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10/31/05
AE 481W

Structural Technical Report 2

Pro-Con Structural Study of Alternate Floor Systems

Executive Summary

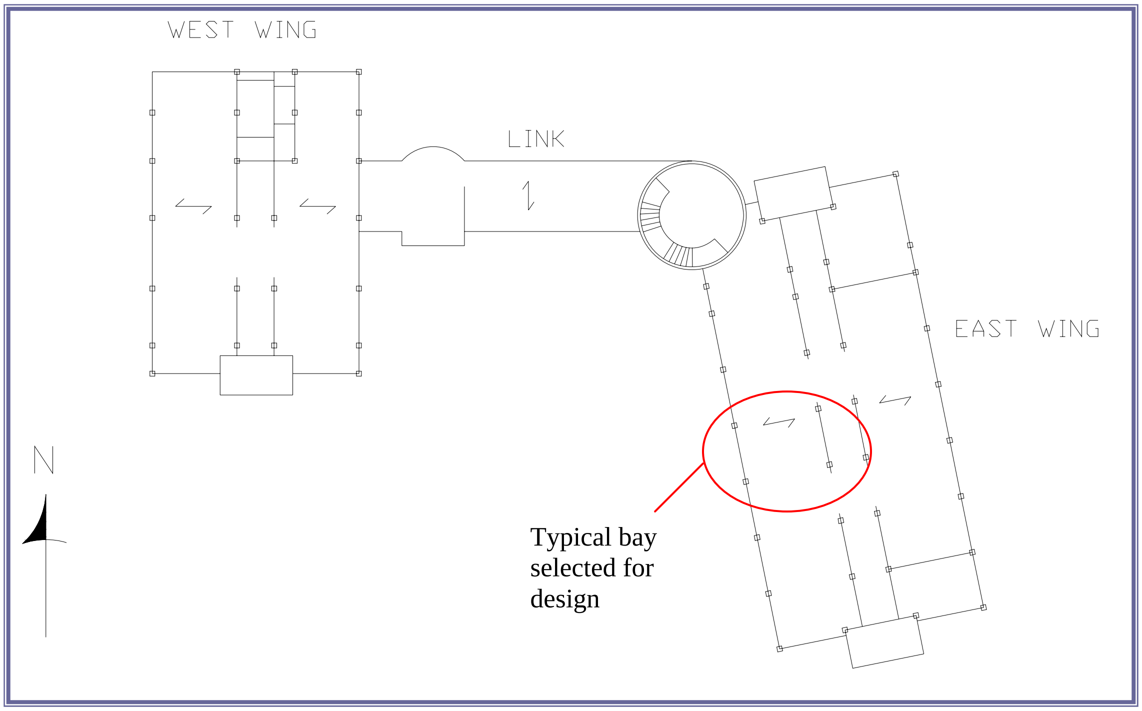
The purpose of this report is to provide a descriptive analysis of, as well as a comparison between, the existing floor system of the Koshland Integrated Natural Science Center and the design of four alternate typical floor systems. The report provides detailed information on the current floor system and also the factored loads that were used to design the existing system. The design loads were taken from the drawings and confirmed with the BOCA 1993 code. The analysis focused on a typical bay of the current floor system which was calculated by hand and was also verified with the PCI Manual for the Design of Hollow Core Slabs (2nd Edition) and the CRSI Handbook (2002). The alternative floor designs were also analyzed by hand calculations which can be reviewed in the Appendices following this report.

The following portion of this report consists of the designing of four floor systems as alternatives to the existing system. All four systems are significantly different from one another. The floor systems that were selected for design include a one-way pan joist system, an open-web steel joist system, a composite beam/composite slab system, and a precast double tee beam system. In this section of the report, each alternative floor system is described, followed by a sketch of the typical bay incorporating the system described. In addition, several advantages and disadvantages of each of the alternative systems are discussed and compared briefly. Also, as referenced in the body of the report, calculations, design tables, and complete design details are included in the report as Appendices.

Finally, the last section of the report compares the advantages and disadvantages of each system in table format. The categories that are compared as the advantages and disadvantages are floor depth, self weight of the system, impact of construction schedule, and cost. With the information from the comparison table, the conclusion of the report emphasizes that the existing system proved to be the most efficient overall for this building. However there are other systems that could be considered as realistic possible alternatives such as the open web steel joist system or the double tee beam system.

Existing Floor System Design

The Koshland Integrated Natural Science Center combines four stories of laboratory and class room spaces with additional office space, library, and mechanical rooms. Laboratory spaces are located on each of the four floors. In addition, mechanical rooms are located on the ground floor and the fourth floor, classrooms are found on the first floor, offices are found on the second floor, and the third floor is the location for the library. The library area has a second mezzanine level which is on the fourth floor. The floor framing is typical throughout the East and West wings of the building, with the exception of the second level library found on the fourth floor of the West Wing. The similarities include member sizes, locations, and spans. Below is a diagram of the typical floor layout for the entire building.



The existing floor design of a typical lab floor layout is comprised of precast concrete hollow core planks that are supported by precast beams. For this analysis, a typical framing bay from the East Wing spanning from column lines 16 to 17 and U to W has been selected. The typical span for the hollow core planks is 31'-5". The beams typically span 21'-0". Currently, the design calls for hollow core planks that are 10" deep with a 2" topping slab. It is assumed that the planks have a length of 4' in the short span direction. Also, since the reinforcement is not called out on the drawings, it is assumed to be (6) – 0.6" diameter prestressed tendons. The precast beams that support the floor system are dimensioned at 20"x12". The reinforcement of the beams is unknown, therefore, it is assumed to be two layers of (7) – 0.6" diameter prestressed strands at the bottom and (6) - #11 bars in compression at the top. The precast concrete is also assumed to have a compressive strength of 5 ksi.

