Butler Memorial Hospital

Butler, PA

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Technical Report #2

Floor System Alternatives

Structural Option

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Executive Summary:

In this second technical report for The Butler Health System – New Inpatient Tower Addition and Remodel the existing floor system and three alternative system designs are investigated and analyzed. These four systems are then compared side-by-side with multiple general conditions and criteria to determine which systems would be a good fit for the structure and which ones deserve further consideration and analysis.

The floor systems proposed are:
1. Existing- composite steel beam with lightweight concrete
2. Non-composite steel beam construction with one additional beam per bay
3. Concrete two-way flat-slab with drop panels
4. Precast hollow plank concrete slabs on steel beams

A final criteria summary chart is depicted in Figure 2.25 which lists fourteen different aspects. Final conclusions show that the non-composite steel system has no merit to be considered as an alternative design for this type of structure. The final conclusions also show that the two way flat-slab with drop panels is a good and viable alternative solution on a general basis. Floor system four is a fair alternative for the existing one.

Systems 3& 4 as listed above both deserve a further analysis with the flat slab being the better choice.

Introduction:

Butler Health System's new addition consists of two sub grade levels which have limited facade and entrances at ground level on the plan west end of the structure. There are five other at or above grade levels that comprise the bulk of the hospitals general facilities. One more final level, the penthouse level, encompasses the mechanical equipment on the roof top.

The structure is approximately 206,000 square feet with floor to floor heights of 14'-8" each. It stands at just a little over 100' tall above the highest grade level and is situated on the middle of a hillside. With the exception of the slightly arcing plan north facade the floor plan is quite regular with typical bay sizes being 28' x30'.

Drilled caissons were used for the foundation system which range from 30" – 78" in diameter and reach depths of up to 79'. Grade beams between the caissons on the below grade level areas transfer wall loads to the foundation system and provide interior perimeter walls for the lower levels as well as provide support for the slab on grade at the second level. The superstructure is composed of steel W-shape members with a steel HSS lateral bracing system. Almost all member connections are shear connections with the exception of a few moment connections at cantilevering beams. These moment connections however do not contribute to the lateral force resisting system.

This report examines the existing floor construction system and three alternative design methods to determine each systems viability and possible implementation into the structure. Factors for this analytical review are system weight, depths, costs, fire ratings, foundation impacts, constructability, changes to the lateral force resisting system, vibration concerns and if each system warrants future consideration for a thesis proposal.

All calculations and designs are purely schematic and are only taking into account a typical bay and therefore are not an exhaustive analysis for each type of floor system.

Structural System:

Existing System:

Existing conditions for the originally designed floor system consists of composite steel decking with lightweight concrete (f'c = 3500psi @28 days). It has 20 gauge steel decking with 3” deep flutes, ¾” diameter 5” long shear studs and an additional 3.5” of concrete. The girders supporting the beams and floor system are typically W21x50, 28' long with 38 shear studs. There are typically four beams per bay including the ones at each column line. The beams are W18x40 evenly spaced at ten foot intervals and are 30 feet long with 28 shear studs each.

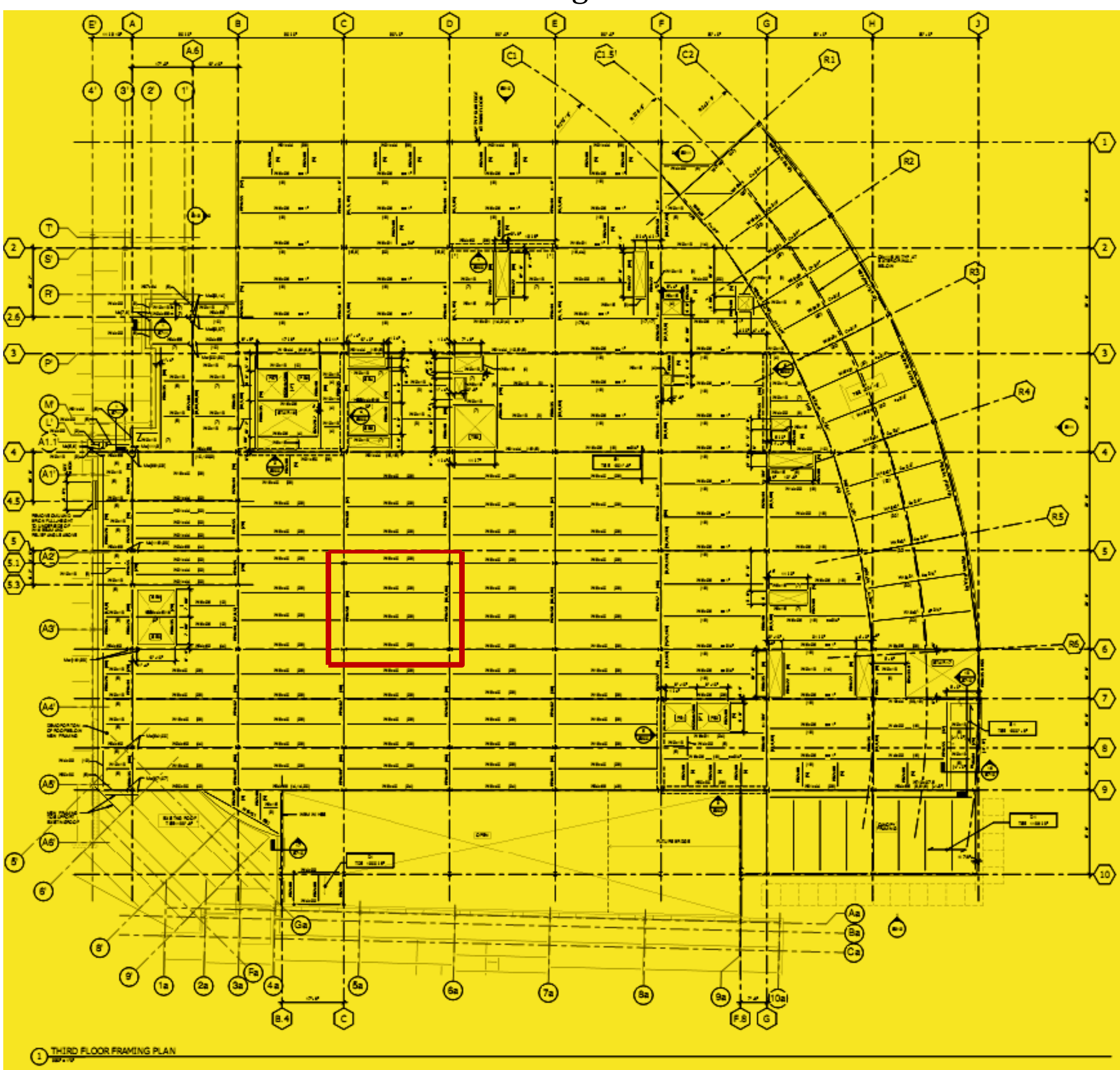


Figure 2.1: Third floor framing plan with typical bay shown

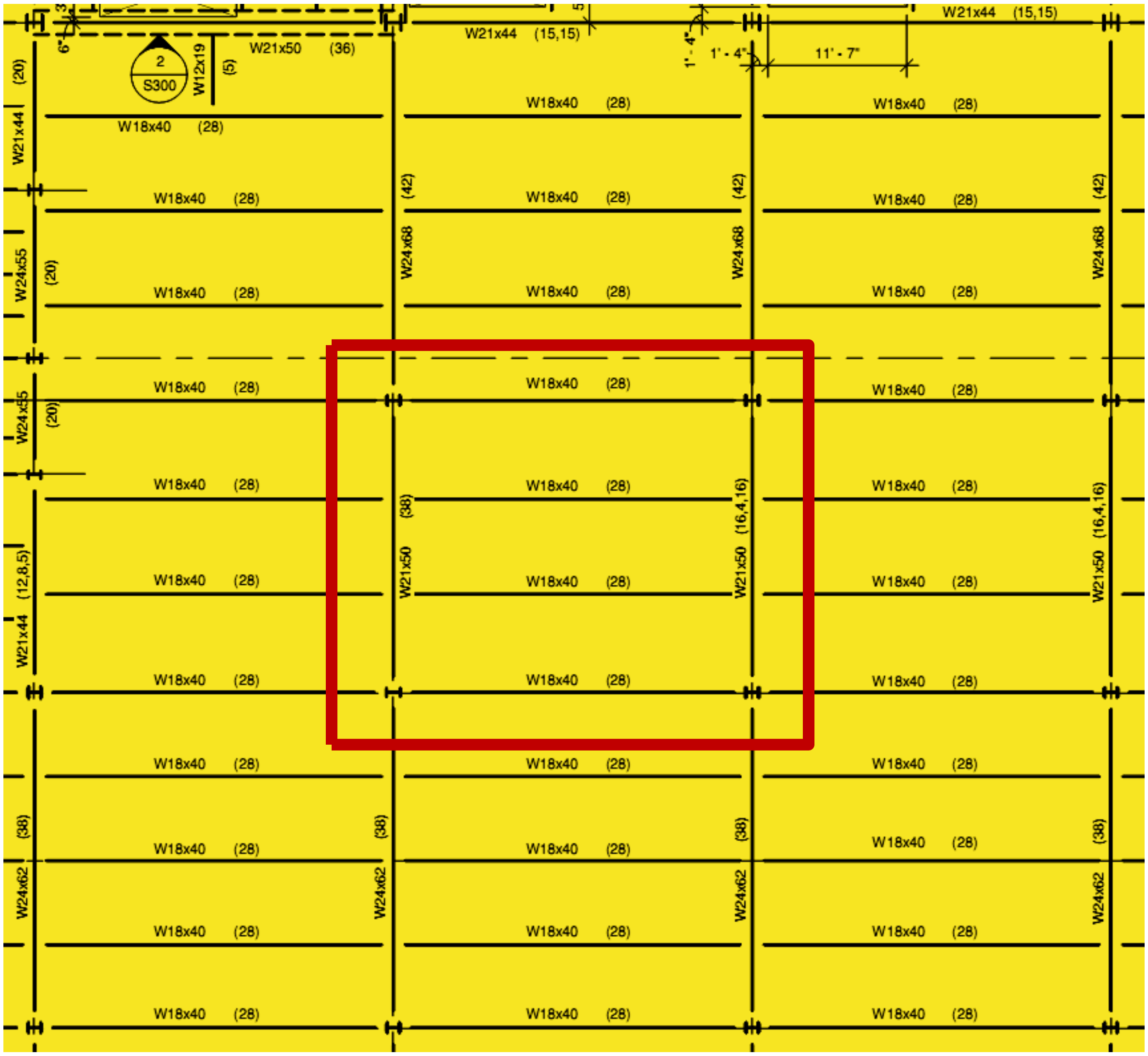


Figure 2.2: Typical bay to be considered enlarged view

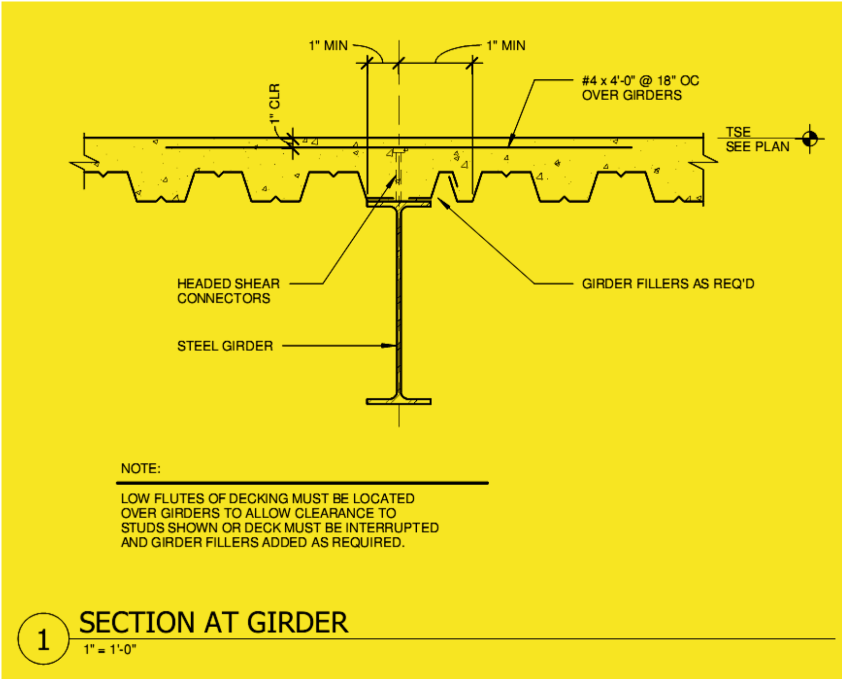


Figure 2.3: existing slab & beam/girder conditions

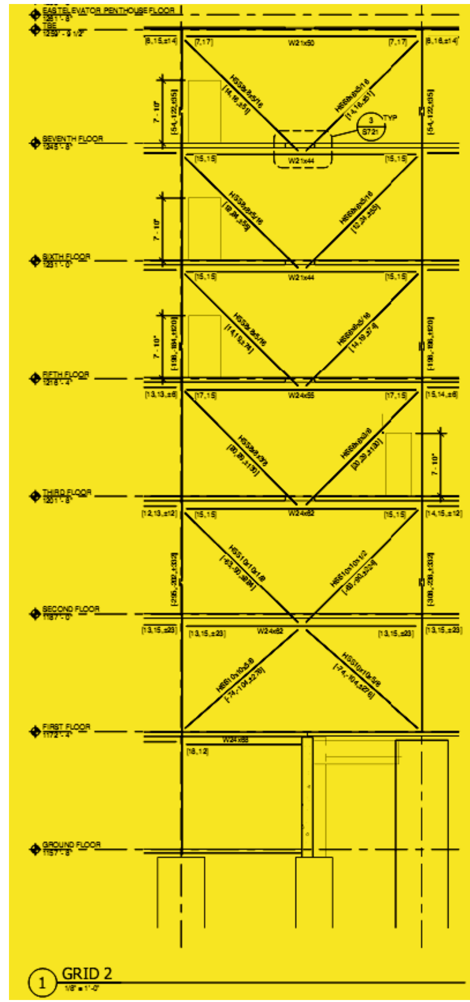


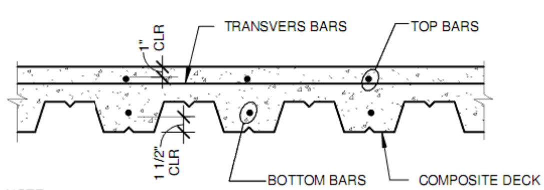
Figure 2.4: Lateral Bracing Elevation

SLAB/DECK SCHEDULE

MARK	TOTAL THICKNESS	TYPE	DECK			CONCRETE		STUD LENGTH	REINFORCING	
			DEPTH	GAGE	FINISH	THICKN	TYPE		REINF	DETAIL
S1	6 1/2"	COMP DECK	3"	20	GALV	3 1/2"	LW	5"	WWF 6x6 W2.1xW2.1	
S2	6 1/2"	COMP DECK	3"	18	GALV	3 1/2"	LW	5"	#5 @ 12"OC T & B #4 @ 12"OC TRANSVERSE	
D1	3"	ROOF DECK	3"	20	GALV	-	-	-	-	

NOTES:

1. ALL COMPOSITE SHEAR CONNECTORS (STUDS) ARE 3/4"Ø UNO.
2. NW=NORMAL WEIGHT CONCRETE; LW=LIGHTWEIGHT CONCRETE.
3. STUD LENGTHS ARE LENGTHS AFTER WELDING.
4. SEE DETAILS 1,2,3/S701 FOR SLAB REINFORCING.
5. SEE 14-16/S700 FOR DECK WELDING.
6. SEE 17/S700 FOR COMPOSITE DECK STUD PLACEMENT.



