2}{8}$$

$$= 563 \text{ ft-k}$$

$M_n^- = 0.65M_o = 0.65(563)$

$\boxed{= -366 \text{ ft-k}}$

$M_n^+ = 0.35M_o = 0.35(563)$

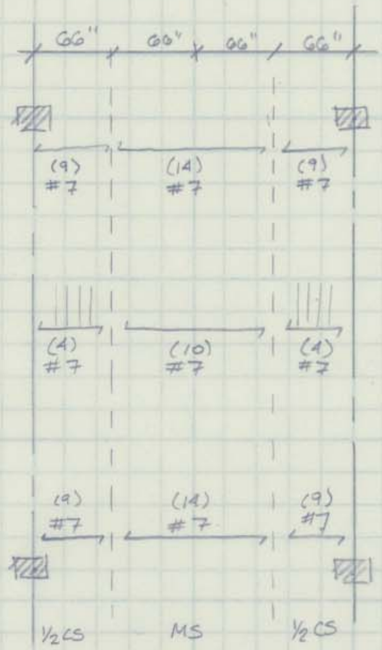
$\boxed{= +197 \text{ ft-k}}$

%CS = 0.75 @ negative   0.60 @ middle

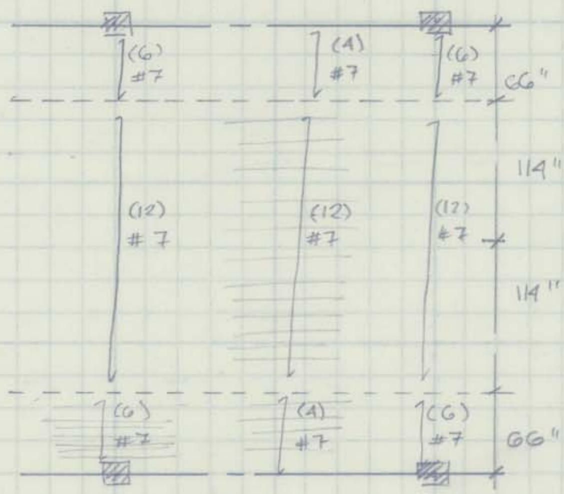
CS width = 132"   ½ MS width = 114"

assuming #7 $d_{short} = 10.5 - 3/4 - 1\frac{1}{2}(0.875)$
$= 8.44"$

	M_{cs}^-	M_{ms}^-	M_{cs}^+	M_{ms}^+
M_n	-275	-91	+118	+79
$M_u / \frac{M_n}{\phi}$	-306	-101	+131	+88
$R = \frac{M_u}{bd^2}$	391	149	167	130
ρ	0.007	0.0025	0.003	0.0023
$A_s = \rho bd$	7.80 ←	2.41 ←	3.34 ←	2.21
$A_s = 0.002bt$	2.77	2.39	2.77	2.39 ←
$N = \frac{A_s}{A_b}$	(13)#7 ←	(5)#7	(6)#7	(4)#7
$N_{min} = \frac{width}{2t}$	(7)#7	(6)#7 ←	(7)#7 ←	(6)#7 ←



LONG DIRECTION Reinforcing



Short Direction reinforcing

Check shear:

WIDE BEAM

$d = 9.31 + 8.44/2$

$= 8.88$

$w_u = 0.357$ ksf

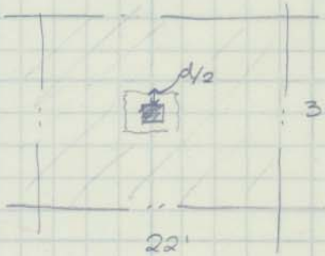
$V_u = 0.357(15 - \frac{1}{2}(\frac{18}{12}) - \frac{8.88}{12})(22)$

$= 195k$

$\phi V_n = \phi 2\sqrt{f'_c}\, b_w d = 0.75(2)\sqrt{5950}\,(30 \times 12)(8.88)$

$= 370\text{ k} > V_u \therefore$ OK

PUNCHING SHEAR



Perimeter of crit. section:

$b_o = (18 + d) \times 4 = (18 + 8.88)(4)$

$= 108$ in

Area loaded $= (22)(30) - [(18 + 8.88)/12]^2$

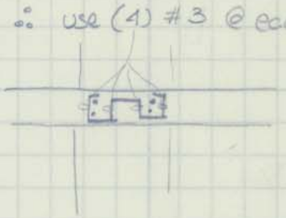
$= 655\text{ ft}^2$

$V_u = 0.357(655) = 234$ k

$b_o/d = 108/8.88 = 12.2$

eqt 1. $V_c = 4\sqrt{f'_c}\, b_o d = 4\sqrt{5950}\,(108)(8.88) = 296$ no good $\phi V_c < 234$

$\therefore$ use (4) #3 @ each face (16 total legs)



$V_n = V_c + V_s \leq 6\sqrt{f'_c}\, b_o d$

$\therefore V_u \leq \phi 6\sqrt{f'_c}\, b_o d$

$V_u = 234 \leq 0.75(6)\sqrt{5950}\,(108)(8.88)$

$V_u = 234k < 333k \therefore$ OK

$\phi V_n = 0.75(V_c + V_s)$

$= 0.75(2\sqrt{f'_c}\, b_o d + \frac{A_v f_y d}{s})$ $s = \frac{d}{2}$

$= 0.75[2\sqrt{5950}\,(108)(8.88) + 16(0.11)(60000)(2)]$

$= 269\text{ K} > V_u \therefore$ (2) double #3 U-stirrups are adequate @ each face

