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Bronx, NY
Structural Option
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Structural Tech Report #2

Pro – Con Structural Study of Alternate Floor Systems

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1.0 Executive Summary

Introduction

This report analyzes and compares alternate floor systems to the existing floor system at Fordham Place. The alternate systems were chosen based on design considerations listed below. Assumptions were made and each system was compared based on the advantages and disadvantages to determine a viable alternate system.

Existing Floor System

Composite steel beams with composite slab on metal deck

This system is suited very well for Fordham Place. It is really quick and easy to erect which can be important in a busy city like Bronx. It is also more economical than concrete systems because there is no height limitation in Bronx, therefore floor sandwich depth is not a factor.

Alternate Floor System

Two – Way flat plate

A two – way flat plate is an average design. Concrete is primarily used to decrease floor sandwich depth. However, as noted before that is not major factor. Also if a two way system would be used, a two – flat slab with drop panels is a more efficient design.

Two – way flat slab with drop panels

As discussed above this system is better than the flat plate because the extra concrete in the drop panels gives the system a higher moment capacity where it is needed (column supports).

Two – way waffle slab

This system was considered because it performs the same as any other two way system, but the geometry allows a design that does not require as much concrete as the other two – way systems. However, it is very unsightly and will not work well with duct work.

One – way pan joist

One – way concrete floor systems are primarily used for bays that are not so square. This bad thing about this design, and the other concrete systems, is they do not work well with duct work and electrical lines.

Open web steel joist

This is a risky design for and office building due to its suseptability to floor vibrations. It is the designers engineering judgment, but I would avoid Open web steel joist.

Non – composite steel beams with concrete slab on metal deck

This is a good design; however a composite design yields smaller members, and therefore is the better design.

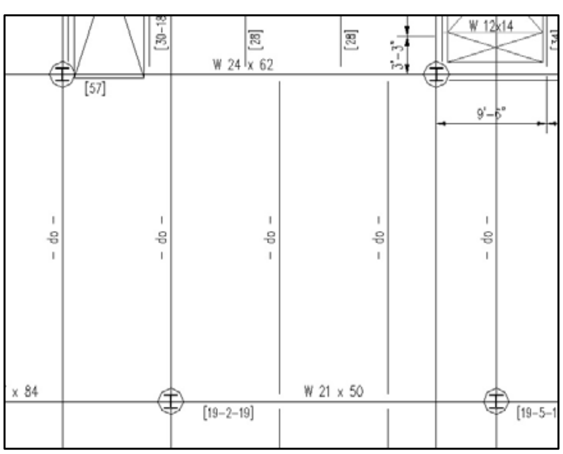
Conclusion: Structurally, all options are viable. However only the steel systems and the flat slab with drop panels are architecturally and structurally viable. The best system is the current composite steel beams with composite slab on metal deck.

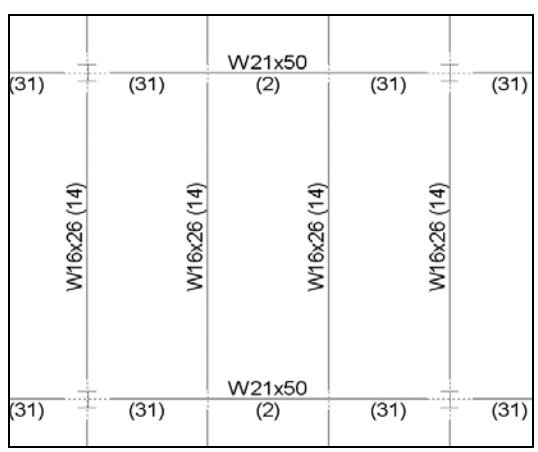
Floor Design Considerations

Cost, Constructability, Floor sandwich depth, Flexibility of floor (holes for ductwork etc.), Vibrations, Weight, Fire Protection, Durability, Erection time.

Building Description

Fordham Place is a 15 story office / retail building that is located at 400 East Fordham Road, Bronx, NY. The 174060 sq. ft tower is going to tie into an existing 6 story SEARS building. In the new tower, structural engineers used modern design, taking advantage of composite action using steel beams with a 6 ¼ " concrete slab. The slab will be supported by 3" composite floor deck with 3" headed shear studs within the slab. Steel columns are used to transfer load to foundation, where it will be supported by a number of 150 ton piles. The main lateral resisting system is made up of steel concentrically loaded chevron braced frames.
The framing plan of a typical office floor at Fordham Place is quite complex. Columns are shifted from bent to bent leaving no "typical" bay. Due to the implications brought about by these "not so typical" bays when comparing floor systems and the difficulty in communicating the results to a client or architect, I decided to try and simplify the framing plan. The main goal in doing this is to be able to produce a typical square bay that is structurally equivalent to the current design. To do this, typical beam spacing and span was kept at 9'-4" and 27'-9" respectively, which in turn did not change tributary areas. Therefore the beam design should theoretically be the same. The girder's span of which I plan analyze will stay at 28'.However columns will be shifted so that the bay will be a 27'-9" x 28' square bay. To prove that this proposed typical bay is indeed the same as the current design, I modeled this typical square bay in RAM using the exact design criteria that was used for the current design. When this was done, the square bay design was very similar as the current bay design. From this point on, I will be analyzing and comparing different floor systems using the proposed square bay. (Pictures show the differences between the two designs)

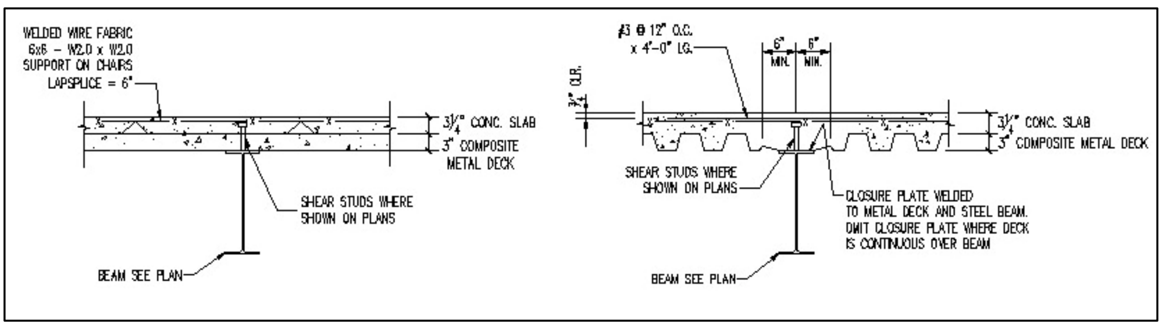




2.0 Existing System

2.1 Composite steel beams with composite slab on metal deck

The floor system of Fordham Place consists of structural steel W sections that support metal deck and concrete slab. The W shape beams and girders are A992 grade 50 and support a light weight concrete (115pcf) slab of 6.25 in. The concrete's compressive strength is f'_c = 3000psi for all floors. Reinforcing of concrete is done with high strength billet deformed steel bars with fy = 60,000psi as a minimum. All floor deck is 20 gage 3" deep galvanized composite deck and is continuous over 2 spans at the joints of the deck. All shear studs are headed studs of grade 1015 or 1020 cold finish carbon steel. Studs, at a maximum are spaced every 12".



<u>**Design Criteria:**</u>

DL = 60psf
LL = 80psf
Wu = 1.2DL + 1.6LL
=200psf

F'c = 4ksi (LWT)
fy = 60ksi
Typical bay size: 28' x 27'9"

<u>Advantages</u>

- Reduces floor vibrations induced by walking
- Composite action yields smaller member sizes
- Easy to erect
- Works well with ductwork and electrical lines running through floor
- Relatively light system
- No cost for formwork

<u>Disadvantages</u>

- Floor sandwich depth is large
- Needs additional fire proofing to meet standards

3.Alternate systems

3.1Two – way flat plate

I chose to consider this system as an option because the bays at Fordham place are square and do not span a very long distance. This geometric setup makes a two way system an efficient design. This system was designed by hand per ACI and using CRSI. When designing with CRSI, load factors were 1.4DL + 1.7LL as opposed to 1.2DL + 1.6LL. There were differences in the two designs but it was just the selection of bar sizes and spacing.

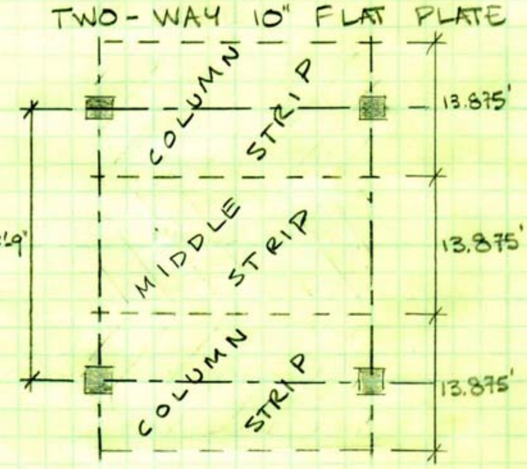
<u>Advantages</u>

- Small Floor sandwich depth compared to that of a steel system
- Reduces floor vibrations induced by walking
- Concrete provides required fire protection rating therefore eliminating the need for any additional fire proofing.

<u>Disadvantages</u>

- A two way concrete floor system is not a preferred design for an office building because office buildings usually use a central heating system which means holes will be cut in the slab for ductwork. These holes decrease the capacity of the concrete significantly.
- This system will be heavier than a typical steel building making the foundations larger. This heavier system will produce a greater seismic loading, increasing the need for a larger lateral force resisting system.
- Construction costs will be larger due to the formwork and shoring than that of a steel building where metal deck is used.