NOTE:
SEE SLAB/DECK SCHEDULE
FOR BAR SIZES & SPACING

1 SLAB/DECK SCHEDULE
1" = 1'-0"

Figure 2.5: Existing slab/deck schedule

Design Standards & Codes: For all four design cases

2006 IBC
2000 NFPA 101
2006 Guidelines for Design & Construction of Health Care Facilities
1998 Pennsylvania Department of Health Rules and Regulations for Hospitals
ASCE 7-05: for wind, seismic, snow and gravity loads
ACI 318-08: for concrete construction
AISC Thirteenth Edition: for steel members
United Steel Deck Catalog #303-16 Copyright 2002
RS Means Square Foot Costs Guide 2008
CRSI Design Handbook 2002
Nitterhouse Concrete Products Inc. design guide
Floor deflections limited to: L/360 for construction load
L/360 for live load,
L/240 total

Fire Protection & Ratings:

FIRE RESISTIVE CONSTRUCTION:

	HR	UL DESIGN
EXTERIOR LOAD BEARING WALLS	2	UL U906
FIRE WALLS	NA	NA
FIRE WALLS (EXITS, SHAFTS, ELEVATORS)	2	UL U438
FIRE BARRIERS (MIXED USE)	2	UL U412
FIRE PARTITIONS	NA	NA
SMOKE BARRIERS	1	UL U465
FIRE BARRIERS (INCIDENTAL USE)	1	UL U465
STRUCTURAL FRAME	3	UL X772
STRUCTURAL FRAME (SUPPORTING ROOF ONLY)	2	UL X772
NON-BEARING EXTERIOR WALLS	0	NA
FLOOR CONSTRUCTION	2	UL D916
ROOF CONSTRUCTION	1	UL P741

Figure 2.6: Table from construction documents

Design Load Summary:

Gravity Loads					
Description/location	DL/ LL	ASCE 7-05/ IBC 1607.9 values	HGA's values	Reduction available/used	**Design value**
Concrete floors	DL	90-115pcf	115pcf	NO/NO	**115pcf**
MEP/partitions/finishes	SDL	20-25psf		NO/NO	**35psf**
1st floor mechanical	LL		125psf	YES/NO	**125psf**
2nd floor/ lobby	LL	100psf	100psf	YES/NO	**100psf**
Hospital floors	LL	40-80psf	80psf	YES/YES	**80psf**
Stairs & exits	LL	100psf	100psf	NO/NO	**100psf**
5th floor roof	LL		115psf	NO/NO	**115psf**
Mech. Penthouse floor	LL		125psf	NO/NO	**125psf**
Elevator Machine room floor	LL		125psf	YES/NO	
Roof top equipment areas	LL		125psf (or actual equipment wt.)	NO/NO	**125psf**
Balconies	LL	100psf	100psf	YES/YES	**psf**
Snow	LL	24-30psf	24-30psf	NO/NO	**24-30psf**

Figure 2.7

Gravity Loads:

Dead loads for the floor area were determined in technical report 1 and were calculated at 48psf for the lightweight concrete, 7psf for the wide flanges, 3psf for the steel decking, and 35psf for MEP/partitions/finishes**. Live loads** are 40-80psf for hospital floors, therefore 80 will be used for calculation purposes and no live load reduction will be taken since there are other areas with larger load criteria and reductions are not permitted.

Total DL= 93psf
Total LL= 80psf

1.2DL + 1.6 LL = 239.6psf

3 x 12" DECK F_y = 33ksi f'_c = 3 ksi 115 pcf concrete

United Steel Deck, Inc.

LRFD

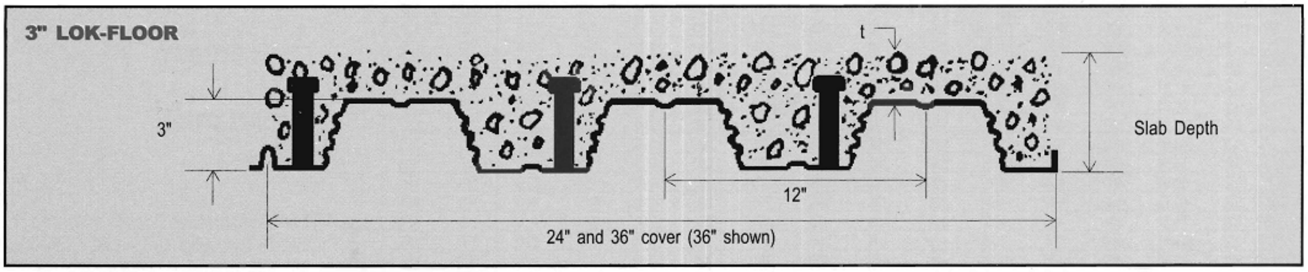


Figure 2.8: Composite steel decking used for existing floor design & non composite beam floor design.

Note: f'c= 3000psi for table values, f'c of 3500psi is used in design, therefore these values will be slightly conservative

	COMPOSITE PROPERTIES												
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
20 gage	5.50	62.81	37.6	0.333	38	1.51	8.1	42.29	5250	9.35	11.75	12.14	0.023
	6.00	71.37	42.0	0.375	43	1.73	10.4	48.61	5870	8.92	11.27	11.65	0.027
	6.25	75.65	44.3	0.396	46	1.85	11.7	51.89	6180	8.73	11.06	11.43	0.029
	6.50	79.92	46.6	0.417	48	1.97	13.0	55.23	6470	8.55	10.85	11.21	0.032
	7.00	88.48	51.3	0.458	53	2.21	16.1	62.07	6800	8.23	10.48	10.82	0.036
	7.25	92.76	53.8	0.479	55	2.34	17.8	65.57	6980	8.08	10.30	10.64	0.038
	7.50	97.03	56.3	0.500	58	2.46	19.6	69.10	7150	7.94	10.13	10.47	0.041
	8.00	105.59	61.3	0.542	62	2.72	23.6	76.28	7500	7.72	9.82	10.15	0.045
	8.25	109.87	63.9	0.563	65	2.85	25.7	79.92	7690	7.64	9.67	9.99	0.047
	8.50	114.15	66.6	0.583	67	2.98	28.0	83.59	7870	7.56	9.53	9.85	0.050

Figure 2.9: Shows the ΦMn (in*k) & the maximum unshored span for a 3span system

The ΦMn value of 79.92 equates to 959ft*k, which is well above the design of Mu=270 ft*k. The design span is equal to 10' which is below the 11.21' specified.

	Slab Depth	ϕMn in.k	L, Uniform Live Service Loads, psf * 9.00	9.50	10.00	10.50	11.00	11.50	12.00	12.50	13.00	13.50	14.00	14.50	15.00
20 gage	5.50	42.29	185	165	145	130	115	105	90	80	75	65	60	55	50
	6.00	48.61	215	190	170	150	135	120	105	95	85	75	70	60	55
	6.25	51.89	230	205	180	160	145	130	115	105	90	85	75	65	60
	6.50	55.23	245	215	195	170	155	135	120	110	100	90	80	70	65
	7.00	62.07	280	245	220	195	175	155	140	125	110	100	90	80	75
	7.25	65.57	295	260	230	205	185	165	145	130	120	105	95	85	80
	7.50	69.10	310	275	245	215	195	175	155	140	125	115	100	90	85
	8.00	76.28	345	305	270	240	215	190	170	155	140	125	115	105	95

Figure 2.10: Shows the uniform live service loads (NO factors) of combined DL & LL for a 10' span as 195psf which is above the 173psf design value.

U.L. Fire Ratings - Composite Deck, cont'd.

RESTRAINED ASSEMBLY RATINGS (HOURLY)	U.L. DES. NO.	F.P.	CONCRETE COVER	USD PRODUCTS
2	D772	C	2 ½ NW,LW	LF2,LF3*
	D773	C	2 ½ LW	BL*
	D774	C	2 ½ LW	LF2*
	D775	C	2 ½ NW,LW	BL,INV. BL,LF2,LF3*
	D779	C	2 ½ NW,LW	BL,LF15,LF2,LF3
	D822	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,NL,NLC*
	D824	F	2 ½ NW,LW	BL,BLC,LF15,LFC1
	D825	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
	D826	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
	D831	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
	D832	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
	D833	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
	D837	F	2 ½ NW	BL,BLC,LF15,LFC1*
	D840	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
	D847	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,NLC*
	D852	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
	D858	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,AWC2,AWC3*
	D859	F	2 NW,LW	LF2,LFC2,LF3,LFC3*
	D860	F	3 ¼ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
	D861	F	2 ½ NW,LW	LF2,LF3*
	D862	F	2 ½ LW	LF2,LF3*
	D870	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
	D902	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D902	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D902	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D906	N	3 ¼ LW	NLC
	D907	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3
	D908	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D913	N	3 ¼ LW	BL,LF15,LF2,LFC2,LF3,LFC3
	D916	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D916	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D916	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
	D918	N	4 ½ NW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC

Figure 2.11: Shows that the flooring system meets the U.L. designation code as specified in the construction documents with 3.5" LW Concrete. Therefore spray-on fireproofing is not needed on the underside of the deck as designated by the N.

Note: Design calculations for the girder and beam can be found in Appendix B. The calculations vary from those of technical report 1 in the fact that the typical bay was chosen from a different area which used W18x40 non-cambered unshored beams instead of the W16x26 ¾" cambered beams. Both typical bays are predominating and found on all levels and this is a good check to see how each one performs given the same loading.

Conclusions from the floor analysis show that construction load deflection controlled the beam size for the W16x26 analyzed for technical report #1as well as the size of the W18x50. The reason for using the smaller beams with camber could be that there are depth limitations in that area, which could be a limiting factor for design #2, non-composite beams with one additional beam per bay.

Non-composite steel beam construction:

The first alternate floor design has the same bay size as that of the existing design. In this configuration I chose to resize the girders and beams to try and get a more even distribution of strength and serviceability requirements. Members will try to be selected so as to minimize depth and still keep costs down. The beams and girders will still have the same lengths and direction. A lighter gage deck will be used for the shorter span and there will be no composite beam action.

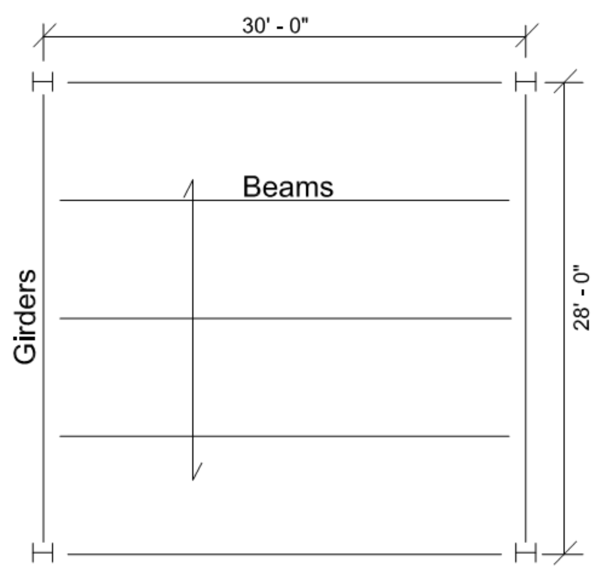


Figure 2.12: Bay beam layout

A 2” LOK-FLOOR using 22 gage steel and a 3.5” LW concrete topping for a total depth of 5.5” is used in this design.

Table values listed below can be found in figures 2.14 – 2.15

ΦMno = 38.29in*k, the factored resisting moment of the composite slab with no shear studs

W=43, the weight of the concrete in psf

ΦVnt=4970lbs, the factored vertical shear resistance of the composite system

Maximum unshored span=7.86 ft, for 3 spans this is the maximum unshored distance

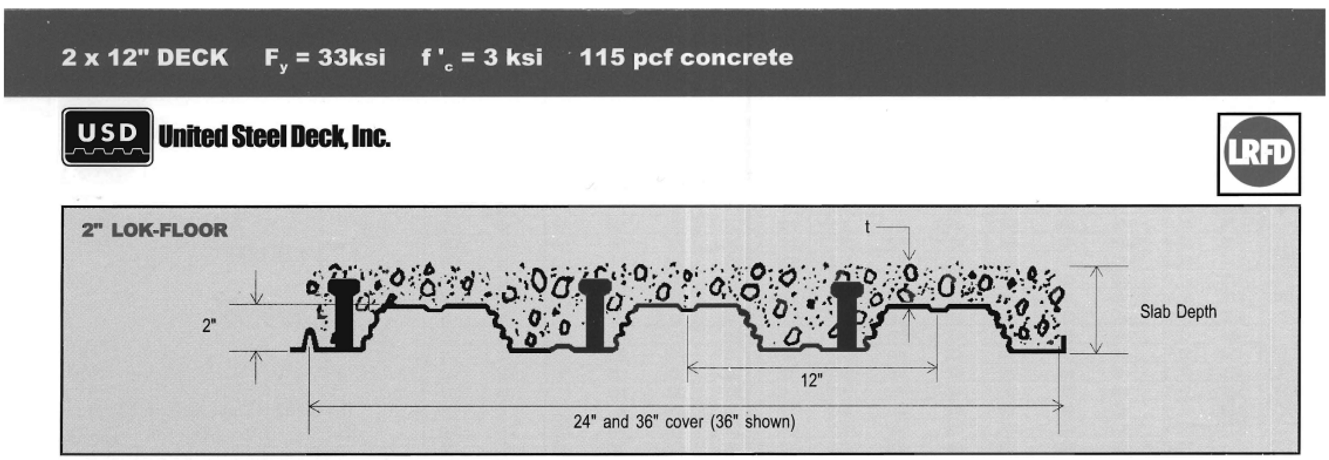


Figure 2.13: Composite steel decking

Note: f'c= 3000psi for table values, f'c of 3500psi is used in design, therefore these values will be slightly conservative

COMPOSITE PROPERTIES (22 gage)

Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
4.50	40.27	32.6	0.292	34	1.00	4.4	28.13	4270	6.32	8.46	8.56	0.023
5.00	46.44	37.5	0.333	38	1.18	6.0	33.12	4610	6.03	8.09	8.19	0.027
5.25	49.53	40.0	0.354	41	1.27	6.9	35.69	4790	5.90	7.93	8.02	0.029
5.50	52.61	42.6	0.375	43	1.36	7.9	38.29	4970	5.77	7.77	7.86	0.032
6.00	58.78	48.0	0.417	48	1.55	10.1	43.58	5340	5.55	7.49	7.58	0.036
6.25	61.87	50.8	0.438	50	1.65	11.3	46.26	5540	5.45	7.36	7.45	0.038
6.50	64.95	53.6	0.458	53	1.75	12.7	48.97	5730	5.36	7.24	7.32	0.041
7.00	71.12	59.5	0.500	58	1.94	15.7	54.44	6150	5.18	7.01	7.10	0.045
7.25	74.21	61.9	0.521	60	2.04	17.4	57.20	6310	5.10	6.91	6.99	0.047
7.50	77.29	64.3	0.542	62	2.14	19.2	59.97	6480	5.05	6.81	6.89	0.050

Figure 2.14: Composite properties

	L, Uniform Live Service Loads, psf *														
	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
22 gage	4.50	28.13	300	250	215	180	155	135	120	105	90	80	70	60	55
	5.00	33.12	355	295	250	215	185	160	140	125	110	95	85	75	65
	5.25	35.69	380	320	270	235	200	175	150	135	115	105	90	80	70
	5.50	38.29	400	345	290	250	215	185	165	145	125	110	100	85	75
	6.00	43.58	400	395	335	285	245	215	185	165	145	130	115	100	90
	6.25	46.26	400	400	355	305	260	230	200	175	155	135	120	105	95
	6.50	48.97	400	400	375	320	280	240	210	185	165	145	130	115	100
	7.00	54.44	400	400	400	360	310	270	235	205	185	160	145	125	115

Figure 2.15: Shows the uniform live service loads (NO factors) of combined DL & LL for a 7.5' span as 250psf which is above the 173psf design value.

