TECHNICAL ASSIGNMENT 2

PRO/CON STRUCTURAL STUDY OF ALTERNATE FLOOR SYSTEMS

THE TOWERS AT THE CITY UNIVERSITY OF NEW YORK

NEW YORK CITY, NEW YORK

ROBIN SCARAMASTRO

STRUCTURAL OPTION

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AE 481 W

DR. MEMARI

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EXECUTIVE SUMMARY

A study of alternate floor framing systems was performed to investigate other possible systems that could have been used for The Towers. The existing framing system and four alternate framing systems were analyzed and compared to each other to determine their feasability. The framing systems included in this report are:

- Cast in place flat plate concrete
- A modified grid with two-way concrete slab with edge beams
- Precast framing
- A modified grid with composite steel deck
- Steel framing with composite steel joists

To analyze the systems, a bay was chosen from the typical floor plan. Gravity loads, which were determined in Technical Report 1, were imposed on the structure. Design aids and RAM models were used to develop the preliminary designs for each framing analysis and to further check hand calculations. Detailed calculations and framing plans can be found in the Appendices at the end of this report. From the analysis, the designs for each system are as follows:

SYSTEM	FLOOR SLAB	BEAMS	GIRDERS
Flat Plate Concrete	10″ NW concrete with #8 bars spaced at 18″ o.c.	-	-
Two Way Slab with Edge Beams	8″ NW concrete with #8 bars spaced 16″ o.c.	C.I.P 18″x26″	-
Precast Concrete Framing	12DT26 precast planks with 2″ concrete topping	12LB36	12LB36
Steel Framing with Composite Deck	6″ NW concrete with 3″ composite metal deck	W10x12	W16x31
Steel Framing with Composite Joists	4″ LW concrete with 2″ composite metal deck	16K7 bar joists	W16x31

After analyzing each framing system, a chart was created to compare advantages and disadvantages of using the system. Comparisons are based on cost, schedule, and architectural impact of the building layout.

LOADS

The following is the list of gravity dead and live loads for a dormitory occupancy used in the design of The Towers. These loads are in accordance with the Building Code of the City of New York and do not include the self weight of the structural members. Lateral loads imposed on The Towers are the result of wind and seismic forces. Per the City Building Code of the City of New York, the wind loads are calculated based on the methods provided in ASCE 7-98 and the seismic loads are calculated based on the UBC Section 2312-1990.

DORMITORY	PSF
Construction Dead Load	
- 8″ normal weight concrete elevated slab	100
Superimposed Dead Load	
- ceiling	4
- floor finish	2
- mechanical/electrical	2
- partitions (100-200 plf)	12
Total Dead Load	120
Live Load	
- for partitioned dormitories	40

LOBBY	PSF
Construction Dead Load	
- 10″ normal weight concrete elevated slab	125
Superimposed Dead Load	
- ceiling	2
- floor finish	2
- mechanical/electrical	6
Total Dead Load	135
Live Load	100

ROOF (MECHANICAL)	PSF
Construction Dead Load	
- 8″ normal weight concrete elevated slab	100
Superimposed Dead Load	
- ceiling	2
- mechanical/electrical	6
- roofing and insulation	6
Total Dead Load	115
Live Load	
- weight of equipment and ponding water	150

EXTERIOR WALL LOADS	PSF
Dead Load	
- prefabricated thin brick panels with metal stud back-up wall	24
- curtain wall system	15

The codes design aids used in the design of all five systems are as follows:

- *CRSI Handbook* and *ACI 18-2005* for concrete
- *PCI Handbook* for precast concrete
- *13th AISC Manual of Steel Construction Edition LRFD* for steel
- *The Building Code of the City of New York*
- *ASCE7-02*

ALTERNATE FRAMING SYSTEMS

The existing and four alternate framing systems were analyzed to determine if any would be feasible for the design of The Towers:

- A **cast-in-place flat plate concrete floor system** was used in the original design to keep floor to ceiling heights at the minimum in accordance with the Building Code of the City of New York and to keep the building height down.
- A **two-way cast-in-place concrete floor slab with edge beams** will be analyzed to determine if a thinner slab can be used. The bays of the existing framing plan were modified to allow for edge beams to determine if a thinner slab could be used.
- A **precast concrete framing system** will be analyzed to determine if the construction schedule and cost of the building can be decreased.
- **Composite steel beams** will be analyzed to determine if the weight of the building can be decreased. The bays of the original structural plan were placed into a more regular grid to see if the number of columns can be decreased.
- **Composite steel joists** will be analyzed to determine if the weight of the steel system can further be decreased without compromising structural integrity.

A typical bay for each analysis was chosen for the 5th floor under gravity loads. The current story height is 8′-8″, but there is an allowance of three extra feet per story to comply with the zoning ordinance.

CAST-IN-PLACE FLAT PLATE CONCRETE ANALYSIS

A flat plate concrete was used to obtain minimum floor to floor heights. The required thickness required for deflection control of the slab is 10″ and the loads imposed on the slab are directly transferred to the cast-in-place columns. The slab is heavily reinforced with #8 bars at 18″ on-center in each direction. See Appendix A for flat plate concrete calculations.

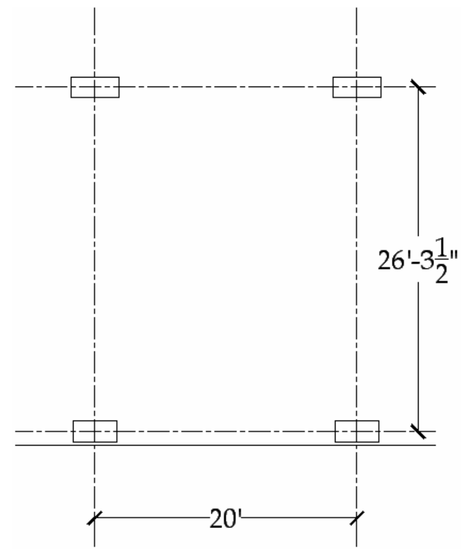


Figure 1: Typical bay used in analysis of flat plate system

ADVANTAGES

- Cost savings with respect to shorter mechanical and plumbing runs and curtain wall spans associated with the low story heights
- Slab provides more than the required fire protection needed by code

DISADVANTAGES

- Slab thicknesses become very large to control deflections for longer spans
- Must shore and re-shore during construction to prevent collapse, which can add to the construction cost

TWO WAY CONCRETE SLAB WITH BEAMS ANALYSIS

A two-way concrete slab was analyzed to determine if a thinner floor slab could be used. The bays were modified to allow for beams to be placed between the columns. An 8 ½" slab reinforced with #8 bars spaced at 16" is capable of carrying the loads. See Appendix B for two-way slab calculations.