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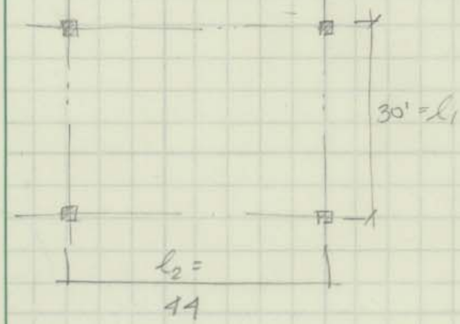
Post Tensioned - flat slab

slab thickness $\frac{l_n}{50} = \frac{44\times12-18}{50} = 10.2$ use 10.5" slab

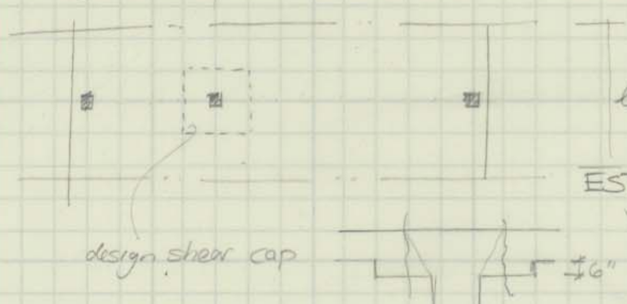


$\therefore$ w_u = 357 psf - same as previous.

f'_c = 5950 psi

- Req'ts for Direct Design 1. atleast three spans - fail
$\therefore$ use equivalent frame method.

- Long direction



ESTIMATE SHEARCAP

Assume mild steel 13 #5

$d = 16.5 - .75 - 0.5$
$= 15.25"$

1. $V_c = 4\sqrt{5950}\ b_o(15.25)$
$= 4.71\ b_o \Rightarrow \phi V_c = 3.53\ b_o$

2. $V_c = (2 + \frac{4}{1})\sqrt{5950}\ b_o(15.25)$
$= 7.1\ b_o$

$V_u = (0.357/144)\left[(30\times44\times144) - \left(\frac{b_o}{4}\right)^2\right]$

$V_u = 471 - 0.00016\ b_o^2$

$b_o \rightarrow 3.53 b_o = 471 - .00016\ b_o^2$

$b_o = 130$ in $\therefore$ $b_o/4 = 32.5$ width col $+d = 33"$

$\therefore$ 6" depth ok for prelim

-check punching for slab (design length of shear cap)

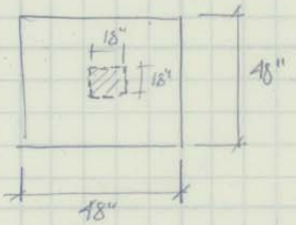
$d = 10.5 - .75 - 0.5$
$= 9.25''$

$V_u = 471 - 0.00016\, b_o^2$

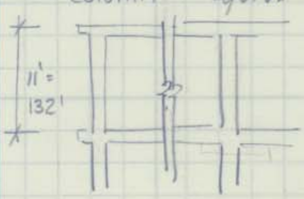
1. $V_c = 4\sqrt{5950}\, b_o(9.25) - 2.85\, b_o$

$2.85\, b_o = 471 - 0.00016\, b_o^2$

$b_o = 163''$ ∴ length of cap $= 40''$ ∴ use $48''$



Column Equiv. Stiffness — all columns the same



column $K_c = \dfrac{4E_cI_c}{L-2t}$ $\qquad I_c = bh^3/12 = \dfrac{18(18)^3}{12} = 8748\ in^4$

$= \dfrac{4E_c(8748)}{132-2(10.5)} = \boxed{315\, E_c}$

Slab torsional

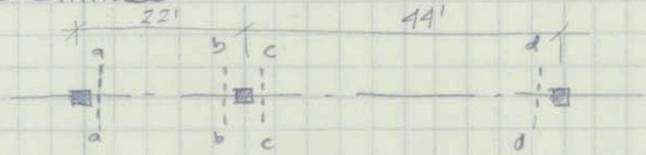
$C = \Sigma\left(1-0.63\dfrac{x}{y}\right)\left(\dfrac{x^3y}{3}\right) = \left(1-0.63\,\dfrac{10.5}{18}\right)\left(\dfrac{10.5^3\cdot 18}{3}\right)$
$= 4393$

$K_t = \Sigma \dfrac{9E_cC}{\ell_2\left(1-\frac{c_2}{\ell_2}\right)^3} = 2\,\dfrac{9E_c(4393)}{(30\times12)\left(1-\frac{18}{30\times12}\right)^3} = \boxed{256\, E_c}$

$\dfrac{1}{K_{ec}} = \dfrac{1}{\Sigma K_c} + \dfrac{1}{K_t} = \dfrac{1}{2(315E_c)} + \dfrac{1}{256E_c}$

$\boxed{K_{ec} = 182\, E_c}$ ← all columns

Slab Stiffness



$$I_s = \frac{(30\times12)(10.5)^3}{12} = 34700\ in^4$$

@ section a-a

$$K_s = \frac{4E_cI_s}{l_n - \frac{c_1}{2}} = \frac{4E_c(34700)}{(22\times12) - \frac{18}{2}} = 544\ E_c \longleftarrow$$

@ section b-b $K_s = 544E_c \longleftarrow$

@ section c-c & d-d

$$K_s = \frac{4E_c(34700)}{(44\times12) - 18/2} = 264\ E_c$$

DISTRIBUTION FACTOR

@ section a-a

$$DF = \frac{K_s}{\Sigma K} = \frac{544E_c}{544E_c + 182E_c} = 0.75 \longleftarrow$$