U.L. Fire Ratings - Composite Deck, cont'd.

		U.L. DES. NO.	F.P.	CONCRETE COVER	USD PRODUCTS
RESTRAINED ASSEMBLY RATINGS (HOURLY)	2	D772	C	2 ½ NW,LW	LF2,LF3*
		D773	C	2 ½ LW	BL*
		D774	C	2 ½ LW	LF2*
		D775	C	2 ½ NW,LW	BL,INV. BL,LF2,LF3*
		D779	C	2 ½ NW,LW	BL,LF15,LF2,LF3
		D822	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,NL,NLC*
		D824	F	2 ½ NW,LW	BL,BLC,LF15,LFC1
		D825	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
		D826	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
		D831	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
		D832	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
		D833	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
		D837	F	2 ½ NW	BL,BLC,LF15,LFC1*
		D840	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
		D847	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,NLC*
		D852	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
		D858	F	2 ½ NW,LW	LF2,LFC2,LF3,LFC3,AWC2,AWC3*
		D859	F	2 NW,LW	LF2,LFC2,LF3,LFC3*
		D860	F	3 ¼ LW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC*
		D861	F	2 ½ NW,LW	LF2,LF3*
		D862	F	2 ½ LW	LF2,LF3*
		D870	F	2 ½ NW,LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3*
		D902	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D902	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D902	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D906	N	3 ¼ LW	NLC
		D907	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3
		D908	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D913	N	3 ¼ LW	BL,LF15,LF2,LFC2,LF3,LFC3
		D916	N	4 ½ NW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D916	N	3 ¼ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D916	N	3 ½ LW	BL,BLC,LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC
		D918	N	4 ½ NW	LF15,LFC1,LF2,LFC2,LF3,LFC3,NL,NLC

Figure 2.16: Shows that the flooring system meets the U.L. designation code as specified in the construction documents with 3.5" LW Concrete. Therefore spray-on fireproofing is not needed on the underside of the deck as designated by the N.

Gravity Loads:

Dead loads for the floor area are determined from figure 2.14 at 43psf for the lightweight concrete, 1.5psf for the steel decking, and 35psf for MEP/partitions/finishes**. Live loads** are 40-80psf for hospital floors, therefore 80 will be used for calculation purposes and no live load reduction will be taken since there are other areas with larger load criteria and reductions are not permitted.

Total DL= 79.5psf
Total LL= 80psf

1.2DL + 1.6 LL = 223.4psf

Floor deflections limited to: L/360 for construction load
L/360 for live load,
L/240 total

Note: Design calculations for the girder and beam can be found in Appendix C.

Conclusions:

The first alternate floor analysis shows that it is not possible to achieve an even use of the member through its strength and serviceability (deflection criteria). Without the use of the composite beam/girder system the members will either have to be deeper or their weight per lineal foot will increase by at least a factor of two. Addressing the depth issue is not a problem for strictly height as there are no code restrictions on floor to floor heights. This does however lead to other issues with an increased ceiling cavity that would require more energy to control, and increase in the amount of fire protection that would be required to protect the structural members. There are also structural concerns to deal with which include increased load on the columns and footings and well as an increase in unbraced lengths both of which would contribute to larger columns. If smaller depths were used there would still be the concern for the above mentioned structural issues. Either way the costs would be the most prohibitive fact. Costs associated with steel tonnage, increased footing sizes, increased connection sizes and number of fasteners, and labor associated with these would all be factors. The composite system is a better overall system.

Concrete two-way flat slab with drop panels:

Alternate floor design number two will utilize the same bay size and configuration as the existing structure. The column layout will remain the same (see figure 2.17 for details), but column sizes will be increased due to the additional loading and type of material used, which will be reinforced concrete. In this design all strength and serviceability requirements will be met while trying to achieve a smaller floor to floor height with the least depth slab and drop panels. A design aid from CRSI 2002 (Figure 2.21) was used to compare hand calculations against tabulated values after an initial floor thickness had been determined using ACI Table 9.5(c).

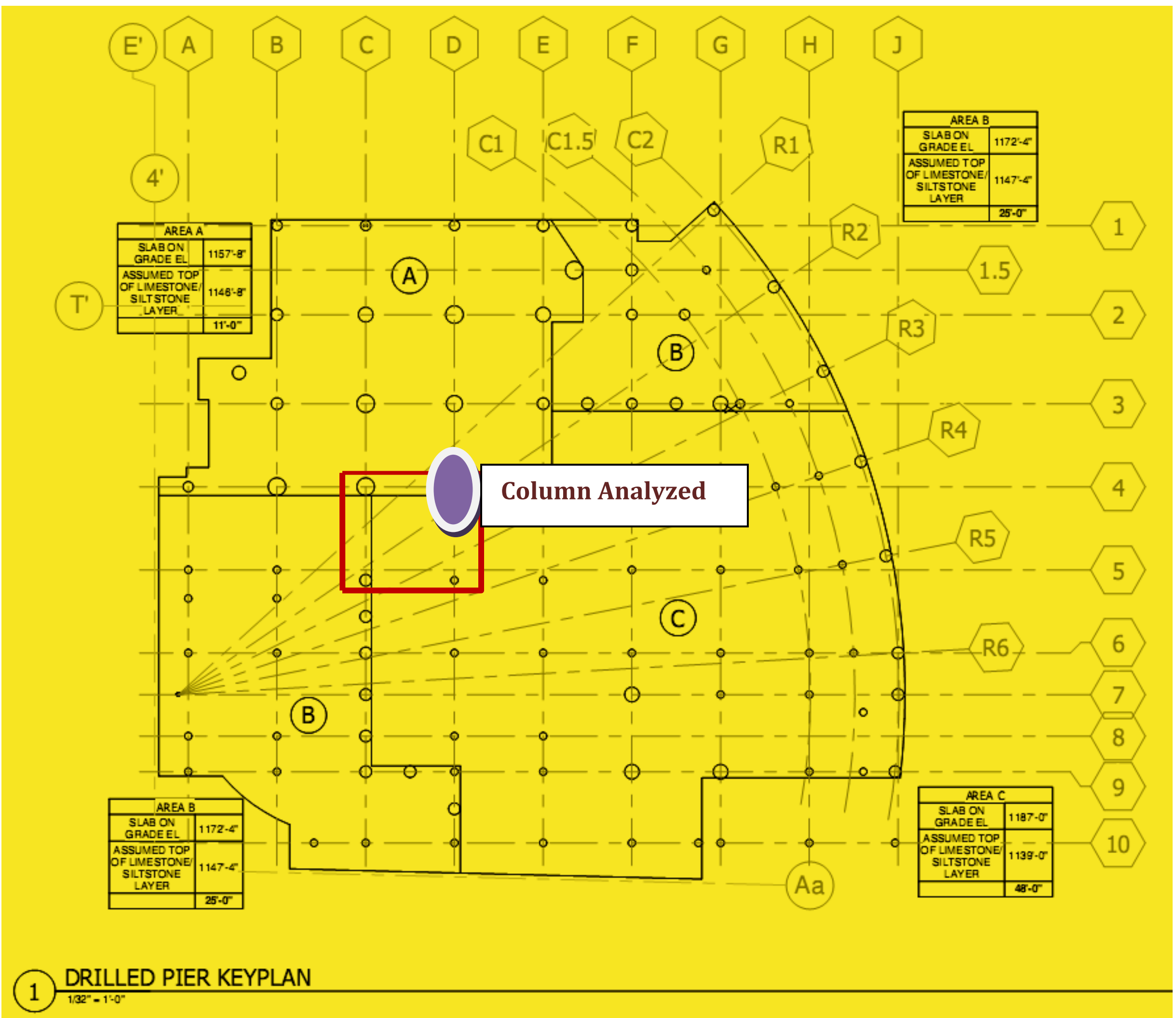


Figure 2.17: Existing and proposed column layout

Note: Footing & column sizes may need to be increased

System description:

This system is an all concrete flat slab with standard size reinforcing, generally #5. Each floor level will need to be formed and shored and then reshored after stripping forms until the concrete reaches its 28day field strength. The system can have edge beams to help carry façade loads and transfer them to the columns; this report however will only consider the inside bays. The system is based on the criteria that the columns carry the entire load from the slab and punching shear will most likely control the thickness and design.

An assumption for the design of this type of system is that the Direct Design Method is going to be used. The actual layout of the current building does not meet all of the requirements for this assumption; therefore, the Equivalent Frame Method should be used. As stated in the start of this report this is not an exhaustive analysis and only one interior bay is being compared so the Direct Design Method will be utilized for simplicity.

To achieve preliminary slab and drop panel thicknesses the 2002 CRSI Design Handbook was used. A minimum slab thickness was first determined using ACI 318-08 §9.5 Table 9.5(c). See figure 2.19 for layout. Calculations are presented in Appendix D.

System Components:

Concrete	F'_c=4000psi	
Steel reinforcement (rebar)	F_y=60,000psi	Typically #5

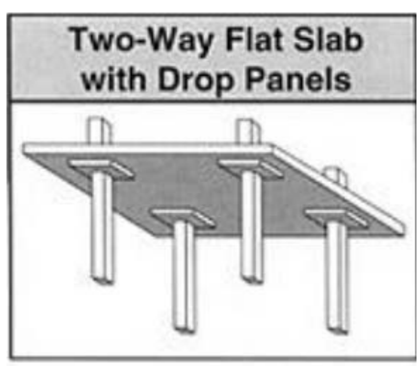


Figure 7: *Two-Way Slab with Drop Panels. Taken from www.crsi.org*

Figure 2.18: System type

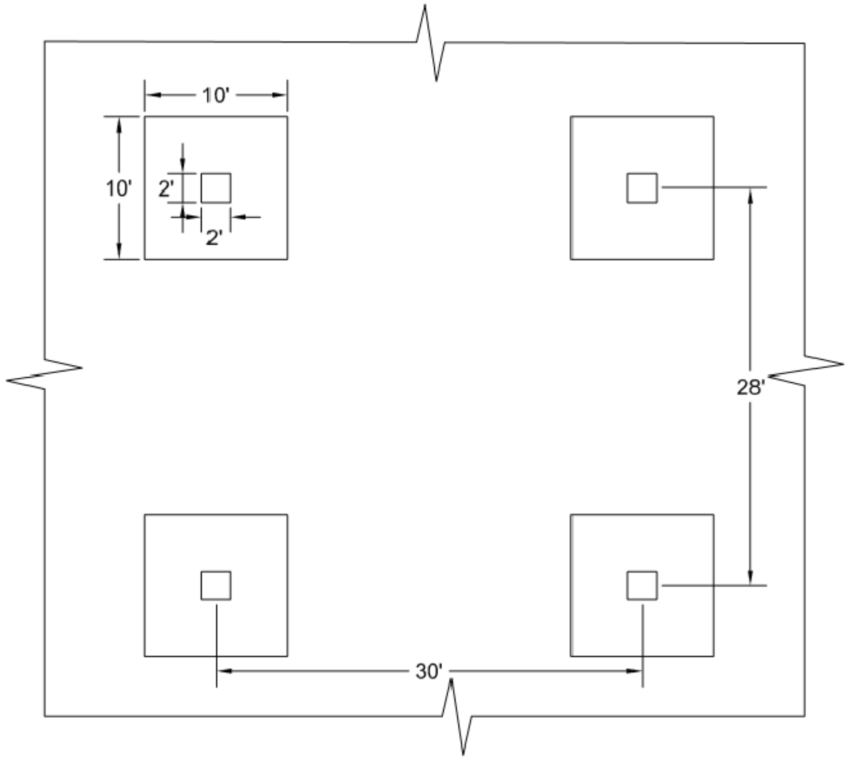


Figure 2.19: Typical interior span layout

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

f_y, psi†	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
60,000	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
75,000	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.
‡Drop panels as defined in 13.2.5.
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

Figure 2.20: Table from ACI 318-08 for minimum slab thickness

f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL With Drop Panels
No Beams

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel Depth (in.)	Square Drop Panel Width (ft)	Square Column (3) Size (in.)	Square Column (3) γ_f	Reinforcing Bars (E. W.) Column Strip (1) Top Ext. +	Column Strip (1) Bottom	Column Strip (1) Top Int.	Middle Strip Bottom	Middle Strip Top Int.	Total Steel (psf)	Moments Edge (–) (ft-k)	Moments Bot. (+) (ft-k)	Moments Int. (–) (ft-k)
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS														
25	100	5.50	8.33	12	0.776	12-#5 2	10-#6	14-#5	9-#5	9-#5	2.39	130.1	260.2	350.3
25	200	5.50	8.33	15	0.809	12-#5 4	13-#6	13-#6	12-#5	10-#5	2.95	171.3	342.6	461.2
25	300	7.00	8.33	18	0.664	12-#5 1	17-#6	15-#6	15-#5	9-#6	3.59	212.4	424.7	571.8
25	400	8.50	8.33	19	0.632	12-#5 1	15-#7	12-#7	10-#7	15-#5	4.25	254.3	508.6	684.6
25	500	8.50	10.00	21	0.744	13-#5 3	11-#9	26-#5	15-#6	10-#7	4.97	295.4	590.8	795.3
26	100	5.50	8.67	12	0.810	12-#5 3	11-#6	16-#5	11-#5	10-#5	2.60	146.8	293.7	395.3
26	200	7.00	8.67	15	0.704	12-#5 1	11-#7	14-#6	10-#6	12-#5	3.17	194.0	388.0	522.3
26	300	8.50	8.67	18	0.633	12-#5 1	11-#8	15-#6	9-#7	15-#5	3.88	240.6	481.1	647.6
26	400	8.50	8.67	19	0.745	13-#5 3	13-#8	18-#6	11-#7	9-#7	4.73	287.7	575.5	774.7
26	500	8.50	10.40	24	0.745	15-#5 4	13-#9	12-#8	10-#8	14-#6	5.49	330.9	661.8	890.9
27	100	7.00	9.00	12	0.746	12-#5 2	18-#5	16-#5	12-#5	10-#5	2.63	165.4	330.8	445.4
27	200	7.00	9.00	15	0.804	12-#5 5	17-#6	15-#6	11-#6	13-#5	3.37	218.2	436.3	587.4
27	300	8.50	9.00	18	0.674	12-#5 2	16-#7	13-#7	19-#5	16-#5	4.12	270.7	541.5	728.9
27	400	8.50	10.80	22	0.756	14-#5 5	12-#9	12-#8	10-#8	19-#5	5.09	321.6	643.2	865.8
27	500	8.50	10.80	27	0.682	16-#5 3	17-#8	13-#8	9-#9	9-#8	5.78	366.6	733.3	987.1
28	100	7.00	9.33	12	0.784	13-#5 2	14-#6	18-#5	13-#5	11-#5	2.76	185.0	370.0	498.1
28	200	8.50	9.33	16	0.714	13-#5 3	11-#8	15-#6	17-#5	15-#5	3.56	243.2	486.4	654.8
28	300	8.50	9.33	19	0.757	13-#5 5	11-#9	14-#7	12-#7	10-#7	4.56	302.4	604.8	814.1
28	400	8.50	11.20	25	0.692	16-#5 3	17-#8	13-#8	11-#8	12-#7	5.47	357.1	714.3	961.5
29	100	8.50	9.67	12	0.737	13-#5 2	22-#5	18-#5	15-#5	12-#5	2.91	206.7	413.4	556.5
29	200	8.50	9.67	16	0.758	13-#5 4	12-#8	13-#7	19-#5	16-#5	3.81	271.2	542.5	730.3
29	300	8.50	9.67	22	0.718	15-#5 4	20-#7	16-#7	10-#8	20-#5	4.92	334.3	668.6	900.1
29	400	8.50	11.60	28	0.639	17-#5 2	15-#9	14-#8	12-#8	10-#8	5.83	392.7	785.4	1057.3
30	100	8.50	10.00	12	0.774	14-#5 2	10-#8	20-#5	16-#5	10-#6	3.16	229.4	458.8	617.6
30	200	8.50	10.00	18	0.744	14-#5 4	11-#9	14-#7	21-#5	10-#7	4.16	299.6	599.1	806.5
30	300	8.50	10.00	24	0.675	16-#5 3	17-#8	14-#8	11-#8	12-#7	5.24	369.5	739.1	994.9

