Gateway Plaza
Wilmington, DE
October 31, 2005

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Technical Report 2:
Pro-Con Structural Study of Alternate Floor Systems

EXECUTIVE SUMMARY

The goal of this report was to determine if any other floor systems should have been considered in the design of Gateway Plaza. Five different framing schemes were studied and analyzed to determine whether they could have been used.

- Modified existing bays
- Composite steel joists
- One-way concrete slab with beams & girders
- Concrete pan Joists
- Pre-cast, pre-stressed double tees

In order to study these floor systems, a typical bay from a typical office floor was designed according to the loads set forth in Technical Report 1. In order to implement a few of the alternate schemes, alterations had to be made to the existing column layout. Several design aides were referenced to speed the design process. These preliminary designs were further checked in RAM models for accuracy. Designs are discussed at further length in the main body of the report, and design calculations can be found in the appropriate Appendices. The following chart summarizes their designs.

System	Slab	Beams	Girders
Modified existing bays	3.25” LW concrete on 3” composite deck	W14x26 [15]	W33x116 [118]
Composite steel joists	3.25” LW concrete on 2” composite deck	24VC	
One-way concrete slab	6” NW concrete	16x24	16x24
Concrete pan joists	4.5” NW concrete	30” pan, 6”x20” rib	30x24.5
Pre-stressed double tees	2” NW concrete topping	12DT32	24IT32

Finally, a comparison chart lists the advantages and disadvantages of each system to determine whether their implementation should be seriously considered. This chart is based on cost, constructability, local availability, and architectural impact. According to this chart, the only systems which warrant further investigation are the composite steel joist and the pre-cast, pre-stressed double tees.

INTRODUCTION

Codes

The existing conditions include loadings set forth in IBC 2003 and follow typical procedures for finding wind and seismic loads according to ASCE 7-02. For the purpose of this report, a typical office floor was analyzed. These typical office floors can be found on levels 6-15 of the building and are very similar to those on 2-5.

For steel redesigns, provisions set forth in the AISC Specification, latest edition, were followed. For concrete redesigns, design procedures outlined in the CRSI Handbook, 1992 ed., were used. These design simplifications follow codes and standards set forth in the ACI Code.

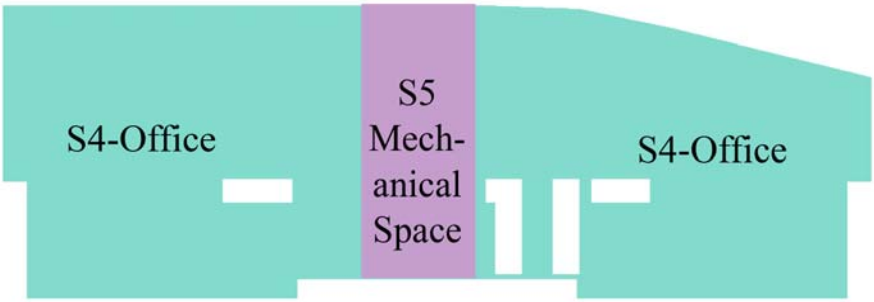
Loads

Loads outlined in the following chart provide a break down of component loads that comprise a typical office floor loading. For the live load, although a 60 psf + 20 psf (partitions) is the minimum required loading condition, a 100 psf live load is designed for due to the fact that these are tenant fit-out spaces. None of the final occupancy is known, at the time being, so a worst case of 100 psf is used in the design to account for the fact that corridors could be placed anywhere.

LOADING IN POUNDS/SQUARE FOOT

	Steel & Joists	Ceiling	Collateral	3-1/4" LW conc. on deck	Total DL	Total LL
Typ. Office Loading	10	5	5	46	66	100

A typical office load was accounted for in all redesigns. Actual loading conditions for the typical office floor are illustrated in the diagram below.



ALTERNATE FRAMING SYSTEMS

Five different framing schemes were explored to determine which option is best suited for Gateway Plaza:

- **Modified existing bays** were chosen because of their anticipated decrease in weight of the current structure, since shorter span will yield lighter beams.
- **Composite steel joists** from Vulcraft were analyzed because of their possibility of decreasing the floor depth.
- **One-way concrete slab with beams & girders** were analyzed to look at capabilities of concrete as a viable framing option, and to determine whether floor depth could be decreased.
- **Concrete Pan Joists** were considered in the redesign as another concrete possibility because of their quick construction, since formwork is pre-manufactured.
- **Pre-cast, Pre-stressed Double Tees** were analyzed because they give a better solution to a concrete system because they are much more quickly erected. More importantly, it is able span 52' without having to add another column line.

In order to aide in the analysis of these different floor systems, a variety of different references and software programs were utilized:

- References:
    - CRSI Handbook, 1992 ed.
    - AISC Design Specification Manual
    - RS Means, 2005 ed. Assemblies Cost Data
    - Nitterhouse Precast concrete literature
    - Vulcraft Composite Steel Joist Manual
- Software:
    - RAM Structural System
    - Concise Beam

To help with steel redesigns, RAM Structural System and composite joist tables from Vulcraft were used. These designs were first done through preliminary hand calculations, and later checked with the RAM Software.

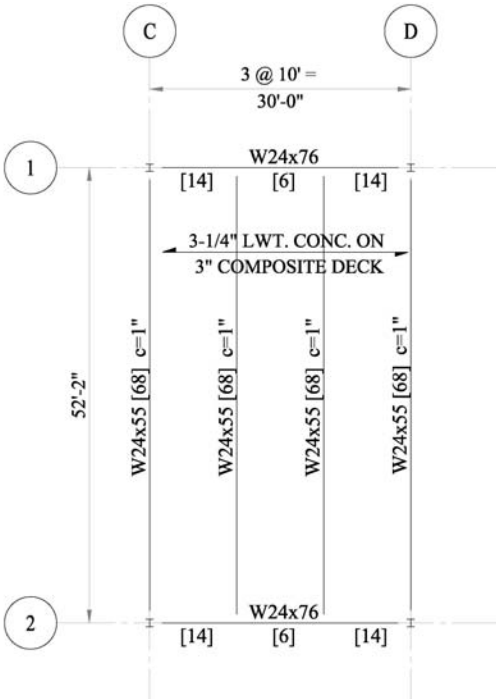
To aide in concrete redesigns, The Concrete Reinforcing Steel Institute (CRSI) Handbook, Nitterhouse Precast Systems product literature, RAM Structural System, and Concise Beam Software were consulted. The designs for the one-way systems were first determined through consultation of the CRSI Handbook, and further design was analyzed in RAM. The pre-cast system was initially chosen from the Nitterhouse literature, but was further designed using Concise Beam software.

When checking the accuracy of the designs provided by these references, models of a typical floor subjected to an office live loading of 100 psf were constructed. RAM output was also used in order to compile a takeoff of material required. In order to compare costs, RS Means 2005, Assemblies Cost Data was referenced. A cost analysis was performed and can be found in *Appendix F-Cost Analysis.*

EXISTING FRAMING SYSTEM

The existing framing plan utilizes 30’x52’ and 30’x36’ bays using composite steel members to support a 3” metal deck and 3-1/4” lightweight concrete slab. Beams span in the long direction in both bays and are supported by 30’-long girders on either side. Currently, the structural floor sandwich depth is 30.25”, and the floor to floor height is set at 13’-6” with little restriction on floor depth. The current weight of a typical floor is approximately 182,000#.

A typical bay is illustrated to the right. Refer to *Appendix F: Framing Plans* for the existing framing plan.



Advantages

Open Floor Plan: The oversized bays will allow the architect to work with a much more open floor plan because it is nearly a column free space. This layout is not stringent; the office floors are tenant fit-out spaces and have not yet been designed.

Floor Depth: All of the gravity members in this scheme are 24” deep and yield a total floor depth of 30.25” (with deck and slab). This is a relatively shallow depth and is looked favorably upon by the architect when using a glazed curtain wall façade.

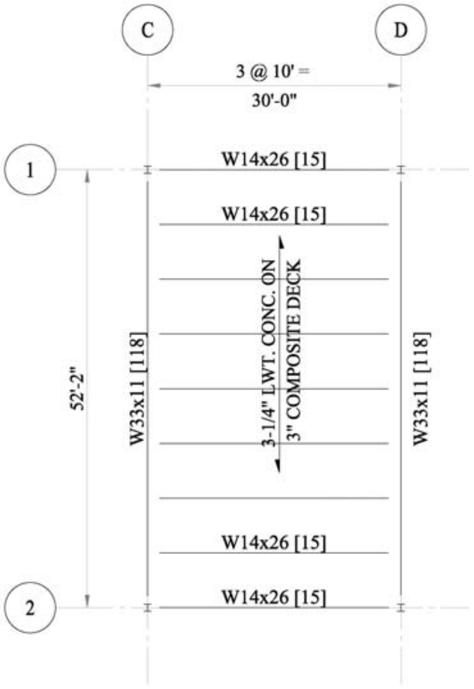
Overall Weight: A steel structure keeps the overall building weight to a relative minimum compared to concrete construction. The weight of the building greatly impacts the foundation size, which is critical in this case due to the moderately poor soil on site. Additionally, the weight has a large impact on lateral system design if seismic conditions were to govern.

Disadvantages

Fabrication: The cost to fabricate and ship 52’ long beams that are 2900# is great compared to a shorter span system. Additionally, the small amount of camber needed for these beams to work sufficiently is difficult to control in a fabrication shop.

Cost: Composite steel construction is the most expensive, at $20.05/square foot, it cost s approximately $438,854 per floor. See *Appendix F-Cost Analysis* for further comparison. Although the volatility in the steel market has settled, somewhat, construction of composite steel framing is a labor intensive process both before and during construction.

Alternate 1: Modified Existing Bays



The first alternate framing system will utilize the existing column grids and bay layout. However, the beams in the bays will span in the short, 30', direction rather than the long, 52', direction and be spaced approximately 9' o.c. The slab will consist of the existing 3-1/4" lightweight concrete on 3" Lok-Floor deck.

Details of the redesign can be found in *Appendix A: Alternate 1: Modified Existing Bays*. This redesign called for W14x26 beams with W33x116 girders, typically. This yielded a weight of 161,000# per typical floor.

Advantages

Overall Weight: The main advantage to this framing layout is overall weight. There is a 21,000# savings per floor over the existing framing system, resulting in a total savings 315,000# on the entire building. This weight will have a significant impact on the foundation design

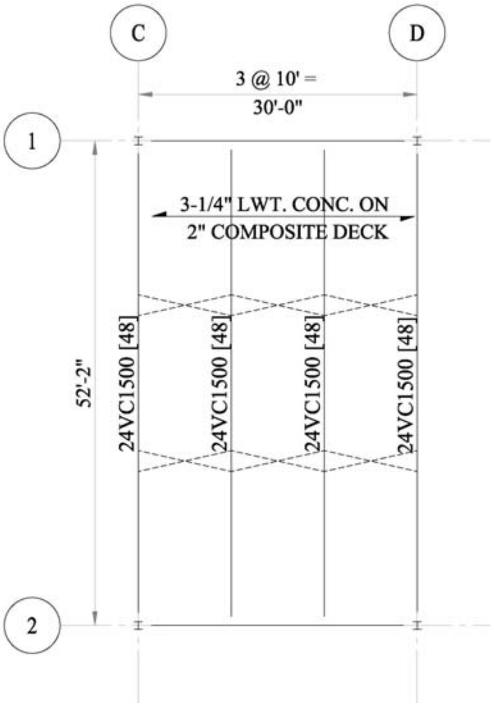
Disadvantages

Floor Depth: In this alternate scheme, the average girder size is a W33x116. This decreases the floor depth by 9". This huge loss in ceiling height is likely to anger an architect.

Connections: The number of connections needed to achieve this framing layout has increased by nearly three times. The additional shear connections will increase costs in the aspects of design, material, erection, and fabrication.

Cost: Similarly to the existing system, composite steel construction is the most expensive type of construction. See *Appendix F-Cost Analysis* for further details.

Alternate 2: Composite Steel Joists

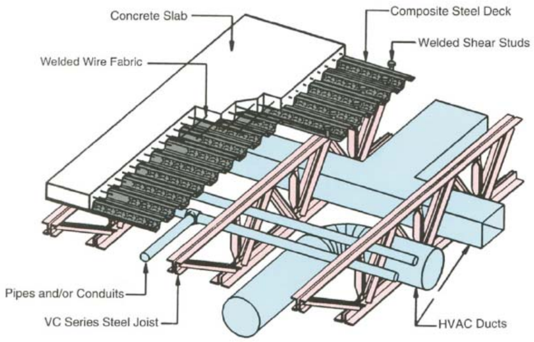


The layout of the composite steel joist framing system is identical to that of the existing framing plan. The 52’-2”x30’ bays were kept, but an altered slab will be used: 3-1/4” lightweight concrete on 2” VLI deck.

The design details are available in *Appendix B-Alternate 2: Composite Steel Joists*. The re-design resulted in 52’-long 24VC1500/714/300 joists with (48) 3/4"ϕ shear studs.

Advantages

Floor depths: Composite steel joists, manufactured by Vulcraft, will greatly decrease floor-floor heights in two ways. First, the floor deck is shallower by 1”. Second, the open-web joists allow MEP ducts and conduit to be routed through the openings in the joist.



Overall Weight: The composite joists weigh 44 pounds per foot compared to the 55 plf of the existing wide-flange beams. Similar to the first alternative, the weight savings will have a significant impact on the foundation and lateral designs. Also, weight limitations restrict the amount of beams that can be carried by one truck load. Lighter joists correlate to a shorter quicker transportation time because more can fit on a truck bed in a single load.

Disadvantages

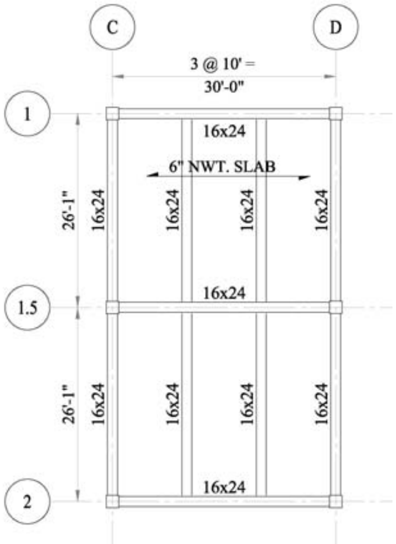
Erection time: Since joists are connected to their supporting members through tack-welds and tack welds take longer to complete than a bolted connection, erection takes longer.

Cost: According to cost data available in RS Means, 2005 ed., composite steel joists are one of the most expensive floor systems available. At $18.55 per square foot, a typical office floor in Gateway Plaza would cost around $406,000 to construct. This is to be expected because they are steel, however they do offer a cheaper solution to traditional wide flange, composite construction.

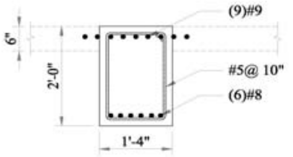
Alternate 3: One-way Concrete Slab

This framing system utilizes a one-way concrete slab with beams and girders. Since cast-in-place concrete often cannot span much more than 35’ without being subjected to great deflections, column line 1.5 was added to give added support to girders in the 52’ direction. The framing layout will use beams to span 26’-1” and 36’ and girders to span 30’-0”.