@ sectio b-b

$$DF = \frac{K_s}{\Sigma K} = \frac{544}{544+264+182} = 0.55 \longleftarrow$$

@ section c-c

$$DF = \frac{264}{544+264+182} = 0.27 \longleftarrow$$

@ section d-d

$$DF = \frac{264}{264+182} = 0.60 \longleftarrow$$

Fixed END moment

$$FEM\ a\text{-}a\ \&\ b\text{-}b = wl_2 l_n^2/12 = \frac{(0.357)(30)(22)^2}{12} = \underline{432\ ft\text{-}k}$$

$$FEM\ c\text{-}c\ \&\ d\text{-}d = wl_2 l_n^2/12 = \frac{(.357)(30)(44)^2}{12} = \underline{1728\ ft\text{-}k}$$

Equiv. Frame Short Direction (interior frame)

controls design of uniform tendons

COLUMN STIFNESS

$K_c = 315\,E_c$

$C = 4393$ $\quad K_t = \Sigma \dfrac{9E_c C}{\ell_2(1-\frac{c_2}{\ell_2})^3} = \dfrac{2 \times 9E_c(4393)}{(44\times12)(1-\frac{18}{44\times12})^3}$

$= 166\,E_c$

$$\frac{1}{K_{EC}} = \frac{1}{2K_c} + \frac{1}{K_t} = \frac{1}{2(315E_c)} + \frac{1}{166\,E_c}$$

$$K_{EC} = 131\,E_c$$

SLAB STIFFNESS

all sections the same I_s, ℓ_n, c_1

$$I_s = \frac{(33\times12)(10.5)^3}{12} = 38202\,in^4$$

$$K_s = \frac{4E_c I_s}{\ell_n - \frac{c_1}{2}} = \frac{4E_c(38202)}{(30\times12)-\frac{18}{2}}$$

$= 435\,E_c$

Distribution Factor

exterior $\quad D_F = \dfrac{K_s}{\Sigma K} = \dfrac{435}{435+131} = 0.77$

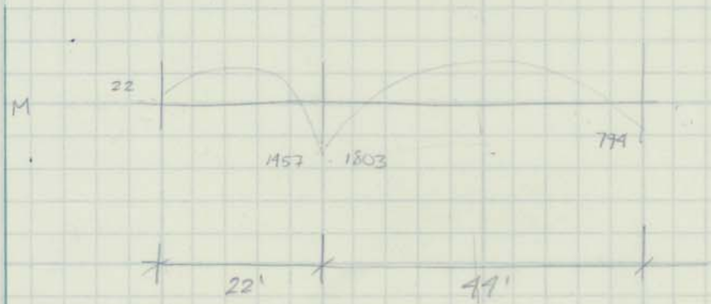
interior $\quad D_F = \dfrac{K_s}{\Sigma K} = \dfrac{435}{435+435+131} = 0.43$

FEM $\quad w\ell_2\ell_n^2/12 = \dfrac{(0.357)(33)(30)^2}{12} = 884$ ft-k

excel for moment distribution

Moment Dist.

D.F.	.75	.55	.27	0.60
FEM	-432	+432	-1728	+1728
	+324	+162	-518	-1036
Sum		(-1652)		
DIST.		+909	+446	+223
C.O.	+455			
DIST.	-341			-134
C.O		-171	-67	
Sum		(-238)		
DIST		+131	+64	
C.O.	+66			+32
DIST	-49.5			-19.2
C.O.		-25	-9.6	
		(-34.6)		
DIST		+19	+9.34	
Moment	22.5	+1457	-1803	+794



PT Design Parameters

Class U

$f'_{ci} = 3000\text{psi}$

Compression @ jacking $= 0.60 f'_{ci} = 0.6(3000) = 1{,}800\text{psi}$

Tension @ jacking $= 3\sqrt{f'_c} = 3\sqrt{3000} = 164\text{psi}$

AT service loads

$f'_c = 5950\text{ psi}$

Compression $= 0.45 f'_c = 0.45(5950) = 2680\text{ psi}$

Tension $= 6\sqrt{f'_c} = 6\sqrt{5950} = 463\text{psi}$

Average precompression limits

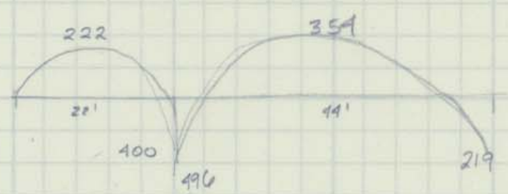
$P/A = 125\text{ psi min}$
300 psi max

Target load balance

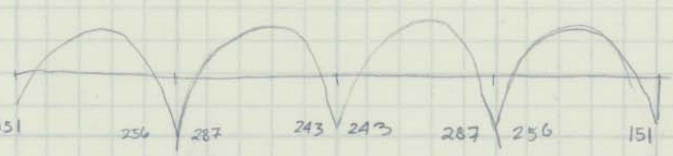
$0.75 w_{DC} = 0.75(\frac{10.5}{12} \cdot 150) = 98\text{psf}$

Long Direction

M due to $0.75 w_{DC}$



short direction



Force needed in tendons to counteract moment

long Direction $Pe = M$ assume $\left(e_{e_{max}} = \frac{t_{slab}}{2} - 1''\right)$

$P = 496 \text{ ft k} / (3.75/12) = 1590 \text{ k}$

Short Direction

$P = 287 / (3.75/12) = 918 \text{ k}$

Precompression Allowance

# of tendon in long dir.

$= 1590 \text{ k} / 26.6 \text{k/tendon} = 60$ ∴ use 60 tendons

tendons in short dir.