SQUARE INTERIOR PANEL
With Drop Panels(2)
No Beams

SPAN (ft)	Factored Superimposed Load (psf)	(3) Square Column Size (in.)	Reinforcing Bars (E. W.) Column Strip Top	Column Strip Bottom	Middle Strip Top	Middle Strip Bottom	Total Steel (psf)	Concrete (cu. ft / sq. ft)
h = 10 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS								
25	100	12	13-#5	9-#5	9-#5	9-#5	2.19	0.884
25	200	18	12-#6	12-#5	10-#5	9-#5	2.63	0.884
25	300	21	14-#6	15-#5	12-#5	10-#5	3.10	0.898
25	400	23	15-#6	18-#5	10-#6	12-#5	3.63	0.912
25	500	25	13-#7	15-#6	16-#5	10-#6	4.26	0.947
26	100	12	15-#5	11-#5	10-#5	10-#5	2.40	0.884
26	200	18	17-#5	14-#5	11-#5	10-#5	2.73	0.898
26	300	21	14-#6	9-#7	13-#5	11-#5	3.31	0.912
26	400	23	13-#7	11-#7	16-#5	10-#6	4.17	0.912
26	500	25	27-#5	10-#8	10-#7	16-#5	4.65	0.947
27	100	12	15-#5	12-#5	10-#5	10-#5	2.37	0.898
27	200	18	14-#6	11-#6	12-#5	10-#5	2.92	0.898
27	300	21	12-#7	19-#5	15-#5	9-#6	3.56	0.912
27	400	24	26-#5	10-#8	10-#7	15-#5	4.35	0.947
27	500	27	16-#7	11-#8	11-#7	18-#5	5.02	0.947
28	100	12	17-#5	13-#5	10-#5	10-#5	2.42	0.898
28	200	19	14-#6	17-#5	13-#5	12-#5	3.02	0.912
28	300	21	13-#7	22-#5	12-#6	10-#6	3.85	0.912
28	400	24	16-#7	11-#8	20-#5	12-#6	4.71	0.947
29	100	12	17-#5	15-#5	12-#5	11-#5	2.58	0.912
29	200	19	16-#6	19-#5	15-#5	13-#5	3.27	0.912
29	300	21	15-#7	10-#8	10-#7	16-#5	4.34	0.912
29	400	26	13-#8	12-#8	12-#7	10-#7	5.06	0.947
30	100	12	14-#6	12-#6	13-#5	11-#5	2.77	0.912
30	200	19	18-#6	22-#5	12-#6	10-#6	3.57	0.912
30	300	21	16-#7	11-#8	11-#7	18-#5	4.56	0.912

CONCRETE REINFORCING STEEL INSTITUTE

Figure 2.21: Table from CRSI 2002, to obtain preliminary sizes along with figure 2.20. These figures are for 30' square bays (designed is 30'x28'), therefore numerical values should be conservative.

System Loading & Deflection Criteria:

Gravity Loads: Dead loads for the floor area are 125psf for normal weight reinforced concrete @ 10", and 35psf for MEP/partitions/finishes**. Live loads** are 40-80psf for hospital floors, therefore 80 will be used for calculation purposes and no live load reduction will be taken since there are other areas with larger load criteria and reductions are not permitted.

Total DL= 160psf
Total LL= 80psf

1.2DL + 1.6 LL = 320psf

Floor deflection calculations are not required since ACI 9.5.3 Table 9.5c was used.

Advantages:

- It is possible to reduce the overall floor to floor heights by approximately 10" per level, over the total height of the structure this would equate to about five feet in overall height.
- A reduction in height would reduce some of the lateral forces caused by wind and would improve lateral resistance itself.
- Reduction in the cost of all vertical elements such as exterior walls, elevators, stairs, mechanical system components
- Additional unobstructed ceiling space for MEP's.
- Eliminate the need for spray on fireproofing of the structural frame
- Increased mass would reduce vibrational concerns
- Reduce noise transmission from floor to floor
- Works well with the current foundation and column layout
- There are no large lead times with this type of system
- Simple construction and formwork
- Can use flying forms
- Span range up to 40 feet
- Ideal for offices, hospitals, parking decks, warehouses, & industrial plants

Disadvantages:

- A different shear wall lateral system would have to be designed
- Increases the overall weight of the building, therefore making another analysis of the foundation system necessary.
- Longer to complete each level and weather could play a significant role (cold & rain) in western Pennsylvania.
- This method is also not very conducive to letting other trades get in behind (below) to start other work until at least three levels are complete and the concrete has reached sufficient enough strength so falsework and shoring can be removed. In a building of this size that is nearly half of the structure.
- Increased column sizes
- The increased weight dramatically increases the seismic load and analysis
- Mechanical component adjustments for two different slab thicknesses

System Conclusions:
Costs and associated construction time frames would be the two biggest factors affecting whether or not this system should be used. From the list of advantages on the previous page it can be seen that this system is a good viable solution. A cost analysis of savings due to the advantages such as no spray on fireproofing, lower material and labor costs, and less MEP clashes due to more open space would have to be compared to the additional costs of increased foundation bearing and construction schedule timelines as discussed earlier. Another seismic analysis would have to be done and determined if this might control.
Note: Values obtained for this system taken from CRSI Found in Figure 2.21 do not match the calculated numerical values found in Appendix D. Possible reasons for the differences could be 1) Bay sizes in the table are 30' x 30', calculated is 28' x 30' 2) Calculated moments do not include the moment due to the increased size of the drop panels, and 3) The table values may be upsizing the rebar to account for the need for additional shear reinforcement instead of adding additional bars.

Precast hollow plank concrete slabs on steel beams:

System Description:
The idea behind the use of this system is to reduce the overall floor depth, while trying to develop a quicker to install and less expensive system. For the purpose of this design Nitterhouse Concrete Products Inc. published load tables with the 10”x4’ hollow core plank with 2” topping with .5”Ø 270K Lo-relaxation strand will be used for the floor decking. Typically the planks are placed on top of the steel members and the joints are grouted along with the topping keeping the floor system as a rigid diaphragm and the ability to use the existing lateral system. However, to cut down on total system height a wide flange will be designed to carry the moment and more importantly control the deflection. To achieve this, a section will be selected and an angle with the long leg placed out will be secured to the girder to carry the planks. (See figure 2.22 for details). The angle leg will have to be longer than ½ of the top flange of the supporting member to be able to place and support the plank.

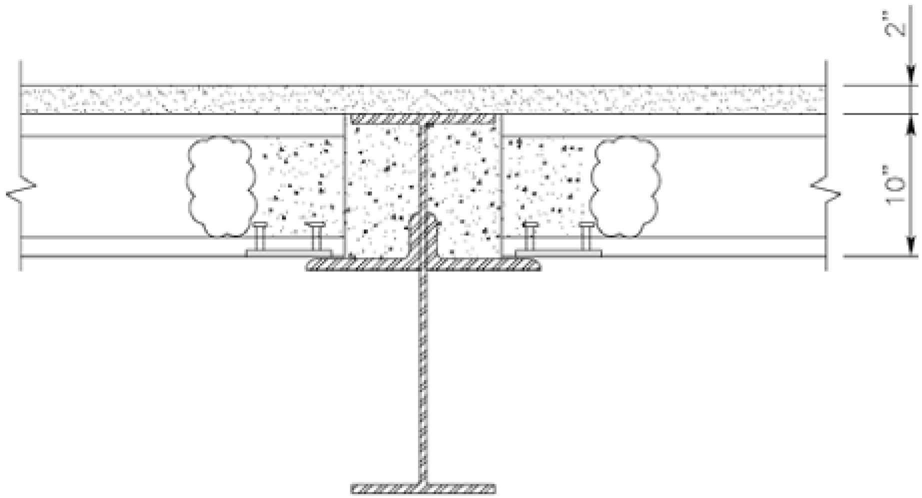
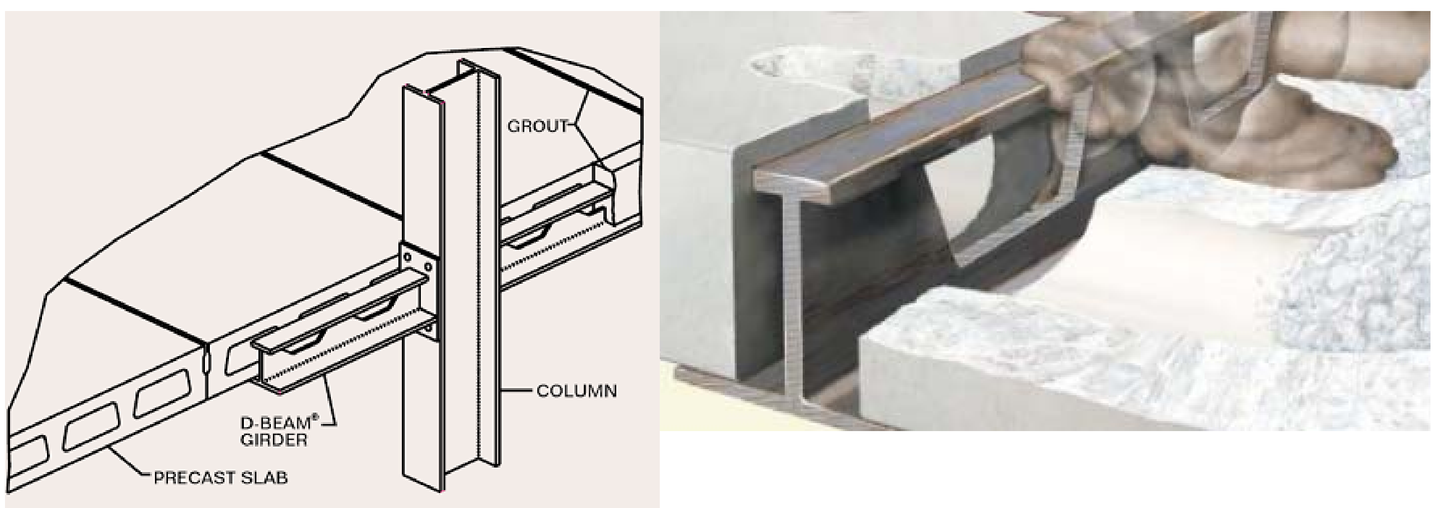


Figure 2.22: Modified wide flange to reduce depth

A better way to achieve this would be to use a modified castellated section with a shorter top flange that can resist the applied moments and control the deflections to within acceptable limits. (See figure 2.23 & 2.24 for details)



Figures 2.23 & 2.24: Modified castellated sections

The infill beams used in the existing design are eliminated except for the beams between the columns. These beams are not used in the gravity load system and therefore will not be analyzed here.

Advantages:

- Easy & fast to install
- The lateral system can still be utilized
- No formwork required and concrete slabs are already at usable capacity when they arrive
- No intermediate beams in interior of bays needed
- Can be installed in any type of weather
- Other trades can start work underneath almost immediately
- Additional unobstructed ceiling space for MEP's.
- Meets or exceeds floor fireproofing requirements
- Reduce noise transmission from floor to floor through baffled cavities
- Can work with the current foundation and column layout
- No increase in floor to floor heights
- Reduces overall weight of the structure

Disadvantages:

- Large lead times with this type of system
- Girders and columns would need fireproofing
- Much more efficient and cost effective at shorter spans
- Column spacing may have to be reduced, increasing footing requirements
- Floor penetrations must be well coordinated with the slab designer/manufacture

System Conclusions:

The advantages outweigh the disadvantages for this system if the girders that support the loading can be designed and manufactured at a cost that could be offset in time and labor savings as well as the need for no intermediate beams.

Summary:

Floor Comparison Summary Table

Floor System Comparison of a Typical Bay				
	Floor systems			
Criteria	Existing Composite Steel	Steel Non-Composite	Concrete Two-way Flat-slab	Precast hollow-core concrete planks on steel beams
System weight (psf)	**58**	**63**	**125**	**75**
Slab depth (in)	6.5	5.5	10	10
Total depth (in)	28	32.5	18.5	27
Column size	W14	W14	24x24	W14
Fire rating (hr)	2	2	3	2
Additional Fire Proofing required	Yes	Yes	No	Yes
Column (cost/V.L.Ft)	**161.20**	**185.65**	**105.00**	**161.20**
Material (cost/sq.ft)	13.95	19.05	8.20	8.45
Labor (cost/sq.ft)	6.10	8.70	9.15	2.05
Total (cost/sq.ft)	**181.25**	**213.40**	**122.35**	**171.70**
Foundation impact	None	Minimal	Moderate	None
Constructability	Easy	Easy	Moderate	Easy
Vibration concerns	Some	Some	No	Some
Lateral force resisting system changes	N/A	No	Yes	No
Alternative	N/A	No	Yes	Yes
Additional study	N/A	No	Yes	Yes

Figure 2.25: Comparison summary

Good

Fair

Poor

Conclusions:

By comparing the three alternate floor systems to the existing composite slab and composite beam system a determination can be made if each system is a viable option to replace the existing system or at least a good candidate for further analysis.

Alternate system one, the non-composite steel beam and steel column system, has no apparent advantages over the existing design in respect to any of the criteria listed in Figure 2.25 and is therefore not a candidate for future analysis.

Figure 2.25 shows that while the two way flat-slab with drop panels may have a significant impact on the foundation system with respect to bearing capacity, the potential cost savings as discussed earlier in the system conclusions could very well out weigh this impact. As discussed with the architect this was the initial design intent for the structure before the geotechnical report came in. The increased time in the foundation completion due to the deep socketed piers did not allow for the increased time for superstructure completion; therefore, a quicker to install steel frame with composite action was decided on. This would still be a good choice for an alternate system if it were not for this fact.

The third alternate design of the precast hollow core planks recessed down into the girders appears to also be a fair to somewhat reasonable design if a cost analysis as mentioned in the system conclusions shows that it would have minimal impact. All other aspects of the design criteria shown in Figure 2.25 are relatively comparable in numerical values. If a girder can be manufactured to carry the required moment at the larger span economically this would be a fair candidate for further study.