Floor Loads

The loads used in the analysis of the existing floor system were taken as typical floor loads for the chosen bay. These floor loads used for this typical bay may not be representative of the entire structure due to specified increased loading requirements for other locations throughout the building. The following is a breakdown of the dead and live loads that were used for the typical bay.

Live Loads (psf):

- Typical floors 100

Dead Loads (psf):

- Ceiling 5
- Mech., elec., & plumb. 10
- Framing 15
- 10" hollow core planks w/2" topping 91
- Partitions 30

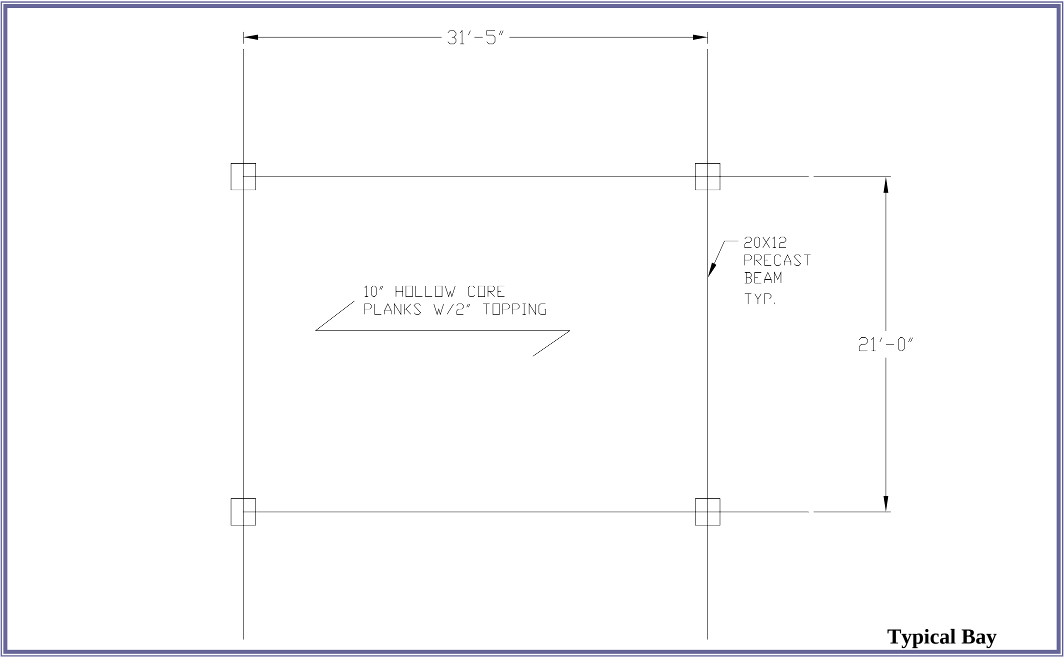
Hollow Core Floor System Design

	Dead Loads (psf)		Live Loads (psf)
Ceiling	5	Typical floor	100
MEP	10		
Framing	15		
Hollow core planks w/topping	91		
Partitions	30		
Total:	151	Total:	100

Span (ft.)	Tributary Width (ft.)	Tributary Area (sq. ft.)	T_{LL} (psf)	T_{DL} (psf)	w_u (psf)	w_u (klf)	M_{max} (ft. kips)
31.42	4	125.68	100	151	341.2	1.36	142.9

Analysis

The existing floor system was analyzed by hand calculations. The results of these calculations are available for review in Appendix B. Certain information about the existing floor system was not available on the drawings. Therefore, assumptions had to be made to carry out the analysis, which are noted in the calculations. These assumptions include the width of the hollow core planks, the strength of the precast concrete, and the prestressed reinforcement used in the planks.



Typical Bay

From the drawings, it is noted that the floor system currently used is a 10" precast hollow core plank system with a 2" topping slab from SpanDeck. However, when running the analysis it was found that there is no 10" plank section from SpanDeck in the PCI Manual for the Design of Hollow Core Planks, 2nd Edition. Several manufacturers' websites were reviewed for a similar section. Ultimately, the most similar section found was manufactured by DyCore and was used in the calculations. The results from the analysis of the existing floor system showed that the 10" hollow core planks with a 2" topping slab is sufficient to carry the applied floor loads. The required reinforcement in the planks was determined to be (6) – 0.6" diameter prestressed tendons.