$= 918 / 26.6 = 34.5$ ∴ use 35 tendons

actual force in long dir.

$= 1590 \text{ k}$

actual force in short dir

$(35 \times 26.6) = 931 \text{ k}$

$1590 / (30 \times 12)(10.5) = 420 >> 300$

need 15" slab to work?

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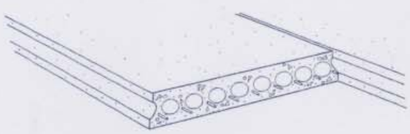
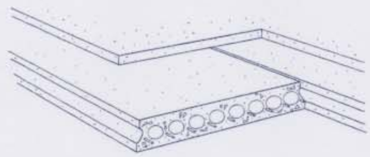
B10 Superstructure

B1010 Floor Construction



Precast Plank with No Topping



Precast Plank with 2″ Concrete Topping

B1010 229 — Precast Plank with No Topping

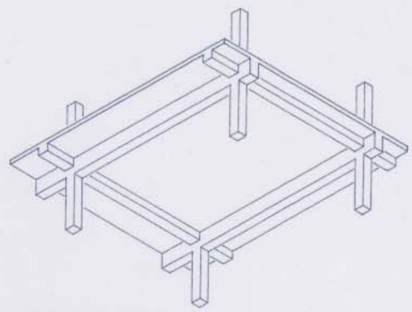
	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
0720	10	40	4	50	90	5.90	3.06	8.96
0750		75	6	50	125	7.25	2.63	9.88
0770		100	6	50	150	7.25	2.63	9.88
0800	15	40	6	50	90	7.25	2.63	9.88
0820		75	6	50	125	7.25	2.63	9.88
0850		100	6	50	150	7.25	2.63	9.88
0950	25	40	6	50	90	7.25	2.63	9.88
0970		75	8	55	130	7.90	2.30	10.20
1000		100	8	55	155	7.90	2.30	10.20
1200	30	40	8	55	95	7.90	2.30	10.20
1300		75	8	55	130	7.90	2.30	10.20
1400		100	10	70	170	8.45	2.05	10.50
1500	40	40	10	70	110	8.45	2.05	10.50
1600		75	12	70	145	9.70	1.84	11.54
1700	45	40	12	70	110	9.70	1.84	11.54

B1010 230 — Precast Plank with 2″ Concrete Topping

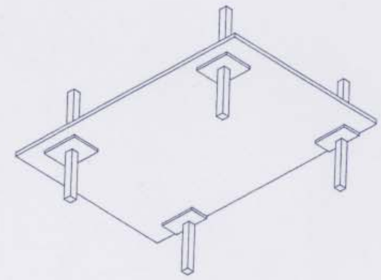
	SPAN (FT.)	SUPERIMPOSED LOAD (P.S.F.)	TOTAL DEPTH (IN.)	DEAD LOAD (P.S.F.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2000	10	40	6	75	115	6.80	5.05	11.85
2100		75	8	75	150	8.15	4.61	12.76
2200		100	8	75	175	8.15	4.61	12.76
2500	15	40	8	75	115	8.15	4.61	12.76
2600		75	8	75	150	8.15	4.61	12.76
2700		100	8	75	175	8.15	4.61	12.76
3100	25	40	8	75	115	8.15	4.61	12.76
3200		75	8	75	150	8.15	4.61	12.76
3300		100	10	80	180	8.80	4.28	13.08
3400	30	40	10	80	120	8.80	4.28	13.08
3500		75	10	80	155	8.80	4.28	13.08
3600		100	10	80	180	8.80	4.28	13.08
4000	40	40	12	95	135	9.35	4.03	13.38
4500		75	14	95	170	10.60	3.82	14.42
5000	45	40	14	95	135	10.60	3.82	14.42

264

B1010 Floor Construction



General: Solid concrete two way slab cast monolithically with reinforced concrete support beams and girders.



General: Flat Slab: Solid uniform depth concrete two way slabs with drop panels at columns and no column capitals.

B1010 220 Cast in Place Beam & Slab, Two Way

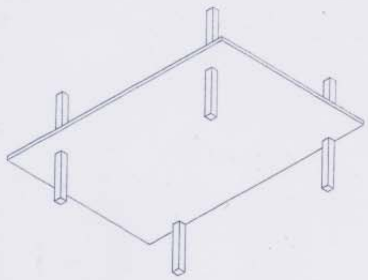
	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB THICKNESS (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
4000	20 x 25	40	12	7	141	6.05	9.60	15.65
4300		75	14	7	181	6.90	10.50	17.40
4500		125	16	7	236	7	10.80	17.80
5100	25 x 25	40	12	7-1/2	149	6.30	9.70	16
5200		75	16	7-1/2	185	6.85	10.40	17.25
5300		125	18	7-1/2	250	7.45	11.30	18.75
7600	30 x 35	40	16	10	188	8.30	11	19.30
7700		75	18	10	225	8.80	11.40	20.20
8000		125	22	10	282	9.75	12.35	22.10
8500	35 x 35	40	16	10-1/2	193	8.90	11.25	20.15
8600		75	20	10-1/2	233	9.35	11.70	21.05
9000		125	24	10-1/2	287	10.40	12.55	22.95

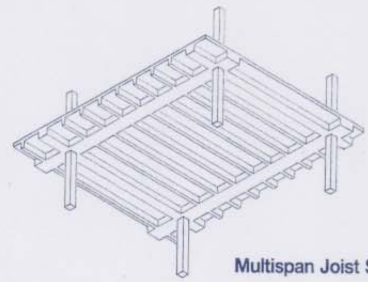
B1010 222 Cast in Place Flat Slab with Drop Panels