TWO-WAY 10" FLAT PLATE SYSTEM



TYPICAL INTERIOR BAY

ASSUMPTIONS:

24" COLUMNS
$f'_c = 4$ KSI
$f_y = 60$ KSI

$0.25\,l_2 = 0.25(27'\text{-}9") = 6.9375'$

C.S = 6.9375' x 2 = 13.875'
M.S = 27.75' - 13.875 = 13.875'

LOADING

LL = 80 PSF
SDL = 60 PSF
SELF = (10/12) 150 PCF = 125 PSF

$(t_s)_{MIN} = \ell_n/36 = 26/33 = 9.5" \leq 10"$ OK

$w_u = 1.2(125 + 60) + 1.6(80)$
$w_u = 350$ PSF

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{(350)(27.75)(26)^2}{8}$$

$\ell_n = 28' - 2' = 26'$
$\ell_2 = 27.75'$

$M_o = 821$ k-ft

M_o	NEG. SUPPORT $= 0.65 M_o$	C.S. = (0.65)(0.75) M_o
		M.S. = (0.65)(0.25) M_o
	POS. MIDSPAN $= 0.35 M_o$	C.S. = (0.35)(0.6) M_o
		M.S. = (0.35)(0.4) M_o

* THE FOLLOWING TABLE LISTS THE COMPUTATED TOTAL MOMENT AT THE GIVEN LOCATIONS

$821^{k\text{-}ft}$	$553.7^{k\text{-}ft}$	$400.3^{k\text{-}ft}$
		$133.4^{k\text{-}ft}$
	$287.4^{k\text{-}ft}$	$172.4^{k\text{-}ft}$
		$115.0^{k\text{-}ft}$

TWO-WAY 10" FLAT PLATE SYSTEM CON'T

DESIGN OF SLAB WILL BE DONE ON A PER FOOT BASIS

NEGATIVE SUPPORT COLUMN STRIP MOMENT

$M_u = 400.3/13.875$

$M_u = 28.8\ ^{k\text{-}ft}/_{ft}$

NEGATIVE SUPPORT MIDDLE STRIP MOMENT

$M_u = 133.4/13.875$

$M_u = 9.6\ ^{k\text{-}ft}/_{ft}$

POSITIVE MIDSPAN COLUMN STRIP MOMENT

$M_u = 172.4/13.875$

$M_u = 12.4\ ^{k\text{-}ft}/_{ft}$

POSITIVE MIDSPAN MIDDLE STRIP MOMENT

$M_u = 115.0/13.875$

$M_u = 8.3\ ^{k\text{-}ft}/_{ft}$

COVER ASSUME #8 TO MIDDLE OF #8 BAR

$d = 10'' - 0.75 - 1 - 0.5$

$d = 7.75$ IN

ASSUME $a = 2$ IN

$M_n = \frac{M_u}{\phi} = A_s f_y (d - a/2)$

$\frac{(12)(28.8)}{0.9} = A_s (60)(7.75 - 2/2)$

$A_s = 0.95\ in^2$ TRY 2-#7

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

$(1.2)(60) = 0.85(4)(12)\, a$

$a = 1.76$ IN

$M_n = A_s f_y (d - a/2)$
$= (1.2)(60)(7.75 - 1.76/2)$

$M_n = 41.2^{k\text{-}ft}$ $\quad \phi M_n = 37.1^{k\text{-}ft} \geq 28.8$ TRY 2-#7 @ 6"

TWO - WAY 10" FLAT PLATE SYSTEM CONT'

CHECK MAX SPACING: $S_{MIN} = 2t_s = 20"$ OK

CHECK DUCTILITY: $c \leq 0.375d$
$2.07 \leq 2.9$ OK

USE 2-#7's @ 6" SPACING FOR COLUMN STRIP AT SUPPORTS (TOP BARS)

$M_n = \frac{M_u}{\phi} = A_s f_y (d - a/2)$

$= \frac{(12.4)(12)}{0.9} = A_s (60)(7.75 - 1/2)$ ASSUME $a = 1"$

$A_s = 0.38 \text{ in}^2$

TRY 1-#6 @ 12"

$A_s f_y = 0.85 f'_c b a$
$(0.4)(60) = 0.85(4)(12) a$

$a = 0.59$

$M_n = A_s f_y (d - a/2)$
$= (0.4)(60)(7.75 - 0.59/2)$

$M_n = 14.9^{k\text{-}ft}$ $\phi M_n = 13.4^{k\text{-}ft} \geq 12.4$

MAX SPACING OK

CHECK DUCTILITY: $c \leq 0.375d$
$0.69 \leq 2.9$ OK

USE 1-#6 @ 12" SPACING FOR: MIDDLE STRIP AT SUPPORTS (TOP BARS)
MIDDLE STRIP AT MIDSPAN (BOTTOM BARS)
COLUMN STRIP AT MIDSPAN (BOTTOM BARS)

DESIGN IS BASED ON CRITICAL DIRECTION.
SINCE OTHER DIRECTION IS JUST 3" SHORTER,
USE SAME DESIGN.

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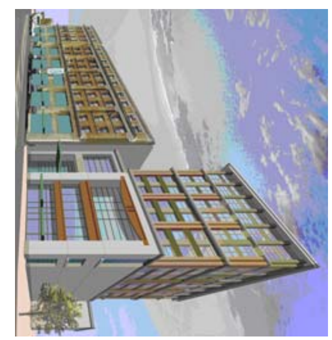


FLAT PLATE SYSTEM (WITHOUT SHEARHEADS) — f'_c = 4,000 psi, Grade 60 Bars

SQUARE EDGE PANEL

SPAN c.-c. Cols. $\ell_1 = \ell_2$	Factored Superimposed Load	(1) Min. Square Column		Total Panel Moments			Reinforcing Bars					End Panel Steel (psf)		
				−M Ext.	+M Int.	−M 1st. int.	Each Column Strip			Each Middle Strip		Location of Panel		
							Top Ext. +	Bottom	Top Int.	Bottom	Top Int.	E	EC	C
(ft)	(psf)	(in.)	γ_f	(ft-kip)	(ft-kip)	(ft-kip)								
10 in. = TOTAL THICKNESS OF SLAB														0.833 c.f./s.f.
26	50	20	0.762	115	230	309	12-# 5 4	12-# 5	15-# 6	10-# 5	10-# 5	2.72	2.74	2.74
26	100	24	0.724	136	272	367	12-# 5 5	14-# 5	13-# 7	10-# 5	10-# 5	2.96	2.98	2.96
26	150	28	0.685	157	313	421	14-# 5 4	9-# 7	12-# 8	11-# 5	10-# 5	3.33	3.37	3.42
26	200	32	0.677	175	350	471	16-# 5 5	10-# 7	13-# 8	12-# 5	10-# 5	3.70	3.73	3.83
26	250	36	0.612	192	384	517	12-# 6 2	20-# 5	15-# 8	10-# 6	11-# 5	4.11	4.14	4.29
26	300	41	0.613	205	411	553	13-# 6 3	9-# 8	16-# 8	11-# 6	12-# 5	4.47	4.52	4.71
26	350	47	0.610	216	431	580	19-# 5 3	10-# 8	17-# 8	11-# 6	9-# 6	4.72	4.77	4.96
27	50	22	0.741	128	256	345	12-# 5 5	10-# 6	12-# 7	10-# 5	10-# 5	2.80	2.80	2.74
27	100	26	0.708	151	303	407	13-# 5 5	16-# 5	12-# 8	11-# 5	10-# 5	3.11	3.16	3.17
27	150	31	0.675	173	346	466	15-# 5 6	10-# 7	13-# 8	12-# 5	10-# 5	3.50	3.54	3.66
27	200	35	0.652	194	387	521	12-# 6 4	11-# 7	15-# 8	10-# 6	12-# 5	4.02	4.05	4.20
27	250	40	0.611	211	422	568	19-# 5 3	12-# 7	16-# 8	11-# 6	9-# 6	4.41	4.42	4.65
27	300	46	0.610	224	447	602	14-# 6 3	10-# 8	17-# 8	11-# 6	13-# 5	4.61	4.67	4.95
27	350	53	0.609	233	466	628	15-# 6 2	9-# 9	18-# 8	9-# 7	10-# 6	5.13	5.18	5.32
28	50	24	0.706	142	283	381	13-# 5 4	11-# 6	14-# 7	10-# 5	10-# 5	2.86	2.89	2.95
28	100	28	0.722	168	335	451	15-# 5 6	10-# 7	13-# 8	12-# 5	10-# 5	3.33	3.36	3.50
28	150	33	0.665	192	383	516	17-# 5 5	20-# 5	15-# 8	10-# 6	11-# 5	3.80	3.81	3.94
28	200	37	0.668	214	428	576	19-# 5 6	12-# 7	16-# 8	11-# 6	13-# 5	4.24	4.26	4.50
28	250	44	0.616	230	460	619	20-# 5 5	10-# 8	18-# 8	16-# 5	10-# 6	4.49	4.56	4.82
28	300	52	0.609	241	483	650	15-# 6 3	11-# 8	19-# 8	12-# 6	10-# 6	4.90	4.97	5.18
28	350	59	0.608	252	504	678	16-# 6 2	11-# 8	20-# 8	10-# 7	11-# 6	5.25	5.32	5.46
29	50	26	0.730	156	312	420	14-# 5 7	12-# 6	15-# 7	11-# 5	11-# 5	3.03	3.05	3.07
29	100	31	0.665	184	369	496	16-# 5 5	14-# 6	14-# 8	13-# 5	11-# 5	3.49	3.53	3.69
29	150	36	0.644	210	421	566	13-# 6 4	12-# 7	16-# 8	11-# 6	13-# 5	4.03	4.08	4.14
29	200	42	0.611	233	466	627	15-# 6 2	10-# 8	18-# 8	16-# 5	10-# 6	4.39	4.45	4.69
29	250	50	0.609	248	496	667	16-# 6 2	11-# 8	19-# 8	10-# 7	11-# 6	4.91	4.97	5.11
29	300	57	0.608	261	521	702	23-# 5 4	10-# 9	20-# 8	10-# 7	11-# 6	5.22	5.29	5.59
29	350	65	0.607	270	541	728	17-# 6 2	10-# 9	21-# 8	10-# 7	16-# 5	5.43	5.50	5.88
30	50	28	0.699	171	343	462	15-# 5 6	10-# 7	17-# 7	12-# 5	11-# 5	3.21	3.23	3.26
30	100	33	0.692	203	406	546	18-# 5 7	12-# 7	15-# 8	10-# 6	12-# 5	3.77	3.78	3.86
30	150	39	0.642	231	462	622	20-# 5 6	10-# 8	18-# 8	16-# 5	10-# 6	4.21	4.26	4.41
30	200	47	0.616	251	502	676	16-# 6 4	11-# 8	19-# 8	10-# 7	11-# 6	4.74	4.79	4.95
30	250	55	0.608	267	534	718	17-# 6 2	10-# 9	21-# 8	10-# 7	11-# 6	5.14	5.20	5.46
30	300	63	0.607	280	560	754	18-# 6 1	10-# 9	22-# 8	14-# 6	12-# 6	5.35	5.42	5.77
30	350	71	0.607	290	579	780	19-# 6 0	13-# 8	23-# 8	20-# 5	12-# 6	5.58	5.68	6.00
31	50	30	0.707	188	376	506	17-# 5 7	14-# 6	14-# 8	13-# 5	11-# 5	3.28	3.33	3.46
31	100	35	0.705	222	444	597	14-# 6 6	13-# 7	17-# 8	11-# 6	13-# 5	3.92	3.97	4.16
31	150	43	0.655	250	500	673	16-# 6 5	11-# 8	19-# 8	13-# 6	11-# 6	4.53	4.58	4.75
31	200	52	0.609	270	541	728	17-# 6 4	12-# 8	21-# 8	14-# 6	16-# 5	4.93	4.99	5.14
31	250	61	0.608	287	573	772	18-# 6 3	13-# 8	22-# 8	20-# 5	12-# 6	5.19	5.26	5.62
31	300	69	0.607	300	600	808	19-# 6 1	17-# 7	23-# 8	15-# 6	13-# 6	5.50	5.58	5.88
31	350	78	0.606	310	620	835	20-# 6 0	11-# 9	24-# 8	12-# 7	13-# 6	5.89	5.95	6.34