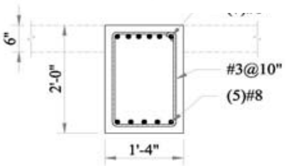
The design calculations can be found in *Appendix C-Alternate 3: One-way Concrete Slab*. The redesign yielded an overall building weight of 1,370,000# per floor including rebar. Design for a 30’x36’ bay using beams spaced at 10’ o.c. yielded the following element design:



Element	Dimension	Flexural Reinforcement	Transverse Reinforcement
Slab	6” thk.	Top: #4 @12” Bottom: #4 @ 18”	
Beams	16”x24” (incl. slab)	Top: (7) #8 Bottom: (5) #8	(17) #3 @ 10”
Girders	16”x24” (incl. slab)	Top: (9) #9 Bottom: (6)8	(16) #5 @ 10”



GIRDER



BEAM

Advantages

Floor Depth: The beams and girders of this system are 24” deep, including the thickness of the slab. Therefore, the floor depth is 6.25” shallower than the original steel design. If floor to floor were critical, this could offer a significant advantage over the original design.

Disadvantages

Additional Column Line: Since cast-in-place concrete is unable to span the 52’ necessary, another column line was added to break the 52’ dimension into 26’.

Overall Weight: The additional weight of a concrete system is 7.5 times the original steel design. The foundation redesign will have a significant impact on the cost of the building.

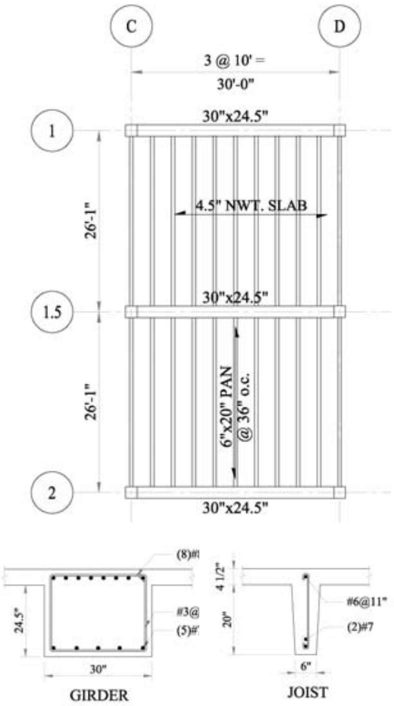
Lateral System: Shearwalls cannot be placed in the current locations of lateral resisting elements because of architectural necessity. Consequently, a redesign could have major cost implications on the structure.

Alternate 4: Concrete Pan Joists

This framing system utilizes the same column, girder, and beam layout of Alternate 3, the one-way concrete slab system. Again, since concrete is unable to efficiently span 52’, an additional column line was implemented.

Design calculations are in *Appendix D-Alternate 4: Concrete Pan Joists*. The resulting members are summarized below. The weight of this type of structure would be approximately 2,000,000# per floor including rebar.

Element	Dimensions	Flexural Reinforcement	Transverse Reinforcement
Slab	4.5” thk.		
Joists	6”x20” deep rib 30” form	Top: #6 @ 11” Bottom: (2) #7	N/A
Girders	30”x24.5”	Top: (5) #7 Bottom: (8) #8	#3 Closed stirrups @ 6”



Advantages

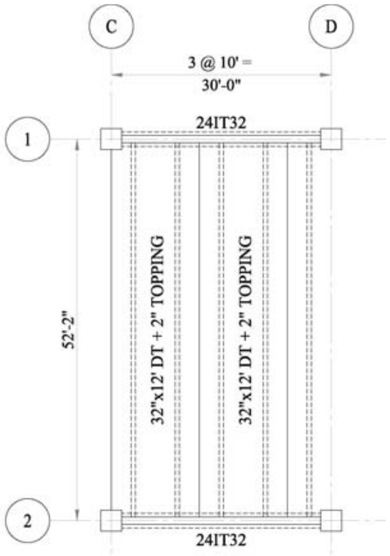
Floor Depth: The major advantage to the pan joist layout is the shallow floor sandwich. The depth of the joists and girders is 24.5”, including the slab thickness. This is a 6” advantage of the current design.

Constructibility: Pre-manufactured formwork that is much easier to assemble makes the concrete pan-joist system much easier to construct. Additionally, there is much less rebar to tie because of the layout of flexural and type of transverse reinforcement.

Disadvantages

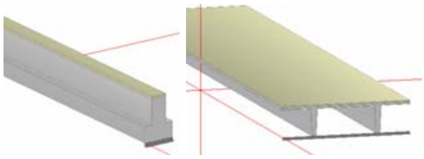
Refer to the Disadvantages listed for the One-way Concrete Slab system.

Alternate 5: Pre-cast, pre-stressed concrete double-tees



The pre-cast system is designed using the existing column layout. Pre-cast, pre-stressed double-tee members with a 2" concrete topping span 52' and are supported by inverted tee members. Although they are not designed in this exercise, it is assumed that cast-in-place concrete columns will transmit loads to the foundation and that pre-cast shear walls will provide lateral stability for the system. See *Appendix E-Alternate 5: Pre-cast, pre-stressed concrete double-tees* for design procedures.

The design requires 12' wide by 32" deep double-tee beams and 24" wide by 32" deep inverted tee girders. This system will weigh approximately 1,815,000# per floor.



Advantages

Span: Pre-cast is well known for its ability to span long directions with little deflection. This system is able to span 52' without the additional column line of the previously designed concrete framing systems.

Erection time: Since a majority of the labor is performed off-site, erection time is cut in half because there is no need for forming or curing of concrete or placing and tying rebar.

Cost: Pre-cast concrete offers the cheapest alternative solution, at $206,000/floor. See *Appendix F-Cost Analysis* for further details.

Disadvantages

Availability: Some locations are unable to consider pre-cast as an alternative solution because a manufacturer may not be nearby. The building material of choice in Wilmington, DE is steel because of the reputable fabrication shops and erectors in the area.

Overall Weight: The floor weight of this system is average compared to the other concrete systems, weighing 1,815,000#. The additional weight will require that the foundation be redesigned.

Connections: As with the composite steel joists, the tee-beams are connected to girders through overhead spot welds. On-site welding should be avoided where possible because it is costly and difficult to perform.

Floor Depth: The floor depth of this system is 32" overall, and is a 1.75" increase from the original system. Though it is not an issue, floor depth could be restricted in some cases.

COMPARISON

Framing System	Advantages	Disadvantages	Cost/ft^2	Further Investigation
Existing Framing	• Open floor plan • Lightweight	• Cost	$20.05	–
Alternate 1: Short span beams	• Open floor plan • Lightweight • 30.25” Floor depth	• Increased floor depth • Number of connections • Cost	$20.05	No
Alternate 2: Composite Steel Joists	• MEP can be routed through joists • Very light weight	• Long erection time • Local availability	$18.55	**Yes**
Alternate 3: One-way Concrete Slab	• 24” floor depth	• cannot span required bay • heavy, requires foundation redesign • requires formwork to be made on site	$17.95	No
Alternate 4: Concrete Pan Joists	• 24.5” floor depth • pre-manufactured formwork	• cannot span required bay • heavy, requires foundation redesign	$15.75	No
Alternate 5: Pre-cast, Pre-stressed concrete	• able to span 52’ • quick erection time (compared to other concrete systems) • cheapest alternative	• Local availability • Heavy, requires foundation redesign • On-site welded connections	$9.44	**Yes**

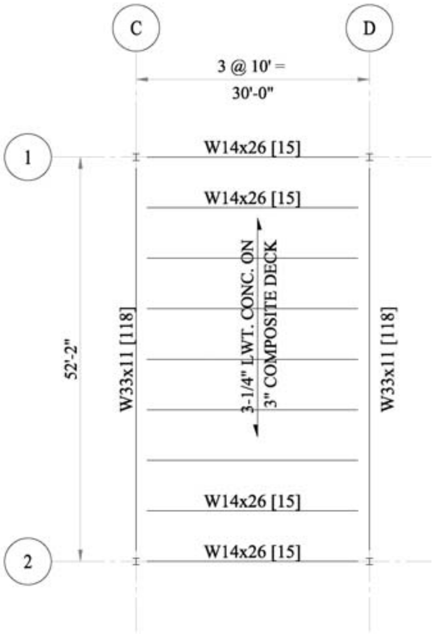
CONCLUSIONS

According to the designs and analyses performed on each of the previously mentioned systems, only two seem to be viable alternatives that should be further researched. Based on the impact of both overall weight and the additional column line added to the existing layout, both cast-in-place concrete alternatives should not be further considered in this building’s design. Since the modified framing layout of shorter spanning beams has such an impact on floor depth and the increased number of connections, it too was ruled out of further consideration. The ability of both composite joists and pre-cast concrete’s ability to span the 52’ of the existing column grid, give them the best advantages over the other systems. Additionally, they provide a more cost-effective alternative to the existing condition. Although the two solutions are feasible alternatives to the existing framing system, it is my belief that the existing system is still the one best suited for this building’s size, architectural needs, and location.

APPENDICES

Appendix A – Alternative 1: Modified Framing

Beam Check



Given

$f_y = 50ksi$

$f'_c = 3000ksi, NWT concrete$

$q_n = 17.7k$

$l_n = 30'$

$s = 9'$

Factored Load: $w_u = 1.2(66) + 1.6(100) = 239.2\,psf$

Factored Moment: $M_u = \dfrac{239.2\,psf\,(9')(30'\text{–}0")^2}{8} = 242'k$

Assuming: $a = 1"$

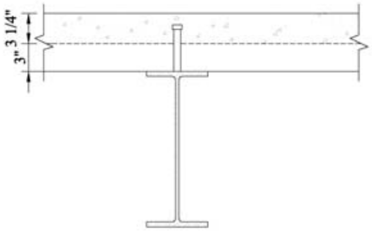
$Y2 = 6.25" - .5" = 5.75"$

Use 5.5”

Trial Sizes:

Size	ΦM_p	ΦM_{pc}	ΣQ_n	# Studs	Weight (k)
W12x22	110	251	281	32	980
W12x26	140	265	259	30	1080
W12x30	162	242	153	18	1080
W14x22	125	251	241	28	940
W14x26	151	254	135	15	930

Try W 14x26



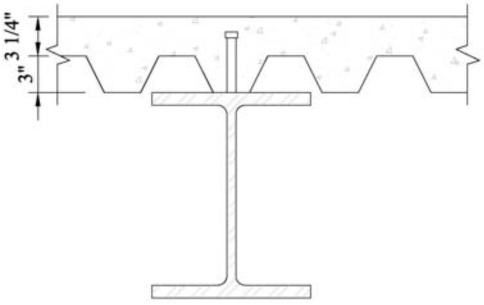
$$b_{eff} = \min\begin{cases} \dfrac{l}{4} = \dfrac{30'\text{–}0"}{4} = 90" \\ s = 9' = 108" \end{cases} = 90"$$

$$a = \frac{135k}{.85(3)(90")} = 0.588" < 1"$$

$\Phi M_n > M_u = 242'k$ ✓ W14x26 [15] okay for prescribed loading

Girder Check

$$b_{eff} = \min\begin{cases} \frac{l}{4} = \frac{52'-2''}{4} = 156.5'' \\ s = 30' = 360'' \end{cases} = 156.5''$$



Factored Load: $P_u = [1.2(66)+1.6(100)](9')(30') = 64.58k$

Resolve point loads into a distributed load:

$$\omega_u = \frac{5(64.58k)}{52'-2''} = 6.19klf$$

Factored Moment: $M_u = \frac{6.19klf\,(52'-2'')^2}{8} = 2106'k$

Assuming: $a = 1''$

$Y2 = 6.25''-.5'' = 5.75''$

Use 5.5”

Trial Sizes:

Size	ΦM_p	ΦM_{pc}	ΣQ_n	# Studs	Weight (k)
W30x108	1300	2130	1190	136	6994
W30x116	1420	2210	1040	118	7231
W33x130	1750	2260	479	54	7322
W33x141	1930	2480	520	58	7935

Try W30x108

$$a = \frac{1190k}{.85(3)(156.5'')} = 2.98'' > 1''$$

$$Y2 = 6.25'' - \frac{2.98''}{2} = 4.76''$$

$\Phi M_n(Y2 = 4.76) = 2070'k$

$\Phi M_n < M_u = 2021'k$

Try again…

Try W30x116

$$a = \frac{1040k}{.85(3)(156.5'')} = 2.61'' > 1''$$

$$Y2 = 6.25'' - \frac{2.61''}{2} = 4.95''$$

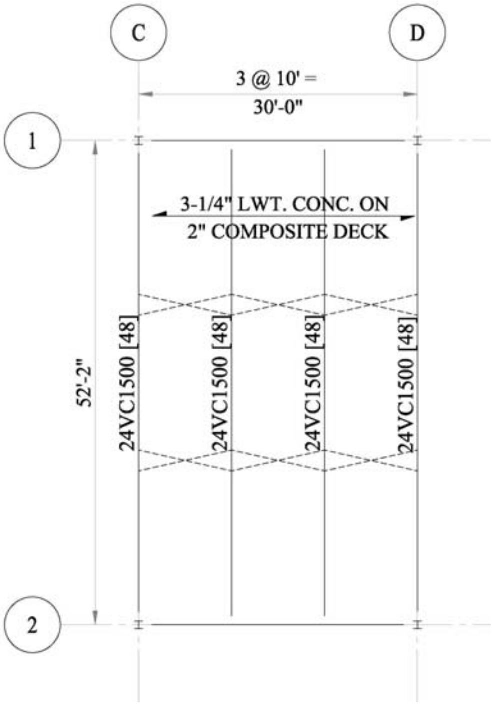
$\Phi M_n(Y2 = 4.95) = 2165'k$

$\Phi M_n > M_u = 2021'k$

✓ W33x116 [118] okay for prescribed loading.