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB & DROP (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
1960	20 x 20	40	12	7 - 3	132	5.35	7.65	13
1980		75	16	7 - 4	168	5.65	7.85	13.50
2000		125	18	7 - 6	221	6.25	8.15	14.40
3200	25 x 25	40	12	8-1/2 - 5-1/2	154	6.25	8.05	14.30
4000		125	20	8-1/2 - 8-1/2	243	7.05	8.60	15.65
4400		200	24	9 - 8-1/2	329	7.40	8.85	16.25
5000	25 x 30	40	14	9-1/2 - 7	168	6.80	8.35	15.15
5200		75	18	9-1/2 - 7	203	7.25	8.70	15.95
5600		125	22	9-1/2 - 8	256	7.60	8.90	16.50
6400	30 x 30	40	14	10-1/2 - 7-1/2	182	7.35	8.55	15.90
6600		75	18	10-1/2 - 7-1/2	217	7.80	8.90	16.70
6800		125	22	10-1/2 - 9	269	8.20	9.15	17.35
7400	30 x 35	40	16	11-1/2 - 9	196	8	8.90	16.90
7900		75	20	11-1/2 - 9	231	8.55	9.30	17.85
8000		125	24	11-1/2 - 11	284	8.90	9.50	18.40
9000	35 x 35	40	16	12 - 9	202	8.25	9	17.25
9400		75	20	12 - 11	240	8.85	9.45	18.30
9600		125	24	12 - 11	290	9.15	9.60	18.75

Scott M. Rabold

B1010 Floor Construction





Multispan Joist Slab

General: Flat Plates: Solid uniform depth concrete two way slab without drops or interior beams. Primary design limit is shear at columns.

General: Combination of thin concrete slab and monolithic ribs at uniform spacing to reduce dead weight and increase rigidity.

B1010 223 Cast in Place Flat Plate

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	SLAB THICKNESS (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
3000	15 x 20	40	14	7	127	5.05	7.35	12.40
3400		75	16	7-1/2	169	5.35	7.55	12.90
3600		125	22	8-1/2	231	5.90	7.75	13.65
3800		175	24	8-1/2	281	5.95	7.75	13.70
4200	20 x 20	40	16	7	127	5.05	7.35	12.40
4400		75	20	7-1/2	175	5.40	7.55	12.95
4600		125	24	8-1/2	231	5.90	7.70	13.60
5000		175	24	8-1/2	281	5.95	7.75	13.70
5600	20 x 25	40	18	8-1/2	146	5.85	7.70	13.55
6000		75	20	9	188	6.10	7.80	13.90
6400		125	26	9-1/2	244	6.55	8.05	14.60
6600		175	30	10	300	6.80	8.15	14.95
7000	25 x 25	40	20	9	152	6.05	7.80	13.85
7400		75	24	9-1/2	194	6.45	7.95	14.40
7600		125	30	10	250	6.80	8.20	15

B1010 226 Cast in Place Multispan Joist Slab

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	MINIMUM COL. SIZE (IN.)	RIB DEPTH (IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2000	15 x 15	40	12	8	115	6.05	9	15.05
2100		75	12	8	150	6.05	9.05	15.10
2200		125	12	8	200	6.20	9.15	15.35
2300		200	14	8	275	6.40	9.50	15.90
2600	15 x 20	40	12	8	115	6.15	9	15.15
2800		75	12	8	150	6.30	9.50	15.80
3000		125	14	8	200	6.50	9.65	16.15
3300		200	16	8	275	6.80	9.80	16.60
3600	20 x 20	40	12	10	120	6.30	8.90	15.20
3900		75	14	10	155	6.55	9.40	15.95
4000		125	16	10	205	6.60	9.55	16.15
4100		200	18	10	280	6.95	10	16.95
6200	30 x 30	40	14	14	131	7	9.30	16.30
6400		75	18	14	166	7.20	9.60	16.80
6600		125	20	14	216	7.65	10.15	17.80
6700		200	24	16	297	8.20	10.55	18.75

05 12 Structural Steel Framing

05 12 23 – Structural Steel for Buildings

05 12 23.77 Structural Steel Projects

	05 12 23.77 Structural Steel Projects	Crew	Daily Output	Labor-Hours	Unit	Material	2008 Bare Costs Labor	Equipment	Total	Total Incl O&P
4700	Maximum	E-2	7	8	Ton	2,525	335	224	3,084	3,625
4900	Heavy sections, over 50# per L.F., minimum		11.70	4.786		2,425	200	134	2,759	3,150
5000	Maximum	↓	7.80	7.179		2,650	300	201	3,151	3,650
5390	For projects 75 to 99 tons, add					10%				
5392	50 to 74 tons, add					20%				
5394	25 to 49 tons, add					30%	10%			
5396	10 to 24 tons, add					50%	25%			
5398	2 to 9 tons, add					75%	50%			
5399	Less than 2 tons, add				↓	100%	100%			

05 12 23.80 Subpurlins

	05 12 23.80 Subpurlins	Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P
0010	**SUBPURLINS** R051223-50									
0020	Bulb tees, shop fabricated, painted, 32-5/8" O.C., 40 psf L.L.									
0100	Type 178, max 8'-9" span, 2.15 plf, 2" high x 1-5/8" wide	E-1	4200	.006	S.F.	1.53	.24	.03	1.80	2.14
0200	Type 218, max 10'-2" span, 3.19 plf, 2-1/8" high x 2-1/8" wide	"	3100	.008		1.77	.32	.04	2.13	2.58
1420	For 24-5/8" spacing, add					33%	33%			
1430	For 48-5/8" spacing, deduct				↓	50%	50%			