SQUARE INTERIOR PANEL

(2) Span c.-c.	(3) Load	(1) Min. Sq. Col.	Reinforcing Bars				Steel (psf)		
			Column Strip		Middle Strip		Location of Panel		
(ft)	(psf	(in.)	Top	Bottom	Top	Bottom	I	IE	IC
10 in. = TOTAL THICKNESS OF SLAB									0.833 c.f./s.f.
26	50	14	14-# 6	10-# 5	10-# 5	10-# 5	2.76	2.77	2.79
26	100	19	13-# 7	10-# 5	10-# 5	10-# 5	3.01	3.04	3.04
26	150	23	14-# 7	11-# 5	10-# 5	10-# 5	3.23	3.27	3.31
26	200	28	13-# 8	9-# 6	10-# 5	10-# 5	3.69	3.69	3.69
26	250	33	14-# 8	10-# 6	11-# 5	10-# 5	3.98	4.01	4.04
26	300	40	15-# 8	11-# 6	11-# 5	10-# 5	4.26	4.30	4.35
26	350	48	15-# 8	11-# 6	12-# 5	10-# 5	4.35	4.43	4.50
27	50	15	12-# 7	10-# 5	10-# 5	10-# 5	2.82	2.81	2.81
27	100	21	14-# 7	11-# 5	10-# 5	10-# 5	3.10	3.14	3.18
27	150	26	12-# 8	9-# 6	10-# 5	10-# 5	3.42	3.45	3.48
27	200	31	14-# 8	10-# 6	11-# 5	10-# 5	3.81	3.85	3.89
27	250	37	15-# 8	11-# 6	12-# 5	10-# 5	4.12	4.16	4.20
27	300	46	16-# 8	16-# 5	12-# 5	11-# 5	4.37	4.42	4.46
27	350	55	17-# 8	9-# 7	9-# 6	11-# 5	4.72	4.77	4.82
28	50	17	13-# 7	10-# 5	10-# 5	10-# 5	2.81	2.83	2.85
28	100	23	15-# 7	12-# 5	10-# 5	10-# 5	3.14	3.19	3.23
28	150	28	14-# 8	10-# 6	11-# 5	10-# 5	3.66	3.69	3.72
28	200	34	15-# 8	11-# 6	12-# 5	10-# 5	3.95	3.99	4.03
28	250	43	16-# 8	16-# 5	13-# 5	11-# 5	4.23	4.31	4.38
28	300	53	17-# 8	12-# 6	13-# 5	11-# 5	4.49	4.57	4.65
28	350	62	18-# 8	10-# 7	10-# 6	12-# 5	4.93	5.01	5.09
29	50	19	14-# 7	11-# 5	11-# 5	11-# 5	2.98	3.00	3.02
29	100	25	13-# 8	13-# 5	11-# 5	11-# 5	3.39	3.42	3.45
29	150	31	15-# 8	11-# 6	12-# 5	11-# 5	3.87	3.90	3.94
29	200	38	17-# 8	12-# 6	13-# 5	11-# 5	4.25	4.29	4.33
29	250	49	18-# 8	10-# 7	10-# 6	12-# 5	4.68	4.72	4.76
29	300	60	18-# 8	10-# 7	10-# 6	12-# 5	4.74	4.81	4.89
29	350	70	19-# 8	10-# 7	11-# 6	13-# 5	5.05	5.11	5.18
30	50	21	16-# 7	13-# 5	11-# 5	11-# 5	3.18	3.20	3.22
30	100	27	15-# 8	11-# 6	11-# 5	11-# 5	3.68	3.69	3.70
30	150	33	17-# 8	12-# 6	13-# 5	11-# 5	4.08	4.12	4.16
30	200	44	18-# 8	10-# 7	10-# 6	12-# 5	4.49	4.53	4.57
30	250	56	19-# 8	10-# 7	11-# 6	13-# 5	4.79	4.85	4.91
30	300	67	20-# 8	14-# 6	11-# 6	13-# 5	5.00	5.08	5.15
30	350	78	20-# 8	20-# 5	11-# 6	13-# 5	5.05	5.16	5.27
31	50	23	17-# 7	14-# 5	11-# 5	11-# 5	3.21	3.24	3.26
31	100	30	16-# 8	16-# 5	12-# 5	11-# 5	3.71	3.75	3.78
31	150	38	18-# 8	13-# 6	14-# 5	12-# 5	4.25	4.29	4.33
31	200	50	19-# 8	14-# 6	11-# 6	13-# 5	4.63	4.68	4.74
31	250	62	20-# 8	20-# 5	11-# 6	14-# 5	4.84	4.91	4.98
31	300	74	21-# 8	11-# 7	16-# 5	14-# 5	5.13	5.22	5.31
31	350	85	22-# 8	12-# 7	12-# 6	14-# 5	5.46	5.51	5.61

(1) Columns same above and below plate. (2) Center-to-center of columns; $\ell_1 = \ell_2$. (3) Superimposed factored load (factored dead load has been deducted).

3.2Two – way slab with drop panels

This system was considered because the drop panels allow a smaller slab depth compared to that of a flat plate. This was hand calculated and designed per ACI 02.

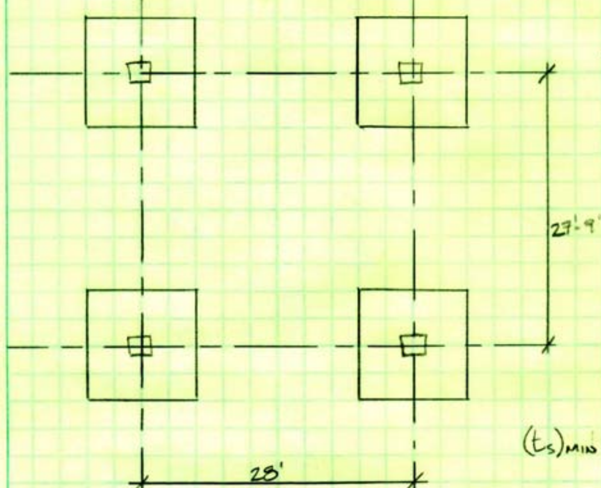
<u>Advantages</u>

- The drop panels provide extra strength to resist negative moment at the supports as opposed to just a flat plate
- Punching shear at the columns are a common controlling design, making drop panels a very efficient design with respect to the amount concrete used.
- Reduces floor vibrations induced by walking.
- Small floor sandwich depth compared to that of a steel system
- Concrete provides required fire protection rating therefore eliminating the need for any additional fire proofing.

<u>Disadvantages</u>

- A two way concrete floor system is not a preferred design for an office building because office buildings usually use a central heating system which means holes will be cut in the slab for ductwork. These holes decrease the capacity of the concrete significantly.
- This system will be heavier than a typical steel building making the foundations larger. This heavier system will produce a greater seismic loading, increasing the need for a larger lateral force resisting system.
- Construction costs will be larger due to the formwork and shoring than that of a steel building where metal deck is used.

TWO-WAY 9" FLAT PLAT WITH 5'x5'x3½" DROP PANELS



TYPICAL INTERIOR BAY

ASSUMPTIONS:
- 24" COLUMNS
- $f'_c = 4$ ksi
- $f_y = 60$ ksi
- 5'x5' DROP PANELS
- 3½" DROPS

COLUMN AND MIDDLE STRIPS SAME AS 10" FLAT PLATE SYSTEM

$(t_s)_{min} = l_n/36 = 26/36 = 8.6 \Rightarrow 9"$

ASSUME WEIGHT FROM DROP PANELS IS UNIFORMLY DISTRIBUTED OVER ENTIRE BAY.