Appendix B – Alternative 2: Composite Steel Joists

Joist Design



Given:

- LWT, $f'_c = 3$ ksi
- $T_c = 3.25''$ above 2” rib
- Spacing = 10’

Design:

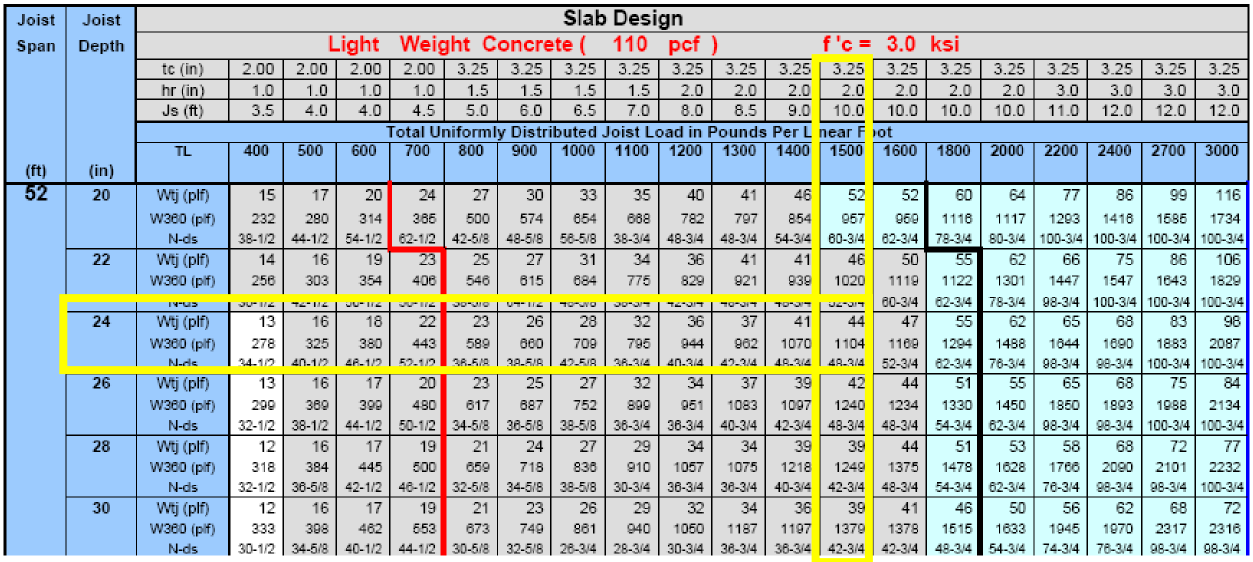
- Design Loads:
    - Non-composite Dead Load:

Concrete	39 psf
Joists	45 plf
Deck	2.4 psf
Bridging	0.1 psf
Total	46 psf

    - Construction Live Load: 8 psf
    - Composite Dead Load:

Partitions	20 psf
MEP	5 psf
Ceiling/flooing	5 psf
Total	30 psf

    - Composite Live Load:
        - Design Live Load 100 psf
        - Reduction: $R=\left(0.25+\frac{15}{\sqrt{2(52'-2'')(10')}}\right)=.714>0.4$
        - Reduced Live Load 71.4 psf
    - Total Non-Composite & Composite Loads: 147.4 psf = 1500 plf



Joist Span	Joist Depth	Slab Design																			
		Light Weight Concrete (110 pcf)												f 'c = 3.0 ksi							
		tc (in)	2.00	2.00	2.00	2.00	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25	3.25
		hr (in)	1.0	1.0	1.0	1.0	1.5	1.5	1.5	1.5	2.0	2.0	2.0	2.0	2.0	2.0	2.0	3.0	3.0	3.0	3.0
		Js (ft)	3.5	4.0	4.0	4.5	5.0	6.0	6.5	7.0	8.0	8.5	9.0	10.0	10.0	10.0	10.0	11.0	12.0	12.0	12.0
		Total Uniformly Distributed Joist Load in Pounds Per Linear Foot																			
(ft)	(in)	TL	400	500	600	700	800	900	1000	1100	1200	1300	1400	1500	1600	1800	2000	2200	2400	2700	3000
52	20	Wtj (plf)	15	17	20	24	27	30	33	35	40	41	46	52	52	60	64	77	86	99	116
		W360 (plf)	232	280	314	365	500	574	654	668	782	797	854	957	959	1116	1117	1293	1416	1585	1734
		N-ds	38-1/2	44-1/2	54-1/2	62-1/2	42-5/8	48-5/8	56-5/8	38-3/4	48-3/4	48-3/4	54-3/4	60-3/4	62-3/4	78-3/4	80-3/4	100-3/4	100-3/4	100-3/4	100-3/4
	22	Wtj (plf)	14	16	19	23	25	27	31	34	36	41	41	46	50	55	62	66	75	86	106
		W360 (plf)	256	303	354	406	546	615	684	775	829	921	939	1020	1119	1122	1301	1447	1547	1643	1829
		N-ds	36-1/2	42-1/2	50-1/2	56-1/2	38-5/8	64-1/2	48-5/8	38-3/4	42-3/4	48-3/4	48-3/4	52-3/4	60-3/4	62-3/4	78-3/4	98-3/4	100-3/4	100-3/4	100-3/4
	24	Wtj (plf)	13	16	18	22	23	26	28	32	36	37	41	44	47	55	62	65	68	83	98
		W360 (plf)	278	325	380	443	589	660	709	795	944	962	1070	1104	1169	1294	1488	1644	1690	1883	2087
		N-ds	34-1/2	40-1/2	46-1/2	52-1/2	36-5/8	38-5/8	42-5/8	36-3/4	40-3/4	42-3/4	48-3/4	48-3/4	52-3/4	62-3/4	76-3/4	98-3/4	98-3/4	100-3/4	100-3/4
	26	Wtj (plf)	13	16	17	20	23	25	27	32	34	37	39	42	44	51	55	65	68	75	84
		W360 (plf)	299	369	399	480	617	687	752	899	951	1083	1097	1240	1234	1330	1450	1850	1893	1988	2134
		N-ds	32-1/2	38-1/2	44-1/2	50-1/2	34-5/8	36-5/8	38-5/8	36-3/4	36-3/4	40-3/4	42-3/4	48-3/4	48-3/4	54-3/4	62-3/4	98-3/4	98-3/4	100-3/4	100-3/4
	28	Wtj (plf)	12	16	17	19	21	24	27	29	34	34	39	39	44	51	53	58	68	72	77
		W360 (plf)	318	384	445	500	659	718	836	910	1057	1075	1218	1249	1375	1478	1628	1766	2090	2101	2232
		N-ds	32-1/2	36-5/8	42-1/2	46-1/2	32-5/8	34-5/8	38-5/8	30-3/4	36-3/4	36-3/4	40-3/4	42-3/4	48-3/4	54-3/4	62-3/4	76-3/4	98-3/4	98-3/4	100-3/4
	30	Wtj (plf)	12	16	17	19	21	23	26	29	32	34	36	39	41	46	50	56	62	68	72
		W360 (plf)	333	398	462	553	673	749	861	940	1050	1187	1197	1379	1378	1515	1633	1945	1970	2317	2316
		N-ds	30-1/2	34-5/8	40-1/2	44-1/2	30-5/8	32-5/8	26-3/4	28-3/4	30-3/4	36-3/4	36-3/4	42-3/4	42-3/4	48-3/4	54-3/4	74-3/4	76-3/4	98-3/4	98-3/4

- Trial Size:
    - Joist Depth 24”
    - W_{tj} 44 plf
    - W_{360} 1104 plf
    - N-d_s (48) ¾”ϕ

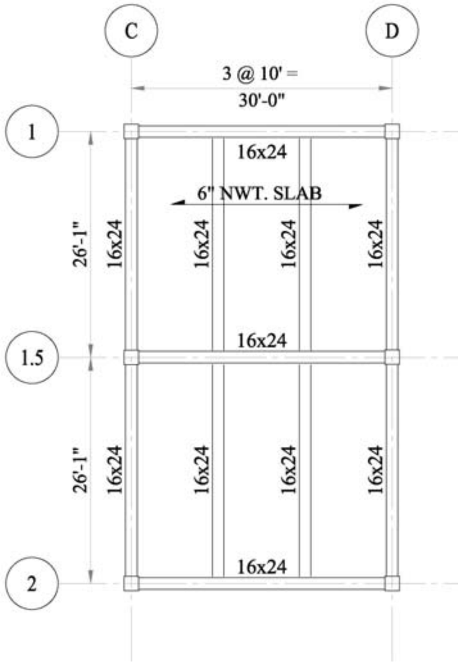
- Deflection Calculations
    - $I_{ESTIMATEDNON-COMPOSITE} = 0.0488(W_{TJ})d_j^{\,2} = 0.0488(44)(24)^2 = 1237in^4$
    - $\Delta_{ESTIMATEDNON-COMPOSITEDL} = \dfrac{1.15(5)W_{NON-COMP.DL}(SPAN_{theoretical})^4 1728}{384E_{steel}I_{EST.NON-COMP}}$

      $= \dfrac{1.15(5)(0.46klf)(51'-10")^4 1728}{384(29{,}000ksi)(1237)} = 2.39"$
    - $\Delta_{COMPOSITEDl} = \dfrac{W_{COMP.DL}}{W_{L/360}}\left[\dfrac{L}{360}\right] = \dfrac{300plf}{1104plf}\left[\dfrac{(51'-10")12}{360}\right] = 0.47" = \dfrac{L}{1332}$
    - $\Delta_{COMPOSITE.LL} = \dfrac{W_{COMP.DL}}{W_{L/360}}\left[\dfrac{L}{360}\right] = \dfrac{714plf}{1104plf}\left[\dfrac{(51'-10")12}{360}\right] = 1.12" = \dfrac{L}{560}$
    - $\Delta_{TL} = \Delta_{EST.NON-COMP.DL} + \Delta_{COMP.DL} + \Delta_{COMP.LL} = 2.39"+.47"+1.12" = 3.98" = \dfrac{L}{560}$

- Camber
    - Camber for full $\Delta_{EST.NON-COMP.DL} + 50\%\Delta_{COMP.DL} + 20\%\Delta_{COMP.LL}$
    - $= 2.85" \therefore 2-7/8"$

- Design Summary
    - 24VC 1500/714/300
    - (48)-3/4”ϕ studs
    - Joist Weight = 44 plf
    - 2 rows of bridging
    - Joist bearing depth = 5”

Appendix C – Alternative 3: One-way Concrete Slab System



Given:

- **Loading**:
  - Superimposed Dead Load=10 psf
  - Live Load = 100 psf
  - Spacing = 10’, l_n=36’
  - $\omega_u = 1.4(10)+1.7(100)=184\,psf$
- **Slab Design**: *CRSI Handbook*, Chapter 7
  - 6” thk. (see table)
- **Beam Design**: *CRSI Handbook*, Chapter 12
  - $$w_{slab} = 6''(150\,pcf)(10') = 750\,plf$$ $$w_{beam} = \frac{(18''\times 24'')(150\,pcf)}{144} = 450\,plf$$ $$w_{SDL} = 10\,psf\,(10') = 100\,plf$$ $$w_{LL} = 100\,psf\,(10') = 1000\,plf$$ $$w_u = 1.4(750+450+100)+1.7(1000) = 3.52\,klf$$

Trial Size	Flexural Reinforcement		Transverse Reinforcement
	Bottom	Top	
14x24	(5) #8	(8) #8	(18) #4 @ 10”
16x24	(5) #8	(7) #8	(17) #3 @ 10”

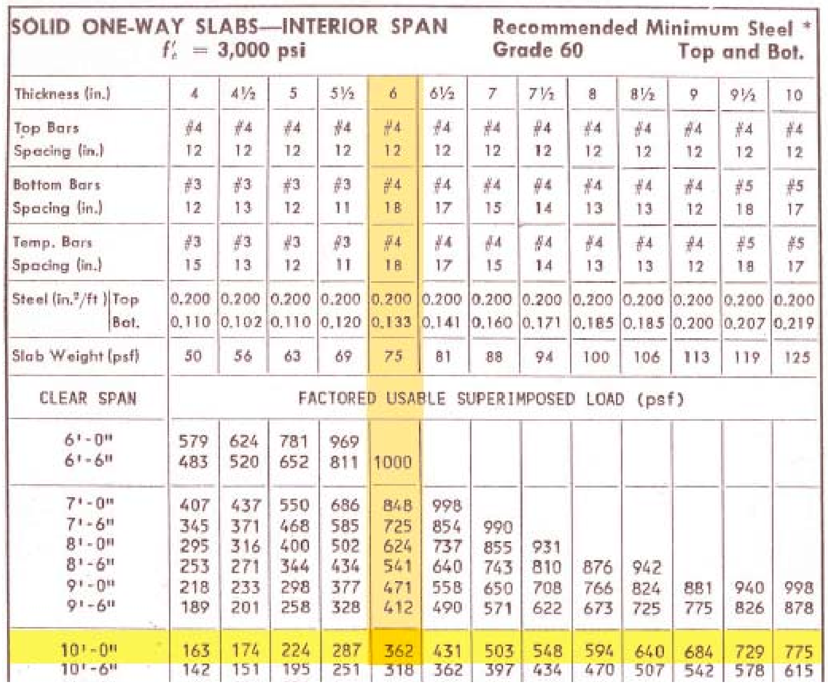
  - Use 16x24 beam

- **Girder Design:**
  - Beam Point Loads: $P_u = 3.52\,klf\,(31') = 109.12k$
  - Estimated Stem Weight: $\frac{(18''\times 28'')}{144}(150\,pcf) = 525\,plf$
  - $w_u = 1.4(525\,plf) = 735\,plf$
  - Concentrated Load Moment: $M = \frac{109.1k(30')}{9} = 363.6'\,k$
  - Equivalent Uniform Load: $w_u = \frac{11(30')}{30^2} = 4.4\,klf$
  - Total Uniform Load: $w_u = 4.4+0.735 = 5.14\,klf$

Trial Size	Flexural Reinforcement		Transverse Reinforcement
	Bottom	Top	
14x24	(5) #8	2 layers: (8) #8	(24) #5 @ 5”
16x24	(6) #8	(9) #9	(16) #5 @ 10”

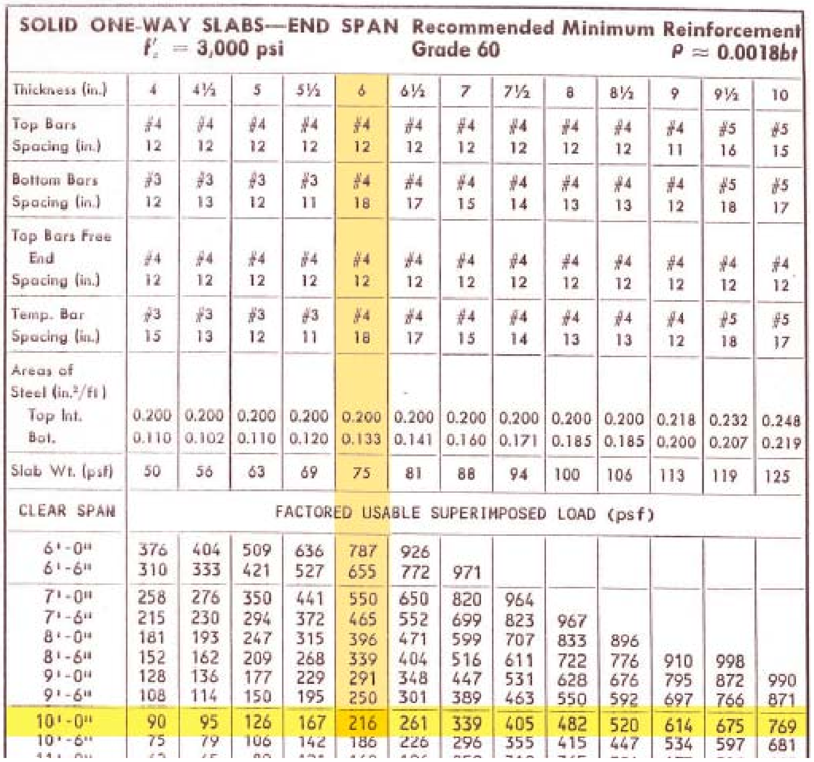
  - Use 16x24 Girder

Slab Tables



SOLID ONE-WAY SLABS—INTERIOR SPAN Recommended Minimum Steel *
f'_c = 3,000 psi Grade 60 Top and Bot.