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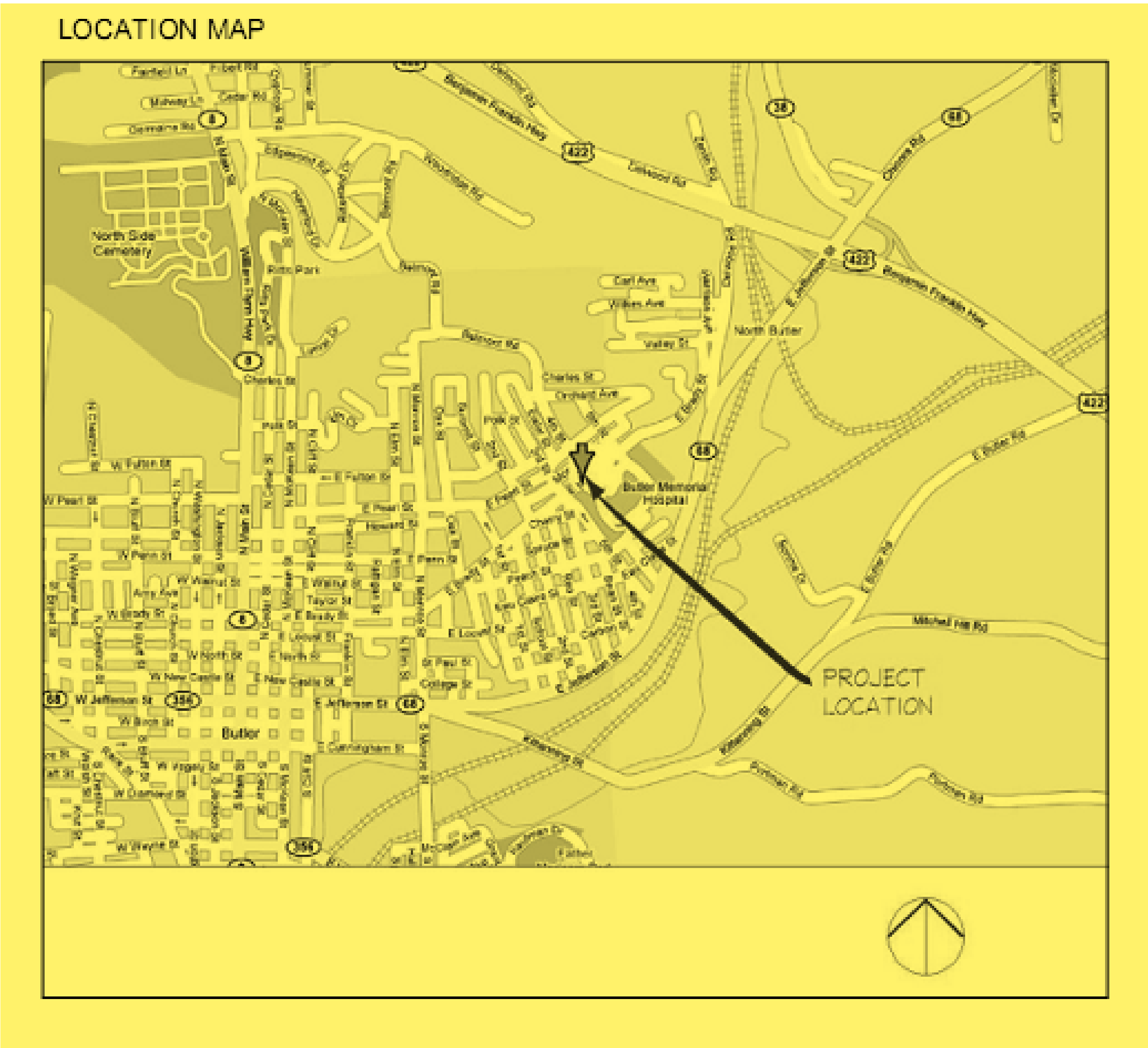
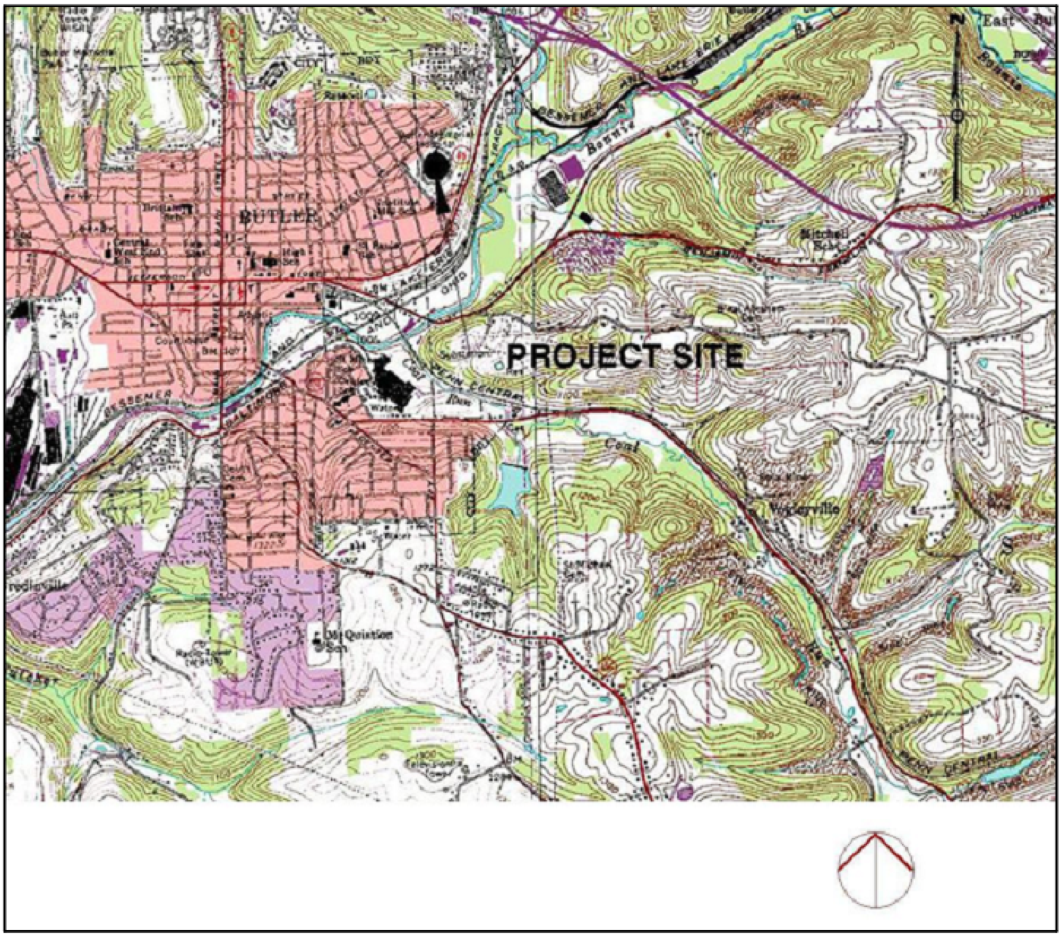
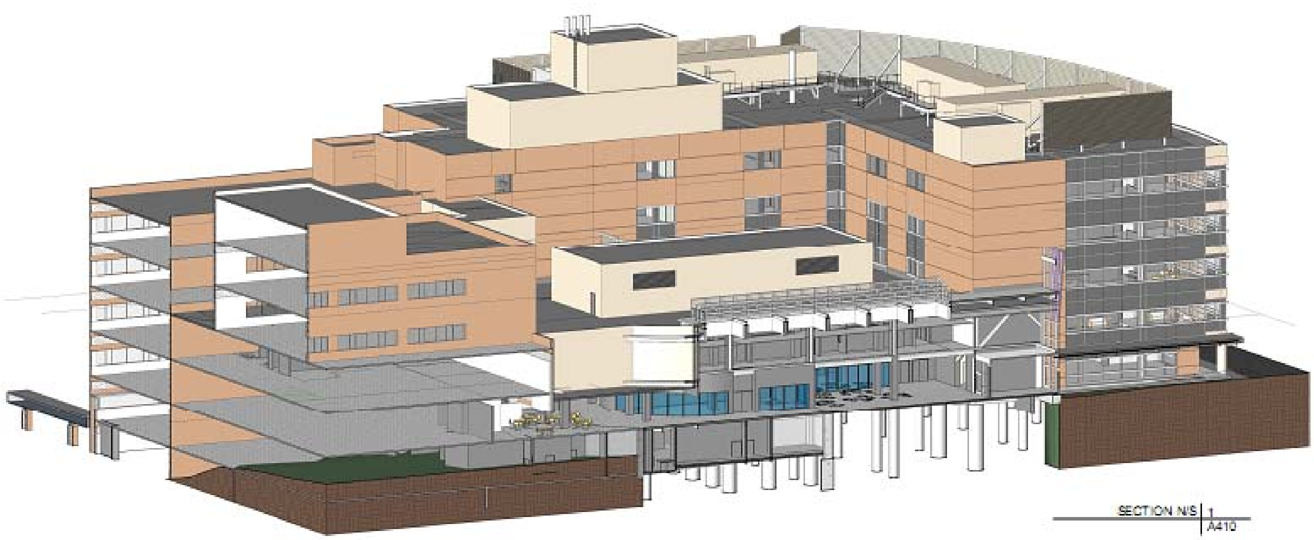
Appendix: A

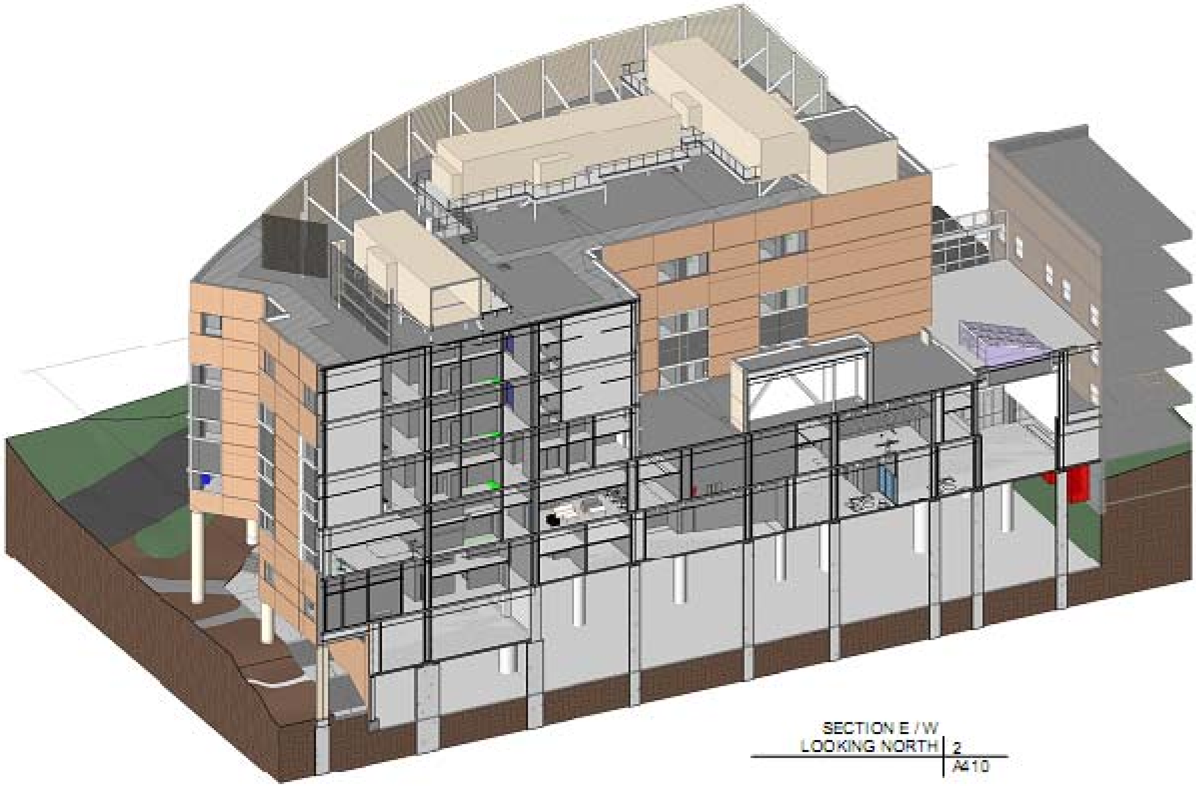
View looking from magnetic north


LOCATION MAP
PROJECT
LOCATION
Butler Memorial
Hospital
North Side
Cemetery
North Butler
Butler

VICINITY MAP







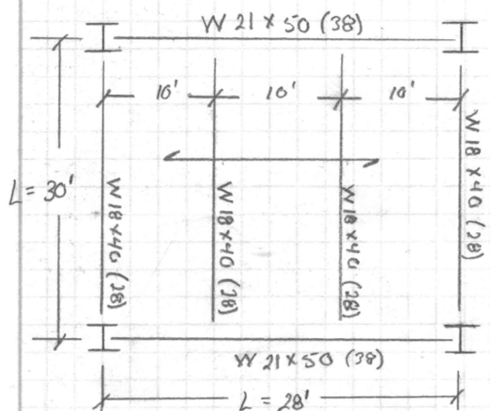
SECTION E / W
LOOKING NORTH 2
A410

MAIN FACADE
NORTHWEST CORNER 3
A400

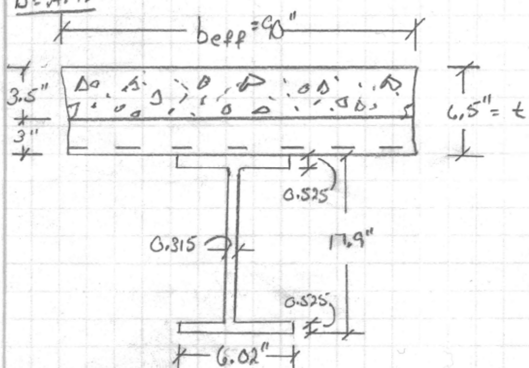
Appendix B

TECH II | 10-28-09 | Existing Beam Calc. | JIM ROTUNNO

COMPOSITE SLAB WITH COMPOSITE BEAM CONSTRUCTION



BEAM:



W18x40:

$A_s = 11.8 \text{ in}^2$

$d = 17.9''$

$b_f = 6.02''$

$t_w = 0.315''$

$t_f = 0.525''$

$I_{xx} = 612 \text{ in}^4$

$F_y = 50$ ksi

$b_{eff} = L/4 < S = 7.5' = 90''$

Concrete

$f'_c = 3500$ psi

$A = 3.5(90'') = 315 \text{ in}^2$

$C_c = 0.85 f'_c A_c = 0.85(3.5)(3.5)(90'') = 937.125^k$

$T_s = A_s f_y = 11.8(50) = 590^k$

Table 3-21
$Q_n = 19.15$

$C_c > T_s \therefore$ PNA is in the concrete

$$a = \frac{T_s}{0.85 f'_c b_{eff}} = \frac{590}{0.85(3.5)(90)} = 2.204''$$

Assuming full composite action and ΣQ_n is at least 590^k

$Y_1 = TFL$, $Y_2 = 5.0 \approx 6.5'' - a/2 = 5.4 < 5.5$ $\therefore 5$

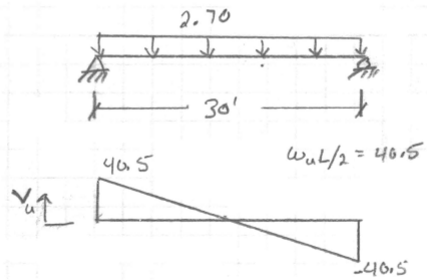
$\phi M_n = 615$ k·ft Table 3-19 AISC - 13

$\phi V_n = 169$ k Table 3-2 AISC - 13

(1)

TECH II 10-28-09 | Existing Beam Calc. | JIM ROTUNNO

W_u = (1.2DL + 1.6LL) Tributary width
= (1.2(93) + 1.6(80)) 10'
= 2.396 k/ft



$M_u = W_u L^2/8 = 2.396(30^2)/8$
= 269.55 k·ft

$M_u < \phi M_n = 615$ ∴ OK

$V_u = 40.5k < \phi V_n = 169^k$ ∴ OK

$\Delta_{max} = \frac{5 \omega l^4}{384 EI}$ check with $I_{LB} = 1760\ in^4$ Table 3-20 AISC-13

$\Delta_{LL} = \frac{5(1.6)(0.80)(30^4)(1728)}{384(29000)(1760)} = 0.457'' < 1''$ ∴ OK