05 14 Structural Aluminum Framing

05 14 23 – Non-Exposed Structural Aluminum Framing

05 14 23.05 Aluminum Shapes

	05 14 23.05 Aluminum Shapes	Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P
0010	**ALUMINUM SHAPES**									
0020	Structural shapes, 1" to 10" members, under 1 ton	E-2	1050	.053	Lb.	2.66	2.23	1.49	6.38	8.60
0050	1 to 5 tons		1330	.042		2.54	1.76	1.18	5.48	7.25
0100	Over 5 tons		1330	.042		2.45	1.76	1.18	5.39	7.15
0300	Extrusions, over 5 tons, stock shapes		1330	.042		2.65	1.76	1.18	5.59	7.40
0400	Custom shapes	↓	1330	.042	↓	2.70	1.76	1.18	5.64	7.45

05 15 Wire Rope Assemblies

05 15 16 – Steel Wire Rope Assemblies

05 15 16.05 Accessories for Steel Wire Rope

	05 15 16.05 Accessories for Steel Wire Rope	Crew	Daily Output	Labor-Hours	Unit	Material	Labor	Equipment	Total	Total Incl O&P
0010	**ACCESSORIES FOR STEEL WIRE ROPE**									
1500	Thimbles, heavy duty, 1/4"	E-17	160	.100	Ea.	.60	4.40		5	8.90
1510	1/2"		160	.100		2.64	4.40		7.04	11.15
1520	3/4"		105	.152		6	6.70		12.70	19.15
1530	1"		52	.308		12	13.55		25.55	38.50
1540	1-1/4"		38	.421		18.45	18.55		37	55
1550	1-1/2"		13	1.231		52	54		106	159
1560	1-3/4"		8	2		107	88		195	283
1570	2"		6	2.667		156	117		273	390
1580	2-1/4"		4	4		211	176		387	560
1600	Clips, 1/4" diameter		160	.100		2.70	4.40		7.10	11.20
1610	3/8" diameter		160	.100		2.96	4.40		7.36	11.50
1620	1/2" diameter		160	.100		4.76	4.40		9.16	13.50
1630	3/4" diameter		102	.157		7.70	6.90		14.60	21.50
1640	1" diameter		64	.250		12.85	11		23.85	34.50
1650	1-1/4" diameter		35	.457		21	20		41	60.50
1670	1-1/2" diameter		26	.615		28.50	27		55.50	82.50
1680	1-3/4" diameter		16	1		66	44		110	156
1690	2" diameter	↓	12	1.333	↓	73.50	58.50		132	191

05 12 23 – Structural Steel for Buildings

05 12 23.75 Structural Steel Members

		Crew	Daily Output	Labor-Hours	Unit	2008 Bare Costs Material	Labor	Equipment	Total	Total Incl O&P
0010	STRUCTURAL STEEL MEMBERS R051223-10									
0020	Shop fab'd for 100-ton, 1-2 story project, bolted connections									
0102	W 6 x 9 R051223-15	E-2	600	.093	L.F.	10.90	3.91	2.61	17.42	22
0302	W 8 x 10		600	.093		12.10	3.91	2.61	18.62	23
0502	x 31		550	.102		37.50	4.26	2.85	44.61	52.50
0702	W 10 x 22		600	.093		26.50	3.91	2.61	33.02	39.50
0902	x 49		550	.102		59.50	4.26	2.85	66.61	76
1102	W 12 x 14		880	.064		16.95	2.66	1.78	21.39	25.50
1302	x 22		880	.064		26.50	2.66	1.78	30.94	36.50
1502	x 26		880	.064		31.50	2.66	1.78	35.94	41.50
1702	x 72		640	.088		87	3.66	2.45	93.11	105
1902	W 14 x 26		990	.057		31.50	2.37	1.58	35.45	40.50
2102	x 30		900	.062		36.50	2.60	1.74	40.84	46.50
2302	x 34		810	.069		41	2.89	1.93	45.82	53
2502	x 120		720	.078		145	3.26	2.18	150.44	168
2702	W 16 x 26		1000	.056		31.50	2.34	1.57	35.41	40.50
2902	x 31		900	.062		37.50	2.60	1.74	41.84	48
3102	x 40		800	.070		48.50	2.93	1.96	53.39	60.50
3302	W 18 x 35	E-5	960	.083		42.50	3.53	1.77	47.80	55
3502	x 40		960	.083		48.50	3.53	1.77	53.80	61.50
3702	x 50		912	.088		60.50	3.72	1.86	66.08	75.50
3902	x 55		912	.088		66.50	3.72	1.86	72.08	82
4102	W 21 x 44		1064	.075		53	3.19	1.60	57.79	66
4302	x 50		1064	.075		60.50	3.19	1.60	65.29	74
4502	x 62		1036	.077		75	3.27	1.64	79.91	90.50
4702	x 68		1036	.077		82.50	3.27	1.64	87.41	98.50
4902	W 24 x 55		1110	.072		66.50	3.06	1.53	71.09	80
5102	x 62		1110	.072		75	3.06	1.53	79.59	89.50
5302	x 68		1110	.072		82.50	3.06	1.53	87.09	97.50
5502	x 76		1110	.072		92	3.06	1.53	96.59	108
5702	x 84		1080	.074		102	3.14	1.57	106.71	119
5902	W 27 x 94		1190	.067		114	2.85	1.43	118.28	132
6102	W 30 x 99		1200	.067		120	2.83	1.42	124.25	139
6302	x 108		1200	.067		131	2.83	1.42	135.25	151
6502	x 116		1160	.069		140	2.93	1.46	144.39	161
6702	W 33 x 118		1176	.068		143	2.89	1.45	147.34	164
6902	x 130		1134	.071		157	2.99	1.50	161.49	180
7102	x 141		1134	.071		171	2.99	1.50	175.49	195
7302	W 36 x 135		1170	.068		163	2.90	1.45	167.35	187
7502	x 150		1170	.068		182	2.90	1.45	186.35	207
7702	x 194		1125	.071		235	3.02	1.51	239.53	265
7902	x 230		1125	.071		278	3.02	1.51	282.53	310
8102	x 300		1035	.077		365	3.28	1.64	369.92	410
8490	For projects 75 to 99 tons, add					10%				
8492	50 to 74 tons, add					20%				
8494	25 to 49 tons, add					30%	10%			
8496	10 to 24 tons, add					50%	25%			
8498	2 to 9 tons, add					75%	50%			
8499	Less than 2 tons, add					100%	100%			
9000	Minimum labor/equipment charge	E-2	2	28	Job		1,175	785	1,960	2,950