SDL = 60 PSF
LL = 80 PSF
SELF = $(9/12)\,150\text{ PCF} + (5')(5')(3.5/12)(150)/(28)(27.75) = 102$ PSF

$W_u = 1.2(60 + 102) + 1.6(80)$

$W_u = 337$ PSF

$M_o = \dfrac{w_u l_2 l_n^2}{8} = \dfrac{(337)(27.75)(26)^2}{8}$

$M_o = 789^{k\text{-}ft}$

M_o	NEG SUPPORT $= 0.65\,M_o$	C.S $= (0.65)(0.75)\,M_o$
		M.S. $= (0.65)(0.25)\,M_o$
	POS, MIDSPAN $= 0.35\,M_o$	C.S. $= (0.35)(0.6)\,M_o$
		M.S $= (0.35)(0.4)\,M_o$

* THE FOLLOWING TABLE LISTS THE COMPUTATED TOTAL MOMENT AT THE GIVEN LOCATIONS

$789^{k\text{-}ft}$	$513^{k\text{-}ft}$	$385^{k\text{-}ft}$
		$128^{k\text{-}ft}$
	$276^{k\text{-}ft}$	$166^{k\text{-}ft}$
		$110^{k\text{-}ft}$

TWO-WAY 9" FLAT PLATE W 5'x5'x3½ DROP PANELS CON'T

DESIGN OF REBAR WILL BE DONE ON A PER FOOT BASIS

NEGATIVE SUPPORT COLUMN STRIP MOMENT

$M_u = 385/13.875$

$M_u = 27.7^{k\text{-}ft}/ft$

NEGATIVE SUPPORT MIDDLE STRIP MOMENT

$M_u = 128/13.875$

$M_u = 9.2^{k\text{-}ft}/ft$

POSITIVE MIDSPAN COLUMN STRIP MOMENT

$M_u = 166/13.875$

$M_u = 12.0^{k\text{-}ft}/ft$

POSITIVE MIDSPAN MIDDLE STRIP MOMENT

$M_u = 110/13.875$

$M_u = 7.9^{k\text{-}ft}/ft$

$M_n = \frac{M_u}{\phi} = A_s f_y (d - a/2)$

$\frac{(27.7)(12)}{0.9} = A_s (60)(10.25 - 1.5/2)$

$d = 12\frac{1}{2} - 0.75 - 1" - 0.5"$

$d = 10.25$ IN

ASSUME $a = 1.5$ IN

$A_s = 0.65$ IN²

TRY 2-#6's @ 6"

$A_s f_y = 0.85 f'_c b a$

$(0.88)(60) = 0.85(4)(12) a$

$a = 1.29$

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

$= (0.88)(60)(10.25 - 1.29/2)$

$M_n = 42.2^{k\text{-}ft}$ $\phi M_n = 38.0 \geq 27.7^{k\text{-}ft}$

TWO-WAY 9" FLAT PLATE WITH 5'×5'×3½" DROP PANELS CON'T

CHECK DUCTILITY: $c \leq 0.375d$

$1.52 \leq 3.8$ OK

CHECK MAX SPACING: $S_{MAX} = t_s(2) = 18" \leq 6"$

USE 2-#6's @ 6" SPACING FOR COLUMN STRIP AT SUPPORTS (TOP BARS)

$M_R = \frac{M_U}{\phi} = A_s f_y (d - a/2)$

$d = 9" - 0.75 - 1" - 0.5"$

$d = 6.75"$

$\frac{(12.0)(12)}{0.9} = A_s (60)(6.75" - 1/2)$

ASSUME $a = 1"$

$A_s = 0.43\ in^2$

TRY 1-#6

$A_s f_y = 0.85 f'_c b a$

$(0.44)(60) = 0.85(4)(12)\ a$

$a = 0.65\ in$

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

$= (0.44)(60)(6.75 - 0.65/2)$

$M_n = 14.1^{k\text{-}ft}$ $\quad \phi M_n = 12.7 \geq 12.0$ OK

CHECK DUCTILITY: $c \leq 0.375d$

$0.76 \leq 2.5$ OK

MAX SPACING OK

USE 1-#6 @ 12" SPACING FOR: MIDDLE STRIP AT SUPPORTS (TOP BARS)
MIDDLE STRIP AT MIDSPAN (BOTTOM BARS)
COLUMN STRIP AT MIDSPAN (BOTTOM BARS)

DESIGN IS BASED ON CRITICAL DIRECTION
SINCE OTHER DIRECTION IS JUST 3" SHORTER,
USE SAME DESIGN.

3.3 Two – way waffle slab

A two – way waffle slab provides the all of the same advantages as that of a two way flat plate or flat slab with drop panels. The waffle shape of the slab reduces the amount of concrete required. The voids are 19" x 19" with a 5" rib totaling 24" width per member. Total depth of the slab is 11"(8"rib with 3" topping). I selected this size because there will be (28' - 0" / 24") = 14 joist per bay. This will eliminate the need for any kind of extra labor and equipment cost needed to form the slab to fit the bays. This system was designed using CRSI's load tables.

<u>Advantages</u>

- At column supports, the waffle slab is a solid concrete slab. This is done to resist punching shear and negative moment.
- Less concrete used than that in a flat plate or flat slab with drop panels.
- Reduces floor vibrations induced by walking.
- Concrete provides required fire protection rating therefore eliminating the need for any additional fire proofing.

<u>Disadvantages</u>

- The waffle slab is not a favorable design because most architects don't like way it looks.
- A two way concrete floor system is not a preferred design for an office building because office buildings usually use a central heating system which means holes will be cut in the slab for ductwork. These holes decrease the capacity of the concrete significantly.
- This system will be heavier than a typical steel building making the foundations larger. This heavier system will produce a greater seismic loading, increasing the need for a larger lateral force resisting system.
- Construction costs will be larger due to the formwork than that of a steel building where metal deck is used.

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Conclusion: A waffle slab is primarily used in apartment buildings because costs and durability are major design considerations. It is not a preferred design for an office building.

WAFFLE FLAT SLAB SYSTEM 19″ X 19″ Voids: 5″ Ribs @ 24″ — f'_c = 4,000 psi, Grade 60 Bars

Span c-c Columns $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	SQUARE EDGE PANELS: (1) Steel (psf)	Square Edge Column: $c_1 = c_2$ (in.)	Square Edge Column: γ_f	Square Edge Column: (2) Stirrups	Column Strip: Top Edge No.-size +	Column Strip Bottom: No. Ribs	Column Strip Bottom: Bars per Rib	Column Strip: Top Interior No.-size	Middle Strip Bottom: No. Ribs	Middle Strip Bottom: Long Bars	Middle Strip Bottom: Short Bars	Middle Strip: Top Interior No.-size	Moments: −M Edge (ft-k)	Moments: +M Bot. (ft-k)	Moments: −M Int. (ft-k)	SQUARE INTERIOR PANELS: (1) Steel (psf)	Square Interior Column: $c_1 = c_2$ (in.)	Square Interior Column: (2) Stirrups	Column Strip Bottom: No. Ribs	Column Strip Bottom: Bars per Rib	Column Strip: Top Interior No.-size	Middle Strip Bottom: No. Ribs	Middle Strip Bottom: Long Bars	Middle Strip Bottom: Short Bars	Middle Strip: Top Interior No.-size
Edge panels: Total Depth = 11 in. Rib Depth = 8 in. Total Slab Depth = 3 in.																	Interior panels: Total Depth = 11 in. Rib Depth = 8 in. Total Slab Depth = 3 in.									
21′-0″ D=8 1/2 RIB ON COLUMN LINE 0.571 CF SF	50	2.26	12	0.855		18-#5 + 0	5	2-#4	18-#5	7	#4	#4	7-#5	67	135	181	2.23	12		5	2-#4	18-#5	7	#4	#4	7-#5
	100	2.39	12 *	0.889		18-#5 + 0	5	2-#5	18-#5	7	#4	#4	7-#5	88	182	237	2.23	12		5	2-#4	18-#5	7	#4	#4	7-#5
	150	2.54	12 *	0.926		18-#5 + 1	5	1-#5 and 1-#6	18-#5	7	#4	#4	7-#5	109	251	292	2.35	12 *	3 S 4 1	5	1-#4 and 1-#5	18-#5	7	#4	#4	7-#5
	200	2.96	13 *	0.629		18-#5 + 0	5	1-#6 and 1-#7	21-#5	7	#4	#5	7-#5	129	315	347	2.53	12 *	3 S 4 1	5	2-#5	19-#5	7	#4	#4	7-#5
	300	3.73	18 *	0.623	3 S 4 1	18-#5 + 0	5	1-#7 and 1-#8	19-#6	7	#5	#5	9-#5	166	417	446	3.21	14 *	3 S 4 2	5	2-#6	18-#6	7	#4	#4	8-#5
[illegible] D=10 1/2 RIB NOT ON COLUMN LINE 0.585 CF SF	50	2.32	13	0.863		19-#5 + 0	6	1-#4 and 1-#5	19-#5	7	#4	#4	8-#5	86	173	233	2.23	13		6	2-#4	19-#5	7	#4	#4	8-#5
	100	2.40	13 *	0.911		19-#5 + 2	6	2-#5	19-#5	7	#4	#4	8-#5	113	227	303	2.23	13		6	2-#4	19-#5	7	#4	#4	8-#5
	150	2.85	14 *	0.936		19-#5 + 4	6	2-#6	22-#5	7	#4	#5	8-#5	138	315	373	2.52	13 *	3 S 4 1	6	2-#5	20-#5	7	#4	#4	8-#5
	200	3.31	17 *	0.624		19-#5 + 2	6	1-#6 and 1-#7	26-#5	7	#5	#5	8-#5	163	393	439	2.94	13 *	3 S 4 1	6	1-#5 and 1-#6	18-#6	7	#4	#4	8-#5
[illegible] D=10 1/2 RIB NOT ON COLUMN LINE 0.577 CF SF	50	2.39	14 *	0.881		21-#5 + 1	6	2-#5	21-#5	8	#4	#4	8-#5	107	215	289	2.23	14		6	2-#4	21-#5	8	#4	#4	8-#5
	100	2.55	14 *	0.931		21-#5 + 4	6	1-#5 and 1-#6	22-#5	8	#4	#4	8-#5	140	280	377	2.35	14 *	3 S 4 1	6	1-#4 and 1-#5	21-#5	8	#4	#4	8-#5
	150	3.14	17 *	0.624		21-#5 + 0	6	1-#6 and 1-#7	20-#6	8	#4	#5	9-#5	171	378	461	2.81	14 *	3 S 4 1	6	1-#5 and 1-#6	26-#5	8	#4	#4	8-#5
	200	3.45	23 *	0.619		21-#5 + 2	6	2-#7	23-#6	8	#4	#5	10-#5	198	414	533	3.13	17 *	3 S 4 2	6	1-#5 and 1-#6	22-#6	8	#4	#4	10-#5
30′-0″ D=10 1/2 RIB NOT ON COLUMN LINE 0.571 CF SF	50	2.50	15 *	0.893		22-#5 + 2	6	1-#5 and 1-#6	22-#5	9	#4	#4	9-#5	132	263	354	2.33	15		6	1-#4 and 1-#5	22-#5	9	#4	#4	9-#5
	100	2.77	16 *	0.934		22-#5 + 6	6	2-#6	28-#5	9	#4	#4	9-#5	171	343	461	2.60	15 *	3 S 4 1	6	2-#5	26-#5	9	#4	#4	9-#5
	150	3.37	23 *	0.619		22-#5 + 1	6	2-#7	24-#6	9	#4	#5	11-#5	207	417	556	3.06	16 *	3 S 4 2	6	1-#5 and 1-#6	23-#6	9	#4	#4	10-#5