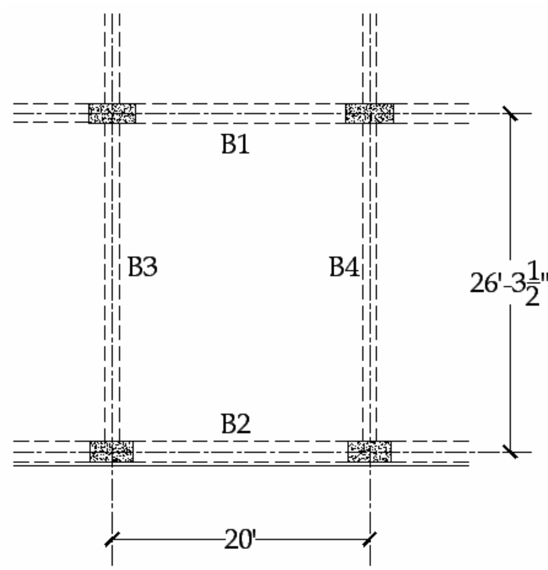


Figure 2: Typical bay used in analysis of two-way slab system

ADVANTAGES

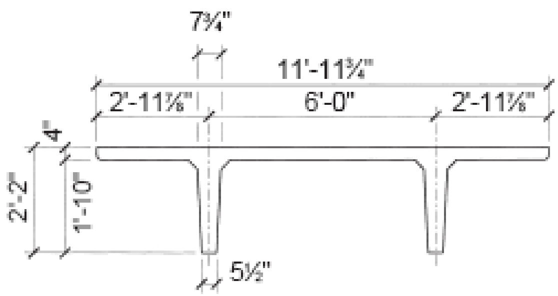
- Thinner slab to allow for smaller floor to floor heights
- Beams provide extra stiffness
- Concrete provides required fire rating for the floor assembly

DISADVANTAGES

- Formwork can become expensive due to irregularity of bay, beam and column sizes
- Since the original column layout of the entire building is irregular, in order to create rectangular bays, the spacing between columns can become very large. This will cause the beam sizes to be very large.

PRECAST CONCRETE FRAMING ANALYSIS

A precast system was analyzed to determine if construction time and cost could be reduced. Using technical data obtained from High Concrete Products, it was determined that a 12DT26 prestressed double tee is capable of spanning the entire width of the building and meet the deflection criteria. See Appendix C for precast concrete calculations.



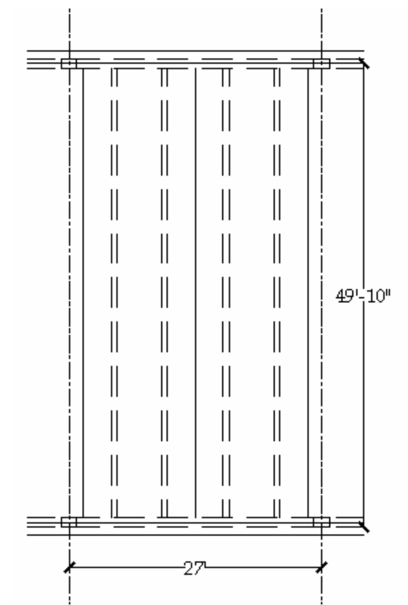


Figure 4: Typical bay used in analysis of precast concrete system

ADVANTAGES

- Controlled fabrication process for planks, beams and columns allows for stronger structural members
- Quick to erect
- Uses high strength concrete and prestressing cables

DISADVANTAGES

- Extensive welding of steel to connect tees, girders and columns. This can cause an increase in labor costs.

COMPOSITE STEEL BEAM FRAMING ANALYSIS

A composite steel beam system was analyzed in RAM to determine if steel would be a feasible structural system fro The Towers. The bays were modified to allow for simple design of the system.

It was determined that a 6″ thick composite concrete slab and 3″ metal deck would be capable to span across the infill beams and carry the live and superimposed dead loads. W10x12 beams are carried by W16x31 girders for the bay shown. See Appendix D for detailed calculations.

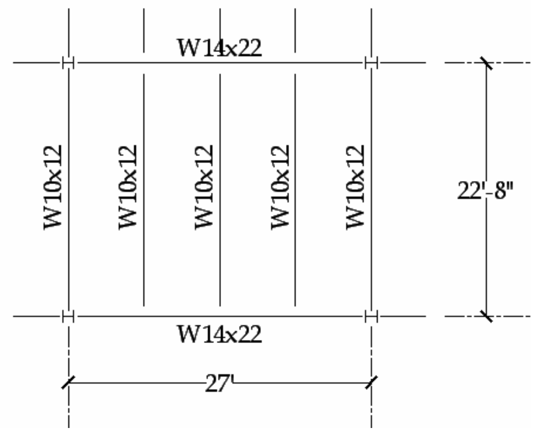


Figure 5: Typical bay used in analysis of a composite steel beam system

ADVANTAGES

- Lighter system than cast-in-place concrete
- Steel construction is a popular method in New York City
- High strength to weight ratio
- Can act as an acoustical barrier between floors

DISADVANTAGES

- Fireproofing is an important consideration when calculating the loads
- Increases floor to floor height

COMPOSITE OPEN WEB STEEL JOIST ANALYSIS

Composite open web steel joists were also analyzed to see if the weight of the structure can be decreased more. Open web joists can span large depths, so the number of beams will decrease.

It was determined that for the loading conditions, 16K7 joists spaced at 4′ on center would be capable to carry the load and stay within the deflection criteria. The slab consists of 4″ lightweight concrete with 2″ metal decking.

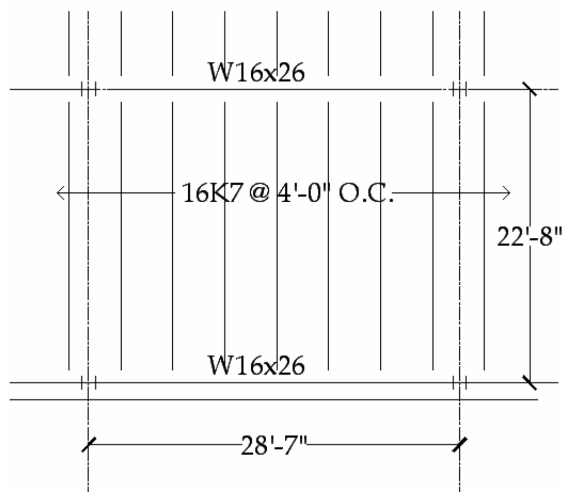


Figure 6: Typical bay used in analysis of steel bar joist system

ADVANTAGES

- Lightweight system
- Joists are readily available and easy to erect
- Can act as an acoustical barrier between floors

DISADVANTAGES

- Fireproofing of joists must be provided by fibrous spray-on fireproofing or by a gypsum board ceiling later suspended from the joist
- Long spans can get very deep members under loading conditions
- Added floor to floor height with joist and composite deck

CONCLUSIONS

The results of the five analyses are provided in the following table. Listed for each system is a summary of advantages and disadvantages. The 27th Edition of the RS Means square cost data was used to estimate cost per square foot of each system.

STRUCTURAL SYSTEM	ADVANTAGES	DISADVANTAGES	COST PER SQ.FT.	FURTHER INVESTIGATION
Existing flat-plate slab	• Low floor to floor depth • Slab provides code required fire rating	• Slab thickness can become very large • Columns are large and impact floor layout	$13.85	-
Two-way concrete slab with edge beams	• Low floor to floor height • Thinner slab can be attained with stiff columns and beams • Slab provides code required fire rating	• Irregularity of the building affects the placement of columns and beams, potentially causing a problem with the floor layout	$18.70	NO
Precast concrete system	• Fast erection times • Controlled fabrication of members • Uses high strength concrete and prestressing cables	• Requires extensive welding of members	$16.46	YES
Composite steel beams	• Light system with a high strength to weight ratio • Can use a grid with larger spacing between columns to best fit the irregular building layout • Acoustical barrier	• Will increase story height • Fireproofing is required	$22.85	YES
Composite steel joists	• Has the ability to span long distances • Acoustical barrier	• Fireproofing is difficult to apply to bar joists • Will increase story height	$17.65	NO

APPENDIX A - FLAT PLATE CALCULATIONS

f'c = 4,000 psi (for slabs)
f'c = 5,000 psi (for columns)
fy = 60,000 psi
Superimposed Dead Load = 20 psf
Live Load = 40 psf (dormitory bay)
Story Height = 8'

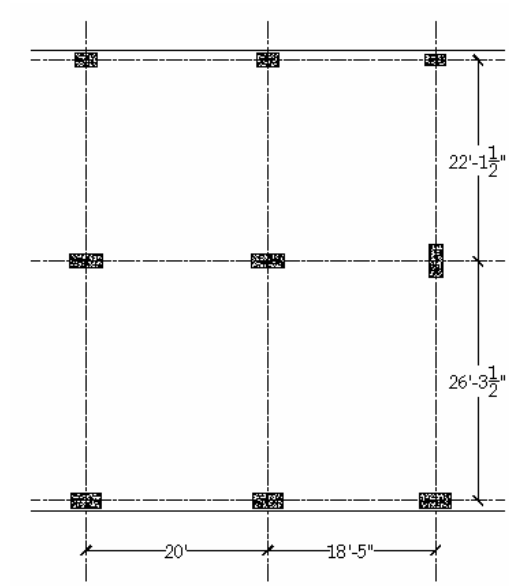


Figure 7: Typical bay used in analysis of flat plate system

The Equivalent Frame Method was used to design the slab. All distribution multipliers were obtained from Tables A.1 and A.7 from the ACI 318-05 code reference.