05 12 23.77 Structural Steel Projects

0010	STRUCTURAL STEEL PROJECTS R050516-30									

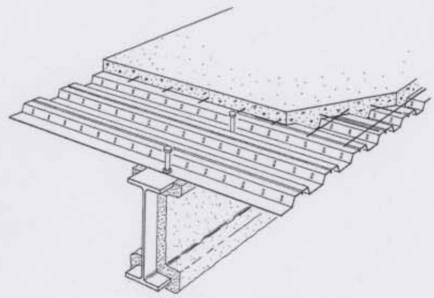
05 1
05 12 2
05 12 23.
0020 0200 0300 0400 0500 0700 0800 0900 1000 1100 1300 1400 1500 1510 1600 1700 1900 2000 2200 2300 2800 2900 2950 3000 3040 3070 3100 3200 3210 3220 3400 3500 3700 3800 3810 3820 3830 3840 3850 3860 3870 3880 3900 3950 4000 4100 4200 4210 4220 4230 4300 4400 4600

05 12 Structural Steel Framing

05 12 23 – Structural Steel for Buildings

05 12 23.77 Structural Steel Projects			Crew	Daily Output	Labor-Hours	Unit	2008 Bare Costs Material	Labor	Equipment	Total	Total Incl O&P
0020	Shop fab'd for 100-ton, 1-2 story project, bolted connections										
0200	Apartments, nursing homes, etc., 1 to 2 stories	R050523-10	E-5	10.30	7.767	Ton	2,200	330	165	2,695	3,200
0300	3 to 6 stories		"	10.10	7.921		2,250	335	168	2,753	3,275
0400	7 to 15 stories	R051223-10	E-6	14.20	9.014		2,300	380	132	2,812	3,375
0500	Over 15 stories		"	13.90	9.209		2,375	390	134	2,899	3,500
0700	Offices, hospitals, etc., steel bearing, 1 to 2 stories	R051223-15	E-5	10.30	7.767		2,200	330	165	2,695	3,200
0800	3 to 6 stories		E-6	14.40	8.889		2,250	375	130	2,755	3,300
0900	7 to 15 stories	R051223-20		14.20	9.014		2,300	380	132	2,812	3,375
1000	Over 15 stories			13.90	9.209		2,375	390	134	2,899	3,500
1100	For multi-story masonry wall bearing construction, add	R051223-25						30%			
1300	Industrial bldgs., 1 story, beams & girders, steel bearing		E-5	12.90	6.202		2,200	263	132	2,595	3,050
1400	Masonry bearing		"	10	8		2,200	340	170	2,710	3,225
1500	Industrial bldgs., 1 story, under 10 tons,										
1510	steel from warehouse, trucked		E-2	7.50	7.467	Ton	2,650	315	209	3,174	3,700
1600	1 story with roof trusses, steel bearing		E-5	10.60	7.547		2,600	320	160	3,080	3,600
1700	Masonry bearing		"	8.30	9.639		2,600	410	205	3,215	3,825
1900	Monumental structures, banks, stores, etc., minimum		E-6	13	9.846		2,200	420	144	2,764	3,350
2000	Maximum		"	9	14.222		3,650	605	208	4,463	5,350
2200	Churches, minimum		E-5	11.60	6.897		2,050	292	146	2,488	2,950
2300	Maximum		"	5.20	15.385		2,725	650	325	3,700	4,550
2800	Power stations, fossil fuels, minimum		E-6	11	11.636		2,200	495	170	2,865	3,500
2900	Maximum			5.70	22.456		3,300	955	330	4,585	5,725
2950	Nuclear fuels, non-safety steel, minimum			7	18.286		2,200	775	267	3,242	4,150
3000	Maximum			5.50	23.273		3,300	985	340	4,625	5,800
3040	Safety steel, minimum			2.50	51.200		3,200	2,175	745	6,120	8,325
3070	Maximum			1.50	85.333		4,225	3,625	1,250	9,100	12,600
3100	Roof trusses, minimum		E-5	13	6.154		3,075	261	131	3,467	4,025
3200	Maximum			8.30	9.639		3,750	410	205	4,365	5,100
3210	Schools, minimum			14.50	5.517		2,200	234	117	2,551	2,975
3220	Maximum			8.30	9.639		3,200	410	205	3,815	4,500
3400	Welded construction, simple commercial bldgs., 1 to 2 stories		E-7	7.60	10.526		2,250	445	241	2,936	3,550
3500	7 to 15 stories		E-9	8.30	15.422		2,600	655	268	3,523	4,350
3700	Welded rigid frame, 1 story, minimum		E-7	15.80	5.063		2,300	215	116	2,631	3,050
3800	Maximum		"	5.50	14.545		2,975	615	335	3,925	4,775
3810	Fabrication shop costs (included in project material cost, above)										
3820	Mini mill base price, A992					Ton	725			725	800
3830	Mill extra for delivery to shop						220			220	242
3840	Shop extra for shop drawings and detailing						240			240	264
3850	Shop fabricating and handling						800			800	880
3860	Shop sandblasting and primer coat of paint						125			125	138
3870	Shop delivery to the job site						90			90	99
3880	Total material cost, shop fabricated, primed, delivered						2,200			2,200	2,425
3900	High strength steel mill spec extras: A242, A441,										
3950	A529, A572 (42 ksi) and A992: same as A36 steel										
4000	Add to A992 price for A572 (50, 60, 65 ksi)					Ton	100			100	110
4100	A588 Weathering					"	92.50			92.50	102
4200	Mill size extras for W-Shapes: 0 to 30 plf: no extra charge										
4210	Member sizes 31 to 65 plf, add					Ton	.10			.10	.11
4220	Member sizes 66 to 100 plf, add						.10			.10	.11
4230	Member sizes 101 to 387 plf, add						56			56	61.50
4300	Column base plates, light, up to 150 lb		2 Sswk	2000	.008	Lb.	1.21	.34		1.55	1.97
4400	Heavy, over 150 lb		E-2	7500	.007	"	1.27	.31	.21	1.79	2.18
4600	Castellated beams, light sections, to 50#/L.F., minimum			10.70	5.234	Ton	2,300	219	146	2,665	3,100