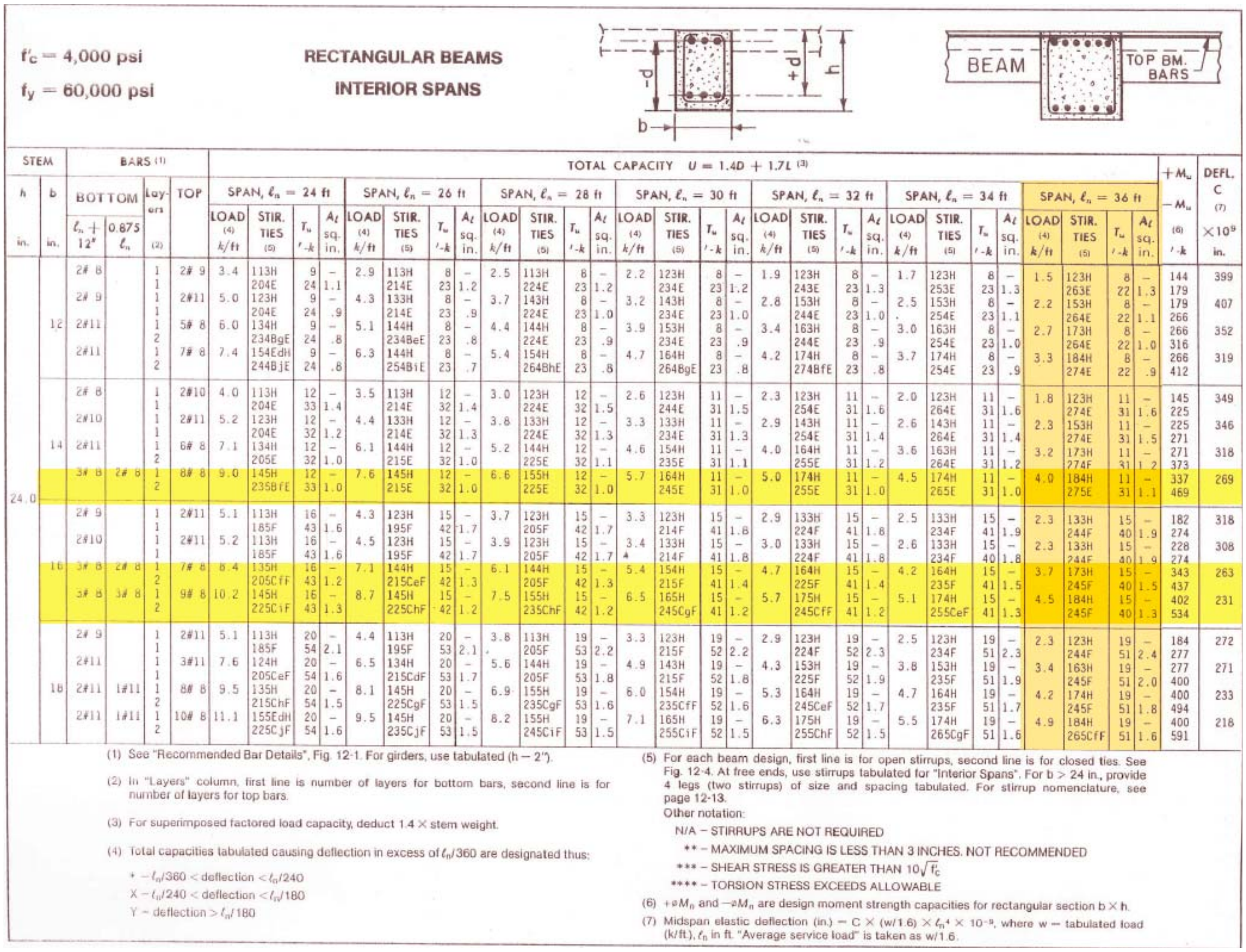
Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars Spacing (in.)	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12
Bottom Bars Spacing (in.)	#3 12	#3 13	#3 12	#3 11	#4 18	#4 17	#4 15	#4 14	#4 13	#4 13	#4 12	#5 18	#5 17
Temp. Bars Spacing (in.)	#3 15	#3 13	#3 12	#3 11	#4 18	#4 17	#4 15	#4 14	#4 13	#4 13	#4 12	#5 18	#5 17
Steel (in.²/ft) Top	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200
Bot.	0.110	0.102	0.110	0.120	0.133	0.141	0.160	0.171	0.185	0.185	0.200	0.207	0.219
Slab Weight (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	FACTORED USABLE SUPERIMPOSED LOAD (psf)												
6'-0"	579	624	781	969									
6'-6"	483	520	652	811	1000								
7'-0"	407	437	550	686	848	998							
7'-6"	345	371	468	585	725	854	990						
8'-0"	295	316	400	502	624	737	855	931					
8'-6"	253	271	344	434	541	640	743	810	876	942			
9'-0"	218	233	298	377	471	558	650	708	766	824	881	940	998
9'-6"	189	201	258	328	412	490	571	622	673	725	775	826	878
10'-0"	163	174	224	287	362	431	503	548	594	640	684	729	775
10'-6"	142	151	195	251	318	362	397	434	470	507	542	578	615



SOLID ONE-WAY SLABS—END SPAN Recommended Minimum Reinforcement
f'_c = 3,000 psi Grade 60 $P \simeq 0.0018bt$

Thickness (in.)	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Top Bars Spacing (in.)	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 11	#5 16	#5 15
Bottom Bars Spacing (in.)	#3 12	#3 13	#3 12	#3 11	#4 18	#4 17	#4 15	#4 14	#4 13	#4 13	#4 12	#5 18	#5 17
Top Bars Free End Spacing (in.)	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12	#4 12
Temp. Bar Spacing (in.)	#3 15	#3 13	#3 12	#3 11	#4 18	#4 17	#4 15	#4 14	#4 13	#4 13	#4 12	#5 18	#5 17
Areas of Steel (in.²/ft) Top Int.	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.200	0.218	0.232	0.248
Bot.	0.110	0.102	0.110	0.120	0.133	0.141	0.160	0.171	0.185	0.185	0.200	0.207	0.219
Slab Wt. (psf)	50	56	63	69	75	81	88	94	100	106	113	119	125
CLEAR SPAN	FACTORED USABLE SUPERIMPOSED LOAD (psf)												
6'-0"	376	404	509	636	787	926							
6'-6"	310	333	421	527	655	772	971						
7'-0"	258	276	350	441	550	650	820	964					
7'-6"	215	230	294	372	465	552	699	823	967				
8'-0"	181	193	247	315	396	471	599	707	833	896			
8'-6"	152	162	209	268	339	404	516	611	722	776	910	998	
9'-0"	128	136	177	229	291	348	447	531	628	676	795	872	990
9'-6"	108	114	150	195	250	301	389	463	550	592	697	766	871
10'-0"	90	95	126	167	216	261	339	405	482	520	614	675	769
10'-6"	75	79	106	142	186	226	296	355	415	447	534	597	681

Beam Table



$f'_c = 4{,}000$ psi

$f_y = 60{,}000$ psi

RECTANGULAR BEAMS

INTERIOR SPANS

STEM		BARS (1)				TOTAL CAPACITY $U = 1.4D + 1.7L$ (3)																												+M_u −M_u (6) ′-k	DEFL. C (7) $\times 10^9$ in.
h in.	b in.	BOTTOM		Layers (2)	TOP	SPAN, ℓ_n = 24 ft				SPAN, ℓ_n = 26 ft				SPAN, ℓ_n = 28 ft				SPAN, ℓ_n = 30 ft				SPAN, ℓ_n = 32 ft				SPAN, ℓ_n = 34 ft				SPAN, ℓ_n = 36 ft					
		ℓ_n + 12"	0.875 ℓ_n			LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.		
24.0	12	2# 8		1 1	2# 9	3.4	113H 204E	9 24	– 1.1	2.9	113H 214E	8 23	– 1.2	2.5	113H 224E	8 23	– 1.2	2.2	123H 234E	8 23	– 1.2	1.9	123H 243E	8 23	– 1.3	1.7	123H 253E	8 23	– 1.3	1.5	123H 263E	8 22	– 1.3	144 179	399
		2# 9		1 1	2#11	5.0	123H 204E	9 24	– .9	4.3	133H 214E	8 23	– .9	3.7	143H 224E	8 23	– 1.0	3.2	143H 234E	8 23	– 1.0	2.8	153H 244E	8 23	– 1.0	2.5	153H 254E	8 23	– 1.1	2.2	153H 264E	8 22	– 1.1	179 266	407
		2#11		1 2	5# 8	6.0	134H 234BgE	9 24	– .8	5.1	144H 234BeE	8 23	– .8	4.4	144H 224E	8 23	– .9	3.9	153H 234E	8 23	– .9	3.4	163H 244E	8 23	– .9	3.0	163H 254E	8 23	– 1.0	2.7	173H 264E	8 22	– 1.0	266 316	352
		2#11		1 2	7# 8	7.4	154EdH 244BjE	9 24	– .8	6.3	144H 254BiE	8 23	– .7	5.4	154H 264BhE	8 23	– .8	4.7	164H 264BgE	8 23	– .8	4.2	174H 274BfE	8 23	– .8	3.7	174H 254E	8 23	– .9	3.3	184H 274E	8 22	– .9	266 412	319
	14	2# 8		1 1	2#10	4.0	113H 204E	12 33	– 1.4	3.5	113H 214E	12 32	– 1.4	3.0	123H 224E	12 32	– 1.5	2.6	123H 244E	11 31	– 1.5	2.3	123H 254E	11 31	– 1.6	2.0	123H 264E	11 31	– 1.6	1.8	123H 274E	11 31	– 1.6	145 225	349
		2#10		1 1	2#11	5.2	123H 204E	12 32	– 1.2	4.4	133H 214E	12 32	– 1.3	3.8	133H 224E	12 32	– 1.3	3.3	133H 234E	11 31	– 1.3	2.9	143H 254E	11 31	– 1.4	2.6	143H 264E	11 31	– 1.4	2.3	153H 274E	11 31	– 1.5	225 271	346
		2#11		1 2	6# 8	7.1	134H 205E	12 32	– 1.0	6.1	144H 215E	12 32	– 1.0	5.2	144H 225E	12 32	– 1.1	4.6	154H 235E	11 31	– 1.1	4.0	164H 255E	11 31	– 1.2	3.6	163H 264E	11 31	– 1.2	3.2	173H 274E	11 31	– 1.2	271 373	318
		3# 8	2# 8	1 2	8# 8	9.0	145H 235BfE	12 33	– 1.0	7.6	145H 215E	12 32	– 1.0	6.6	155H 225E	12 32	– 1.0	5.7	164H 245E	11 31	– 1.0	5.0	174H 255E	11 31	– 1.0	4.5	174H 265E	11 31	– 1.0	4.0	184H 275E	11 31	– 1.1	337 469	269
	16	2# 9		1 1	2#11	5.1	113H 185F	16 43	– 1.6	4.3	123H 195F	15 42	– 1.7	3.7	123H 205F	15 42	– 1.7	3.3	123H 214F	15 41	– 1.8	2.9	133H 224F	15 41	– 1.8	2.5	133H 234F	15 41	– 1.9	2.3	133H 244F	15 40	– 1.9	182 274	318
		2#10		1 1	2#11	5.2	113H 185F	16 43	– 1.6	4.5	123H 195F	15 42	– 1.7	3.9	123H 205F	15 42	– 1.7	3.4 *	133H 214F	15 41	– 1.8	3.0	133H 224F	15 41	– 1.8	2.6	133H 234F	15 40	– 1.8	2.3	133H 244F	15 40	– 1.9	228 274	308
		3# 8	2# 8	1 2	7# 8	8.4	135H 205CfF	16 43	– 1.2	7.1	144H 215CeF	15 42	– 1.3	6.1	144H 205F	15 42	– 1.3	5.4	154H 215F	15 41	– 1.4	4.7	164H 225F	15 41	– 1.4	4.2	164H 235F	15 41	– 1.5	3.7	173H 245F	15 40	– 1.5	343 437	263
		3# 8	3# 8	1 2	9# 8	10.2	145H 225CiF	16 43	– 1.3	8.7	145H 225ChF	15 42	– 1.2	7.5	155H 235ChF	15 42	– 1.2	6.5	165H 245CgF	15 41	– 1.2	5.7	175H 245CfF	15 41	– 1.2	5.1	174H 255CeF	15 41	– 1.3	4.5	184H 245F	15 40	– 1.3	402 534	231
	18	2# 9		1 1	2#11	5.1	113H 185F	20 54	– 2.1	4.4	113H 195F	20 53	– 2.1	3.8	113H 205F	19 53	– 2.2	3.3	123H 215F	19 52	– 2.2	2.9	123H 224F	19 52	– 2.3	2.5	123H 234F	19 51	– 2.3	2.3	123H 244F	19 51	– 2.4	184 277	272
		2#11		1 1	3#11	7.6	124H 205CeF	20 54	– 1.6	6.5	134H 215CdF	20 53	– 1.7	5.6	144H 205F	19 53	– 1.8	4.9	143H 215F	19 52	– 1.8	4.3	153H 225F	19 52	– 1.9	3.8	153H 235F	19 51	– 1.9	3.4	163H 245F	19 51	– 2.0	277 400	271
		2#11	1#11	1 2	8# 8	9.5	135H 215ChF	20 54	– 1.5	8.1	145H 225CgF	20 53	– 1.5	6.9	155H 235CgF	19 53	– 1.6	6.0	154H 235CfF	19 52	– 1.6	5.3	164H 245CeF	19 52	– 1.7	4.7	164H 235F	19 51	– 1.7	4.2	174H 245F	19 51	– 1.8	400 494	233
		2#11	1#11	1 2	10# 8	11.1	155EdH 225CjF	20 54	– 1.6	9.5	145H 235CjF	20 53	– 1.5	8.2	155H 245CiF	19 53	– 1.5	7.1	165H 255CiF	19 52	– 1.5	6.3	175H 255ChF	19 52	– 1.5	5.5	174H 265CgF	19 51	– 1.6	4.9	184H 265CfF	19 51	– 1.6	400 591	218

(1) See "Recommended Bar Details", Fig. 12-1. For girders, use tabulated (h − 2").

(2) In "Layers" column, first line is number of layers for bottom bars, second line is for number of layers for top bars.

(3) For superimposed factored load capacity, deduct 1.4 × stem weight.

(4) Total capacities tabulated causing deflection in excess of $\ell_n/360$ are designated thus:

* − $\ell_n/360 <$ deflection $< \ell_n/240$
X − $\ell_n/240 <$ deflection $< \ell_n/180$
Y − deflection $> \ell_n/180$

(5) For each beam design, first line is for open stirrups, second line is for closed ties. See Fig. 12-4. At free ends, use stirrups tabulated for "Interior Spans". For b > 24 in., provide 4 legs (two stirrups) of size and spacing tabulated. For stirrup nomenclature, see page 12-13.