See the notes on Page 11-19

3.4 One – way pan joist

The typical bay at Fordham Place is square, making a two way system a much more preferred design. However I chose to look at a one – way pan joist system also to compare differences. The joists are 30" forms with a 7" rib totaling 37" width per member. Total depth of the members are 19"(16"rib with 3" topping). I selected this size because there will be (27'-9" / 37") = 9 joist per bay. This will eliminate the need for any kind of extra labor and equipment cost needed to cut the joist to fit the bays. Material strengths are f'c = 4ksi and fy = 60 ksi for concrete and steel respectively. This system was designed using CRSI.

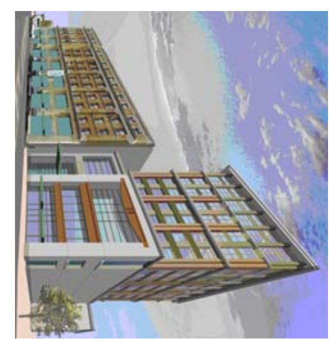
<u>Advantages</u>

- Reduces floor vibrations induced by walking.
- Concrete provides required fire protection rating therefore eliminating the need for any additional fire proofing.
- Joists are shop fabricated which eliminates additional cost for formwork.

<u>Disadvantages</u>

- Floor sandwich height (19") is a little deeper than that of the two way systems, but still significantly less than that of a steel building.
- This system will be heavier than a typical steel building making the foundations larger. This heavier system will produce a greater seismic loading, increasing the need for a larger lateral force resisting system.

Conclusion: The one way joist system is not a reasonable design because of the duct work that needs to travel through the floors in a ordinary office building.



STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS

30" Forms + 6" Rib @ 36" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)

f'_c = 4,000 psi, f_y = 60,000 psi

16" Deep Rib + 3.0" Top Slab = 19.0" Total Depth

TOP BARS	Size @	# 4 12	# 4 10	# 4 8	# 5 10.5	# 5 8.5	End Span Defl. Coeff. (3)	# 4 11	# 4 9	# 5 11	# 5 9	# 6 11	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7		# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	
Steel (psf)		.69	.85	1.04	1.24	1.46		.75	.93	1.18	1.42	1.70	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
25'-0"		127	177	236	295	330*	4.703	161	234	308	375*	379*	2.894
		0	0	0	0	365		0	0	0	393	478*	
26'-0"		109	156	210	264	308*	5.502	141	208	276	353*	356*	3.386
		0	0	0	0	330		0	0	0	355	434	
27'-0"		93	136	186	237	289*	6.398	123	185	248	321	335*	3.938
		0	0	0	0	298		0	0	0	0	395	
28'-0"		79	119	166	213	269	7.400	106	164	223	291	316*	4.554
		0	0	0	0	0		0	0	0	0	359	
29'-0"		66	104	147	191	243	8.516	92	146	201	264	298*	5.240
		0	0	0	0	0		0	0	0	0	327	
30'-0"		55	90	130	171	220	9.752	79	129	180	239	282*	6.001
		0	0	0	0	0		0	0	0	0	299	
31'-0"		44	77	115	153	199	11.119	67	114	162	217	267*	6.842
		0	0	0	0	0		0	0	0	0	273	
32'-0"			65	101	137	180	12.625	56	100	145	197	249	7.769
			0	0	0	0		0	0	0	0	0	
33'-0"			55	88	122	163	14.278	46	88	130	179	228	8.787
			0	0	0	0		0	0	0	0	0	
34'-0"			45	77	109	147	16.089		76	116	162	208	9.901
			0	0	0	0			0	0	0	0	
35'-0"				66	97	133	18.067		66	103	147	190	11.118
				0	0	0			0	0	0	0	
36'-0"				57	85	119	20.222		56	92	133	174	12.445
				0	0	0			0	0	0	0	
37'-0"				48	75	107	22.565		47	81	120	159	13.886
				0	0	0			0	0	0	0	
38'-0"				40	65	96	25.105			71	108	145	15.449
				0	0	0				0	0	0	

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .52 CF/SF) (4)

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.60	.72	.90	1.06	1.31		.65	.80	1.01	1.24	1.44	
STEEL % (UNIFORM)	.45	.54	.68	.80	.99		.49	.60	.77	.94	1.09	
(TAPERED)	.27	.33	.41	.48	.60		.30	.36	.46	.56	.66	
EFF. DEPTH, IN.	17.8	17.8	17.8	17.7	17.7		17.8	17.8	17.7	17.7	17.6	
– ICR/IGR	.144	.167	.199	.225	.263		.155	.181	.217	.252	.279	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.51	.62	.75	.88	1.04		.40	.51	.62	.75	.88	
STEEL %	.08	.10	.12	.14	.16		.06	.08	.10	.12	.14	
EFF. DEPTH, IN.	17.7	17.7	17.6	17.6	17.6		17.8	17.7	17.7	17.6	17.6	
+ICR/IGR	.153	.184	.217	.251	.290		.123	.153	.184	.217	.251	

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS

30" Forms + 7" Rib @ 37" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)

f'_c = 4,000 psi, f_y = 60,000 psi

16" Deep Rib + 3.0" Top Slab = 19.0" Total Depth

TOP BARS	Size @	# 4 10	# 4 8.5	# 5 11	# 5 9	# 5 8	End Span Defl. Coeff. (3)	# 4 9	# 5 11	# 5 9.5	# 5 8	# 6 9.5	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	# 7 # 7		# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6	# 6 # 7	
Steel (psf)		.83	1.00	1.20	1.41	1.63		.92	1.16	1.36	1.63	1.93	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
25'-0"		163	219	277	346	366*	4.834	218	290	372	419*	425*	2.975
		0	0	0	0	416		0	0	0	456	536*	
26'-0"		142	194	247	311	342*	5.655	193	259	335	394*	399*	3.480
		0	0	0	0	376		0	0	0	413	499*	
27'-0"		123	171	221	280	321*	6.576	170	232	302	371*	376*	4.047
		0	0	0	0	340		0	0	0	374	460	
28'-0"		106	151	197	252	301*	7.606	150	207	273	340	354*	4.691
		0	0	0	0	308		0	0	0	0	420	
29'-0"		91	133	176	227	279	8.752	132	185	247	309	335*	5.396
		0	0	0	0	0		0	0	0	0	393	
30'-0"		77	117	157	205	253	10.023	116	165	223	281	317*	6.168
		0	0	0	0	0		0	0	0	0	351	
31'-0"		65	102	139	184	230	11.428	101	148	201	256	300*	7.033
		0	0	0	0	0		0	0	0	0	321	
32'-0"		54	88	124	166	208	12.975	88	131	182	233	285*	7.985
		0	0	0	0	0		0	0	0	0	294	
33'-0"		44	76	109	149	189	14.675	75	116	164	212	269	9.031
		0	0	0	0	0		0	0	0	0	0	
34'-0"			65	96	133	171	16.536	64	103	148	193	247	10.176
			0	0	0	0		0	0	0	0	0	
35'-0"			55	84	119	155	18.569	54	91	133	175	227	11.127
			0	0	0	0		0	0	0	0	0	
36'-0"			45	73	106	140	20.784	45	79	119	159	208	12.790
			0	0	0	0		0	0	0	0	0	
37'-0"				63	94	126	23.191		69	106	145	190	14.272
				0	0	0			0	0	0	0	
38'-0"				54	83	114	25.802		59	95	131	174	15.878
				0	0	0			0	0	0	0	

(1) For gross section properties, see Table 8-1.
(2) First load is for standard square joist ends; second load is for special tapered joist ends.
(3) Computation of deflection is not required above horizonal line (thickness ≥ $\ell_n/18.5$ for end spans, $\ell_n/21$ for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = $\ell_n/360$.

PROPERTIES FOR DESIGN (CONCRETE .55 CF/SF) (4)

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.74	.87	1.04	1.27	1.43		.82	1.04	1.21	1.43	1.71	
STEEL % (UNIFORM)	.49	.58	.70	.85	.96		.55	.70	.81	.96	1.15	
(TAPERED)	.31	.36	.44	.53	.60		.34	.44	.51	.60	.72	
EFF. DEPTH, IN.	17.8	17.8	17.7	17.7	17.7		17.8	17.7	17.7	17.7	17.6	
– ICR/IGR	.159	.181	.207	.241	.264		.173	.207	.232	.264	.298	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.62	.75	.88	1.04	1.20		.51	.62	.75	.88	1.01	
STEEL %	.09	.12	.13	.16	.18		.08	.09	.12	.13	.16	
EFF. DEPTH, IN.	17.7	17.6	17.6	17.6	17.6		17.7	17.7	17.6	17.6	17.6	
+ICR/IGR	.166	.197	.227	.262	.298		.139	.166	.197	.227	[illegible]	

3.5 Open web steel joist

Open web steel joist were a design option because they are significantly less than other options. Since floor sandwich depth or cost control most designs, I will design and compare two different sizes of steel joists. One will be designed based on the smallest depth of a member, and the other will be designed based on economy. This system will be designed using The New Columbia Joist Company design guide.