B1010 Floor Construction



Composite Beam, Deck & Slab

The table below lists costs per S.F. for floors using composite steel beams with welded shear studs, composite steel deck and light weight concrete slab reinforced with W.W.F.

B1010 256 Composite Beams, Deck & Slab

	BAY SIZE (FT.)	SUPERIMPOSED LOAD (P.S.F.)	SLAB THICKNESS (IN.)	TOTAL DEPTH (FT.-IN.)	TOTAL LOAD (P.S.F.)	COST PER S.F.		
						MAT.	INST.	TOTAL
2400	20x25	40	5-1/2	1 - 5-1/2	80	10.60	5.30	15.90
2500		75	5-1/2	1 - 9-1/2	115	11	5.35	16.35
2750		125	5-1/2	1 - 9-1/2	167	13.40	6.25	19.65
2900		200	6-1/4	1 - 11-1/2	251	15.15	6.75	21.90
3000	25x25	40	5-1/2	1 - 9-1/2	82	10.40	5.05	15.45
3100		75	5-1/2	1 - 11-1/2	118	11.55	5.15	16.70
3200		125	5-1/2	2 - 2-1/2	169	12.10	5.55	17.65
3300		200	6-1/4	2 - 6-1/4	252	16.30	6.50	22.80
3400	25x30	40	5-1/2	1 - 11-1/2	83	10.65	5	15.65
3600		75	5-1/2	1 - 11-1/2	119	11.45	5.10	16.55
3900		125	5-1/2	1 - 11-1/2	170	13.25	5.75	19
4000		200	6-1/4	2 - 6-1/4	252	16.35	6.55	22.90
4200	30x30	40	5-1/2	1 - 11-1/2	81	10.70	5.20	15.90
4400		75	5-1/2	2 - 2-1/2	116	11.55	5.40	16.95
4500		125	5-1/2	2 - 5-1/2	168	13.95	6.10	20.05
4700		200	6-1/4	2 - 9-1/4	252	16.75	7.05	23.80
4900	30x35	40	5-1/2	2 - 2-1/2	82	11.20	5.35	16.55
5100		75	5-1/2	2 - 5-1/2	117	12.25	5.50	17.75
5300		125	5-1/2	2 - 5-1/2	169	14.35	6.20	20.55
5500		200	6-1/4	2 - 9-1/4	254	16.90	7.05	23.95
5750	35x35	40	5-1/2	2 - 5-1/2	84	11.90	5.40	17.30
6000		75	5-1/2	2 - 5-1/2	121	13.55	5.75	19.30
7000		125	5-1/2	2 - 8-1/2	170	15.90	6.60	22.50
7200		200	5-1/2	2 - 11-1/2	254	18.15	7.30	25.45
7400	35x40	40	5-1/2	2 - 5-1/2	85	13.15	5.80	18.95
7600		75	5-1/2	2 - 5-1/2	121	14.25	6	20.25
8000		125	5-1/2	2 - 5-1/2	171	16.30	6.70	23
9000		200	5-1/2	2 - 11-1/2	255	19.70	7.60	27.30

277

Steel Tonnage

Cellular Beam

4	LB	27	x	35	@	22	ft	=	3080
4	LB	28	x	43	@	44	ft	=	7568
1	LB	27	x	35	@	30	ft	=	1050
1	LB	27	x	106	@	30	ft	=	3180
1	LB	27	x	55	@	30	ft	=	1650
325	studs	@		10	lb			=	3250

9.889 tons

Composite Steel Framing

12	W	10	x	19	@	22	ft	=	5016
2	W	14	x	53	@	30	ft	=	3180
2	W	16	x	31	@	30	ft	=	1860
428	studs	@		10	lb			=	4280

7.168 tons

Hollowcore Plank

2	W	24	55	19	@	22	ft	=	836
2	W	24	x	162	@	44	ft	=	14256
2	W	8	x	24	@	30	ft	=	1440
1	LB	24	x	31	@	30	ft	=	930

8.731 tons