- Deflection control for flat plate slab:

$$h = \frac{l_n}{30} = \frac{315.5'' - 20''}{30}$$

$$h = 9.85''$$

Use slab thickness of 10"

- Shear strength of slab:

Assume #8 bars and ¾" of cover
d = 10" – ¾" – 1"
d = 8.25"

Factored DL = 1.2((150 pcf)(10"/12) + 20 psf)
174 psf
Factored LL = 1.6(40 psf)
64 psf
Total Load = 238 psf

For wide beam action:

$$V_U = (0.238\text{ksf})\left(\frac{\left(290.5''/2\right) - \left(20''/2\right) - 8.25''}{12''/1'}\right)$$

$$V_U = 2.62^k$$

$$\phi Vc = \phi 2\sqrt{f'c}\,b_w d = (0.75)(2)\left(\sqrt{4000\text{psi}}\right)(12'')(8.75'')$$

$$\phi Vc = 10^k$$

$\phi Vc > V_U \therefore \text{ok}$

For two-way action:

$$V_U = (0.238\text{ksf})[(24'-2.5")(19'-2.5") - (3'-8.25")(2'-4.25")]$$

$$V_U = 108.6^k$$

$$\phi Vc = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'c}\,b_o d$$

$$b_o = (2)(20"+8.25") + (2)(36"+8.25")$$

$$b_o = 145"$$

$$\phi Vc = (0.75)\left(\frac{(40)(8.25")}{145"} + 2\right)\sqrt{4000\text{psi}}(145")(8.25")$$

$$\phi Vc = 242.6^k$$

$\phi Vc > V_U \therefore \text{ok}$

- Flexural stiffness of slab-beams

$$\frac{C_{N1}}{l_1} = \frac{20"}{290.5"} = 0.0688$$

$$\frac{C_{N2}}{l_2} = \frac{36"}{230.5"} = 0.156$$

C_{N2}/l_2	k_{NF}
0.1	4.18
0.156	4.2808
0.2	4.36

$$K_{SB} = k_{NF}\frac{E_{CS}I_S}{l_1}$$

$$I_S = \frac{(230.5")(10")^3}{12} = 19210\text{in}^4$$

$$E_{CS} = 57000\sqrt{4000\text{psi}} = 3.6\text{x}10^6\text{ psi}$$

$$K_{SB} = (4.2808)\frac{(19210)(3.6\text{x}10^6\text{ psi})}{290.5"} = 1.02\text{x}10^9\text{ in}-\text{lb}$$

- Equivalent flexural stiffness of columns

$$\frac{t_a}{t_b} = \frac{5"}{5"} = 1.0$$

$$\frac{H}{H_c} = \frac{8'-10"}{8'-0"} = 1.104$$

H/Hc	k_{AB}
1.1	5.09
1.104	5.1396
1.15	5.71

$$K_C = k_{AB}\frac{E_{CS}I_S}{l_1}$$

$$I_{C\,int} = \frac{bh^3}{12} = \frac{(20'')(36'')^3}{12} = 77760\text{in}^4$$

$$I_{Cext} = \frac{bh^3}{12} = \frac{(20'')(40'')^3}{12} = 106700\text{in}^4$$

$$E_{CS} = 57000\sqrt{5000\text{psi}} = 4.1\text{x}10^6\,\text{psi}$$

$$K_{C\,int} = (5.1396)\frac{(77760\text{in}^4)(4.1\text{x}10^6\,\text{psi})}{290.5''} = 5.64\text{x}10^9\,\text{in} - \text{lb}$$

$$K_{Cext} = (5.1396)\frac{(106700\text{in}^4)(4.1\text{x}10^6\,\text{psi})}{290.5''} = 7.74\text{x}10^9\,\text{in} - \text{lb}$$

$$K_t = \frac{9E_{CS}C}{\left[l_2(1 - c_2/l_2)^3\right]}$$

$$C = \Sigma(1 - 0.63\,x/y)(x^3 y/3) = (1 - (0.63)(10''/20''))((10'')^3(20'')/3)$$

$$C = 4570\text{in}^4$$

$$K_t = \frac{9(3.6\text{x}10^6\,\text{psi})(4570\text{in}^4)}{\left[(230.5''(1 - 36''/230.5'')^3)\right]} = 1.02\text{x}10^9\,\text{in} - \text{lb}$$

$$K_{EC} = \frac{\Sigma K_C \Sigma K_t}{\Sigma K_C + \Sigma K_t}$$

$$K_{ECext} = \frac{((2)(7.74\text{x}10^9))((2)(1.02\text{x}10^9))}{(2)(7.74\text{x}10^9) + (2)(1.02\text{x}10^9)} = 1.8\text{x}10^9\,\text{in} - \text{lb}$$

$$K_{EC\,int} = \frac{((2)(5.64\text{x}10^9))((2)(1.02\text{x}10^9))}{(2)(5.64\text{x}10^9) + (2)(1.02\text{x}10^9)} = 1.73\text{x}10^9\,\text{in} - \text{lb}$$

- Carry over factors for moment distribution

Slab-beams

C_{N2}/l_2	C_{NF}
0.1	0.5
0.156	0.5056
0.2	0.51

Columns

H/Hc	C_{AB}
1.1	0.57
1.104	0.5692
1.15	0.56

- Distribution factors

At exterior joint

$$DF = \frac{1.02}{1.02 + 1.8} = 0.362$$

At interior joint

$$DF = \frac{1.02}{1.02 + 1.73} = 0.371$$

- Moment distribution

Fixed end moments:

$$FEM = m_{NF} w_u l_2 l_1^2$$

$$FEM_1 = (0.0854)(238psf)(19.208')(24.208')^2$$

$$FEM = 175.6'^k$$

$$FEM_2 = (0.0854)(238psf)(19.208')(20.46')^2$$

$$FEM_2 = 163.4'^k$$

Joint	A	B		C
Member	AB	BA	BC	CB
DF	0.36	0.37	0.37	0.36
COF	0.51	0.51	0.51	0.51
FEM	175.60	-175.60	163.40	-163.40
	-63.57	-32.10		
	8.30	16.44	16.44	8.30
	-3.00	-1.52	28.35	56.15
	-5.03	-9.96	-9.96	-5.03
	1.82	0.92	0.92	1.82
SUM	114.12	-201.82	199.15	-102.16
Mo	334.90		334.90	
$M_{midspan}$	176.93		184.24	

- Distribution of factored moments

	Factored Moment	Column Strip %	Column Strip Moment	Moment in Two Half-Middle Strips
End Span:				
Exterior Neg	114.12	100	114.12	0.00
Positive	176.93	60	106.16	70.77
Interior Neg	201.83	75	151.37	50.46
Second Span:				
Negative	199.15	75	149.36	49.79
Positive	184.24	60	110.54	73.70