<u>Advantages</u>

- Very inexpensive system
- Easy to construct
- Not as heavy as other systems therefore reducing foundation sizes and seismic loads
- Construction time will be short

<u>Disadvantages</u>

- Open web steel joist are prone to floor vibrations
- Floor sandwich depth is large
- Additional fire proofing required

<u>Design Parameters:</u>

Span: 28' – 0" (taken from centerline to centerline of supporting members)
Spacing: 24"
LL = 1.6*80psf*2ft = 256psf
Wu = 1.6*80psf*2ft + 1.2*60psf*2ft = 400psf
$\Delta LL = l/360$

Design Results:
18K9 spaced at 2ft with two rows of bridging (shallowest member)
22K6 spaced at 2ft with two rows of bridging (most economic member)
Both have 20 gage 2" Lok-floor metal deck with 1 shear stud per foot.
Total slab thickness = 5"

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Fordham Place
Bronx, NY
Structural Option
Advisor - Dr. Hanagan

Conclusion: Open web steel joist is a risky design due to floor vibrations. Because Fordham Place will house numerous occupants, floor vibrations will be unacceptable. Therefore I would stay away from this design.

STANDARD LOAD TABLE/OPEN WEB STEEL JOISTS, K-SERIES
Based on a Maximum Allowable Tensile Stress of 30 ksi

Joist Designation	18K3	18K4	18K5	18K6	18K7	18K9	18K10	20K3	20K4	20K5	20K6	20K7	20K9	20K10	22K4	22K5	22K6	22K7	22K9	22K10	22K11
Depth (In.)	18	18	18	18	18	18	18	20	20	20	20	20	20	20	22	22	22	22	22	22	22
Approx. Wt. (lbs./ft.)	6.6	7.2	7.7	8.5	9	10.2	11.7	6.7	7.6	8.2	8.9	9.3	10.8	12.2	8	8.8	9.2	9.7	11.3	12.6	13.8
Span (ft.) ↓ 18	550 550	550 550	550 550	550 550	550 550	550 550	550 550														
19	514 494	550 523	550 523'	550 523	550 523	550 523	550 523														
20	463 423	550 490	550 490	550 490	550 490	550 490	550 490	517 517	550 550	550 550	550 550	550 550	550 550	550 550							
21	420 364	506 426	550 460	550 460	550 460	550 460	550 460	468 453	550 520	550 520	550 520	550 520	550 520	550 520							
22	382 316	460 370	518 414	550 438	550 438	550 438	550 438	426 393	514 461	550 490	550 490	550 490	550 490	550 490	550 548	550 548	550 548	550 548	550 548	550 548	550 548
23	349 276	420 323	473 362	516 393	550 418	550 418	550 418	389 344	469 402	529 451	550 468	550 468	550 468	550 468	518 491	550 518	550 518	550 518	550 518	550 518	550 518
24	320 242	385 284	434 318	473 345	526 382	550 396	550 396	357 302	430 353	485 396	528 430	550 448	550 448	550 448	475 431	536 483	550 495	550 495	550 495	550 495	550 495
25	294 214	355 250	400 281	435 305	485 337	550 377	550 377	329 266	396 312	446 350	486 380	541 421	550 426	550 426	438 381	493 427	537 464	550 474	550 474	550 474	550 474
26	272 190	328 222	369 249	402 271	448 299	538 354	550 361	304 236	366 277	412 310	449 337	500 373	550 405	550 405	404 338	455 379	496 411	550 454	550 454	550 454	550 454
27	252 169	303 198	342 222	372 241	415 267	498 315	550 347	281 211	339 247	382 277	416 301	463 333	550 389	550 389	374 301	422 337	459 367	512 406	550 432	550 432	550 432
28	234 151	282 177	318 199	346 216	385 239	463 282	548 331	261 189	315 221	355 248	386 269	430 298	517 353	550 375	348 270	392 302	427 328	475 364	550 413	550 413	550 413
29	218 136	263 159	296 179	322 194	359 215	431 254	511 298	243 170	293 199	330 223	360 242	401 268	482 317	550 359	324 242	365 272	398 295	443 327	532 387	550 399	550 399
30	203 123	245 144	276 161	301 175	335 194	402 229	477 269	227 153	274 179	308 201	336 218	374 242	450 286	533 336	302 219	341 245	371 266	413 295	497 349	550 385	550 385
31	190 111	229 130	258 146	281 158	313 175	376 207	446 243	212 138	256 162	289 182	314 198	350 219	421 259	499 304	283 198	319 222	347 241	387 267	465 316	550 369	550 369
32	178 101	215 118	242 132	264 144	294 159	353 188	418 221	199 126	240 147	271 165	295 179	328 199	395 235	468 276	265 180	299 201	326 219	363 242	436 287	517 337	549 355
33	168 92	202 108	228 121	248 131	276 145	332 171	393 201	187 114	226 134	254 150	277 163	309 181	371 214	440 251	249 164	281 183	306 199	341 221	410 261	486 307	532 334
34	158 84	190 98	214 110	233 120	260 132	312 156	370 184	176 105	212 122	239 137	261 149	290 165	349 195	414 229	235 149	265 167	288 182	321 202	386 239	458 280	516 314
35	149 77	179 90	202 101	220 110	245 121	294 143	349 168	166 96	200 112	226 126	246 137	274 151	329 179	390 210	221 137	249 153	272 167	303 185	364 219	432 257	494 292
36	141 70	169 82	191 92	208 101	232 111	278 132	330 154	157 88	189 103	213 115	232 125	259 139	311 164	369 193	209 126	236 141	257 153	286 169	344 201	408 236	467 269
37								148 81	179 95	202 106	220 115	245 128	294 151	349 178	198 116	223 130	243 141	271 156	325 185	386 217	442 247
38								141 74	170 87	191 98	208 106	232 118	279 139	331 164	187 107	211 119	230 130	256 144	308 170	366 200	419 228
39								133 69	161 81	181 90	198 98	220 109	265 129	314 151	178 98	200 110	218 120	243 133	292 157	347 185	397 211
40								127 64	153 75	172 84	188 91	209 101	251 119	298 140	169 91	190 102	207 111	231 123	278 146	330 171	377 195
41															161 85	181 95	197 103	220 114	264 135	314 159	359 181
42															153 79	173 88	188 96	209 106	252 126	299 148	342 168
43															146 73	165 82	179 89	200 99	240 117	285 138	326 157
44															139 68	157 76	171 83	191 92	229 109	272 128	311 146

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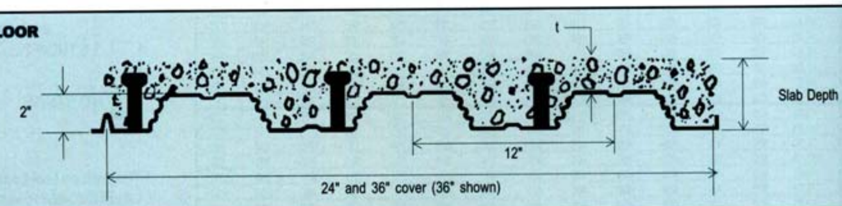
2 x 12" DECK F_y = 33ksi f'_c = 3 ksi 145 pcf concrete

USD United Steel Deck, Inc.

LRFD

2" LOK-FLOOR



The **Deck Section Properties** are per foot of width. The **I** value is for positive bending (in.^{4}); **t** is the gage thickness in inches; **w** is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.^{3}); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

DECK PROPERTIES

Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.36
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.43
19	0.0418	2.1	0.630	0.490	0.445	0.458	1330	2810	0.51
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.57
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3990	0.72

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.^{2} per foot of width. **Vol.** is the volume of concrete in ft.^{3} per ft.^{2} needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.^{2}. S_c is the section modulus of the "cracked" concrete composite slab; in.^{3} per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.^{4} per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