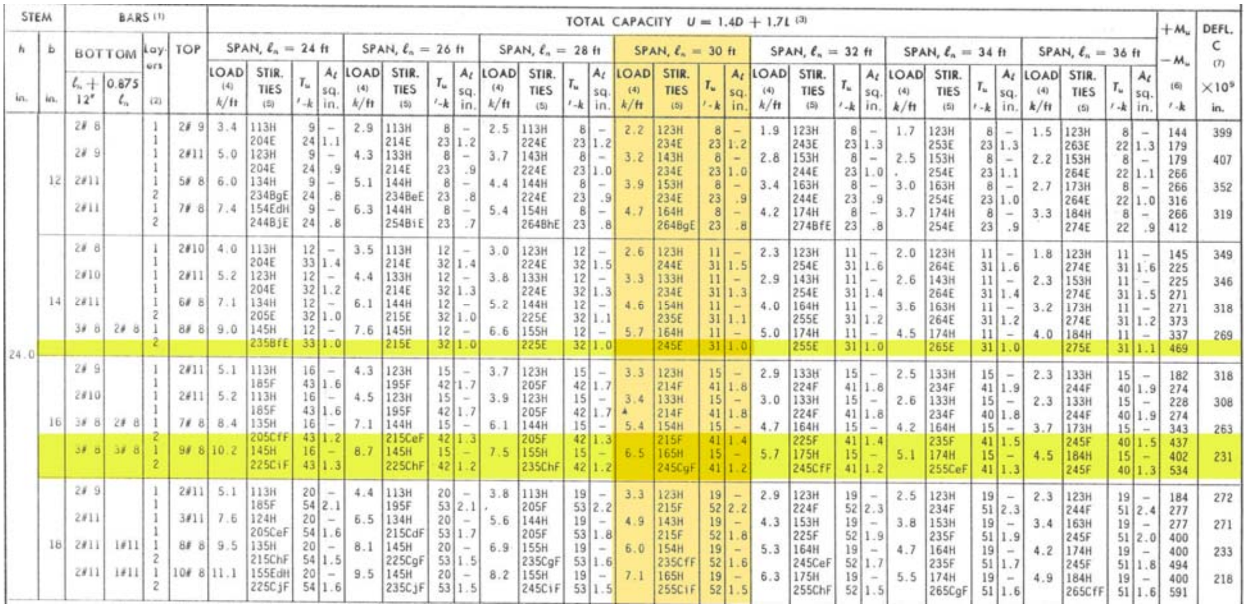
Other notation:

N/A − STIRRUPS ARE NOT REQUIRED
** − MAXIMUM SPACING IS LESS THAN 3 INCHES. NOT RECOMMENDED
*** − SHEAR STRESS IS GREATER THAN $10\sqrt{f'_c}$
**** − TORSION STRESS EXCEEDS ALLOWABLE

(6) $+\phi M_n$ and $-\phi M_n$ are design moment strength capacities for rectangular section b × h.

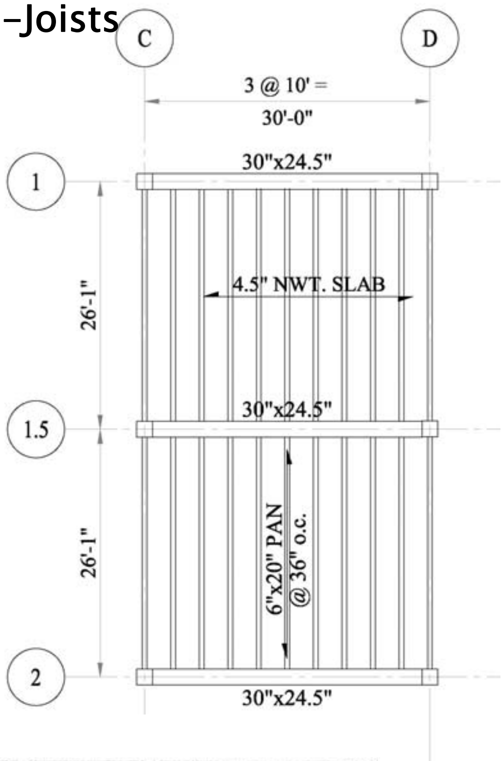
(7) Midspan elastic deflection (in.) = C × (w/1.6) × ℓ_n^4 × 10^{-9}, where w = tabulated load (k/ft.), ℓ_n in ft. "Average service load" is taken as w/1.6.

Girder Table



STEM		BARS (1)				TOTAL CAPACITY $U = 1.4D + 1.7L$ (3)																												$+M_u$ $-M_u$	DEFL. C (7)
h	b	BOTTOM		Layers	TOP	SPAN, ℓ_n = 24 ft				SPAN, ℓ_n = 26 ft				SPAN, ℓ_n = 28 ft				SPAN, ℓ_n = 30 ft				SPAN, ℓ_n = 32 ft				SPAN, ℓ_n = 34 ft				SPAN, ℓ_n = 36 ft					
in.	in.	ℓ_n + 12"	0.875 ℓ_n	(2)		LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	LOAD (4) k/ft	STIR. TIES (5)	T_u ′-k	A_ℓ sq. in.	(6) ′-k	×10⁵ in.
24.0	12	2# 8		1	2# 9	3.4	113H	9	–	2.9	113H	8	–	2.5	113H	8	–	2.2	123H	8	–	1.9	123H	8	–	1.7	123H	8	–	1.5	123H	8	–	144	399
				1			204E	24	1.1		214E	23	1.2		224E	23	1.2		234E	23	1.2		243E	23	1.3		253E	23	1.3		263E	22	1.3	179	
		2# 9		1	2#11	5.0	123H	9	–	4.3	133H	8	–	3.7	143H	8	–	3.2	143H	8	–	2.8	153H	8	–	2.5	153H	8	–	2.2	153H	8	–	179	407
				1			204E	24	.9		214E	23	.9		224E	23	1.0		234E	23	1.0		244E	23	1.0		254E	23	1.1		264E	22	1.1	266	
		2#11		1	5# 8	6.0	134H	9	–	5.1	144H	8	–	4.4	144H	8	–	3.9	153H	8	–	3.4	163H	8	–	3.0	163H	8	–	2.7	173H	8	–	266	352
				2			234BgE	24	.8		234BeE	23	.8		224E	23	.9		234E	23	.9		244E	23	.9		254E	23	1.0		264E	22	1.0	316	
		2#11		1	7# 8	7.4	154EdH	9	–	6.3	144H	8	–	5.4	154H	8	–	4.7	164H	8	–	4.2	174H	8	–	3.7	174H	8	–	3.3	184H	8	–	266	319
				2			244BjE	24	.8		254BiE	23	.7		264BhE	23	.8		264BgE	23	.8		274BfE	23	.8		254E	23	.9		274E	22	.9	412	
	14	2# 8		1	2#10	4.0	113H	12	–	3.5	113H	12	–	3.0	123H	12	–	2.6	123H	11	–	2.3	123H	11	–	2.0	123H	11	–	1.8	123H	11	–	145	349
				1			204E	33	1.4		214E	32	1.4		224E	32	1.5		244E	31	1.5		254E	31	1.6		264E	31	1.6		274E	31	1.6	225	
		2#10		1	2#11	5.2	123H	12	–	4.4	133H	12	–	3.8	133H	12	–	3.3	133H	11	–	2.9	143H	11	–	2.6	143H	11	–	2.3	153H	11	–	225	346
				1			204E	32	1.2		214E	32	1.3		224E	32	1.3		234E	31	1.3		254E	31	1.4		264E	31	1.4		274E	31	1.5	271	
		2#11		1	6# 8	7.1	134H	12	–	6.1	144H	12	–	5.2	144H	12	–	4.6	154H	11	–	4.0	164H	11	–	3.6	163H	11	–	3.2	173H	11	–	271	318
				2			205E	32	1.0		215E	32	1.0		225E	32	1.1		235E	31	1.1		255E	31	1.2		264E	31	1.2		274E	31	1.2	373	
		3# 8	2# 8	1	8# 8	9.0	145H	12	–	7.6	145H	12	–	6.6	155H	12	–	5.7	164H	11	–	5.0	174H	11	–	4.5	174H	11	–	4.0	184H	11	–	337	269
				2			235BfE	33	1.0		215E	32	1.0		225E	32	1.0		245E	31	1.0		255E	31	1.0		265E	31	1.0		275E	31	1.1	469	
	16	2# 9		1	2#11	5.1	113H	16	–	4.3	123H	15	–	3.7	123H	15	–	3.3	123H	15	–	2.9	133H	15	–	2.5	133H	15	–	2.3	133H	15	–	182	318
				1			185F	43	1.6		195F	42	1.7		205F	42	1.7		214F	41	1.8		224F	41	1.8		234F	41	1.9		244F	40	1.9	274	
		2#10		1	2#11	5.2	113H	16	–	4.5	123H	15	–	3.9	123H	15	–	3.4	133H	15	–	3.0	133H	15	–	2.6	133H	15	–	2.3	133H	15	–	228	308
				1			185F	43	1.6		195F	42	1.7		205F	42	1.7		214F	41	1.8		224F	41	1.8		234F	40	1.8		244F	40	1.9	274	
		3# 8	2# 8	1	7# 8	8.4	135H	16	–	7.1	144H	15	–	6.1	144H	15	–	5.4	154H	15	–	4.7	164H	15	–	4.2	164H	15	–	3.7	173H	15	–	343	263
				2			205CfF	43	1.2		215CeF	42	1.3		205F	42	1.3		215F	41	1.4		225F	41	1.4		235F	41	1.5		245F	40	1.5	437	
		3# 8	3# 8	1	9# 8	10.2	145H	16	–	8.7	145H	15	–	7.5	155H	15	–	6.5	165H	15	–	5.7	175H	15	–	5.1	174H	15	–	4.5	184H	15	–	402	231
				2			225CiF	43	1.3		225ChF	42	1.2		235ChF	42	1.2		245CgF	41	1.2		245CfF	41	1.2		255CeF	41	1.3		245F	40	1.3	534	
	18	2# 9		1	2#11	5.1	113H	20	–	4.4	113H	20	–	3.8	113H	19	–	3.3	123H	19	–	2.9	123H	19	–	2.5	123H	19	–	2.3	123H	19	–	184	272
				1			185F	54	2.1		195F	53	2.1		205F	53	2.2		215F	52	2.2		224F	52	2.3		234F	51	2.3		244F	51	2.4	277	
		2#11		1	3#11	7.6	124H	20	–	6.5	134H	20	–	5.6	144H	19	–	4.9	143H	19	–	4.3	153H	19	–	3.8	153H	19	–	3.4	163H	19	–	277	271
				1			205CeF	54	1.6		215CdF	53	1.7		205F	53	1.8		215F	52	1.8		225F	52	1.9		235F	51	1.9		245F	51	2.0	400	
		2#11	1#11	1	8# 8	9.5	135H	20	–	8.1	145H	20	–	6.9	155H	19	–	6.0	154H	19	–	5.3	164H	19	–	4.7	164H	19	–	4.2	174H	19	–	400	233
				2			215ChF	54	1.5		225CgF	53	1.5		235CgF	53	1.6		235CfF	52	1.6		245CeF	52	1.7		235F	51	1.7		245F	51	1.8	494	
		2#11	1#11	1	10# 8	11.1	155EdH	20	–	9.5	145H	20	–	8.2	155H	19	–	7.1	165H	19	–	6.3	175H	19	–	5.5	174H	19	–	4.9	184H	19	–	400	218
				2			225CjF	54	1.6		235CjF	53	1.5		245CiF	53	1.5		255CiF	52	1.5		255ChF	52	1.5		265CgF	51	1.6		265CfF	51	1.6	591	

Appendix D – Alternative 4: Concrete Pan-Joists



CRSI Handbook, Chapter 8

Given:

- $w_u = 184\,psf$ (see previous example)
- $l_n = 36'$

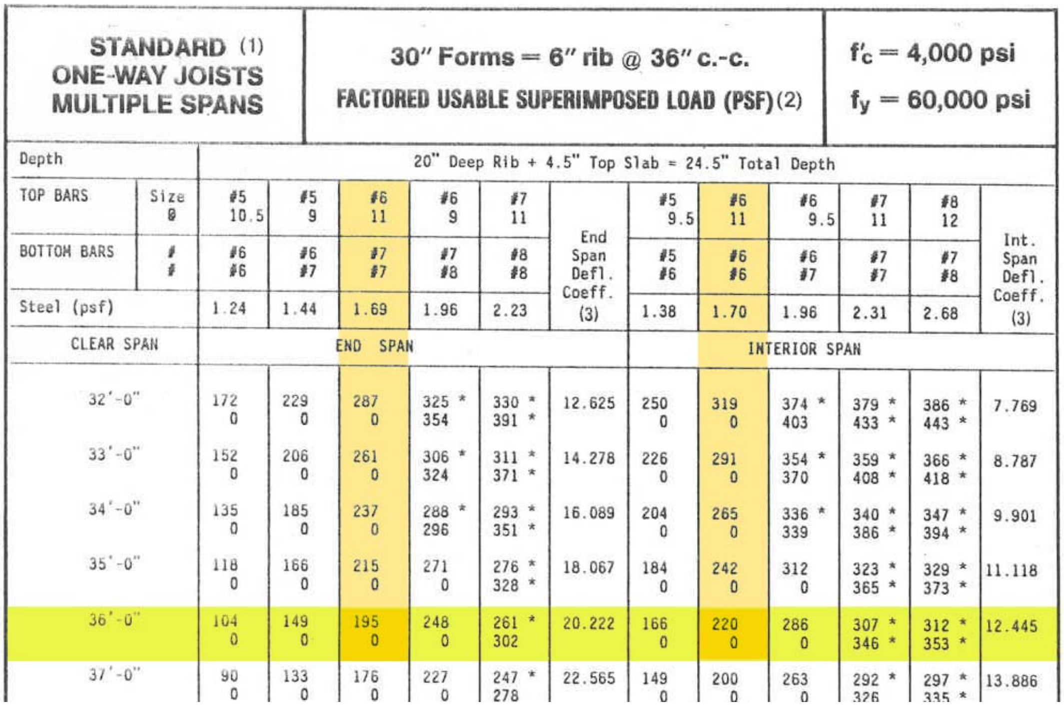
Use 20” deep rib + 4.5” top slab,
30” forms = 6” ribs @ 36” c.-c.