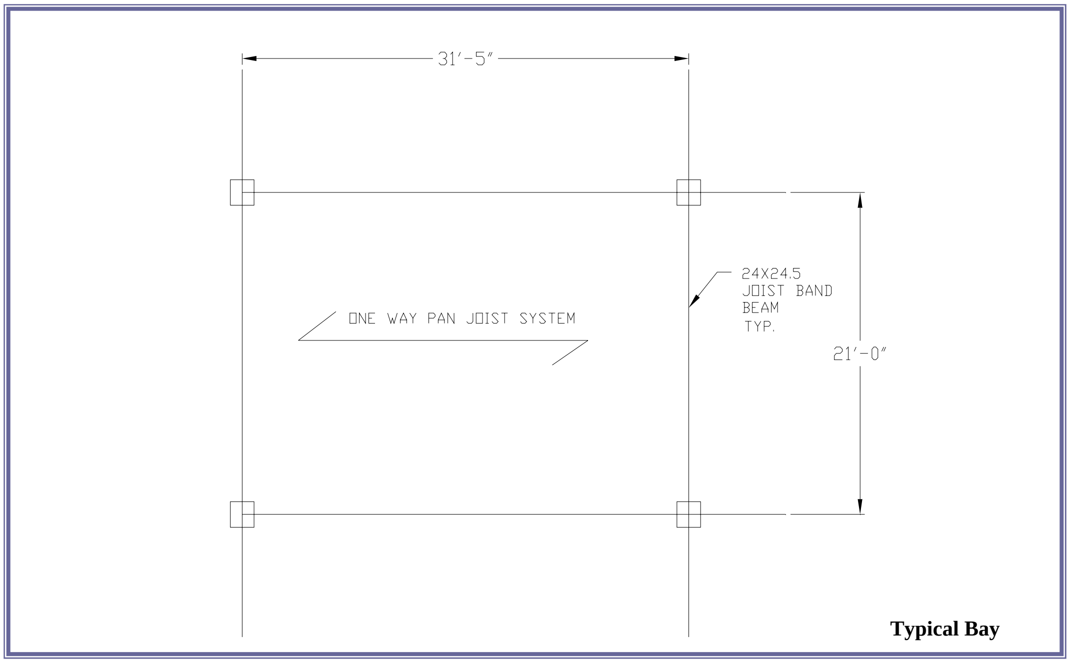
Some advantages to using the existing system may not be seen from the calculations. One of the greatest advantages to using the precast hollow core system is the time efficiency that it allows. The precast concrete does not require the curing time of the concrete that cast-in-place concrete would require. Therefore, it allows for a faster construction schedule which could directly relate to lower costs. Conversely, precast concrete does require more upfront planning. Also, the hollow core system produces a shallower floor depth which provides more room for mechanical or electrical equipment.

Alternate Floor System Designs

During the design of alternate floor systems, both steel and concrete systems were considered. The four alternate systems that were chosen are significantly different than the existing system. The systems that were chosen include a One-way Pan Joist system, a precast Double-T system, open-web steel joists with slab on metal deck, and composite beams with a composite deck. When conducting the designs of possible alternate floor systems, several design aids were used in the process. These design aids are included in the reference section in Appendix A.

One-Way Pan Joist System

The typical bay that was used for these alternative designs has a short span of 21' and a long span of 31'-5". With this in consideration, a one-way pan joist system would work well in this situation. Below is a sketch of the typical floor layout.



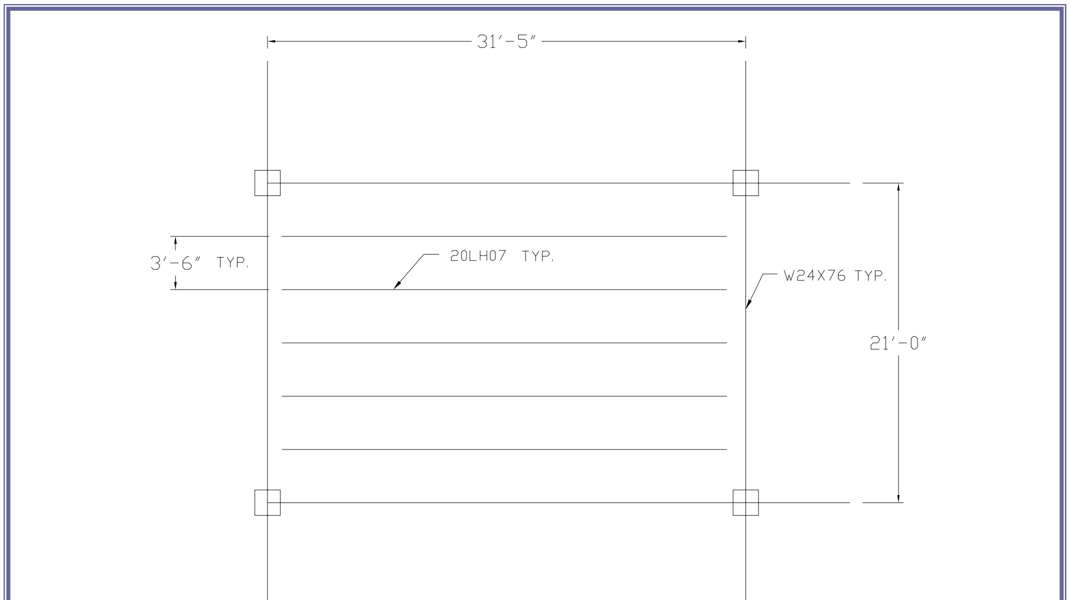
Typical Bay

For the design of this floor, the load tables from the CRSI 2002 handbook were used as design aids to size the slab system. All relevant information that was gathered from the handbook can be reviewed in Appendix C. The design that was chosen is a one-way pan joist system with 30" forms and 6" ribs at 36" on center. The total depth of the system is 23" comprised of 20" deep ribs and a 3" slab. Below is a section of this system. The girders that were designed to support this slab system are 24"x24.5" joist band beams.