- Reinforcing to resist factored moments

Assuming tension controlled:
Mu=151.37'k

$$b = \frac{230.5''}{2} = 115.25''$$

$$Ru = \frac{Mu}{\phi bd^2} = \frac{151.27'^{k}(12000)}{(0.9)(115.25'')(8.25'')^2} = 257 psi$$

$$\rho = 0.0045$$

$$As = \rho bd = 0.0045(115.25'')(8.25'') = 4.28 in^2$$

Use (5) #8 Bars

$$s_{max} = 2h = 20'' > 18''$$

$$a = \frac{Asfy}{0.85 f'cb} = \frac{(5 in^2)(60 ksi)}{(0.85)(4 ksi)(115.25'')} = 0.766''$$

$$c = \frac{a}{\beta_1} = \frac{0.766''}{0.85} = 0.9''$$

$$\varepsilon_t = \frac{0.003 d_t}{c} - 0.003 = \frac{(0.003)(8.25'')}{0.9''} - 0.003$$

$$\varepsilon_t = 0.0245 > 0.005 \therefore \text{Tension controlled}$$

- Final Design

Use 10" normal weight concrete flat plate slab with #8 bars spaced 18" in each direction.

APPENDIX B - TWO-WAY SLAB CALCULATIONS

f'c = 4000 psi (slab)
f'c = 5000 psi (beams and columns)
fy = 60,000 psi
Superimposed Dead Load = 20 psf
Live Load = 40 psf (dormitory bay)
Story Height = 8'

Assume:

14"x26" beams
20"x36" columns
8" slab

Equivalent frame method was used to analyze the two way slab system.

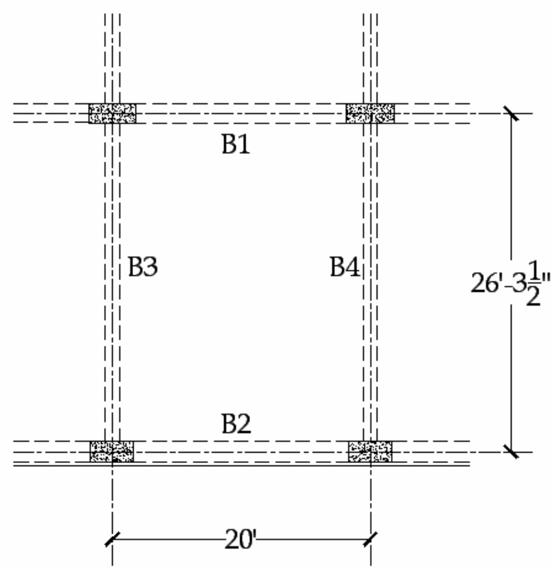


Figure 8: Typical bay used in analysis of two-way slab system

- Slab properties

$$b_E = \begin{matrix} b_w + 4t = 14"+(4)(8") = 46" \\ b_w + h_w = 14"+8" = 22"** \end{matrix}$$

$$\frac{b_E}{b_w} = \frac{22"}{14"} = 1.57$$

$$\frac{t}{h} = \frac{8"}{26"} = 0.308$$

$$k = 1.23$$

$$I_b = k\frac{bh^3}{12} = 1.23\left(\frac{(14")(28")^2}{12}\right) = 31500 in^4$$

$$I_s = \frac{l_2 t^3}{12} = \frac{(240")(8")^3}{12} = 10240 in^4$$

$$\alpha = \frac{(EI)_b}{(EI)_s} = \frac{\left(\sqrt{5000psi}\right)\left(31500 in^4\right)}{\left(\sqrt{4000psi}\right)\left(10240 in^4\right)} = 3.44$$

- Minimum thickness

$$h = \frac{l_n\left(0.8 + {}^{fy}\!\diagup_{200,000}\right)}{36 + 2\beta}$$

$$l_n = 315.5'' - 10'' = 305.5''$$

$$\beta = \frac{305.5''}{204''} = 1.5$$

$$h = \frac{(305.5'')\left(0.8 + {}^{60,000psi}\!\diagup_{200,000}\right)}{36 + 2(1.5)} = 8.5''$$

- Slab-beam stiffness

$$\frac{C_{N1}}{l_1} = \frac{20''}{240''} = 0.083$$

$$\frac{C_{N2}}{l_2} = \frac{36''}{315.5''} = 0.114$$

$$I_{SB} = 2.72\frac{(14'')(28'')^3}{12} = 69700in^4$$

C_{N2}/l_2	k_{NF}
0.1	4.18
0.114	4.2052
0.2	4.36

$$K_{SB} = \frac{(4.2052)\left(57000\sqrt{4000psi}\right)\left(69700in^4\right)}{240''}$$

$$K_{SB} = 4.4x10^9$$

- Equivalent column stiffness

$$t_a = 28'' \qquad H = 10.333'$$

$$t_b = 4.25'' \qquad Hc = 8'$$

$$\frac{t_a}{t_b} = 6.6 \qquad \frac{H}{Hc} = 1.29$$

K_{AB}			
	1.25	1.29	1.30
6.0	8.9		10.7
6.6	8.96	10.131	10.424
7.0	9.0		10.24

C_{AB}			
	1.25	1.29	1.30
6.0	0.50		0.50
6.6	0.50	0.495	0.494
7.0	0.50		0.49

$$I_c = \frac{(20'')(36'')^3}{12} = 77800\text{in}^4$$

$$K_C = 10.131\frac{57{,}000\sqrt{5000\text{psi}}\left(77800\text{in}^4\right)}{124''}$$

$$K_C = 25.6\text{x}10^9$$

$$K_t = \frac{9EcC}{l_2\left(1 - c_2/l_2\right)^3}$$

For interior columns:

$$K_{t\,int} = \frac{9\left(57{,}000\sqrt{5000\text{psi}}\right)\left(14250\text{in}^4\right)}{(204'')\left(1 - 32''/204''\right)^3}$$

$$K_{t\,int} = 4.22\text{x}10^9$$

$$K_{ta-int} = \frac{K_t I_{sb}}{I_s} = \frac{\left(4.33\text{x}10^9\right)\left(69700\text{in}^4\right)}{10240\text{in}^4}$$

$$K_{ta-int} = 29.5\text{x}10^9$$

$$K_{ec} = \frac{((2)(25.6))((2)(29.5))}{(2)(25.6 + 29.5)} = 27.4\text{x}10^9$$

For exterior columns:

$$K_{text} = \frac{9\left(57{,}000\sqrt{5000\text{psi}}\right)\left(10670\text{in}^4\right)}{(204'')\left(1 - 32''/204''\right)^3}$$

$$K_{text} = 3.16\text{x}10^9$$

$$K_{ta-ext} = \frac{K_t I_{sb}}{I_s} = \frac{(3.16x10^9)(69700in^4)}{10240in^4}$$

$$K_{ta-ext} = 21.5x10^9$$

$$K_{ec} = \frac{((2)(25.6))((2)(21.5))}{(2)(25.6+21.5)} = 23.4x10^9$$

- Distribution factors

interior joint

$$DF = \frac{4.4}{4.4+4.4+23.4} = 0.137$$

exterior joint

$$DF = \frac{4.4}{4.4+23.4} = 0.158$$

- Moment distribution

Fixed end moments:

$$FEM = m_{NF} w_u l_2 l_1^{\ 2}$$

$$FEM_1 = (0.0849)(226psf)(20')(26.292')^2$$

$$FEM = 265.3'^k$$

$$FEM_2 = (0.0849)(226psf)(20')(24.125')^2$$

$$FEM_2 = 223.3'^k$$

Joint	A	B		C
Member	AB	BA	BC	CB
DF	0.16	0.14	0.14	0.16
COF	0.51	0.51	0.51	0.51
FEM	175.60	-175.60	163.40	-163.40
	-27.74	-14.19		
	1.85	3.62	3.62	1.85
	-0.29	-0.15	13.05	25.53
	-0.90	-1.77	-1.77	-0.90
	0.14	0.07	0.07	0.14
SUM	148.65	-188.02	178.37	-136.79
Mo	334.90		334.90	