End Span:

- Reinforcing: top-#6 @ 11”, bottom-(2) #7
- $w_n = 195\,psf > 184\,psf$

Interior Span:

- Reinforcing: top-#6 @ 11”, bottom-(6) #6
- $w_n = 220\,psf > 184\,psf$



STANDARD (1) ONE-WAY JOISTS MULTIPLE SPANS

30” Forms = 6” rib @ 36” c.-c.
FACTORED USABLE SUPERIMPOSED LOAD (PSF) (2)

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

Depth		20" Deep Rib + 4.5" Top Slab = 24.5" Total Depth										
TOP BARS	Size @	#5 10.5	#5 9	#6 11	#6 9	#7 11	End Span Defl. Coeff. (3)	#5 9.5	#6 11	#6 9.5	#7 11	#8 12	Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	#6 #6	#6 #7	#7 #7	#7 #8	#8 #8		#5 #6	#6 #6	#6 #7	#7 #7	#7 #8	
Steel (psf)		1.24	1.44	1.69	1.96	2.23		1.38	1.70	1.96	2.31	2.68	
CLEAR SPAN		END SPAN						INTERIOR SPAN					
32'-0"		172 0	229 0	287 0	325 * 354	330 * 391 *	12.625	250 0	319 0	374 * 403	379 * 433 *	386 * 443 *	7.769
33'-0"		152 0	206 0	261 0	306 * 324	311 * 371 *	14.278	226 0	291 0	354 * 370	359 * 408 *	366 * 418 *	8.787
34'-0"		135 0	185 0	237 0	288 * 296	293 * 351 *	16.089	204 0	265 0	336 * 339	340 * 386 *	347 * 394 *	9.901
35'-0"		118 0	166 0	215 0	271 0	276 * 328 *	18.067	184 0	242 0	312 0	323 * 365 *	329 * 373 *	11.118
36'-0"		104 0	149 0	195 0	248 0	261 * 302	20.222	166 0	220 0	286 0	307 * 346 *	312 * 353 *	12.445
37'-0"		90 0	133 0	176 0	227 0	247 * 278	22.565	149 0	200 0	263 0	292 * 326	297 * 335 *	13.886

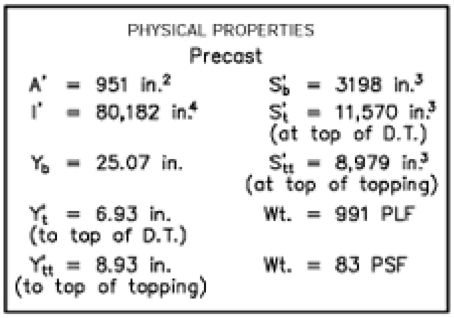
Girder Design: according to RAM output

- Dimension: 30”x24.5” (incl. slab thickness)
- Reinforcing
  - Top: (5) #7
  - Bottom: (8) #8
  - Transverse: #3 Closed stirrups @ 6”

Appendix E – Alternative 5: Pre-cast, Prestressed Concrete System

Beam Design

Input:



PHYSICAL PROPERTIES

Precast

$A' = 951 \text{ in.}^2$

$I' = 80{,}182 \text{ in.}^4$

$Y_b = 25.07$ in.

$Y'_t = 6.93$ in. (to top of D.T.)

$Y'_{tt} = 8.93$ in. (to top of topping)

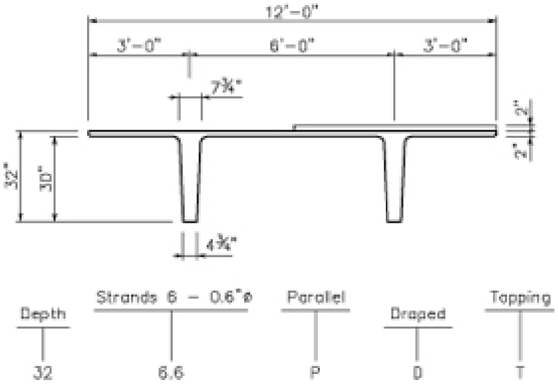
$S'_b = 3198 \text{ in.}^3$

$S'_t = 11{,}570 \text{ in.}^3$ (at top of D.T.)

$S'_{tt} = 8{,}979 \text{ in.}^3$ (at top of topping)

Wt. = 991 PLF

Wt. = 83 PSF



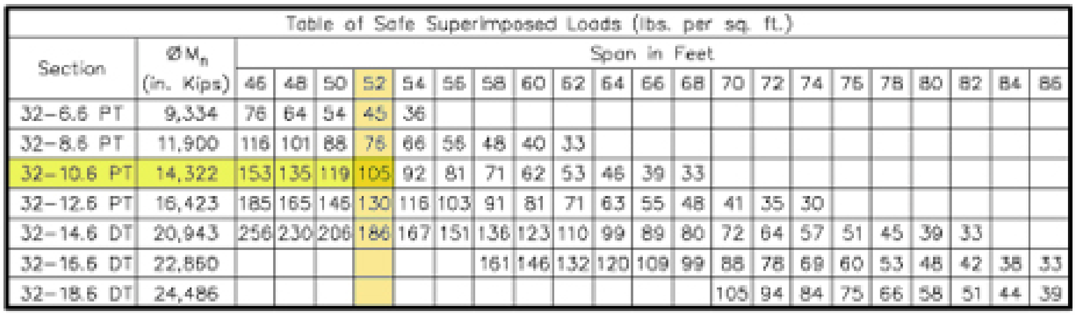
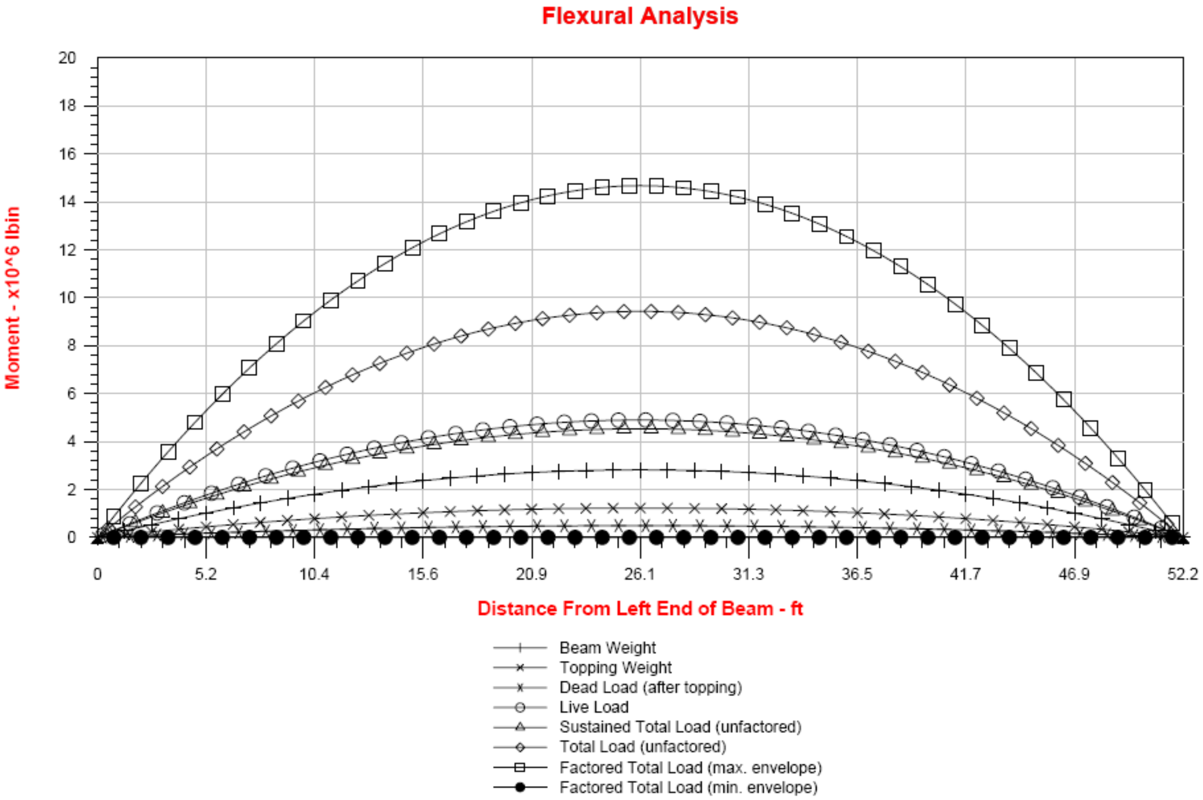


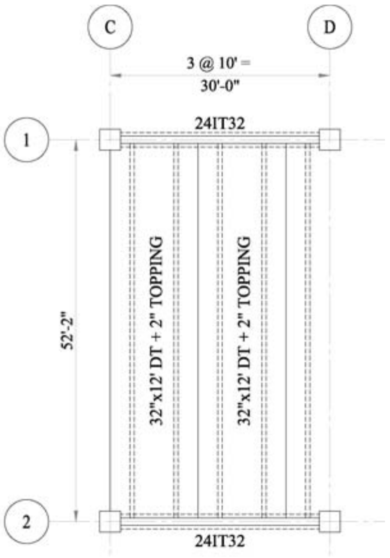
Table of Safe Superimposed Loads (lbs. per sq. ft.)																						
Section	ØM	Span in Feet																				
	(in. Kips)	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86
32–6.6 PT	9,334	76	64	54	45	36																
32–8.6 PT	11,900	116	101	88	76	66	56	48	40	33												
32–10.6 PT	14,322	153	135	119	105	92	81	71	62	53	46	39	33									
32–12.6 PT	16,423	185	165	146	130	116	103	91	81	71	63	55	48	41	35	30						
32–14.6 DT	20,943	256	230	206	186	167	151	136	123	110	99	89	80	72	64	57	51	45	39	33		
32–16.6 DT	22,860							161	146	132	120	109	99	88	78	69	60	53	48	42	38	33
32–18.6 DT	24,486													105	94	84	75	66	58	51	44	39

Design:

```
FLEXURAL DESIGN CHECK

Design Code Used: ACI318/318M-99
Beta Used: for precast beam = 0.750 , for topping = 0.850

             Factored      Design        Minimum      Depth in     Ø      Warnings
              Moment      Strength      Strength    Compression           & Notes
    x           Mu          ØMn          1.2Mcr          c
    ft         lbin         lbin          lbin          in

________ ____________ ____________ ____________ ____________ ____ ____________
    0.00            0            0     2.377E+6         0.00 0.90
    5.22     5.283E+6    10.954E+6    11.363E+6         1.36 0.90
   10.43     9.392E+6    14.688E+6    11.510E+6         1.84 0.90
   15.65    12.326E+6    14.688E+6    11.615E+6         1.84 0.90
   20.87    14.087E+6    14.688E+6    11.678E+6         1.84 0.90
   26.08    14.674E+6    14.688E+6    11.699E+6         1.84 0.90
   31.30    14.087E+6    14.688E+6    11.678E+6         1.84 0.90
   36.52    12.326E+6    14.688E+6    11.615E+6         1.84 0.90
   41.73     9.392E+6    14.688E+6    11.510E+6         1.84 0.90
   46.95     5.283E+6    10.954E+6    11.363E+6         1.36 0.90
   52.17            0            0     2.377E+6         0.00 0.90

________ ____________ ____________ ____________ ____________ ____________
Points of Maximum and Minimum Factored Moment
   26.08    14.674E+6    14.688E+6    11.699E+6         1.84 0.90
    0.00            0            0    -4.824E+6         0.00 0.90
Points of Critical Moment Design
   26.08    14.674E+6    14.688E+6    11.699E+6         1.84 0.90
    0.00            0            0    -4.824E+6         0.00 0.90

Warnings & Notes
```


Flexural Analysis
Moment - x10^6 lbin
20
18
16
14
12
10
8
6
4
2
0
0
5.2
10.4
15.6
20.9
26.1
31.3
36.5
41.7
46.9
52.2
Distance From Left End of Beam - ft
Beam Weight
Topping Weight
Dead Load (after topping)
Live Load
Sustained Total Load (unfactored)
Total Load (unfactored)
Factored Total Load (max. envelope)
Factored Total Load (min. envelope)

Girder Design

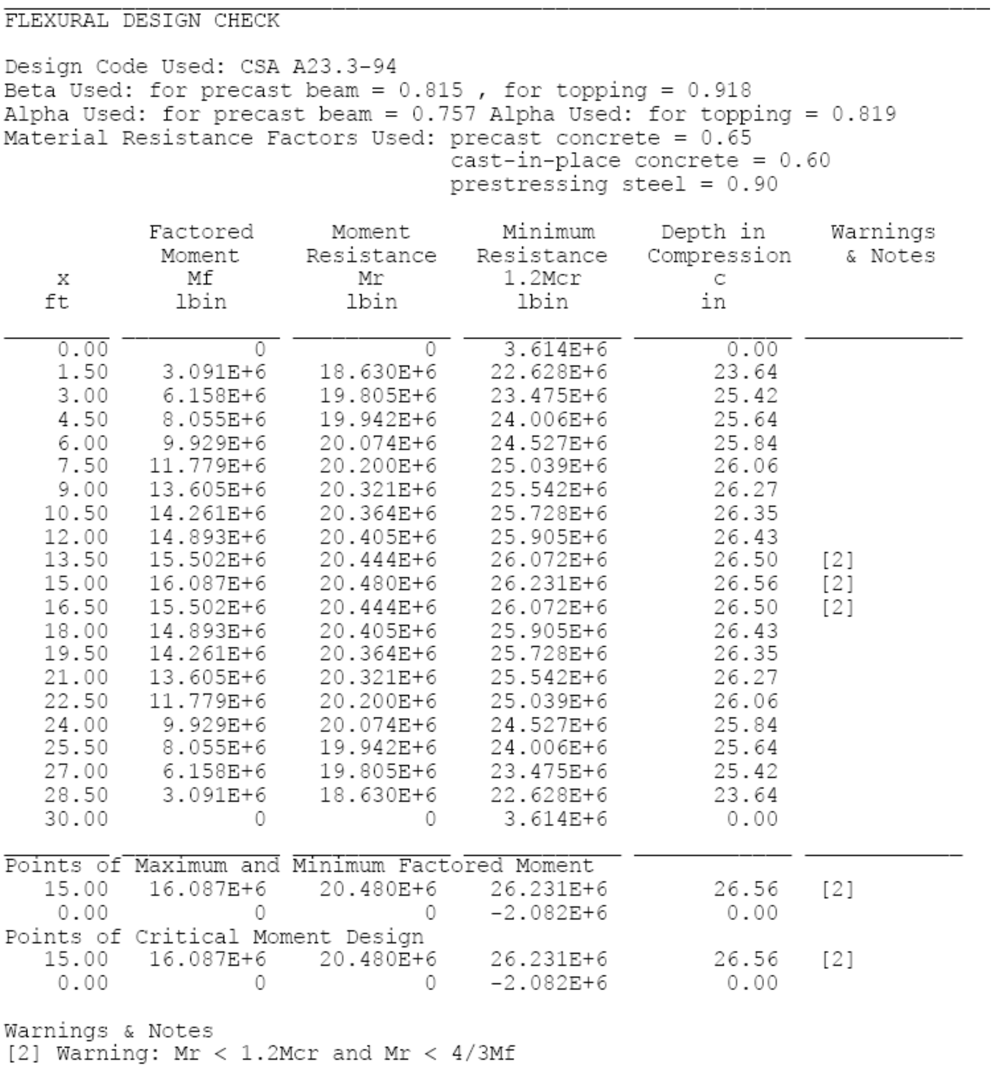


Given:

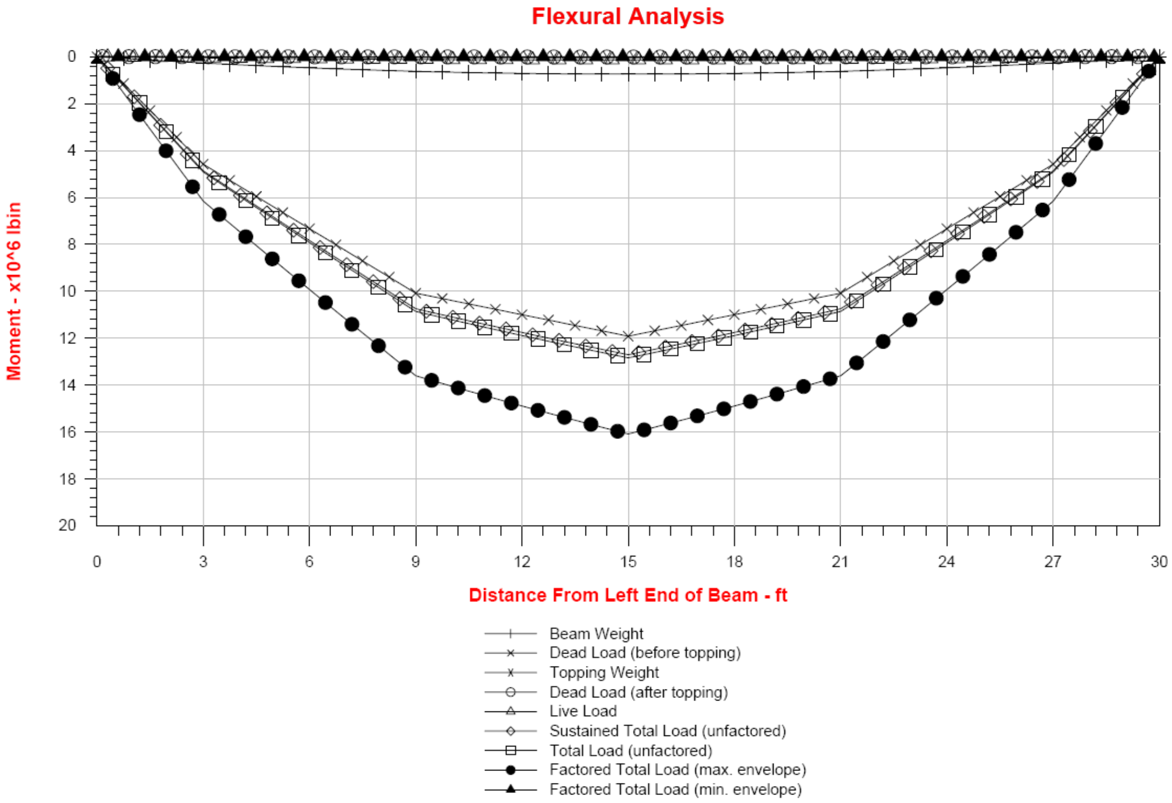
- Concrete:
  - Beams:
    - f'_c=9000 psi, NWT
    - f'_{ci}=6300 psi
  - Topping: f'_c=3000 psi, NWT

Design:

- Inverted T-shape: 24IT32
- Strands: (75) 0.6”ϕ 270k LO-relaxation



```
FLEXURAL DESIGN CHECK

Design Code Used: CSA A23.3-94
Beta Used: for precast beam = 0.815 , for topping = 0.918
Alpha Used: for precast beam = 0.757 Alpha Used: for topping = 0.819
Material Resistance Factors Used: precast concrete = 0.65
                                  cast-in-place concrete = 0.60
                                  prestressing steel = 0.90

           Factored      Moment       Minimum      Depth in     Warnings
            Moment     Resistance   Resistance   Compression    & Notes
    x         Mf           Mr         1.2Mcr          c
   ft        lbin         lbin         lbin          in
_______ ____________ ____________ ____________ ____________ ____________
   0.00            0            0    3.614E+6         0.00
   1.50    3.091E+6    18.630E+6   22.628E+6        23.64
   3.00    6.158E+6    19.805E+6   23.475E+6        25.42
   4.50    8.055E+6    19.942E+6   24.006E+6        25.64
   6.00    9.929E+6    20.074E+6   24.527E+6        25.84
   7.50   11.779E+6    20.200E+6   25.039E+6        26.06
   9.00   13.605E+6    20.321E+6   25.542E+6        26.27
  10.50   14.261E+6    20.364E+6   25.728E+6        26.35
  12.00   14.893E+6    20.405E+6   25.905E+6        26.43
  13.50   15.502E+6    20.444E+6   26.072E+6        26.50     [2]
  15.00   16.087E+6    20.480E+6   26.231E+6        26.56     [2]
  16.50   15.502E+6    20.444E+6   26.072E+6        26.50     [2]
  18.00   14.893E+6    20.405E+6   25.905E+6        26.43
  19.50   14.261E+6    20.364E+6   25.728E+6        26.35
  21.00   13.605E+6    20.321E+6   25.542E+6        26.27
  22.50   11.779E+6    20.200E+6   25.039E+6        26.06
  24.00    9.929E+6    20.074E+6   24.527E+6        25.84
  25.50    8.055E+6    19.942E+6   24.006E+6        25.64
  27.00    6.158E+6    19.805E+6   23.475E+6        25.42
  28.50    3.091E+6    18.630E+6   22.628E+6        23.64
  30.00            0            0    3.614E+6         0.00
_______ ____________ ____________ ____________ ____________ ____________
Points of Maximum and Minimum Factored Moment
  15.00   16.087E+6    20.480E+6   26.231E+6        26.56     [2]
   0.00            0            0   -2.082E+6         0.00
Points of Critical Moment Design
  15.00   16.087E+6    20.480E+6   26.231E+6        26.56     [2]
   0.00            0            0   -2.082E+6         0.00

Warnings & Notes
[2] Warning: Mr < 1.2Mcr and Mr < 4/3Mf
```

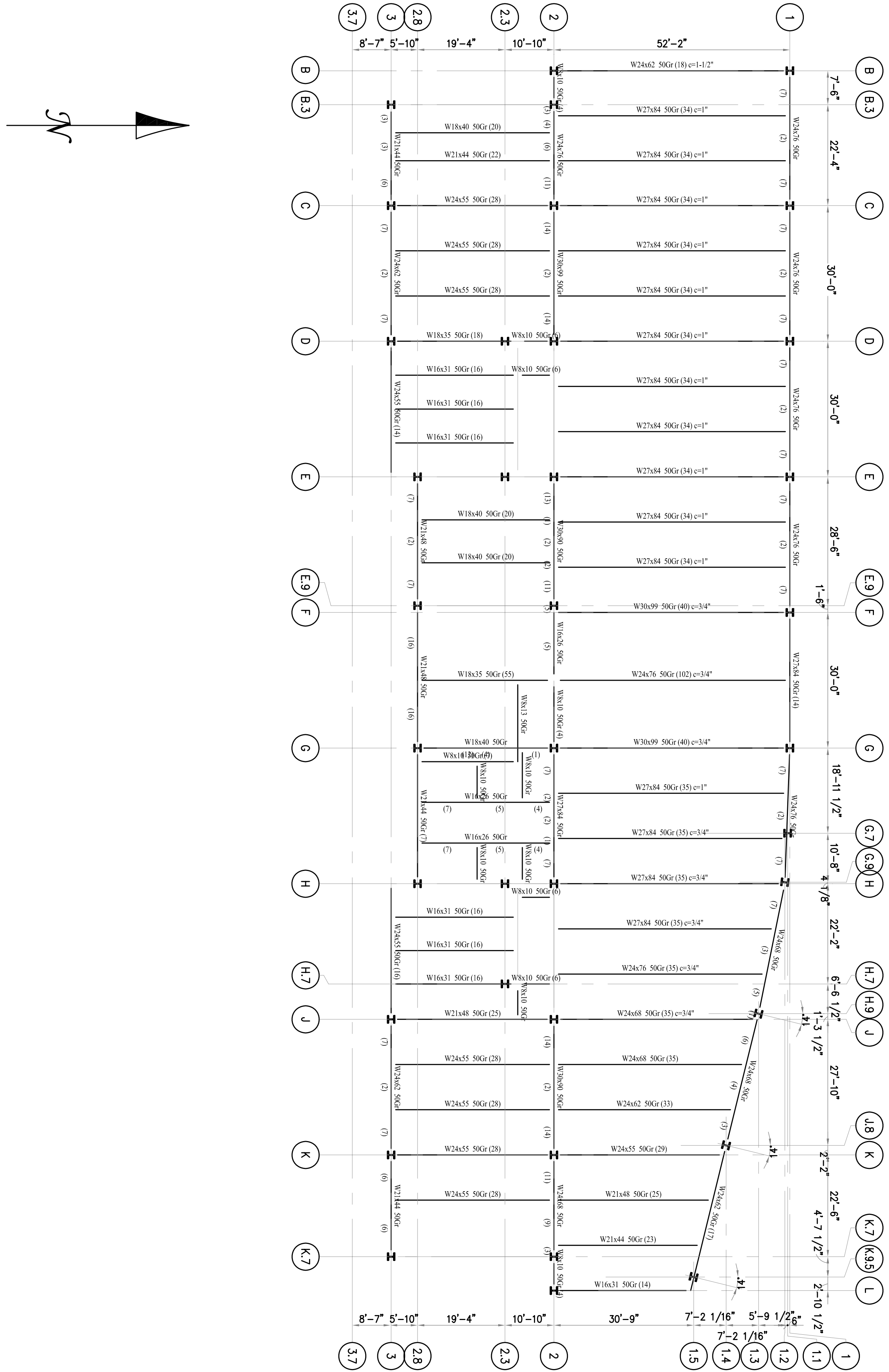

Flexural Analysis
Moment - x10^6 lbin
0
2
4
6
8
10
12
14
16
18
20
0
3
6
9
12
15
18
21
24
27
30
Distance From Left End of Beam - ft
Beam Weight
Dead Load (before topping)
Topping Weight
Dead Load (after topping)
Live Load
Sustained Total Load (unfactored)
Total Load (unfactored)
Factored Total Load (max. envelope)
Factored Total Load (min. envelope)

Appendix F – Cost Analysis

All cost data was found in the RS Means 2005, Assemblies Cost Data and is based on a typical floor area of 21,888 square feet.

System	Cost/Square Foot	Total Cost/Floor
Precast Double-Tee w/ 2” topping: 50’ span	$9.44	$206,623
Composite Steel Joists	$18.55	$406,022
C.I.P. Multi-span joist slab, 35’x30’ bay	$15.75	$344,736
C.I.P. Beam & Slab, One Way, 35’x30’ bay	$17.95	$392,890
Composite Beams, Deck, Slab: 30’x35’ bay	$20.05	$438,854

Appendix G – Framing Plans



GATEWAY PLAZA

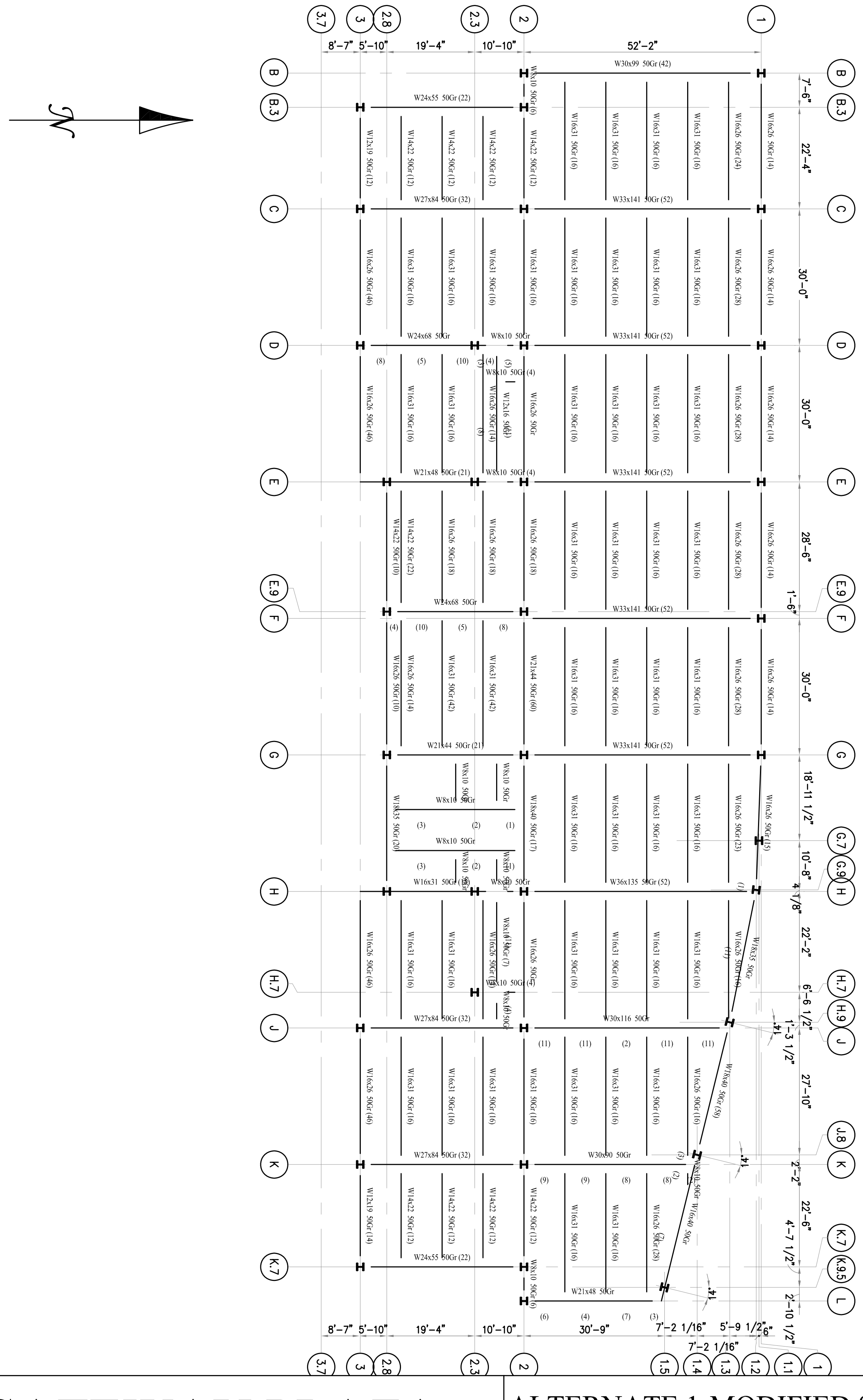
500 DELAWARE AVE.
WILMINGTON, DE

EXISTING FRAMING PLAN

APPENDIX G - FRAMING PLANS

SCALE: 1/32"=1'-0"