This one-way pan joist system is a cast-in-place concrete system, different from the existing system. Therefore curing time for the concrete is necessary during construction. Again, this will increase construction time and possibly project cost. Also, the one way pan joist system has a self weight of 91 psf, as per the CRSI handbook. This is exactly the recorded self weight of the existing hollow core system. Therefore, no increased loading due to self weight will occur. However, as previously stated, this system has a total depth of 23" which is much larger than the existing system. This is one disadvantage of incorporating the one way pan joist system. The increase in depth will take away from ceiling space for mechanical and electrical systems.

Open Web Steel Joist System

When designing the second alternative floor system, an open web steel joist system, the Wheeling Deck Product Catalog was used as a design aid. The design produced a non-composite 3" concrete slab on a 9/16" Tensilform metal deck. The second design aid used with this open web steel joist system was the New Columbia Joist Company (NCJ) Joist Catalog. From the design tables in the catalog, a 20LH07 open web steel joist was selected for the design. The depth of the joist is 20". Three rows of 2x2x1/8 steel angles for bridging are required in the design of the open web joists. The third design aid used for this floor system was the LRFD Manual of Steel Construction (Third Edition) and it was used to design the steel girders to support the open web joists. From the design tables, the girder was sized to be a W24x76. All design details from design aids and calculations are available for review in Appendix D.

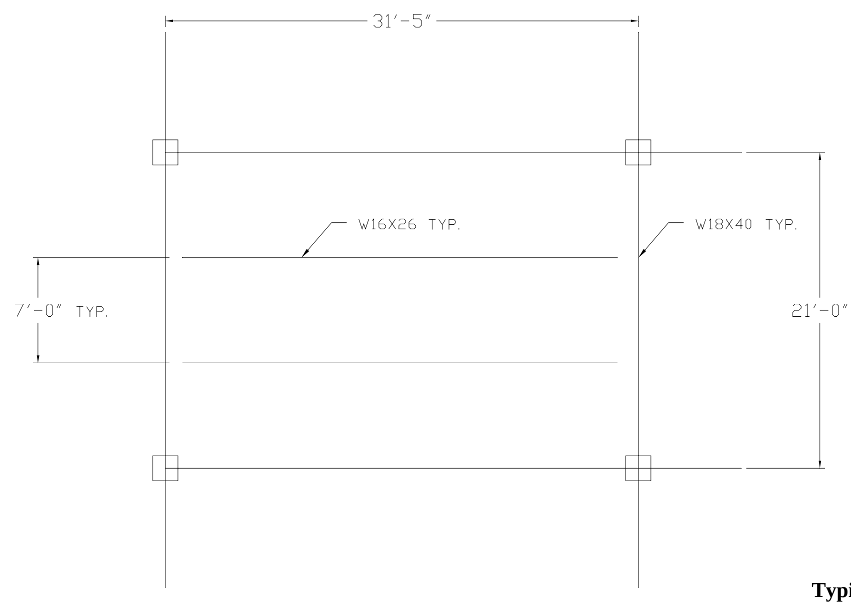


Typical Bay

The depth of the open web steel joist system was designed to be 20". This produces a greater ceiling depth. However, space for mechanical and electric systems may not be compromised. Since the joists are constructed with open webs, the voids in the webs will provide adequate room for MEP systems. In addition, the total weight of the slab on deck and joists is 51 psf. This is significantly less than the 91 psf self weight of the existing floor system. This reduction in self weight could result in smaller framing members. However, this system does require a spacing of 3'-6" for the open web steel joists. For a typical span, this produces seven joists per span. With this in consideration, and the use of steel W24x76 girders, the cost of materials is considerably higher than the existing precast hollow core system.

<u>Composite Beam/Composite Slab</u>

The third alternative floor system that was considered is a composite beam with a composite slab on deck system. Design aids that were used for this floor system design were the Wheeling Deck Products Catalog and the LRFD Manual for Steel Construction (Third Edition). The design for the composite slab resulted in a 4" slab on a 1-1/2" metal deck. The composite beam was designed to be a W16x26 steel beam. From the calculations, a total of 26 shear studs are required for full composite action for the beams. When designing for the steel girders, the results yielded a W18x40 steel girder with 40 shear studs to ensure full composite action. Below is a typical floor layout for this floor system. For full design details and calculations, see Appendix E.