$M_{midspan}$	166.57	177.32

- Distribution of factored moments

	Factored Moment	Column Strip		Moment in Two Half-Middle Strips
		%	Moment	
End Span:				
Exterior Neg	148.65	100	148.65	0.00
Positive	166.47	60	100.0	66.47
Interior Neg	188.02	75	141.0	47.02
Second Span:				
Negative	178.37	75	133.78	44.59
Positive	177.32	60	106.4	70.92

- Reinforcing to resist factored moments

Assuming tension controlled:
Mu=148.65′^{k}

$$b = \frac{240''}{2} = 120''$$

$$Ru = \frac{Mu}{\phi bd^2} = \frac{148.65'^{k}(12000)}{(0.9)(120'')(6.75'')^2} = 362.5\text{psi}$$

$$\rho = 0.0064$$

$$As = \rho bd = 0.0064(120'')(6.75'') = 5.2\text{in}^2$$

Use (6) #8 Bars

$$s_{max} = 2h = 17'' < 18''$$

$$a = \frac{Asfy}{0.85f'cb} = \frac{(6\text{in}^2)(60\text{ksi})}{(0.85)(4\text{ksi})(120'')} = 0.882''$$

$$c = \frac{a}{\beta_1} = \frac{0.882''}{0.85} = 1.04''$$

$$\varepsilon_t = \frac{0.003d_t}{c} - 0.003 = \frac{(0.003)(6.75'')}{1.04''} - 0.003$$

$$\varepsilon_t = 0.0164 > 0.005 \therefore \text{Tension controlled}$$

- Final Design

Use 8 ½″ normal weight concrete two way concrete slab with #8 bars spaced 17″ in each direction.

APPENDIX C - PRECAST CONCRETE CALCULATIONS

f'c=6000 psi
½" diameter strands, Fy = 270ksi

DL = self weight + 20 psf
LL = 40 psf

Span = 50'-0"

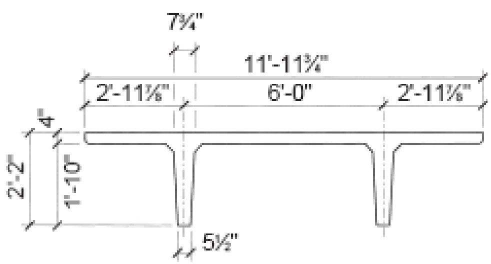


Figure 9: Section of Precast double-tee beam

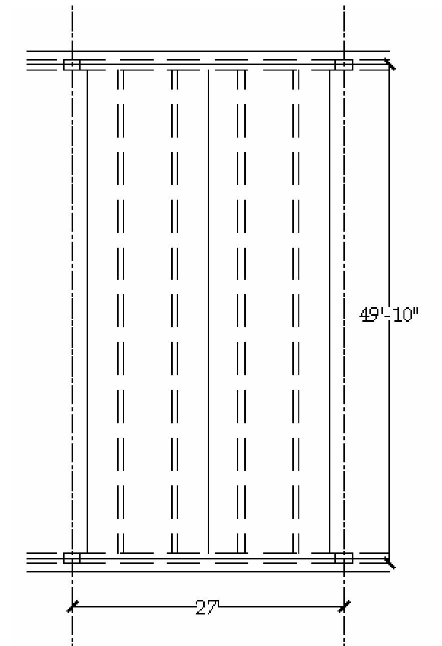


Figure 10: Typical bay used in analysis of Precast concrete system

Strand Pattern	e_e	DESIGN SPAN (FT)																						
		22	24	26	28	30	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66
68-S	14.01	265	213	172	140	114	93	75	60	48	37	28	20											
88-S	14.15			252	208	174	145	121	102	85	71	58	48	38	30	23								
108-S	13.89				270	227	192	163	139	118	101	86	73	61	51	42	34	27	21					
128-S	13.42					276	235	201	172	148	128	110	95	82	70	60	50	42	35	28	22			
148-S	12.84								202	175	152	132	115	100	86	75	65	55	47	40	33	27	22	
168-S	11.97										170	148	129	113	99	86	75	65	56	48	41	34	28	23

-Final Design

Use 12DT26 prestressed double-tees for typical dormitory bays that span 50'. Double-tees are carried by 12LB36 prestressed edge beams and topped with 2"of concrete.

APPENDIX D - COMPOSITE STEEL BEAM CALCULATIONS

Fy = 50ksi
Fu = 65ksi
f'c = 3000 psi

$$w = [1.2(40\text{psf} + 57\text{psf}) + 1.6(40\text{psf})](6.75')$$
$$w = 1.22\text{klf}$$

$$Mu = \frac{(1.22\text{klf})(22.67')^2}{8}$$
$$Mu = 78.4'^{k}$$

$$b_{eff} = \begin{matrix} (22.67')(12)/4 = 68'' \\ (6.75')(12) = 81'' \end{matrix}$$

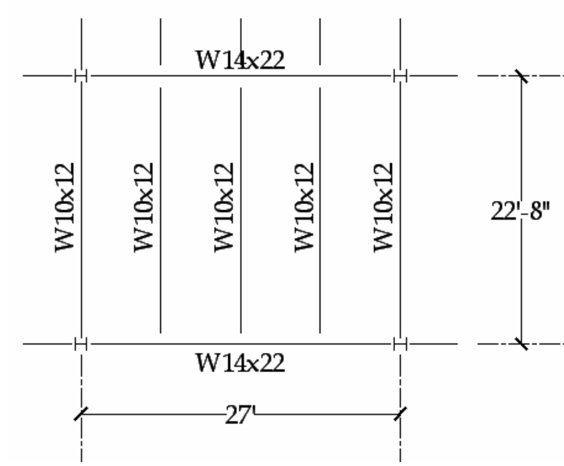


Figure 11: Typical bay used in analysis of composite steel beam system

Assuming 1" of concrete is in compression:

$$y_2 = 6'' - 1'' = 5''$$

@ PNA 2:

$$\phi Mn = 82.4'^{k}$$
$$\Sigma Qn = 156^{k}$$

$$a = \frac{\Sigma Qn}{0.85 f'c b_{eff}} = \frac{156^{k}}{(0.85)(3\text{ksi})(68'')}$$
$$a = 0.899'' < 1'' \therefore \text{ ok}$$

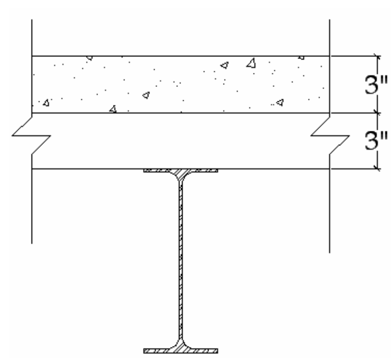


Figure 12: Section showing beam and composite slab construction

W10x12 beams ok

Checking girders:

$$Mu = \frac{(2.04\text{klf})(27')^2}{8}$$
$$Mu = 186'^{k}$$

Checking W14x22:
From Table 3-2 in the AISC 13th Edition Steel Manual

$\phi Mp = 103'^{k} < Mu = 186'^{k}$ W14x22 will not work

Try W16x31 girder

$\phi Mp = 194.9'^{k} > Mu = 186'^{k}$ W16x31 works

- Final Design

Use a 6″ normal weight concrete slab with 3″ composite metal deck on top of W10x12. The beams are carried by W16x31 girders.