PAGE: 1 OF 6



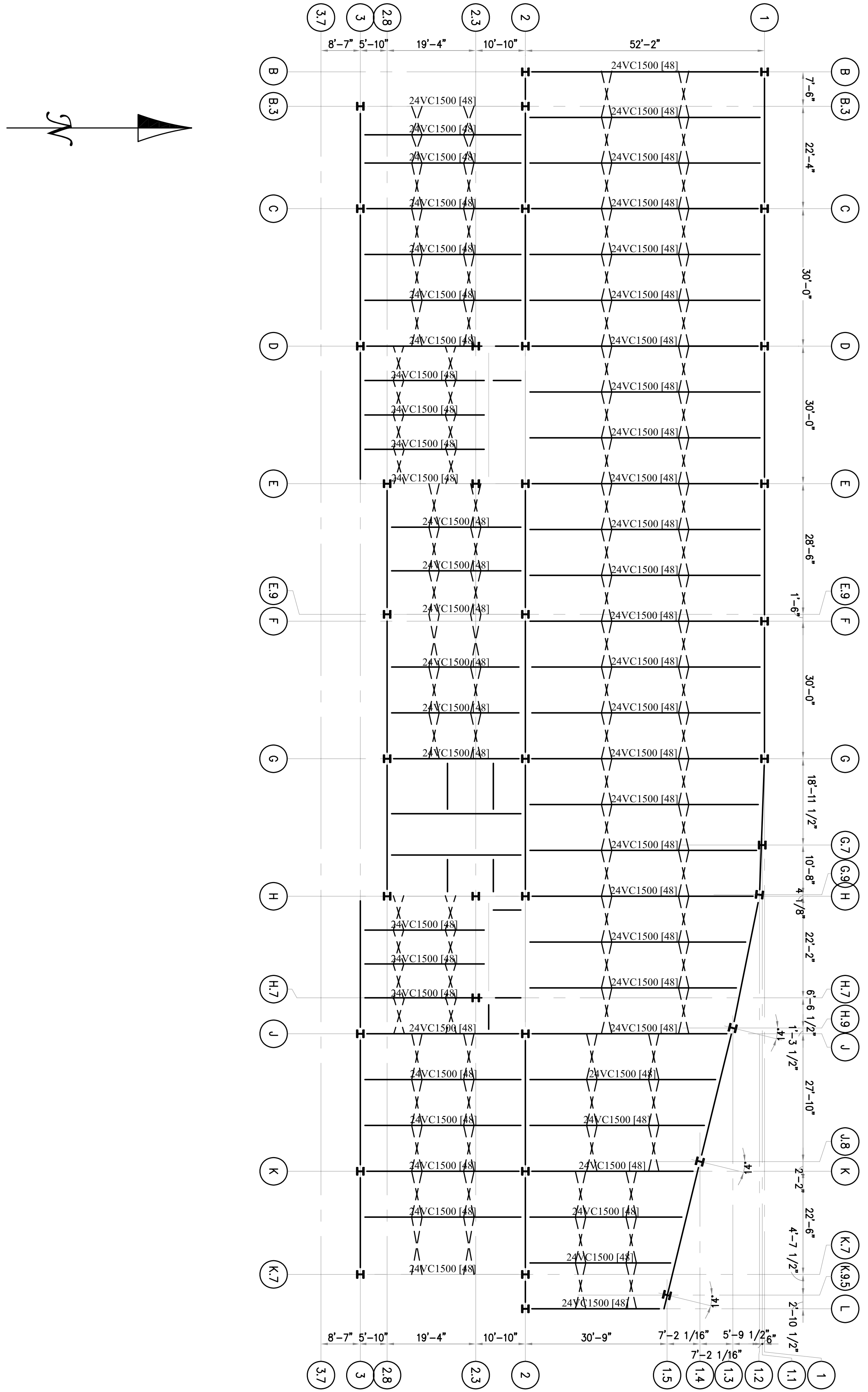
GATEWAY PLAZA

500 DELAWARE AVE.
WILMINGTON, DE

ALTERNATE 1-MODIFIED STEEL

APPENDIX G- FRAMING PLANS

SCALE: 1/32"=1'-0"

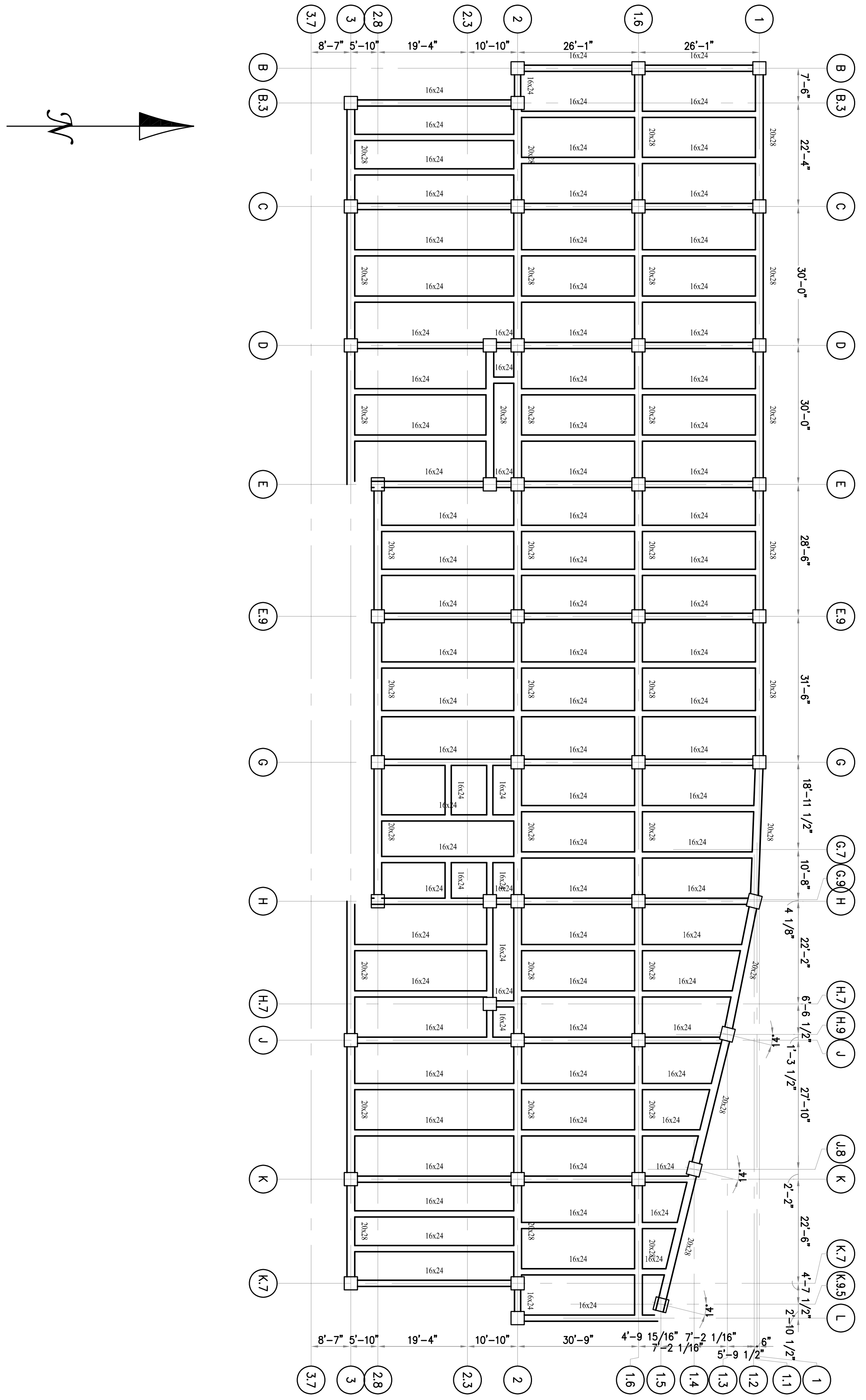


GATEWAY PLAZA
500 DELAWARE AVE.
WILMINGTON, DE

ALTERNATE 2-STEEL JOISTS

APPENDIX G- FRAMING PLANS

SCALE: 1/32"=1'-0"



GATEWAY PLAZA

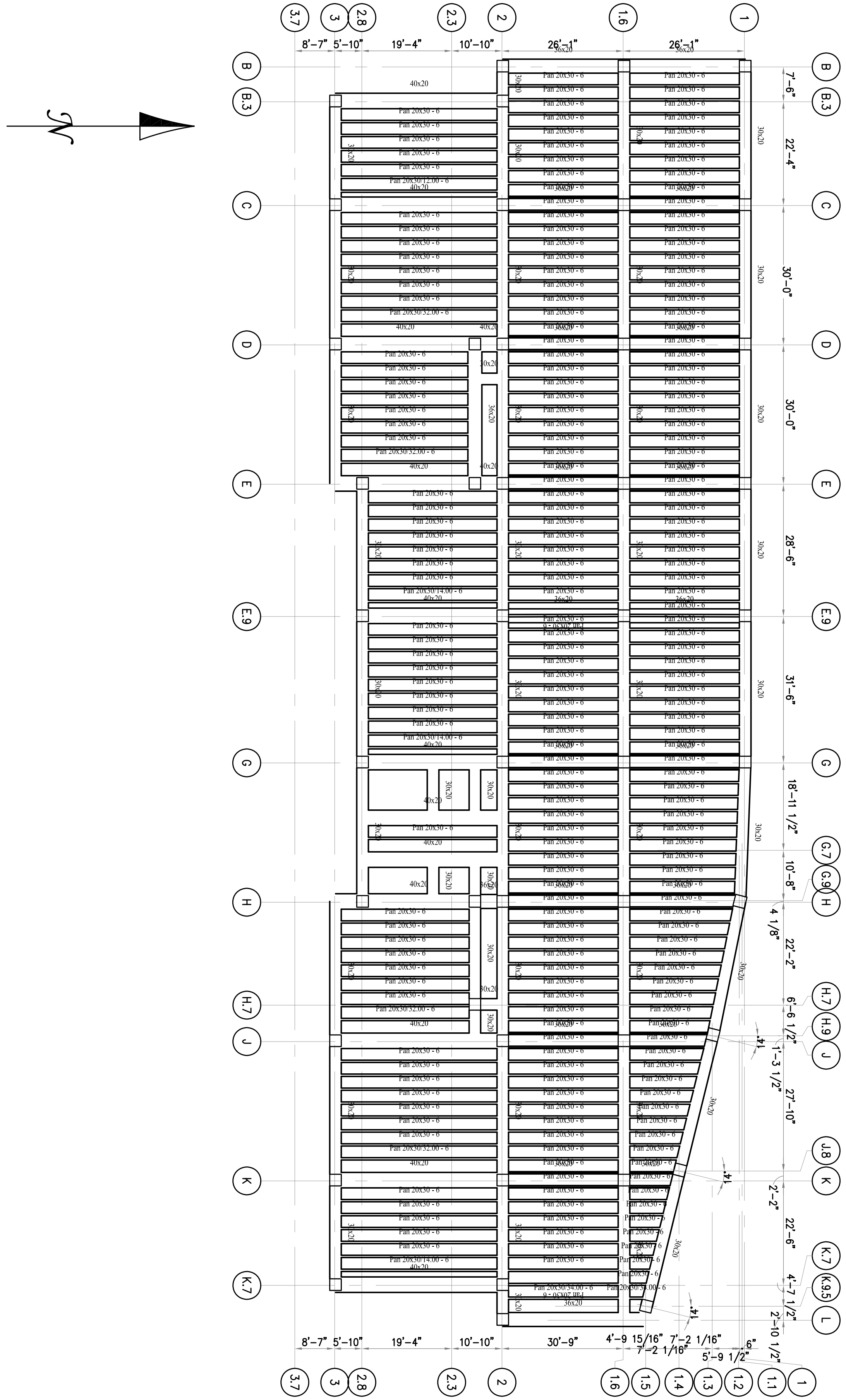
500 DELAWARE AVE.
WILMINGTON, DE

ALTERNATE 3-ONE-WAY CONC. SLAB

APPENDIX G- FRAMING PLANS

SCALE: 1/32"=1'-0"

PAGE: 4 OF 6



GATEWAY PLAZA

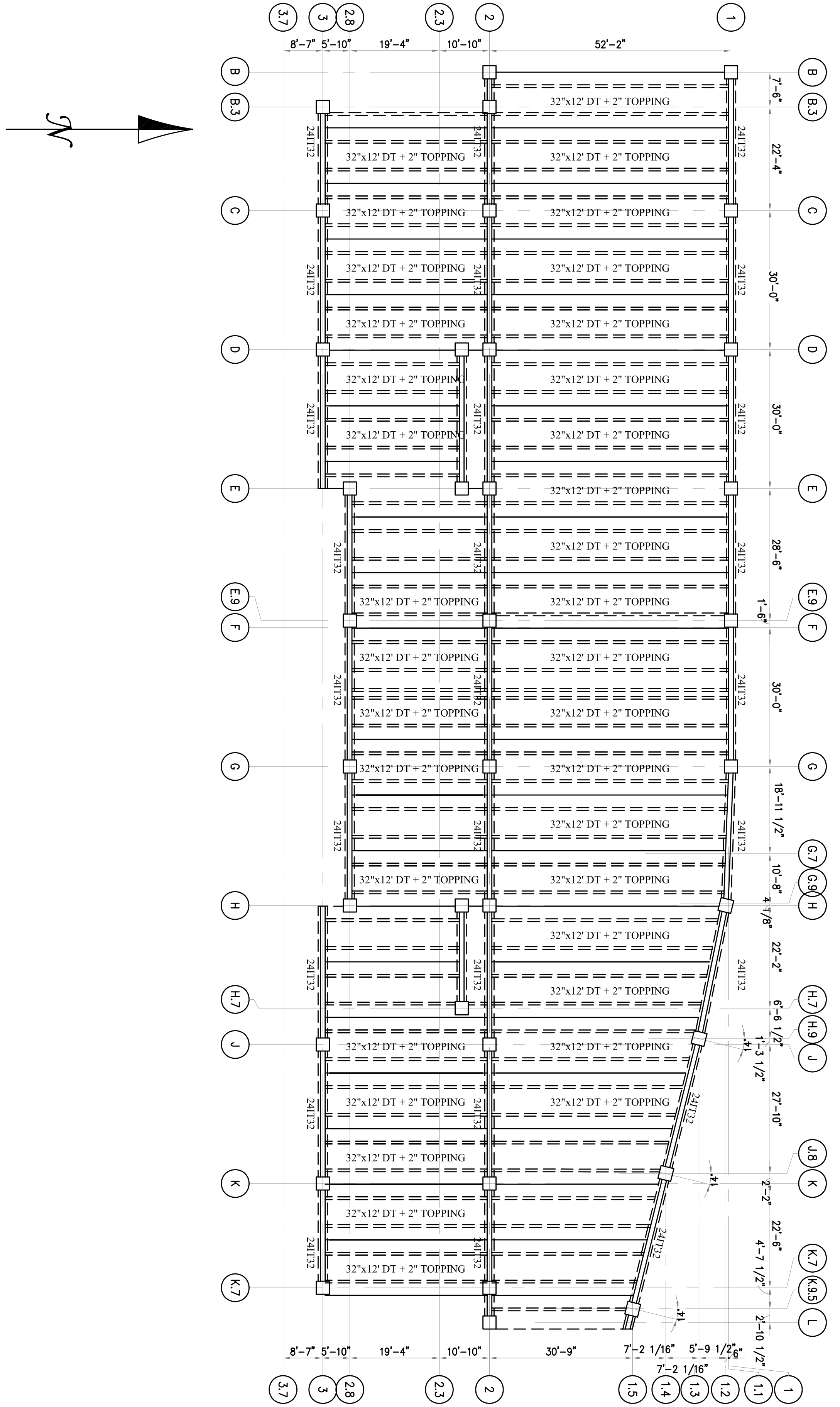
500 DELAWARE AVE.
WILMINGTON, DE

ALTERNATE 4-CONCRETE PAN JOISTS

APPENDIX G- FRAMING PLANS

SCALE: 1/32"=1'-0"

PAGE: 5 OF 6



GATEWAY PLAZA

500 DELAWARE AVE.
WILMINGTON, DE

ALTERNATE 5-PRECAST CONCRETE

APPENDIX G- FRAMING PLANS

SCALE: 1/32"=1'-0"

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