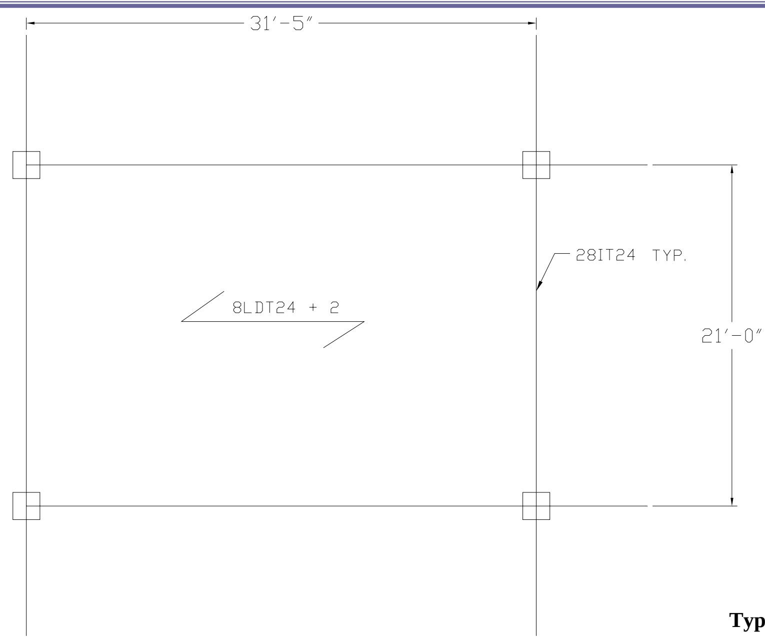


Typical Bay

The total depth of the composite beam/composite slab system is 30". When compared to a depth of 12" from the existing precast hollow core system, this alternative system will certainly decrease ceiling space. This is considered a disadvantage for the location of the MEP systems as well. Also, the self weight of this floor system is substantially larger than that of the existing system. This could cause increased loading on the framing members in flexure, as well as raise the cost of materials for the floor system. However, this system presents a spacing of 7' for the steel beams which results in fewer members. This could actually decrease the cost of the materials for the floor system.

Precast Double Tee Beam System

The final consideration for an alternative floor system was a double tee beam system. The design aid used for this floor system was the PCI Manual. Using the applied floor loads, the design resulted in an 8LDT24 + 2 system. This particular section is an 8' wide double tee beam that is 24" in depth. A two inch topping is applied on top of the double tee beams. The double tee beams are reinforced with (6)-1/2" prestressed strands that run straight through the section. For a girder to support this floor system, a 28IT24 inverted tee beam was selected in the design. This girder has beam seat depth of 12" and an overall depth of 24". It is reinforced with (11)-1/2" diameter low-lax prestressed strands. For full design details and calculations, see Appendix F.



Typical Bay

The overall depth of the double tee beam floor system is 24". This is considerably larger than that of 12" from the existing system. With this increase in depth, the ceiling space will be reduced forcing a larger plenum space for MEP systems. However, this system is still advantageous. Since this floor system is precast concrete, the concrete will not need to cure once it is installed. This will cut down on time in the construction schedule and reduce cost. In addition, the double tee beam system has a self weight of 40 psf, which is lighter than the 91 psf self weight of the existing system. This results in lighter loading on framing members which could reduce the size of the framing members, which, in return, saves cost of materials.

Floor System Design Comparisons

System	Description	Total Depth (in)	Self Weight (psf)	Impact on Schedule	Cost (R.S. Means Values, $/sq.ft.)		
				1-5 (1=worst, 5=best)	Mat'l.	Labor	Total
Existing	Hollow Core Planks	12	91	5	6.10	2.32	8.42
Alternate #1	One Way Pan Joist	23	91	2	4.62	8.15	12.77
Alternate #2	Open Web Steel Joist	20	51	1	8.50	4.63	13.13
Alternate #3	Comp. Beam/Comp. Slab	20	40	3	9.30	5.40	14.70
Alternate #4	Double Tee Beam	24	62.5	4	4.96	1.70	6.83

Conclusion

The results from the comparisons of the existing and alternate floor systems conclude that the existing floor system is the most efficient in physical properties, construction time, and cost. However, some of the alternate systems may be seen as realistic solutions to an alternative floor design. For instance, open web steel joist system offers a lighter solution for the floor system, although it sacrifices some ceiling space. Also, the double tee beam offers a design that is lighter than the existing hollow core system and is almost as time efficient as the existing system. In addition, the double tee beam system is the most cost effective system that was analyzed. However, the overall depth of the double tee beam system is twice that of the existing hollow core system. The above table compares all floor systems that were analyzed in a number of different categories.

Appendix A

References

- PCI Design Handbook, Precast and Prestressed Concrete, 5th Edition
- PCI Manual for the Design of Hollow Core Slabs, 2nd Edition
- CRSI Handbook, 2002
- Wheeling Deck Product Catalog
- New Columbia Joist Catalog
- LRFD Manual for Steel Construction, 3rd Edition
- R.S. Means 2005, Assemblies Cost Estimates

Appendix B

Existing Floor System Design
Precast Hollow Core Planks

G E
210 Arnold Avenue, Suite F
Point Pleasant Beach, New Jersey 08742
Voice: (732) 892-6979 Fax: (732) 892-6976
www.gillespieengineering.com

HOLLOW-CORE PLANK FLOOR SYSTEM - SPOT CHECK

$d = 10''$ SPAN = 31'-6" = 31.5'

$d_{TOPPING} = 2''$ CLEAR SPAN = 29'

ON PLANS, IT IS SPECIFIED THAT A 10" SPANDECK HOLLOW-CORE SYSTEM IS USED. AS PER MANUAL FOR THE DESIGN OF HOLLOW CORE SLABS, 2ND EDITION - PCI, THERE IS NO 10" DEEP SECTION OF SPANDECK.