APPENDIX E - COMPOSITE STEEL JOIST CALCULATIONS

f'c = 3000 psi
Ft = 30ksi

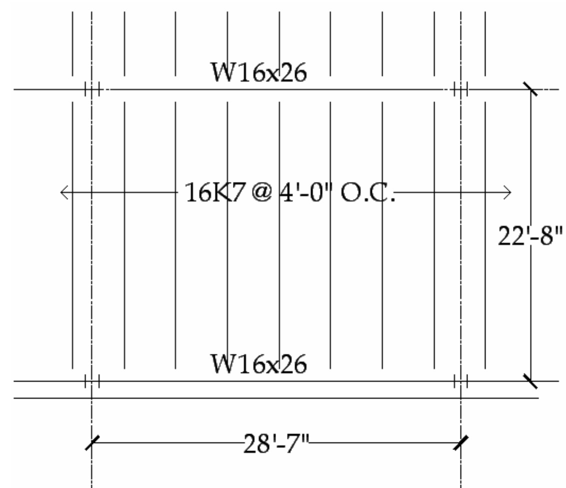


Figure 13: Typical bay used in analysis of composite steel bar joist system

For 16K7 joist:

$$I = 26.767 w_L L^3 \left(x10^{-6}\right)$$
$$I = (26.767)(186\text{plf})(22.67' - 0.33')^3 \left(10^{-6}\right)$$
$$I = 55\text{in}^4$$

$$w_T = ((40\text{psf} + 50\text{psf}) + (40\text{psf}))(4')$$
$$w_T = 320\text{plf}$$
$$w_L = 160\text{plf}$$

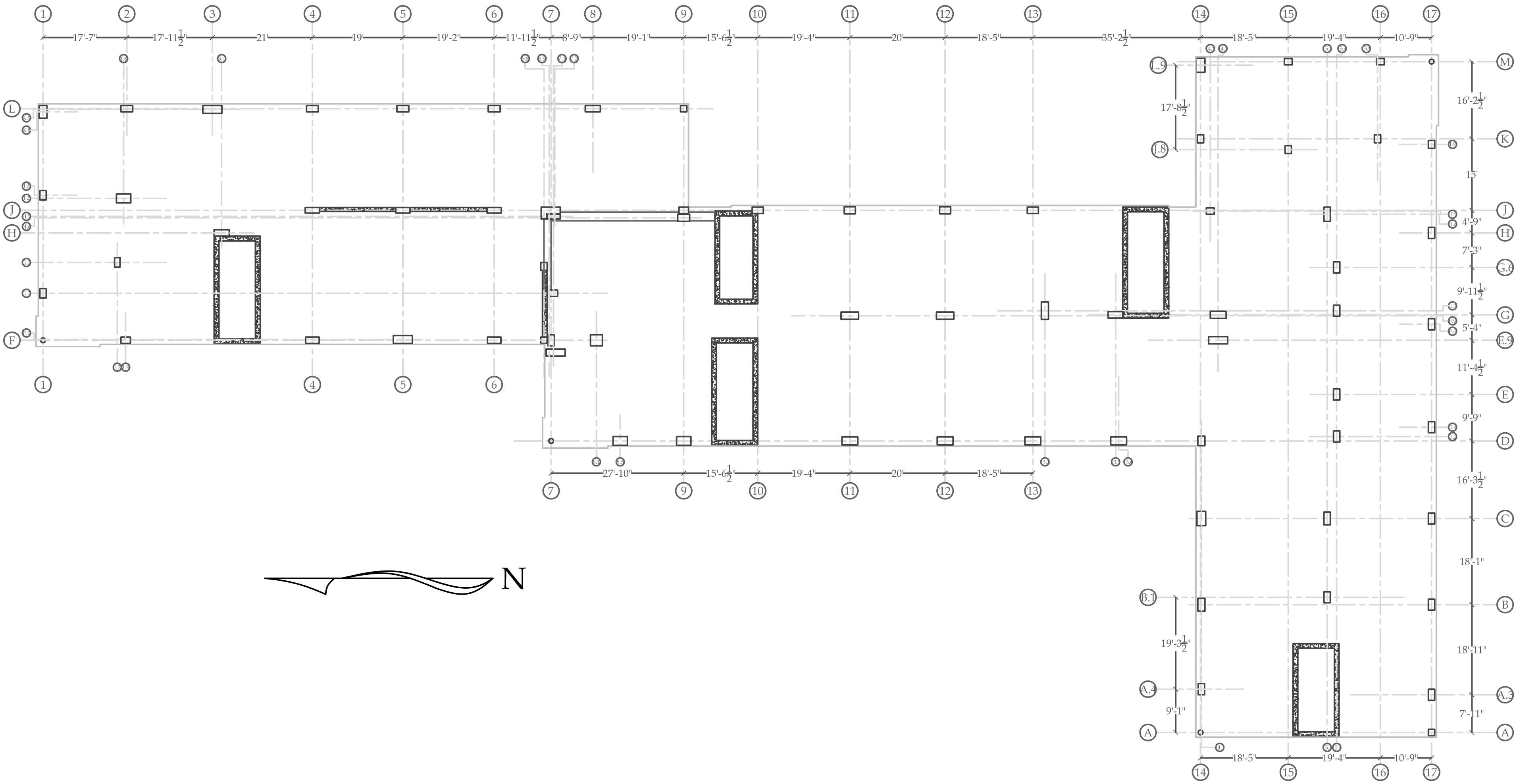
$$\Delta_L = \frac{L}{360} = \frac{22.67'(12)}{360} = 0.76''$$
$$\Delta_T = \frac{L}{240} = \frac{22.67'(12)}{240} = 1.13''$$

$$\Delta = \frac{1.15wL^4(1728)}{384EI}$$
$$\Delta_L = \frac{(1.15)(0.16\text{klf})(22.67')^4(1738)}{384(29000\text{ksi})\left(55\text{in}^4\right)} = 0.14'' < \frac{L}{360} \therefore \text{ok}$$
$$\Delta_L = \frac{(1.15)(0.32\text{klf})(22.67')^4(1738)}{384(29000\text{ksi})\left(55\text{in}^4\right)} = 0.27'' < \frac{L}{240} \therefore \text{ok}$$

- Final Design

Use 16K7 steel bar joists spanning a maximum of 28′ at 4′-0″ on center with a 4″ lightweight concrete slab and 2″composite metal deck. Use W16x26 girders to carry bar joists.