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}
22 gage	4.50	40.27	32.6	0.292	42	1.05	5.9	29.40	5030	5.82	7.83	7.92	0.023
	5.00	46.44	37.5	0.333	48	1.23	8.0	34.53	5480	5.54	7.47	7.56	0.027
	5.25	49.53	40.0	0.354	51	1.32	9.2	37.16	5720	5.41	7.31	7.39	0.029
	5.50	52.61	42.6	0.375	54	1.42	10.5	39.81	5960	5.30	7.16	7.24	0.032
	6.00	58.78	48.0	0.417	60	1.61	13.5	45.21	6460	5.09	6.89	6.97	0.036
	6.25	61.87	50.8	0.438	63	1.71	15.3	47.95	6720	5.03	6.76	6.84	0.038
	6.50	64.95	53.6	0.458	66	1.81	17.1	50.70	6980	4.97	6.65	6.72	0.041
	7.00	71.12	59.5	0.500	73	2.01	21.2	56.26	7530	4.85	6.43	6.51	0.045
	7.25	74.21	61.9	0.521	76	2.11	23.5	59.07	7750	4.79	6.32	6.41	0.047
	7.50	77.29	64.3	0.542	79	2.21	26.0	61.88	7970	4.74	6.22	6.31	0.050
20 gage	4.50	48.60	32.6	0.292	42	1.26	6.3	35.43	5450	6.81	8.97	9.27	0.023
	5.00	56.18	37.5	0.333	48	1.48	8.6	41.65	5900	6.47	8.55	8.83	0.027
	5.25	59.96	40.0	0.354	51	1.60	9.8	44.84	6140	6.32	8.36	8.63	0.029
	5.50	63.75	42.6	0.375	54	1.71	11.3	48.07	6380	6.18	8.18	8.45	0.032
	6.00	71.32	48.0	0.417	60	1.95	14.5	54.63	6880	5.94	7.85	8.11	0.036
	6.25	75.11	50.8	0.438	63	2.07	16.3	57.96	7140	5.86	7.70	7.95	0.038
	6.50	78.90	53.6	0.458	66	2.19	18.2	61.31	7400	5.79	7.56	7.80	0.041
	7.00	86.47	59.5	0.500	73	2.43	22.6	68.09	7950	5.65	7.29	7.53	0.045
	7.25	90.26	61.9	0.521	76	2.55	25.0	71.50	8170	5.58	7.17	7.41	0.047
	7.50	94.05	64.3	0.542	79	2.67	27.6	74.93	8390	5.52	7.05	7.28	0.050
19 gage	4.50	55.85	32.6	0.292	42	1.45	6.7	40.69	5850	7.65	9.76	10.08	0.023
	5.00	64.68	37.5	0.333	48	1.71	9.0	47.87	6300	7.26	9.30	9.61	0.027
	5.25	69.10	40.0	0.354	51	1.84	10.4	51.56	6540	7.09	9.09	9.39	0.029
	5.50	73.52	42.6	0.375	54	1.97	11.9	55.30	6780	6.93	8.90	9.19	0.032
	6.00	82.35	48.0	0.417	60	2.24	15.2	62.90	7280	6.65	8.54	8.83	0.036
	6.25	86.77	50.8	0.438	63	2.38	17.1	66.76	7540	6.56	8.38	8.66	0.038
	6.50	91.19	53.6	0.458	66	2.52	19.2	70.65	7800	6.48	8.23	8.50	0.041
	7.00	100.03	59.5	0.500	73	2.80	23.8	78.50	8350	6.32	7.94	8.20	0.045
	7.25	104.44	61.9	0.521	76	2.94	26.3	82.46	8570	6.24	7.81	8.07	0.047
	7.50	108.86	64.3	0.542	79	3.08	29.0	86.45	8790	6.17	7.68	7.94	0.050
18 gage	4.50	62.08	32.6	0.292	42	1.62	7.0	45.34	6080	8.42	10.48	10.83	0.023
	5.00	72.04	37.5	0.333	48	1.90	9.5	53.36	6670	7.98	9.99	10.32	0.027
	5.25	77.02	40.0	0.354	51	2.05	10.9	57.48	6910	7.79	9.77	10.10	0.029
	5.50	82.00	42.6	0.375	54	2.20	12.4	61.66	7150	7.61	9.56	9.88	0.032
	6.00	91.95	48.0	0.417	60	2.50	15.9	70.18	7650	7.30	9.18	9.49	0.036
	6.25	96.93	50.8	0.438	63	2.66	17.9	74.50	7910	7.20	9.01	9.31	0.038
	6.50	101.91	53.6	0.458	66	2.81	20.0	78.85	8170	7.11	8.85	9.14	0.041
	7.00	111.87	59.5	0.500	73	3.13	24.8	87.66	8720	6.93	8.54	8.82	0.045
	7.25	116.85	61.9	0.521	76	3.28	27.4	92.10	8940	6.85	8.40	8.68	0.047
	7.50	121.83	64.3	0.542	79	3.44	30.2	96.57	9160	6.77	8.26	8.54	0.050
16 gage	4.50	62.08	32.6	0.292	42	1.99	7.7	45.34	6080	9.58	11.63	12.02	0.023
	5.00	72.04	37.5	0.333	48	2.35	10.4	53.36	6980	9.08	11.10	11.47	0.027
	5.25	77.02	40.0	0.354	51	2.53	11.9	57.48	7450	8.85	10.85	11.22	0.029
	5.50	82.00	42.6	0.375	54	2.72	13.6	61.66	7940	8.65	10.63	10.98	0.032
	6.00	91.95	48.0	0.417	60	3.10	17.4	70.18	8460	8.29	10.21	10.55	0.036
	6.25	96.93	50.8	0.438	63	3.29	19.5	74.50	8720	8.17	10.02	10.35	0.038
	6.50	101.91	53.6	0.458	66	3.48	21.8	78.85	8980	8.07	9.84	10.17	0.041
	7.00	111.87	59.5	0.500	73	3.88	27.0	87.66	9530	7.86	9.50	9.82	0.045
	7.25	116.85	61.9	0.521	76	4.08	29.8	92.10	9750	7.77	9.35	9.66	0.047
	7.50	121.83	64.3	0.542	79	4.28	32.8	96.57	9970	7.67	9.20	9.50	0.050

2" LOK-FLOOR

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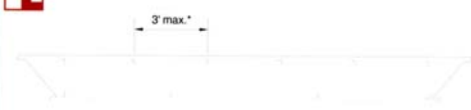
UPLIFT AND BRIDGING

**Bridging at first bottom chord panel point at both ends of joist.

NET UPLIFT BRIDGING



* Spacing of SSR Bridging dependant upon top chord angle size.

STANDING SEAM ROOF SYSTEMS (SSR)

Standing Seam Roof Systems (SSR)

Industry standards are to assume that SSR systems **DO NOT** adequately brace the top chord of joists. In most cases standard SJI bridging is not sufficient for bracing the compression chord. We recommend that the design professional note on the drawings that SSR is being utilized and that the joist manufacturer needs to provide adequate bridging to properly brace the top chord under full design load.

Net Uplift Bridging

When uplift forces from wind are a design consideration the design professional must place the NET UPLIFT values, either in terms of PSF or PLF per joist, on the drawings. These loads must be considered in the design of the joists and bridging. A minimum of an additional row of horizontal bridging near the first bottom chord panel point at each end is required when uplift is a design consideration. Depending on the Net Uplift values additional bottom chord bridging may also be required.

REQUIRED NUMBER OF ROWS OF BRIDGING**

*SECTION NUMBER	ROWS 1	2	3	4	5
1	Up thru 16'	Over 16' thru 24'	Over 24' thru 28'		
2	Up thru 17'	Over 17' thru 25'	Over 25' thru 32'		
3	Up thru 18'	Over 18' thru 28'	Over 28' thru 38'	Over 38' thru 40'	
4	Up thru 19'	Over 19' thru 28'	Over 28' thru 38'	Over 38' thru 48'	
5	Up thru 19'	Over 19' thru 29'	Over 29' thru 39'	Over 39' thru 50'	Over 50' thru 52'
6	Up thru 19'	Over 19' thru 29'	Over 29' thru 39'	Over 39' thru 51'	Over 51' thru 56'
7	Up thru 20'	Over 20' thru 33'	Over 33' thru 45'	Over 45' thru 58'	Over 58' thru 60'
8	Up thru 20'	Over 20' thru 33'	Over 33' thru 45'	Over 45' thru 58'	Over 58' thru 60'
9	Up thru 20'	Over 20' thru 33'	Over 33' thru 46'	Over 46' thru 59'	Over 59' thru 60'
10	Up thru 20'	Over 20' thru 37'	Over 37' thru 51'	Over 51' thru 60'	
11	Up thru 20'	Over 20' thru 38'	Over 38' thru 53'	Over 53' thru 60'	
12	Up thru 20'	Over 20' thru 39'	Over 39' thru 53'	Over 53' thru 60'	

MAXIMUM JOIST SPACING FOR DIAGONAL BRIDGING

Notes: K series joist use 3/8" ϕ ASTM-A307 bridging bolts. Consult shaded portions of SJI load tables to determine when bridging row must be diagonal bolted bridging. Also note OSHA secion 29CFR §1926.757 (c) and (d) have been revised to accept SJI bridging requirements.

JOIST DEPTH	STANDARD EQUAL LEG ANGLES, (additional costs for other bridging sizes) 1 X 7/64 (25 mm x 3 mm) r = .20	1 1/4 X 7/64 (32 mm x 3 mm) r = .25	1 1/2 X 7/64 (38 mm x 3 mm) r = .30	1 3/4 X 7/64 (45 mm x 3 mm) r = .35
8	6'-6 (1981 mm)	8'-3 (2514 mm)	9'-11 (3022 mm)	11'-7 (3530 mm)
10	6'-6 (1981 mm)	8'-3 (2514 mm)	9'-11 (3022 mm)	11'-7 (3530 mm)
12	6'-6" (1981 mm)	8'-3" (2514 mm)	9'-11" (3022 mm)	11'-7" (3530 mm)
14	6'-6" (1981 mm)	8'-3" (2514 mm)	9'-11" (3022 mm)	11'-7" (3530 mm)
16	6'-6" (1981 mm)	8'-2" (2489 mm)	9'-10" (2997 mm)	11'-6" (3505 mm)
18	6-6" (1981 mm)	8'-2" (2489 mm)	9'-10" (2997 mm)	11'-6" (3505 mm)
20	6'-5" (1955 mm)	8'-2" (2489 mm)	9'-10" (2997 mm)	11'-6" (3505 mm)
22	6'-4" (1930 mm)	8'-1" (2463 mm)	9'-10" (2997 mm)	11'-6" (3505 mm)
24	6'-4" (1930 mm)	8'-1" (2463 mm)	9'-9" (2971 mm)	11'-5" (3479 mm)
26	6'-3" (1905 mm)	8'-0" (2438 mm)	9'-9" (2971 mm)	11'-5" (3479 mm)
28	6'-2" (1879 mm)	8'-0" (2438 mm)	9'-8" (2946 mm)	11'-5" (3479 mm)
30	6'-2" (1879 mm)	7'-11" (2413 mm)	9'-8" (2946 mm)	11'-4" (3454 mm)

MAXIMUM JOIST SPACING FOR HORIZONTAL BRIDGING

*SECTION NUMBER	ROUND ROD 1/2" ROUND (13 mm) r = .13	STANDARD EQUAL LEG ANGLES, (additional costs for other bridging sizes) 1 X 7/64 (25 mm x 3 mm) r = .20	1 1/4 X 7/64 (32 mm x 3 mm) r = .25	1 1/2 X 7/64 (38 mm x 3 mm) r = .30	1 3/4 X 7/64 (45 mm x 3 mm) r = .35	2 X 1/8 (51 mm x 3 mm) r = .40	2 1/2 X 5/32 (64 mm x 4 mm) r = .50
1 - 9	3'-3"(991 mm)	5'-0"(1524 mm)	6'-3"(1905 mm)	7'-6"(2266 mm)	8'-7"(2616 mm)	10'-0"(3048 mm)	12'-6"(3810 mm)
10	3'-0"(914 mm)	4'-8"(1422 mm)	6'-3"(1905 mm)	7'-6"(2286 mm)	8'-7"(2616 mm)	10'-0"(3048 mm)	12'-6"(3810 mm)
11, 12	2'-7"(787 mm)	4'-0"(1219 mm)	5'-8"(1727 mm)	7'-6"(2286 mm)	8'-7"(2616 mm)	10'-0"(3040 mm)	12'-6"(3810 mm)

* Last Digit(s) of joist designation. ** Refer to Section 5.11 for additional wind uplift, bridging.
Note: See SJI Load Tables and OSHA Section 29 CFR 1926.757 (d) for additional Diagonal Bolted Bridging requirements.