ALTERNATE SYSTEM USED: Dy-CORE

Section: 4'-0" x 10"

Weight: 81 psf

LOADS

DL = CEILING = 5 psf
MEP = 10 psf
FRAMING = 15 psf
PARTITIONS = 30 psf
60 psf
Self wt = 91 psf
151 psf

Assume: 1) $f'_c = 5000$ psi
2.) low lax tendons
3.) $f_{pu} = 270$ ksi
4.) $f_{pe} = 0.55 f_{pu} = 148.5$

LL = Typ. = 100 psf

$w_u = 1.2(151\ psf(4')) + 1.6(100\ psf(4'))$
$= 1364.8$ plf
$= 1.36$ klf

$M_u = \frac{1.36(29)^2}{8} = 142.9'^k$

TRY (6)-0.6"ϕ PRETENSIONED STRANDS

$A_{ps} = 6(0.216) = 1.296\ in^2$

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Point Pleasant Beach, New Jersey 08742
Voice: (732) 892-6979 Fax: (732) 892-6976
www.gillespieengineering.com

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{pu}}{f'_c}\right)\right]$$

$\gamma_p = 0.28$ for "low-lax"

$\beta_1 = 0.80$

$d_p = 9.0''$

$\rho_p = \frac{A_{ps}}{bd_p} = \frac{1.296}{20(9)} = 0.0008$

$$= 270\left[1 - \frac{0.28}{0.8}\left(0.0008 \frac{270}{5}\right)\right]$$

$$= 265.9 \text{ ksi}$$

$$\omega_p = \frac{\rho_p f_{ps}}{f'_c} = \frac{0.0008(265.9)}{5} = 0.0425 < 0.36\beta_1 = 0.288 \therefore \text{ok}$$

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_c b} = \frac{1.296(265.9)}{0.85(5)(20)}$$

$$= 4.05''$$

$$\phi M_n = \phi\left[A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right)\right] = 0.9\left[1.296(265.9)\left(9 - \frac{4.05}{2}\right)\right]$$

$= 2163.6$ in-k

$= 180.3$ 'k $\simeq 142$ 'k $\therefore$ ok By JUDGEMENT

USE 6 - ½"φ STRANDS in 10" HOLLOW-CORE SYSTEM w/ 2" TOPPING

Appendix C

Alternate Floor System #1
One Way Pan Joist System

ALTERNATE #1 - ONE WAY PAN JOIST SYSTEM

DESIGN AIDS: CRSI Handbook (2002) pg. 8-25, 8-10, 12-90

* CRSI - FACTORED LOADS → $W_u = 1.4DL + 1.7LL$

- ONE WAY PAN JOIST (pg. 8-25, 8-10), MULTIPLE SPANS

SPAN = 31'-5" → use 32' for table values

$W_u = 1.4(45) + 1.7(100) = 233$ psf

END SPAN TABLES

↳ SELECTED:

30" FORMS + 6" RIBS @ 36" C.-C.

20" DEEP RIB + 3.0" TOP SLAB = 23" TOTAL DEPTH

$W_{ALLOW} = 280$ psf > $W_u = 233$ psf ∴ ok

REINFORCEMENT: TOP → #6 @ 11"

BOTTOM → #7, #7

self wt = 91 psf (TABLE 8-1)

ALTERNATE #1 (cont'd) - GIRDER → JOIST-BAND BEAM

$W_u = 1.4(45+91) + 1.7(100) = 360.4$ psf

SPAN = 21' → use 22' for tables

trib. w = 22.5'

$W_u = 360.4$ psf$(22.5') = 8109$ plf $= 8.109$ klf

JOIST-BAND BEAM — END SPAN TABLE (pg. 12-90)

↳ SELECTED:

24" x 24.5" JOIST-BAND BEAM
$W_{ALLOW} = 12.7$ klf $> W_u = 8.109$ klf ∴ ok

REINFORCEMENT : TOP → (4)-#14 bars

BOTTOM → 8#6 / 7#6 — 2 layers

One Way Pan Joist (CRSI 2002)

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

20" Deep Rib + 3.0" Top Slab = 23.0" Total Depth

TOP BARS Size @	#4 10	#4 8	#4 7	#5 9	#6 11	End Span Defl. Coeff. (3)	#4 9	#5 11	#5 9.5	#5 8	#6 9	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)	.85	1.04	1.23	1.44	1.69	(3)	.93	1.18	1.38	1.66	2.01	(3)
CLEAR SPAN	END SPAN						INTERIOR SPAN					
30'-0"	117 0	167 0	218 0	278 0	334* 340	9.752	165 0	228 0	301 0	375 0	395* 458*	6.001
31'-0"	101 0	148 0	196 0	253 0	310 0	11.119	147 0	206 0	274 0	343 0	374* 426	6.842
32'-0"	87 0	131 0	176 0	229 0	283 0	12.625	130 0	185 0	249 0	314 0	355* 391	7.769
33'-0"	74 0	16 0	158 0	208 0	259 0	14.278	114 0	166 0	226 0	287 0	337* 360	8.787
34'-0"	63 0	102 0	141 0	188 0	236 0	16.089	100 0	149 0	206 0	263 0	320* 332	9.901
35'-0"	52 0	89 0	126 0	171 0	216 0	18.067	88 0	134 0	187 0	241 0	305* 306	11.118
36'-0"	42 0	77 0	112 0	154 0	197 0	20.222	76 0	119 0	170 0	221 0	282 0	12.445
37'-0"		66 0	99 0	139 0	180 0	22.565	65 0	106 0	154 0	202 0	261 0	13.886
38'-0"		56 0	87 0	125 0	164 0	25.105	55 0	94 0	139 0	185 0	240 0	15.449
39'-0"		47 0	76 0	112 0	149 0	27.853	46 0	83 0	126 0	169 0	222 0	17.141
40'-0"			66 0	101 0	135 0	30.822		72 0	113 0	155 0	204 0	18.967
41'-0"			57 0	90 0	123 0	34.022		63 0	102 0	141 0	188 0	20.936
42'-0"			48 0	79 0	111 0	37.464		54 0	91 0	128 0	174 0	23.055
43'-0"			40 0	70 0	100 0	41.162		45 0	81 0	117 0	160 0	25.330

(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 ≥ ℓ_n/18.5 for end spans, ℓ_n/21 for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = ℓ_n/360.