APPENDIX F
STRUCTURAL PLANS

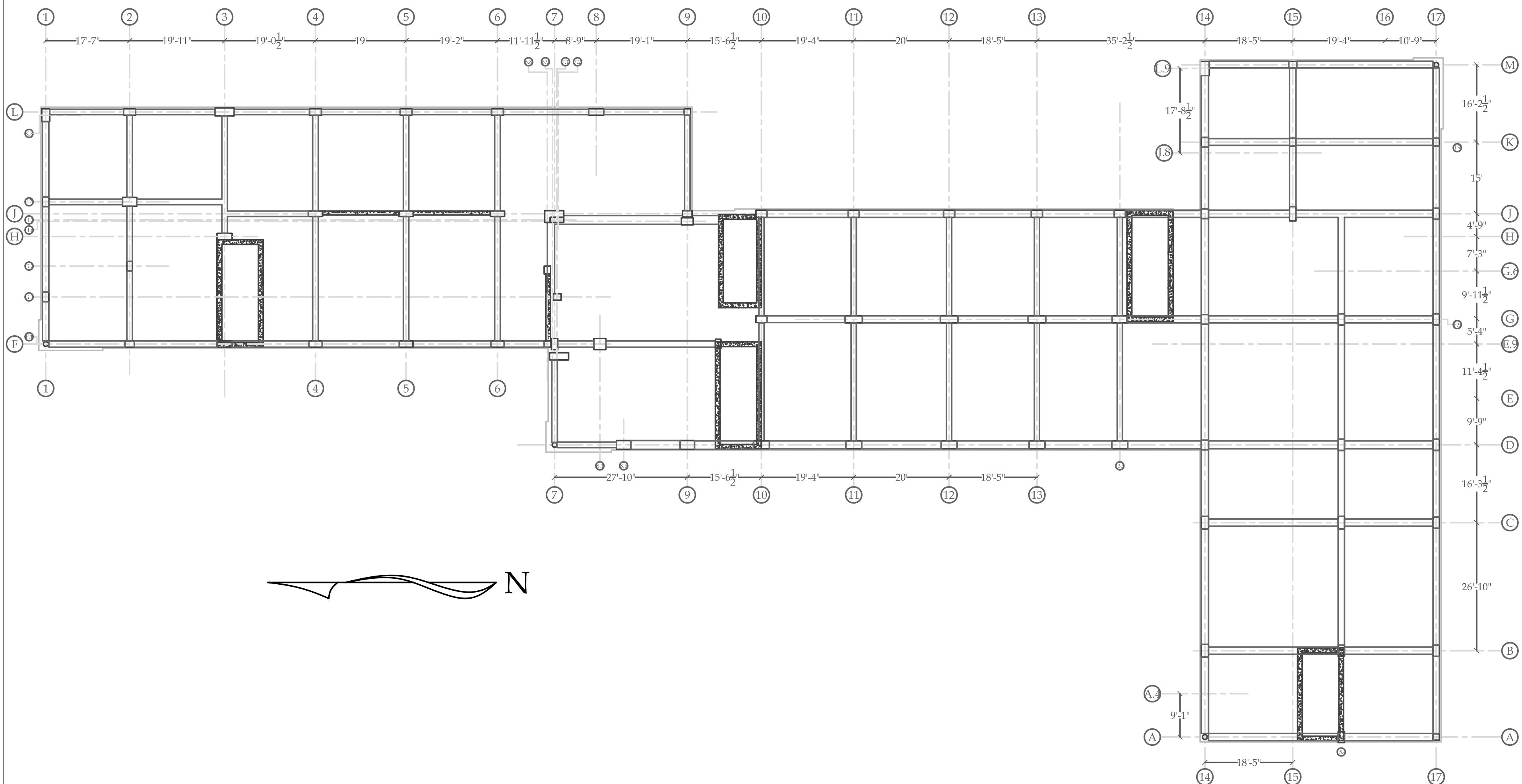


The Towers at The City College of New York

FLAT-PLATE CONCRETE FRAMING
PLAN OF TYPICAL FLOOR

SCALE: $\frac{1}{32}$" = 1'-0"

S.1



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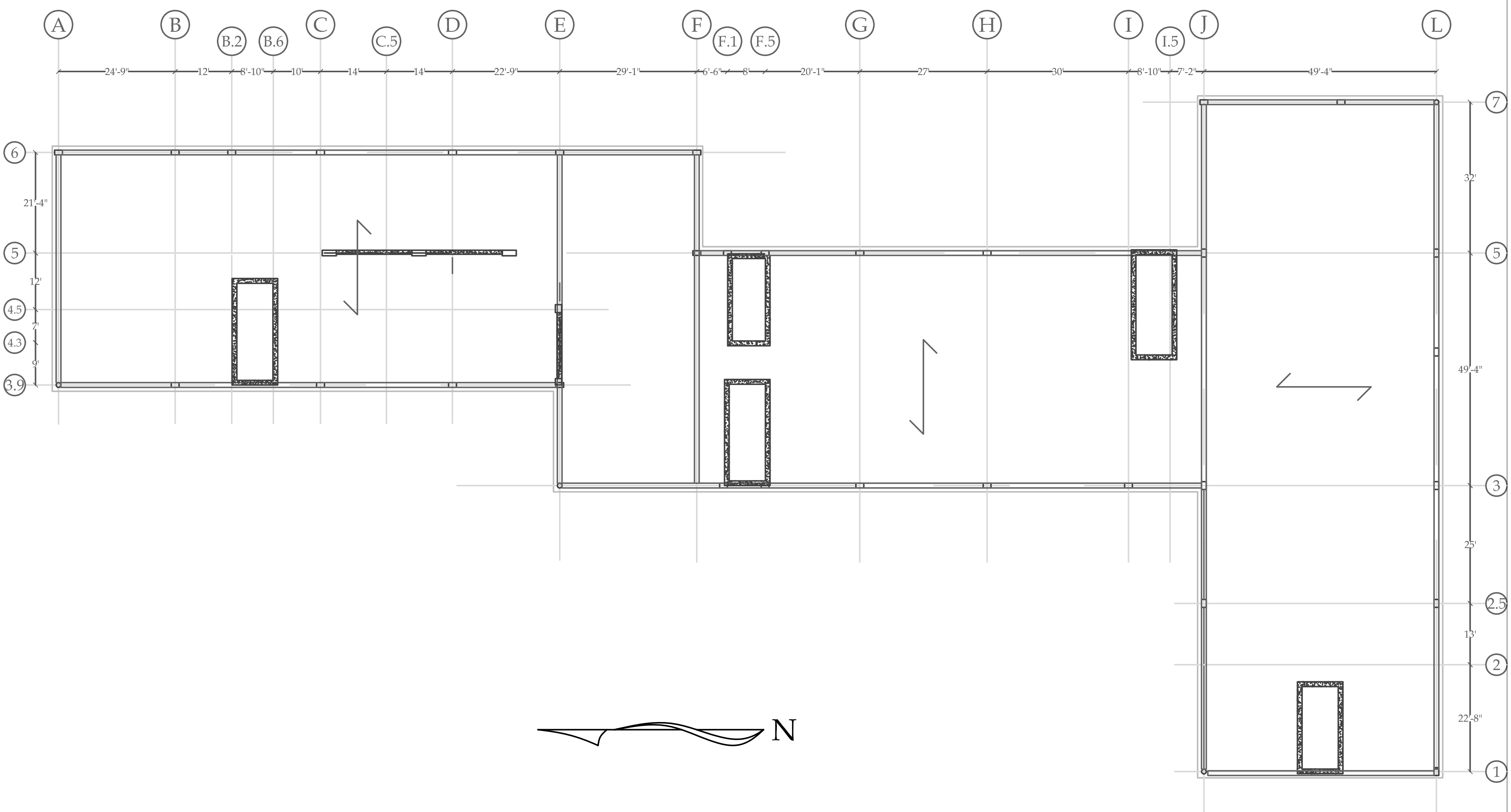
TWO-WAY CONCRETE FRAMING PLAN OF TYPICAL FLOOR

SCALE: $\frac{1}{32}$" = 1'-0"