1. W_{max} = 200 plf + 100 plf = 300 plf < 550 plf.
∴ No special note regarding uniform load required.

2. Place load pattern such that maximum possible moment is obtained.
The maximum moment occurs when $X_{unknown} \cong 15.0$ ft.
$M_{max} \cong 44{,}200$ lb. -ft. $\Rightarrow M_{max} \cong 530.4$ kip – in.

3. Place load pattern such that maximum possible reaction is obtained.
The maximum reaction occurs when $X_{unknown} \cong 4$ ft.
$V_{max} \cong 5{,}000$ lb.

4. KCS tables indicate that a 18KCS3 is adequate for M_{max} and V_{max} found above.
$M_{18KCS3} \cong 532$ kip – in. > $M_{max} \cong 530.4$ kip – in. ∴ Okay
$I_{gross} = 164$ in.^{4}
$V_{18KCS3} \cong 5{,}200$ lb. > $V_{max} \cong 5{,}000$ lb. ∴ Okay
Sect. No.$_{Bridging}$ = 9 (18K9)

5. Check any deflection criteria using I_{gross} being sure to increase deflections by 15% for web elongation.

6. Specify 18KCS3 on plans with bridging required for an 18K9. Indicate that field applied web stiffeners must be installed. Provide size of stiffeners and welding requirements.

K SERIES

3.6 Non – composite steel beams with concrete

This system was considered as an option for Fordham Place because I wanted to see the differences between a Non - composite and composite floor system. This system was designed using RAM Structural System per LRFD.

Advantages

- Less labor cost than composite because there are no shear studs
- Easy to erect
- Construction time will be short
- Reduces floor vibrations induced by walking
- Relatively light building weight

Disadvantages

- Material cost are greater because members need to be bigger
- Floor sandwich depth is large
- Additional fire proofing required

Conclusion: This is a good design however, the composite system is better because it reduces member sizes.

The following is the RAM output of a typical non-composite girder

Gravity Beam Design

RAM Steel v8.1
DataBase: Non - Composite Beams
Building Code: IBC

10/24/05 19:05:13
Steel Code: AISC LRFD

Floor Type: Non Composite Beams **Beam Number = 33**

SPAN INFORMATION (ft): I-End (46.67,55.50) J-End (46.67,83.25)

Beam Size (Optimum) = W18X40 Fy = 50.0 ksi
Total Beam Length (ft) = 27.75
Mp (kip-ft) = 326.67

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.840	0.747	---	NonR
	27.750	0.840	0.747		
2	0.000	0.040	0.000	---	NonR
	27.750	0.040	0.000		

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 31.23 kips 0.90Vn = 152.24 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	216.7	13.9	0.0	1.00	0.90	294.00
Controlling		1.2DL+1.6LL	216.7	13.9	0.0	1.00	0.90	294.00

REACTIONS (kips):

	Left	**Right**
DL reaction	12.21	12.21
Max +LL reaction	10.36	10.36
Max +total reaction (factored)	31.23	31.23

DEFLECTIONS:

Dead load (in)	at	13.87 ft	=	-0.662	L/D =	503
Live load (in)	at	13.87 ft	=	-0.561	L/D =	593
Net Total load (in)	at	13.87 ft	=	-1.223	L/D =	272

The following is the RAM output of a typical non – composite beam

Gravity Beam Design

RAM Steel v8.1
DataBase: Non - Composite Beams — 10/24/05 19:05:13
Building Code: IBC — Steel Code: AISC LRFD

Floor Type: Non Composite Beams **Beam Number = 20**

SPAN INFORMATION (ft): I-End (37.33,55.50) J-End (65.33,55.50)

Beam Size (Optimum) = W24X68 Fy = 50.0 ksi
Total Beam Length (ft) = 28.00
Mp (kip-ft) = 737.50

POINT LOADS (kips):

Dist	DL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%
9.333	12.21	0.00	0.0	10.36	0.00	0.0	0.00	0.0
9.333	12.21	0.00	0.0	10.36	0.00	0.0	0.00	0.0
18.667	12.21	0.00	0.0	10.36	0.00	0.0	0.00	0.0
18.667	12.21	0.00	0.0	10.36	0.00	0.0	0.00	0.0

LINE LOADS (k/ft):

Load	Dist	DL	LL	Red%	Type
1	0.000	0.068	0.000	---	NonR
	27.999	0.068	0.000		

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 63.61 kips 0.90Vn = 265.56 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	Max +	1.2DL+1.6LL	591.0	14.0	9.3	1.00	0.90	613.16
Controlling		1.2DL+1.6LL	591.0	14.0	9.3	1.00	0.90	613.16

REACTIONS (kips):

	Left	**Right**
DL reaction	25.38	25.38
Max +LL reaction	20.72	20.72
Max +total reaction (factored)	63.61	63.61

DEFLECTIONS:

Dead load (in)	at	14.00 ft	=	-0.637	L/D	=	527
Live load (in)	at	14.00 ft	=	-0.526	L/D	=	639
Net Total load (in)	at	14.00 ft	=	-1.163	L/D	=	289

The following is the RAM output of a typical composite beam

Gravity Beam Design

RAM Steel v8.1
DataBase: Composite Beams
Building Code: IBC

10/24/05 18:53:27
Steel Code: ASD 9th Ed.

Floor Type: Composite Beams **Beam Number = 33**

SPAN INFORMATION (ft): I-End (46.67,55.50) J-End (46.67,83.25)

Beam Size (Optimum) = W16X26 Fy = 50.0 ksi
Total Beam Length (ft) = 27.75

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.50	3.50
Decking Orientation	perpendicular	perpendicular
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in)	=	83.25	Y bar(in)	=	17.15
Seff (in3)	=	56.69	Str (in3)	=	72.17
Ieff (in4)	=	794.08	Itr (in4)	=	1211.45
Stud length (in)	=	4.50	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 8.0
# of studs: Max = 54 Partial = 14 Actual = 14
Number of Stud Rows = 1 Percent of Full Composite Action = 29.33

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.840	0.000	0.747	---	NonR	0.000
	27.750	0.840	0.000	0.747			0.000
2	0.000	0.026	0.026	0.000	---	NonR	0.000
	27.750	0.026	0.026	0.000			0.000

SHEAR: Max V (DL+LL) = 22.38 kips fv = 5.96 ksi Fv = 17.89 ksi

MOMENTS:

Span	Cond	Moment	@	Lb	Cb	Tension Flange		Compr Flange	
		kip-ft	ft	ft		fb	Fb	fb	Fb
Center	PreCmp	2.5	13.9	---	---	0.79	33.00	0.79	33.00
	InitDL	2.5	13.9	---	---				
	Max +	155.2	13.9	---	---				
	Mmax/Seff					32.86	33.00	---	---
	Mconst/Sx+Mpost/Seff					33.12	45.00	---	---
Controlling		155.2	13.9	---	---	32.86	33.00	---	---

fc (ksi) = 0.68 Fc = 1.58

REACTIONS (kips):

	Left	Right
Initial reaction	0.36	0.36
DL reaction	12.02	12.02
Max +LL reaction	10.36	10.36
Max +total reaction	22.38	22.38

DEFLECTIONS:

Initial load (in) at 13.87 ft = -0.040 L/D = 8336

The following is the RAM output of a typical composite girder

Gravity Beam Design

RAM Steel v8.1
DataBase: Composite Beams
Building Code: IBC

10/24/05 18:53:27
Steel Code: ASD 9th Ed.

Floor Type: Composite Beams **Beam Number = 20**

SPAN INFORMATION (ft): I-End (37.33,55.50) J-End (65.33,55.50)

Beam Size (Optimum) = W21X50 Fy = 50.0 ksi
Total Beam Length (ft) = 28.00

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Concrete thickness (in)	3.25	3.25
Unit weight concrete (pcf)	115.00	115.00
f'c (ksi)	3.50	3.50
Decking Orientation	parallel	parallel
Decking type	USD 3" Lok-Floor	USD 3" Lok-Floor

beff (in)	=	84.00	Y bar(in)	=	20.14
Seff (in3)	=	155.17	Str (in3)	=	157.28
Ieff (in4)	=	3013.74	Itr (in4)	=	3084.20
Stud length (in)	=	4.50	Stud diam (in)	=	0.75

Stud Capacity (kips) q = 10.7

# of studs per stud segment: Full = 35,1,35
Partial = 31,2,31
Actual = 31,2,31

Number of Stud Rows = 2 Percent of Full Composite Action = 90.78

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	CLL
9.333	12.02	0.36	0.00	0.0	10.36	0.00	0.0	0.00	0.0	0.00
9.333	12.02	0.36	0.00	0.0	10.36	0.00	0.0	0.00	0.0	0.00
18.667	12.02	0.36	0.00	0.0	10.36	0.00	0.0	0.00	0.0	0.00
18.667	12.02	0.36	0.00	0.0	10.36	0.00	0.0	0.00	0.0	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	CLL
1	0.000	0.050	0.050	0.000	---	NonR	0.000
	27.999	0.050	0.050	0.000			0.000

SHEAR: Max V (DL+LL) = 45.45 kips fv = 5.75 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	11.7	14.0	---	---	1.48	33.00	1.48	33.00
	InitDL	11.7	14.0	---	---				
	Max +	422.6	14.0	---	---				
	Mmax/Seff					32.68	33.00	---	---
	Mconst/Sx+Mpost/Seff					33.26	45.00	---	---
Controlling		422.6	14.0	---	---	32.68	33.00	---	---

fc (ksi) = 1.04 Fc = 1.58

REACTIONS (kips):