CONSTRUCTION LOADING
assuming bare beam with full lateral support from decking

W_D = 51 psf (10') + 40 lb/ft = 550 lb/ft
W_{LL} = 20 psf (10') = 200 lb/ft

W_u = 1.2DL + 1.6LL = 1.2(0.55) + 1.6(0.2) = 0.98 k/ft

$M_u = W_u l^2/8 = 0.98(30^2)/8 = 110.25$ k·ft

ϕM_n of the bare beam = 294 k·ft > 110 ∴ OK

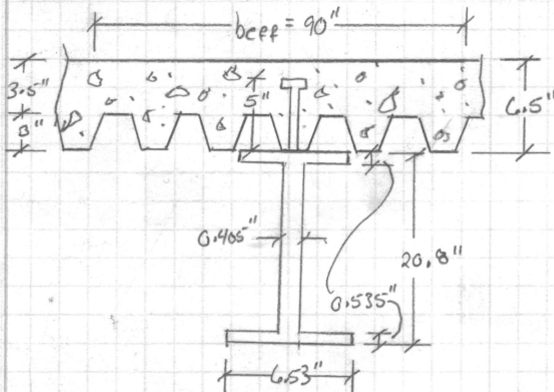
$\Delta_{DL} = \frac{5(0.55)(30^4)(1728)}{384(29000)(612)} = 0.565'' < \frac{L}{360} = \frac{360}{360} = 1''$
∴ OK

②

TECH II | 10-28-09 | Existing Girder Calc | JIM ROTUNNO

COMPOSITE SLAB WITH COMPOSITE GIRDER CONSTRUCTION

GIRDER:



W21x50:

$A_s = 14.7\ in^2$

$d = 20.8\ in$

$b_f = 6.53"$

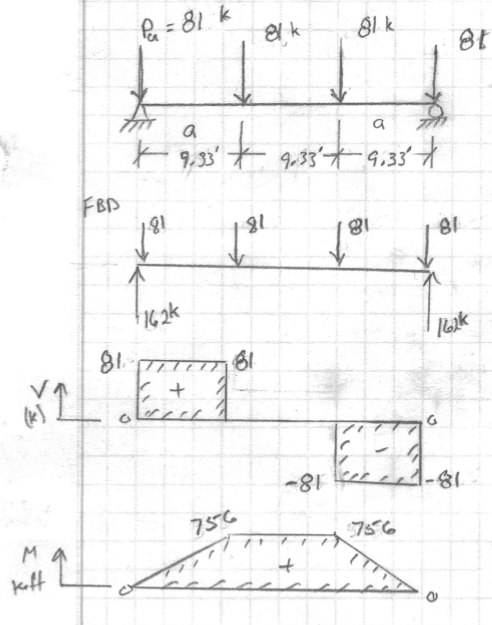
$t_w = 0.405"$

$t_f = 0.535"$

$I_{xx} = 984\ in^4$

$F_y = 50\ ksi$

$b_{eff} = L/4$ or $S \Rightarrow L/4 = 90"$



ϕM_n of bare beam = 413 k·ft

$\phi V_n = 237\ k$

$M_u = 378\ k\text{-}ft$

$A_c = 315\ in^2$

$f'_c = 3500\ psi$

$f_y = 50\ ksi$

$C_c = 937.125^k$

$T_s = 14.7(50) = 735^k$

PNA is in concrete

$a = \frac{T_s}{0.85 f'_c b_{eff}} = \frac{735}{267.75} = 2.75"$

$\Sigma Q_n = 736$ Table 3-19 AISC-13

$Y_1 = TFL \quad Y_2 = 3.5$

$\phi M_n = 768\ k\cdot ft$

$I_{LB} = 2410\ in^4$ Table 3-20 AISC-13

(3)

TECH II 10-28-09 | Existing Girder Calc. | JIM ROTUNNO

GIRDER CONT.

CHECKS

$M_u < \phi M_n \Rightarrow 756 < 768 \therefore OK$

$V_u < \phi V_n \Rightarrow 81 < 237 \therefore OK$

$\Delta_{max} \Rightarrow$ using LL and checked with I_{LB}

$$\Delta_{LL} = \frac{Pa}{24EI_{LB}}(3l^2 - 4a^2) \le L/360$$

P = 43.26
a = 9.33'
I_{LB} = 2410 in^4
l = 28'
L = 28'

$$\Delta_{LL} = \frac{21.63(9.33)(1728)}{24(29000)(2410)}\left(3(28^2) - 4(9.33^2)\right) \le \frac{28(12)}{360}$$

$= 0.0004158(2003.8044) \le \frac{336}{360}$

$= 0.833 < 0.933 \therefore OK$

CONSTRUCTION LOADING

assuming bare girder has full lateral support from decking and shear studs

$M_u = P_u \cdot a \Rightarrow a = 9.33$, $P_u = (1.2)51\,psf(10')(30') + 1.6(20\,psf)(10')(30') = 27.96^k$

$+ W_u l^2/8$ from self weight $= 0.050(28')/8 = 4.9$ k·ft

$M_u = 27.96(9.33) + 4.9$

$= 260.87 + 4.9$

$= 266 < 413$ k·ft $\therefore OK$

$$\Delta_{DL} = \frac{Pa}{24EI}(3L^2 - 4a^2) \le \frac{28(12)}{360}$$

$$= \frac{0.051(10)(30)(9.33)(1728)}{24(29000)(984)}\left(3(28^2) - 4(9.33^2)\right) \le 0.933$$

$= 0.773 < 0.933 \therefore OK$

0.638603869

(4)

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Appendix C

TECH II 10-28-09 | NON COMPOSITE STEEL | JIM ROTUNNO

BEAMS:

Determine loading on beams

concrete = 43 psf
steel deck = 1.5 psf — 79.5 psf
MEP = 35 psf
add in self weight later

LL ⇒ assume 80 psf with no reductions

Load Combination

$1.2DL + 1.6LL \Rightarrow 1.2(79.5) + 1.6(80) = 223.4\text{ psf}$

– Compare this to the 250 psf found in figure 2.15

$223.4 < 250 \therefore$ OK

Total shear in deck and slab at beam edge for a 7.5' span $= 223.4\text{ psf}(7.5')/2 = 837.75$ lb/ft width of slab

– Compare this to the 4970 lbs for ϕV_{nt} in figure 2.14

$837.75 < 4970 \therefore$ OK

$M_{u\,slab} = 223.4\text{ psf}(1')(7.5')^2/8$ assumes simply supported as stated in tables

$= 1570.78$ k·ft

$\approx 1570.78/12 = 0.1309$ k·in $< 38.29 \therefore$ OK

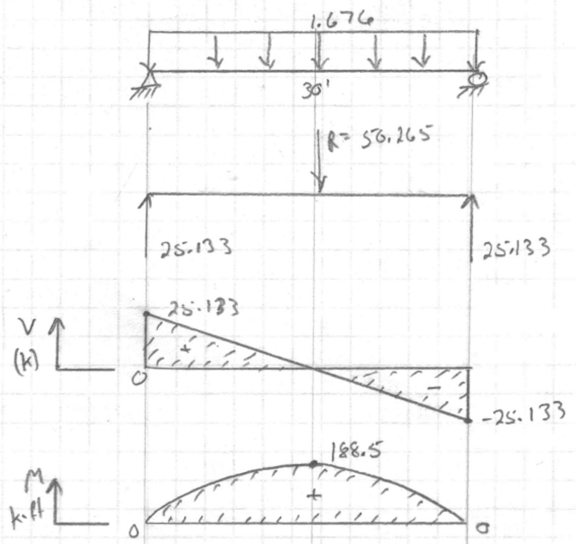
THE DECK IS OK AS DESIGNED

(1)

TECH II 10-28-09 | NON COMPOSITE STEEL | JIM ROTUNNO

Assume simply supported for beam span (shear connected)
tributary area for each beam is 7.5' wide

$w_u = 223.4(7.5') = 1.6755\ k/ft$



$V_u = 25.13^k$

$M_u = 188.5^{k \cdot ft}$

$\Delta_{max} \leq L/360$ for LL

$L/360$ const. Load

$L/240$ Total

$L/360 = 1''$, $L/240 = 1.5''$

$$\Delta_{max_{Total}} = \frac{5(w_u)l^4(1728)}{384EI}$$

⇒ find beam to satisfy I when $\Delta_{max} \leq L/x$ and a M_u of 190 k·ft

$$I_T = \frac{5(1.676)(30^4)(1728)}{384(29000)(1.5)} = 70.2\ in^4$$

construction loading

$$w_u = (43psf + 1.5psf)(7.5')(1.2) + 1.6(20psf)(7.5') = 0.6405\ k/ft$$

$$I_{CL} = \frac{5(0.641)(30^4)(1728)}{384(29000)(1'')} = 403\ in^4$$

w_u for LL ⇒ $1.6(80)(7.5) = 0.96\ k/ft$

$$I_{LL} = \frac{5(0.96)(30^4)(1728)}{384(29000)(1'')} = 603.3\ in^4$$

702 in^4 controls

(2)

TECH II 10-28-09 | NON COMPOSITE STEEL | JIM ROTUNNO

BEAMS CONT.

CHOICES

$W14\times68 \Rightarrow I = 722\ in^4 \qquad \phi M_n = 431\ k\cdot ft$

$W16\times57 \Rightarrow I = 758\ in^4 \qquad \phi M_n = 394\ k\cdot ft$

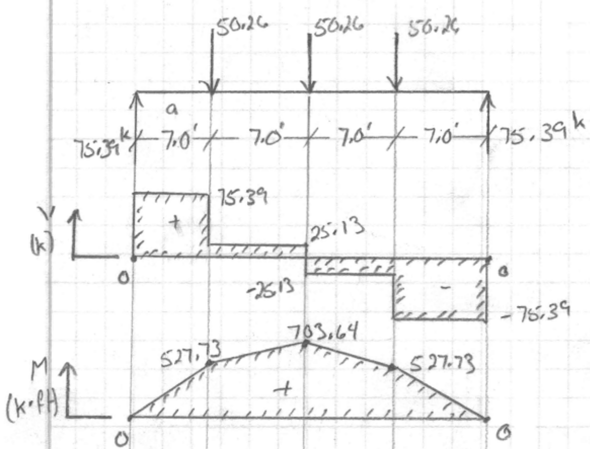
$W18\times46 \Rightarrow I = 712\ in^4 \qquad \phi M_n = 340\ k\cdot ft$

$W21\times44 \Rightarrow I = 843\ in^4 \qquad \phi M_n = 358\ k\cdot ft$

ϕV_n is larger than 75.39 for all

GIRDERS: Assume simply supported @ columns (shear connected)

Reactions from beam ends are 25.13 k each, there is one on both sides of the girder $\therefore 50.26^k = P_u$



$V_u = 75.39\ k$

$M_u = 703.64\ k$

$\Delta_{max_T} \Rightarrow \frac{P_u a}{24EI}(3l^2 - 4a^2)$ for the 2 outer P $+ \frac{Pl^3}{48EI}$ for the center P

$a = 7' \quad P_u = 50.26 \quad l = 28'$

$\Delta_{max_T} \le 1.5''$

$$\frac{50.26^k(7')(1728)}{24(29000^k)(1.5'')}\left(3(28')^2 - 4(7')^2\right) + \frac{50.26(28^3)(1728)}{48(29000)(1.5)} = I_T$$

$I_T = 6.5823(2156) + 913.08 = 2169\ in^4$

(3)

TECH II 10-28-09 | NON COMPOSITE STEEL | JIM ROTUNNO

GIRDER CONT.

I_{CL} ⇒ for construction loading

$P_u \Rightarrow .2[1.2(48+1.5)(15)(7.0) + 1.6(20)(15)(7.0)] = 19.19$

$a = 7'$ $l = 28'$ $\Delta_{CL} \leq 1''$

$$I_{CL} = \frac{19.19(7')(1728)}{24(29000)(1'')}\left(3(28^2) - 4(7^2)\right) + \frac{19.19(28^3)(1728)}{48(29000)(1'')}$$

$= 0.33351(2156) + 522.94$

$= 1242\ in^4$

$I_{LL} \Rightarrow P_u = 1.6(80)(15)(7)(2) = 26.88^k$

$$I_{LL} = \frac{26.88(7')(1728)}{24(29000)(1'')}\left(3(28^2) - 4(7^2)\right) + \frac{26.88(28^3)(1728)}{48(29000)(1'')}$$

$= 0.46716(2156) + 732.5$

$= 1740\ in^4$

$I_T = 2169\ in^4$ controls, $M_u = 763.64$, $V_u = 75.39$

CHOICES

W 18 X 119	$I_x = 2190\ in^4$	$\phi M_n = 983$ k·ft	$\phi V_n = 373$ k
W 21 X 101	$I_x = 2420\ in^4$	$\phi M_n = 949$ k·ft	$\phi V_n = 326$ k
W 24 X 84	$I_x = 2370\ in^4$	$\phi M_n = 840$ k·ft	$\phi V_n = 340$ k
W 27 X 84	$I_x = 2850\ in^4$	$\phi M_n = 915$ k·ft	$\phi V_n = 369$ k

(4)

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Appendix D

Table 2.3—Minimum cover in concrete floors and roof slabs

Aggregate type	Cover*† for corresponding fire resistance, in.					
	Restrained	Unrestrained				
	4 or less	1 hour	1-1/2 hours	2 hours	3 hours	4 hours
Nonprestressed						
Siliceous	3/4	3/4	3/4	1	1-1/4	1-5/8
Carbonate	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Semi-lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Lightweight	3/4	3/4	3/4	3/4	1-1/4	1-1/4
Prestressed						
Siliceous	3/4	1-1/8	1-1/2	1-3/4	2-3/8	2-3/4
Carbonate	3/4	1	1-3/8	1-5/8	2-1/8	2-1/4
Semi-lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4
Lightweight	3/4	1	1-3/8	1-1/2	2	2-1/4

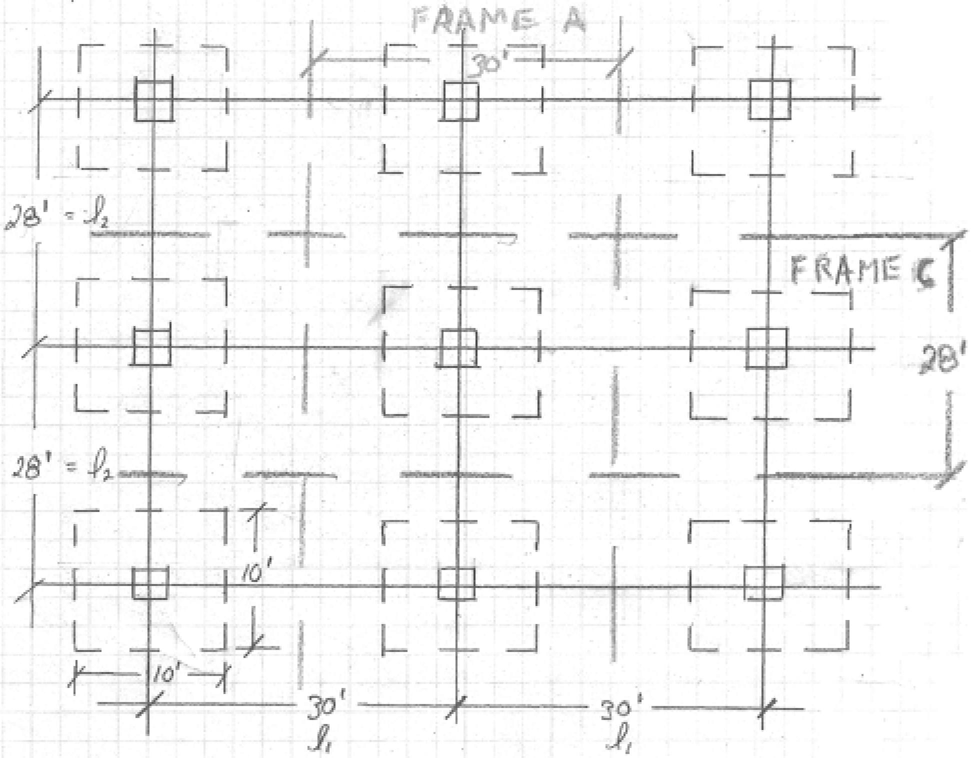
*Shall also meet minimum cover requirements of 2.3.1.