S.2



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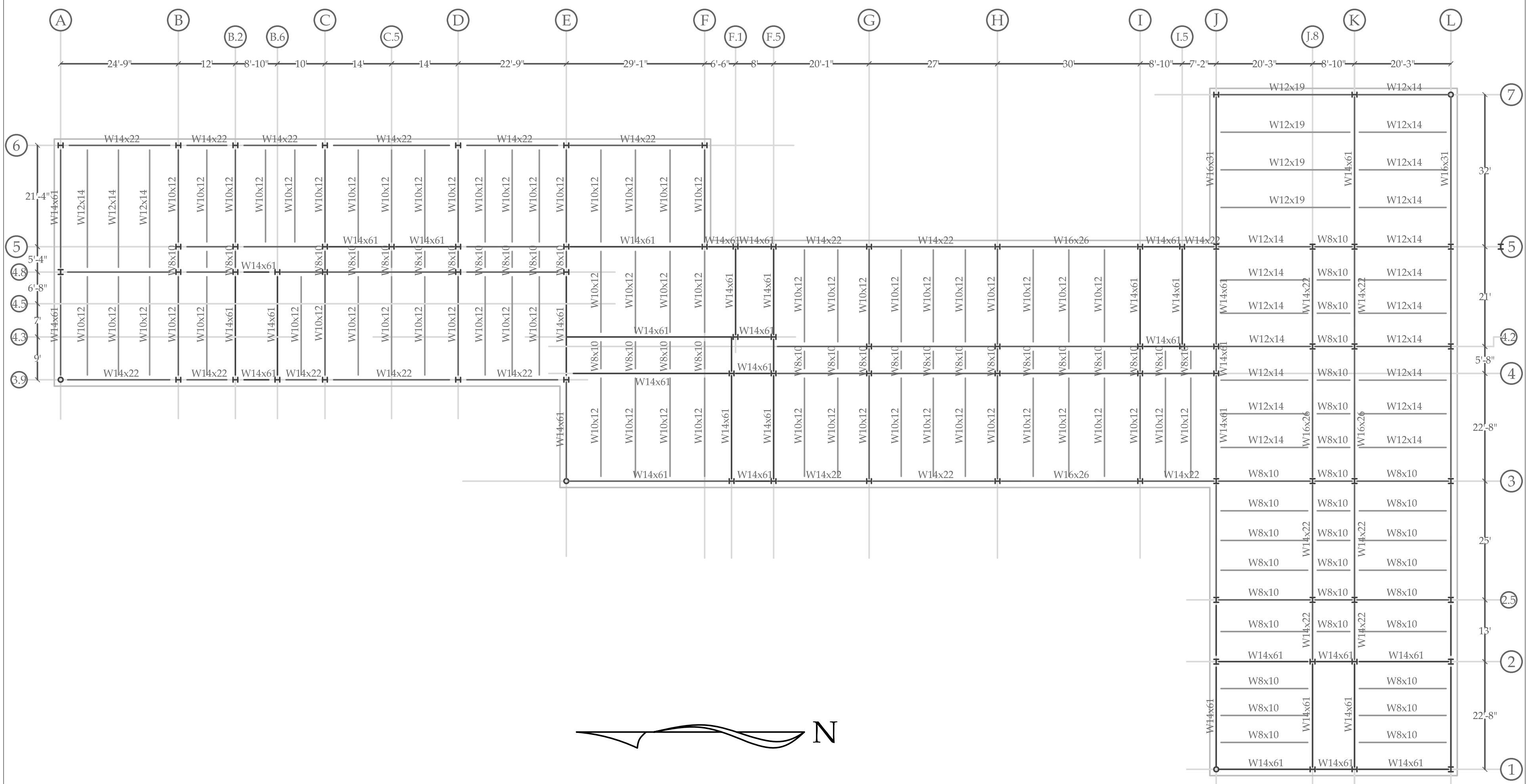
PRECAST CONCRETE FRAMING PLAN OF TYPICAL FLOOR

SCALE: $\frac{1}{32}$" = 1'-0"

S.3



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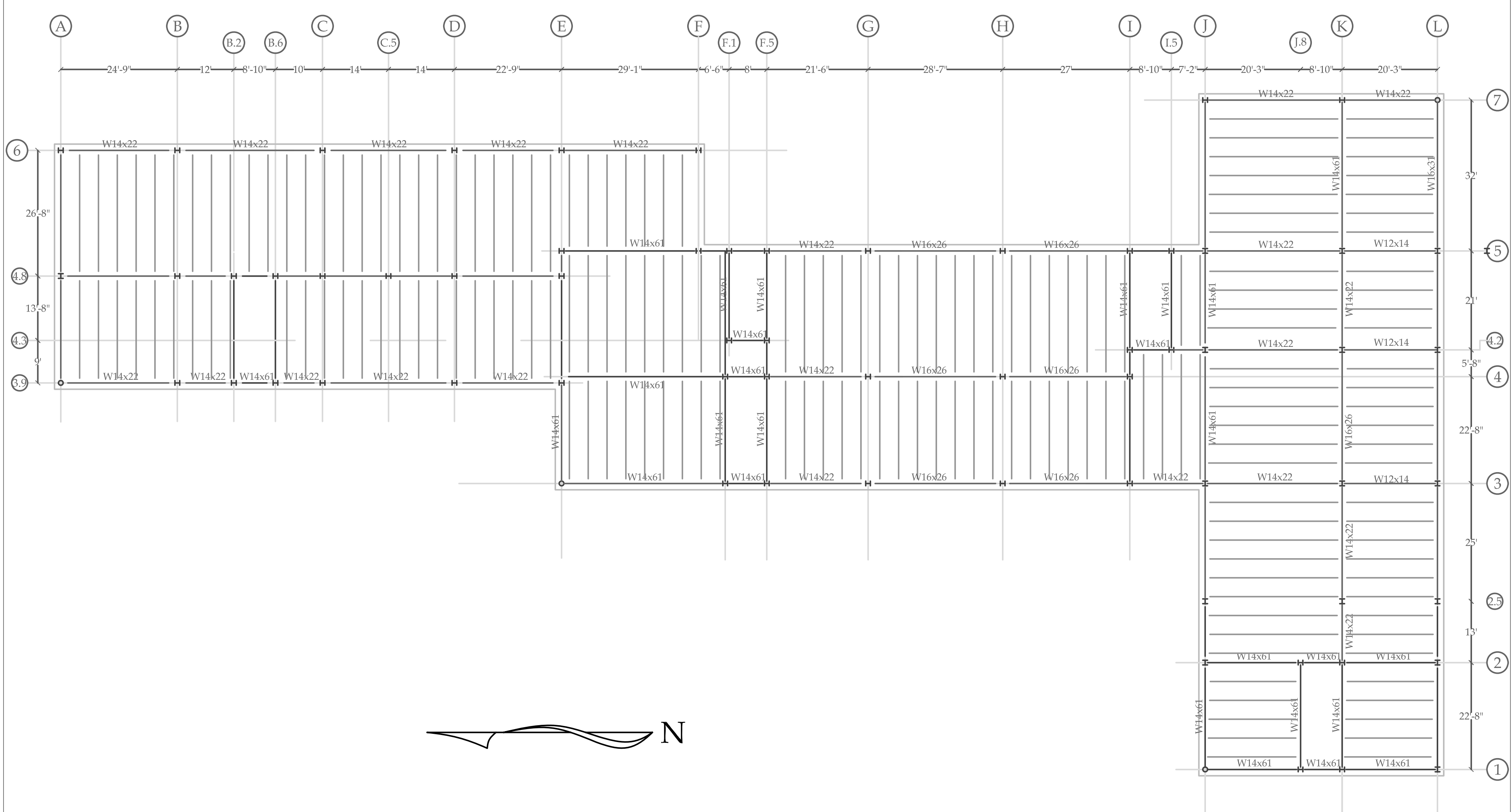
STRUCTURAL STEEL FRAMING PLAN OF TYPICAL FLOOR

SCALE: $\frac{1}{32}$" = 1'-0"

S.4



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STEEL JOIST FRAMING PLAN OF TYPICAL FLOOR

SCALE: $\frac{1}{32}$" = 1'-0"

S.5