PROPERTIES FOR DESIGN (CONCRETE .60 CF/SF) (4)

NEGATIVE MOMENT STEEL AREA (SQ. IN.)	.72	.90	1.03	1.24	1.44		.80	1.01	1.17	1.39	1.76	
STEEL % (UNIFORM)	.42	.53	.61	.73	.85		.47	.60	.69	.82	1.04	
(TAPERED)	.26	.32	.37	.45	.52		.29	.37	.42	.50	.64	
EFF. DEPTH, IN.	21.8	21.8	21.8	21.7	21.6		21.8	21.7	21.7	21.7	21.6	
-ICR/IGR	.149	.179	.199	.228	.254		.162	.195	.219	.250	.295	
POSITIVE MOMENT STEEL AREA (SQ. IN.)	.62	.75	.88	1.04	1.20		.51	.62	.75	.88	1.04	
STEEL %	.08	.10	.11	.13	.15		.07	.08	.10	.11	.13	
EFF. DEPTH, IN.	21.7	21.6	21.6	21.6	21.6		21.7	21.7	21.6	21.6	21.6	
+ICR/IGR	.160	.190	.220	.255	.290		.134	.160	.190	.220	.255	

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

20" Deep Rib + 3.0" Top Slab = 23.0" Total Depth

TOP BARS Size @	#5 12	#5 11	#5 9.5	#5 8	#6 9.5	End Span Defl. Coeff. (3)	#5 12	#5 9.5	#5 8	#5 7	#6 8	Int. Span Defl. Coeff. (3)
BOTTOM BARS # #	#5 #6	#6 #6	#6 #7	#7 #7	#7 #8		#5 #5	#5 #6	#6 #6	#6 #7	#7 #7	
Steel (psf)	1.03	1.20	1.40	1.63	1.89	(3)	1.11	1.36	1.63	1.88	2.29	(3)
CLEAR SPAN	END SPAN						INTERIOR SPAN					
30'-0"	152 0	201 0	261 0	321 0	376* 390	10.023	212 0	282 0	354 0	433* 440	440* 510*	6.168
31'-0"	134 0	180 0	235 0	292 0	354* 357	11.428	190 0	256 0	323 0	404 0	417* 481*	7.033
32'-0"	117 0	161 0	213 0	265 0	326 0	12.975	170 0	232 0	295 0	371 0	396* 447	7.985
33'-0"	102 0	143 0	192 0	241 0	299 0	14.675	151 0	210 0	269 0	340 0	376* 413	9.031
34'-0"	88 0	127 0	173 0	220 0	274 0	16.536	135 0	190 0	246 0	313 0	358* 381	10.176
35'-0"	76 0	112 0	156 0	200 0	251 0	18.569	120 0	172 0	224 0	288 0	341* 352	11.427
36'-0"	64 0	99 0	140 0	181 0	230 0	20.784	106 0	155 0	205 0	265 0	325 0	12.790
37'-0"	54 0	86 0	125 0	165 0	210 0	23.191	93 0	139 0	187 0	243 0	301 0	14.272
38'-0"	44 0	75 0	112 0	149 0	192 0	25.802	81 0	125 0	170 0	224 0	278 0	15.878
39'-0"		64 0	99 0	135 0	176 0	28.627	70 0	112 0	154 0	205 0	257 0	17.617
40'-0"		54 0	88 0	121 0	160 0	31.678	60 0	100 0	140 0	189 0	238 0	19.494
41'-0"		45 0	77 0	109 0	146 0	34.967	51 0	89 0	127 0	173 0	220 0	21.518
42'-0"			67 0	97 0	133 0	38.505	42 0	78 0	115 0	159 0	203 0	23.695
43'-0"			58 0	87 0	121 0	42.305		68 0	103 0	145 0	188 0	26.034

(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 ≥ ℓ_n/18.5 for end spans, ℓ_n/21 for interior spans).
(4) Exclusive of bridging joists and tapered ends.
*Controlled by shear capacity. +Capacity at elastic deflection = ℓ_n/360.

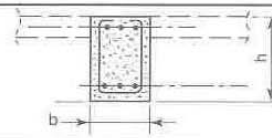
PROPERTIES FOR DESIGN (CONCRETE .64 CF/SF) (4)

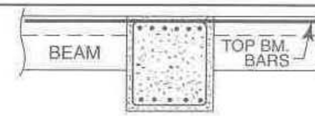
NEGATIVE MOMENT STEEL AREA (SQ. IN.)	.96	1.04	1.21	1.43	1.71		.96	1.21	1.43	1.64	2.04	
STEEL % (UNIFORM)	.50	.55	.63	.75	.90		.50	.63	.75	.86	1.07	
(TAPERED)	.32	.35	.40	.48	.57		.32	.40	.48	.55	.68	
EFF. DEPTH, IN.	21.7	21.7	21.7	21.7	21.6		21.7	21.7	21.7	21.7	21.6	
-ICR/IGR	.174	.186	.209	.239	.272		.174	.209	.239	.265	.309	
POSITIVE MOMENT STEEL AREA (SQ. IN.)	.75	.88	1.04	1.20	1.39		.62	.75	.88	1.04	1.20	
STEEL %	.09	.11	.13	.15	.17		.08	.09	.11	.13	.15	
EFF. DEPTH, IN.	21.6	21.6	21.6	21.6	21.5		21.7	21.6	21.6	21.6	21.6	
+ICR/IGR	.173	.200	.231	.263	.298		.146	.173	.200	.231	.263	