†Measured from concrete surface to surface of longitudinal reinforcement.

Table taken from ACI 216.1 - 07

TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

NO CAPITALS DESIGNED AT TOP OF COLUMNS UNDER DROP PANELS, DROP PANELS WILL BE INCREASE IN DEPTH IF NEEDED TO CONTROL SHEAR.

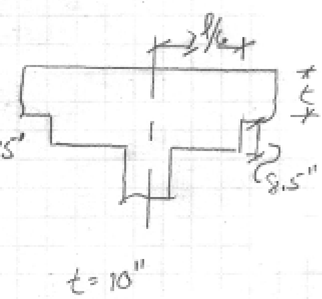


drop panel dimensions on each side of colum

$\geq l/6 \Rightarrow l =$ ℄ - ℄ of columns

long direction $l/6 = 30/6 = 5'$

short direction $l/6 = 28/6 = 4.67 \Rightarrow 5'$



∴ 10'×10' drop panels

thickness of drop = 8.5"

slab thickness = 10"

(1)

TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

TOTAL FACTORED STATIC MOMENT

$M_o = w_u l_2 l_n^2/8$ $l_2 = 28'$ $l_n = 28' = 30'-2'$

$= 320(28)(28^2)/8$ $w_u \Rightarrow$ $LL = 80psf$

$= 878$ k·ft $DL = \frac{10''}{12''}(150psf) = 125psf$

* Note: does not include drop panel thickness $125 + 35 MEP = 160psf$

$w_u = 1.2(160) + 1.6(80) = 320psf$

Determine t:

From Table 9.5c $\Rightarrow$ with drop panels, interior spans

$f_y = 60\,000psi$ $l_n = 30'-2' = 28'$

$t_{min} = l_n/36 = 28(12'')/36 = 9.33in < 10''$ OK

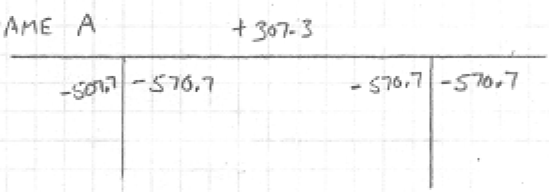
Distribution of Total Static Moment, M_o

ACI 13.6.3.2 In an interior span, total factored static moment, M_o, shall be distributed as follows:

Negative factored Moment $0.65(878) = 570.7$ k·ft

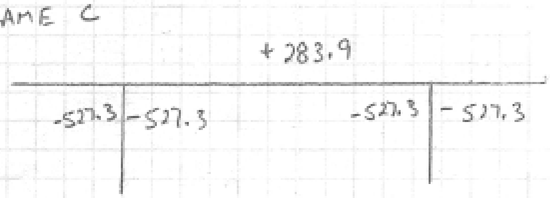
Positive factored Moment $0.35(878) = 307.3$ k·ft

FRAME A



long direction

FRAME C



short direction

(2)

TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

Distribution of longitudinal Moments to C.S + M.S (Column + Middle strips)

From ACI 13.6.4.1
Column strip interior negative factored moment
for $\alpha = 0 \Rightarrow 75\%$

From ACI 13.6.4.4
Column strip interior positive factored moment
for $\alpha = 0 \Rightarrow 60\%$

Moments in C.S. + M.S. Frame A + C

	A M^-	A M^+	C M^-	C M^+
Total Moment	-570.7	+307.3	-527.3	+283.9
% to C.S.	75%	60%	75%	60%
Moment in C.S.	-428	+184.4	-395.5	+170.3
Moment in M.S.	-142.7	+122.9	-131.8	+113.6

C.S. DESIGN OF SLAB REINFORCEMENT FRAME A + C

		A M^-	A M^+	C M^-	C M^+
1	M_u	-428	184.4	-395.5	170.3
2	b width of C.S. or drop panel	120"	180"	120"	168"
3	d effective depth	17.44"	16.81"	17.44"	16.81"
4	$M_n = M_u/\phi$ $\phi = 0.9$	-475.6	204.9	-439.4	189.2
5	$R = M_n/bd^2 = \frac{M_n(12000)}{bd^2}$	156.4	48.34	149.5	47.83
6	ρ required ⇒ From table	0.00267	0.00080	0.00246	0.00080
7	A_s required $= \rho bd$	5.59	2.42	5.15	2.26
8	A_s minimum $= 0.0018bt$	5.076	5.076	4.5	4.5
9	$N = \frac{7 \text{ or } 8}{\#5}$ $\#5 = 0.31$ or $\frac{0.60}{0.79}$	18 #5	17 #5	17 #5	15 #5
10	$N_{minimum} = \frac{\text{width of strip}}{2t}$	4	5	4	5

long direction rebar is lower ∴ $d = 18.5 - 3/4" - 0.625/2 = 17.44"$
short direction rebar $d' = d - 0.625" = 16.81"$

For A $bt = 120"(18.5") + 60"(10") = 2820$
C $bt = 120"(18.5") + 28(10") = 2500$

(3)

TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

M.S. DESIGN OF SLAB REINFORCEMENT FRAMES A & C

		A		C	
		M^-	M^+	M^-	M^+
1	M_u (k.ft)	-142.7	122.9	-131.8	113.6
2	b = width of strip (in)	180"	180"	168"	168"
3	d = effective depth (in)	7.44"	6.81"	7.44"	6.81"
4	$M_n = M_u/\phi$ $\phi = 0.9$ (k.ft)	-158.6	136.6	-146.4	126.2
5	$R = \frac{M_n(12000)}{bd^2}$	191.01	196.37	188.92	194.37
6	$\rho_{required}$ FROM TABLE	0.0031	0.00318	0.00307	0.00315
7	$A_{s\,required} = \rho bd$	4.15	3.90	3.84	3.6
8	$A_{s\,minimum} = 0.0018\,bt$	3.24	3.24	3.024	3.024
9	$N = \frac{\text{larger of 7 or 8}}{\#5 \text{ or } \#7 \text{ or } \#8}$	14 #5	13 #5	13 #5	12 #5
10	$N_{minimum} = \frac{b}{2t}$	9	9	9	9

bt for Frame A = 1800 in^2

bt for Frame C = 1680 in^2

$2t = 2(10") = 20"$

$$R = \rho f_y (1 - 0.59\rho\, f_y/f'_c)$$

$$R = 60000\rho - 531000\rho^2$$

(4)

TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

SHEAR CAPACITY CHECK FOR WIDE BEAM ACTION (PANEL SHEAR)

$V_u \le \phi V_n = \phi V_c = \phi 2 \sqrt{f'_c}\, b_w d$

$= 0.75(2)\sqrt{4000}\,(120'')(18.5'')$

$\phi V_c = 210.61^k$

b_w = width of drop panel = 120''

d = total depth @ drop = 18.5''

$V_u = 320\text{psf}[(15' - 2' - 18.5'')(28') = 162.67^k$

w_u [(½ Frame C – width of Column – d)(ℓ_2)]

$\phi V_n \ge V_u \Rightarrow 210.6^k > 162.7^k \therefore$ OK

SHEAR CAPACITY CHECK FOR PUNCHING SHEAR

drop panel is 10' x 10'

$d/2$ = d in slab (more conservative)/2

$= 7.44''/2 = 3.72''$

$V_u = W_u \times \text{Area} \Rightarrow W_u = 0.32\ ^k/ft^2$ $A = \ell_2$ (Frame C – Col. width – d)

$= 0.320\ (28' \times 30' - 2' - 7.44'')$

$= 245.3^k$

least of

$V_c = 4\sqrt{f'_c}\, b_o d \Rightarrow b_o$ = perimeter $= 4(24'' + d) = 4(24 + 7.44)$ $= 125.76'' = 10.48'$

or $= 4\sqrt{4000}\,(125.76)(7.44) = 236.7^k$

$V_c = (2 + 4/\beta_c)\sqrt{f'_c}\, b_o d \Rightarrow \beta_c = 1$ for square

or $= 6\sqrt{f'_c}\, b_o d > 4\sqrt{f'_c}\, b_o d \therefore$ does not control

$V_c = (\alpha_s/b_o d + 2)\sqrt{f'_c}\, b_o d \Rightarrow \alpha_s$ = 40 for an interior column

$= (40/125.76(7.44) + 2)\sqrt{4000}\,(125.76)(7.44) =$

$= 2.0428(59175.98) = 120.88^k$ governs

(5)

TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

$(\alpha_s / b_o d + 2)\sqrt{f'_c}\, b_o d$ governs

$= 120.88^k$

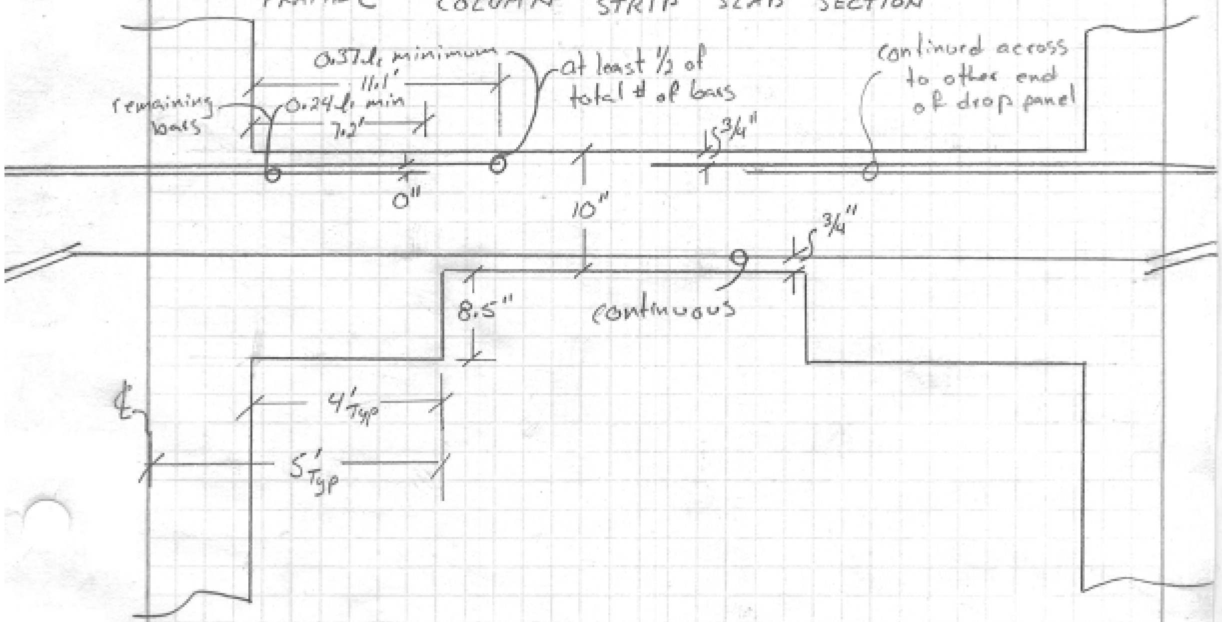
$\phi V_c = \phi V_n = 0.75(120.88) = 90.66^k$ ($\phi V_c = 212.5$ if $d = 17.44$ is used, d of drop panel)

$V_u > \phi V_n \Rightarrow 245.3^k > 90.66^k$ ∴ SHEAR REINFORCEMENT IS REQUIRED

THIS COULD BE ACCOMPLISHED BY UP SIZING THE REBAR ALREADY PRESENT FOR THE FLEXURAL REQUIREMENTS OR BY INCREASING THE NUMBER OF BARS FOR SHEAR REQUIREMENTS.

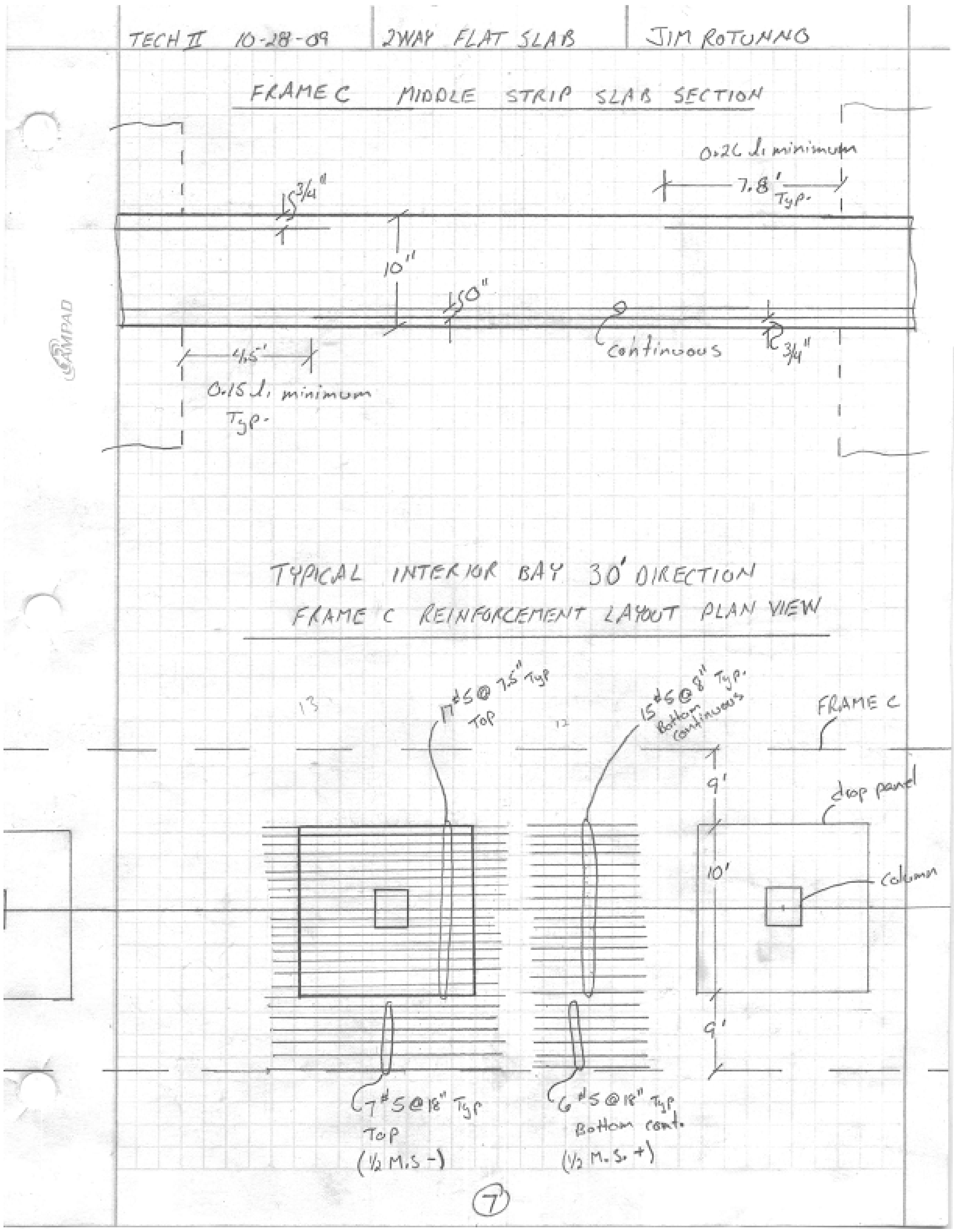
REBAR DESIGN DRAWINGS FOR FLEXURAL REINFOREMENT IN 30' DIRECTION

FRAME C COLUMN STRIP SLAB SECTION



N.T.S.

6


TECH II 10-28-09
2WAY FLAT SLAB
JIM ROTUNNO
FRAME C MIDDLE STRIP SLAB SECTION
0.2C ℓ₁ minimum
7.8' Typ.
3/4"
10"
0"
continuous
3/4"
4.5'
0.15 ℓ₁ minimum Typ.
TYPICAL INTERIOR BAY 30' DIRECTION
FRAME C REINFORCEMENT LAYOUT PLAN VIEW
17 #5 @ 7.5" Typ Top
15 #5 @ 8" Typ. Bottom continuous
FRAME C
9'
drop panel
10'
column
9'
7 #5 @ 18" Typ Top (1/2 M.S −)
6 #5 @ 18" Typ Bottom cont. (1/2 M.S. +)
7

TECH II 10-28-09 | 2WAY FLAT SLAB | JIM ROTUNNO

COLUMN SIZE CALCULATION ⇒ NOT REINFORCEMENT

ACI CODE 13.6.9 REQUIRES INTERIOR COLUMNS RESIST A MOMENT $M = 0.07[(W_d + 0.5W_L) l_2 l_n^2 - W_d' l_2' (l_n')^2]$

the primes refer to the shorter of the two adjacent spans

This is from a more severe loading due to removal of live load.

The column being analyzed is on the 2nd level supporting the 3rd level and above tributary areas and the self weight of columns above.

For the above equation
$l_2 = 28'$
$l_n = 30'$
$l_2' = 28'$
$l_n' = 30'$
$W_d' = W_d$

∴ The equation would reduce to just the W_L term

$M = 0.07(0.5 W_L l_2 l_n^2)$

⇒ W_L = all W_u due to LL on level 3 up to + including the roof level (no LL reduction)

$W_L = 1.6[3(80\text{psf}) + 40\text{psf}]$ 0.448^k

$M = 0.07(0.5)(0.448)(28)(30^2)$
$= 395$ k·ft

roof area has no equipment or access ∴ 40 psf

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TECH II 10-28-09 | 2 WAY FLAT SLAB | JIM ROTUNNO

COLUMN ANALYSIS CONT.

Total gravity load on Column at bottom of 3rd level floor drop panel levels 3, 5, 6 + roof above *(there is no level 4)

$3,5,6 \Rightarrow P_{u_D} = 1.2\left[3\left(\frac{10''}{12''}\right)(150\,pcf)(30')(28') + 3\left(\frac{8.5''}{12''}\right)(10')(10')(150) + 3(35)(30)(28)\right]$ (slabs, drops, MEP)

$= 522^k$

roof $\Rightarrow P_{u_D} \Rightarrow$ assuming same slab as floors with no MEP

$1.2\left[\left(\frac{10}{12}\right)(150)(30)(28) + \left(\frac{18.5}{12}\right)(10')(10')(150)\right]$ this is probably an over estimation

$P_{ud} = 139^k$

Columns $\Rightarrow$ Assuming 24'x24' as per the design aid

$P_{ud} = 3(14'\text{-}8'' - 18.5'')(2')(2')(150)$

$= 22.5^k$

$P_{u_{D\,Total}} = 522 + 139 + 22.5$

$= 684^k$

THE COLUMN SIZE OF 24"x24" WAS CHECKED WITH PCA COLUMN AND FOUND TO BE ADEQUATE

9

```
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OOOOOOO   OOOOO    OOOOO    OOOOO    OOOOO   OO
OO    OO OO   OO  OO   OO  OO   OO  OO   OO  OO
OO    OO OO   OO  OO   OO  OO   OO  OO   OO  OO
OO    OO OO       OO   OO  OO       OO   OO  OO
OO    OO OO       OOOOOOO  OO       OO   OO  OO
OOOOOOO  OO   OO  OO   OO  OO   OO  OO   OO  OO
OO       OO   OO  OO   OO  OO   OO  OO   OO  OO
OO        OOOOO   OO   OO   OOOOO    OOOOO   OOOOO   (TM)


=========================================================================
Computer program for the Strength Design of Reinforced Concrete Sections
=========================================================================
```

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  General Information:
  ====================
     File Name: \\aep.coeaccess.psu.edu\profiles$\jdr274\Desktop\PCA column TECH 2 .col
     Project:  TECH II
     Column:                                     Engineer:
     Code:     ACI 318-02                        Units: English

     Run Option: Design                          Slenderness: Not considered
     Run Axis:   X-axis                          Column Type: Structural

  Material Properties:
  ====================
     f'c    = 4 ksi                              fy    = 60 ksi
     Ec     = 3605 ksi                           Es    = 29000 ksi
     Ultimate strain = 0.003 in/in
     Beta1 = 0.85

  Section:
  ========
     Rectangular: Width = 24 in                  Depth = 24 in

     Gross section area, Ag =  576 in^2
     Ix =  27648 in^4                            Iy =  27648 in^4
     Xo =  0 in                                  Yo =  0 in

  Reinforcement:
  ==============
     Rebar Database: ASTM A615
     Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
     ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
     # 3      0.38        0.11    # 4      0.50        0.20    # 5      0.63        0.31
     # 6      0.75        0.44    # 7      0.88        0.60    # 8      1.00        0.79
     # 9      1.13        1.00    # 10     1.27        1.27    # 11     1.41        1.56
     # 14     1.69        2.25    # 18     2.26        4.00

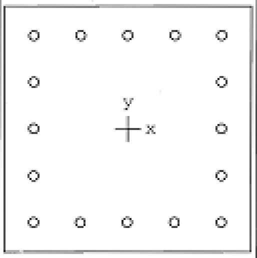
     Confinement: Tied; #3 ties with #8 bars,  #4 with larger bars.
     phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

     Layout: Rectangular
     Pattern: All Sides Equal  (Cover to transverse reinforcement)
     Total steel area, As = 12.64 in^2 at 2.19%
     16 #8    Cover = 2 in

  Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
  =========================================================
                    Pu          Mux          fMnx
     No.           kip         k-ft          k-ft    fMn/Mu
     --- ------------ ------------ ------------ --------
       1        684.0        395.0        539.1    1.365

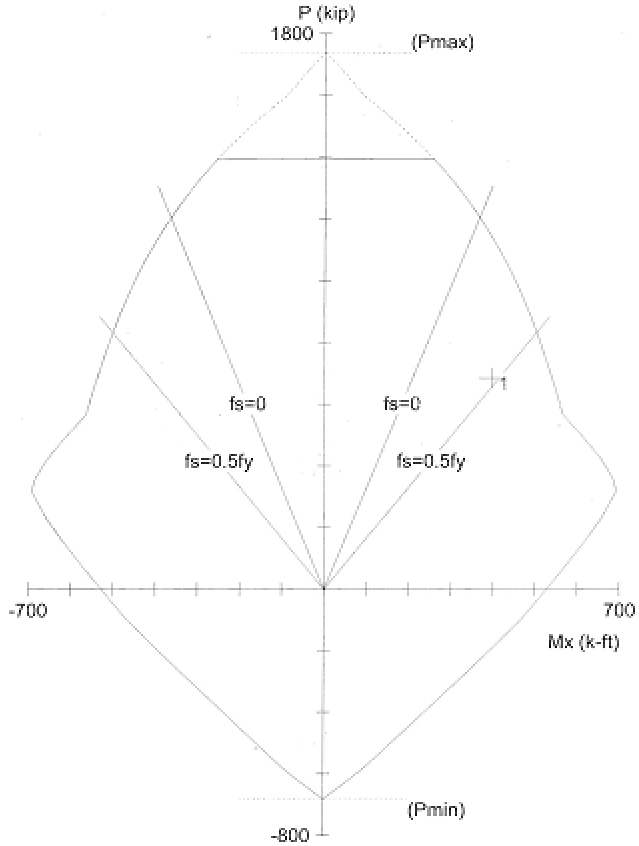
     *** Program completed as requested! ***
```

(11)



24 x 24 in

Code: ACI 318-02
Units: English
Run axis: About X-axis
Run option: Design
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 10/27/09
Time: 00:10:42



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File: \\aep.coeaccess.psu.edu\profiles$\jdr274\Desktop\PCA column TECH 2 .col
Project: TECH II

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	16 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 12.64 in^2	Rho = 2.19%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Clear spacing = 3.56 in	Clear cover = 2.38 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

(12)

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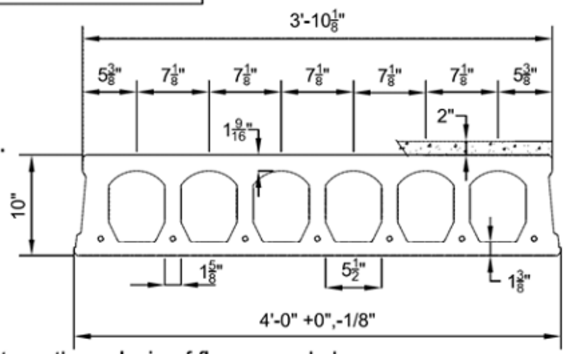
Appendix E

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	170	153	137	123	110	98	87	77	68	60	52					
7 - 0.6"ø	LOAD (PSF)			256	244	233	222	202	185	168	154	140	128	116	106	96	87	78	70	63

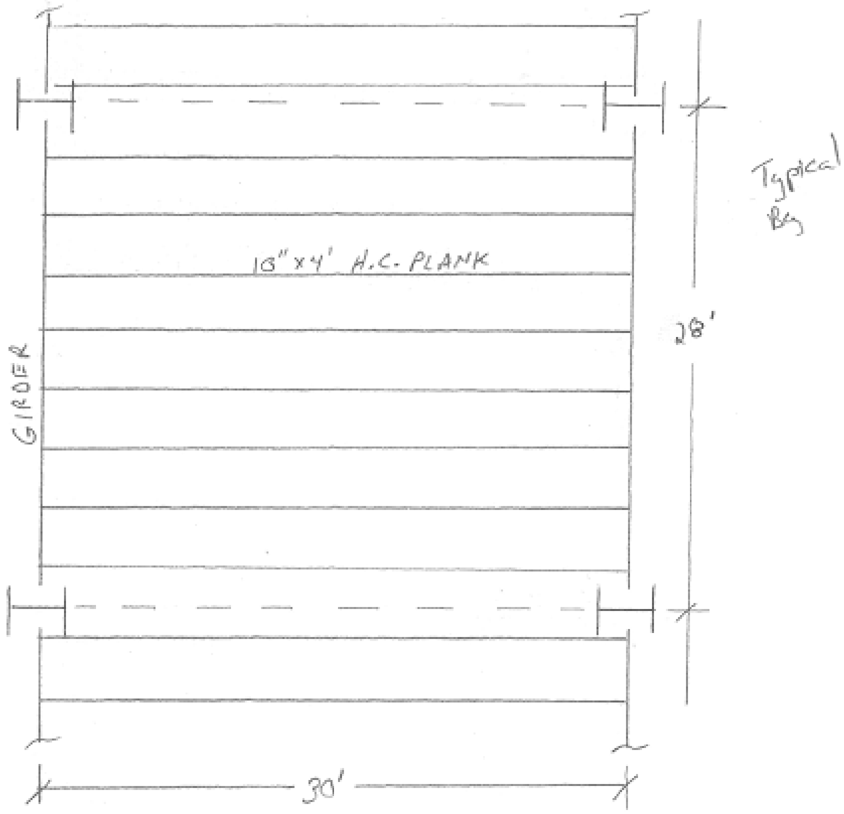
NITTERHOUSE CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
717-267-4505 Fax 717-267-4518

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

TECH II 10-28-09 | HOLLOW PLANK on STEEL | JIM ROTUNNO



PLANK DESIGN

W_{LL} = 80 psf

W_{DL} = 35 psf, MEP, partitions, finishes

W_T = Total Superimposed service load for Nitterhouse Table See Figure 2.22 in report = 115 psf

FROM NITTERHOUSE DESIGN GUIDE 10F2.0T 5/14/07 a safe superimposed service load of 153 psf, which includes live load @ 30' span.

153 > 115 ∴ ok

①

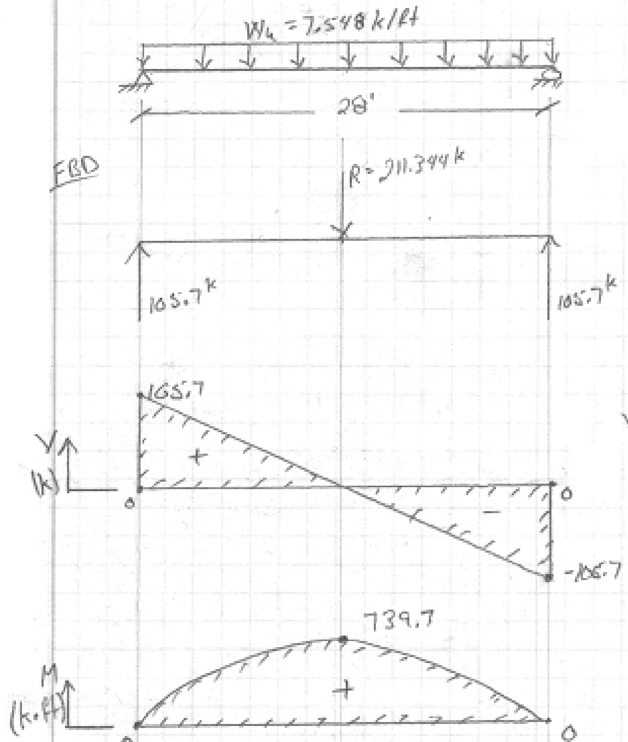
TECH II 10-28-09 | HOLLOW PLANK on STEEL | JIM ROTUNNO

DESIGN of STEEL GIRDER

Span = 28'

The Girder is assumed to be fully braced along its length by the grouted hollowcore plank

The ends of the girder are also assumed to be simply supported by the shear connections to the columns



$W_u \Rightarrow 1.2D + 1.6L$

weight of plank = 68psf

68psf(30') = 2040 lb/ft

MEP / partitions / finishes

35(30') = 1.05 k/ft

LL → 80psf(30') = 2.4 k/ft

1.2D + 1.6L

$W_u = 1.2(2.04 + 1.05) + 1.6(2.4) = 7.548$ k/ft

$\Delta_{LL} \leq L/360 = 28(12)/360 = 0.933 \text{ in} = \frac{5}{384}\left(\frac{w l^4}{EI}\right)$

$\Delta_{Total} \leq L/240 = 28(12)/240 = 1.4 \text{ in}$

(2)

TECH II 10-28-09 | HOLLOW PLANK on STEEL | JIM ROTUNNO

CALCULATE I_x needed

$$I_{x\ \text{for LL}} = \frac{5}{384}\left(\frac{1.6(2.41)(28^4)(1728)}{29000(0.933)}\right) = 1963\ in^4$$

$$I_{x\ \text{for Total}} = \frac{5}{384}\left(\frac{7.548(28^4)(1728)}{29000(1.4)}\right) = 2571\ in^4$$

Need W section will properties of

$\phi M_n \geq 740$ + $I_x \geq 2571\ in^4$. Shear will not Control

Choices

W 18 x 143	$\phi M_n = 1210$ k·ft	$I_x = 2750\ in^4$
W 21 x 111	$\phi M_n = 1050$ k·ft	$I_x = 2670\ in^4$
W 24 x 94	$\phi M_n = 953$ k·ft	$I_x = 2700\ in^4$
W 27 x 84	$\phi M_n = 915$ k·ft	$I_x = 2850\ in^4$

THE W18X143 would be the least depth Member

⇒ The W 27 x84 would be the best choice based on weight and will still not increase floor to floor heights

*Note: The addition of the angles onto the girder were not taken into consideration for the calculation of Ix, which would increase the value ∴ making it possible to select an even smaller member. (Again this was only a preliminary not exhaustive analysis)

③

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