Joist Band Beam – Girder (CRSI 2002)

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

JOIST-BAND BEAMS, END SPANS





STEM		BARS(1)				TOTAL CAPACITY $U = 1.4D + 1.7L$(3)																				$+\phi M_n$ $-\phi M_n$	DEFL (C)
h	b	BOTTOM		Layers	TOP	SPAN, ℓ_n = 20 ft					SPAN, ℓ_n = 22 ft					SPAN, ℓ_n = 24 ft					SPAN, ℓ_n = 26 ft						
in.	in.	ℓ_n + 12 in.	0.875 ℓ_n	(2)		LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	(6) ft-kip	(7) $\times 10^{-9}$ in.
		2#10	1#10	1	4#9	9.2	113H	19	-	546	7.6	123H	19	-	595	6.4	133H	18	-	645	5.4	133H	18	-	685	350	353
				1			125H	76	1.9	858		135H	75	1.9	932		155H	74	1.9	1041		165H	73	1.9	1115	368	
		2#10	1#10	1	4#11	9.6	123H	19	-	671	8.0	123H	19	-	721	6.7	133H	18	-	779	5.7	133H	18	-	828	350	312
				1			125H	76	1.9	974		135H	75	1.9	1058		155H	74	1.9	1175		165H	73	1.9	1256	547	
	24	8#6	7#6	2	4#14	15.4	125H	19	-	1220	12.7	134H	19	-	1194	10.7	154H	18	-	1306	9.1	164H	18	-	1401	560	257
				1			175E	76	1.9	1475		195E	74	1.9	1620		215E	73	1.9	1766		225E	72	1.8	1878	742	
		10#6	9#6	2	5#14	18.9	244C	19	-	1531	15.6	264C	19	-	1663	13.1	155H	18	-	1728	11.2	165H	18	-	1853	685	222
				1			245C	76	1.9	1943		265C	74	1.9	2109		245D	73	1.9	2140		225E	72	1.8	2170	886	
		2#10	2#10	1	5#10	13.0	113H	34	-	751	10.7	123H	34	-	818	9.0	123H	33	-	875	7.7	133H	33	-	942	471	231
				1			175E	137	2.8	1304		195E	135	2.8	1438		155H	133	2.8	1341		523A	131	2.7	1465	580	
		3#10	2#10	1	5#11	16.0	124H	34	-	1035	13.2	134H	34	-	1126	11.1	133H	33	-	1082	9.4	143H	33	-	1163	580	222
				1			175E	137	2.8	1471		195E	135	2.8	1621		215E	133	2.8	1770		225E	131	2.7	1881	697	
24.5	36	4#10	4#10	1	5#14	23.9	135EdH	34	-	1626	19.8	135H	34	-	1735	16.6	155H	33	-	1904	14.2	165H	33	-	2042	885	177
				1			245C	137	2.8	2172		265C	135	2.8	2359		245D	133	2.8	2391		265D	131	2.7	2578	956	
		14#6	14#6	2	7#14	27.8	245C	34	-	2329	23.0	264C	33	-	2244	19.3	165EeH	33	-	2375	16.4	175EdH	33	-	2546	1010	153
				1			405A	136	2.8	3170		445A	134	2.8	3466		295C	132	2.7	3026		315C	131	2.7	3245	1261	
		3#10	3#10	1	6#10	17.5	114H	51	-	1099	14.5	123H	50	-	1081	12.2	123H	49	-	1159	10.4	133H	49	-	1248	701	175
				1			175E	203	3.8	1616		264C	200	3.7	1665		294C	197	3.7	1826		**	195	0.0	1110	701	
		3#10	3#10	1	6#11	19.3	124H	51	-	1212	15.9	124H	50	-	1298	13.4	133H	49	-	1277	11.4	133H	49	-	1362	701	172
				1			305B	203	3.8	2276		195E	200	3.7	1882		294C	197	3.7	1933		314C	195	3.6	2074	845	
	48	6#10	5#10	1	7#14	33.2	155DhH	51	-	2214	27.5	155EfH	50	-	2365	23.1	165EeH	49	-	2549	19.7	175EdH	49	-	2734	1213	129
				1			305B	203	3.8	3038		335B	200	3.7	3320		365B	197	3.7	3603		395B	195	3.6	3886	1329	
		19#6	18#6	2	9#14	36.8	245C	51	-	2889	30.4	265C	50	-	3139	25.6	185DiH	49	-	3065	21.8	185EgH	48	-	3250	1339	116
				1			405A	202	3.8	3859		445A	199	3.7	4218		365B	196	3.6	4054		395B	194	3.6	4369	1633	

(1) See "Recommended Bar Details", Fig. 12-1. For girders, use tabulated beam depth — 2 inches (b — 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 x 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, secondline 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 x (w/1.6) x ℓ_n^4, where w = tabulated load (k/ft.), ℓ_n in ft. "Average service load" is taken as w/1.6.

Appendix D

Alternate Floor System #2
Open Web Steel Joist System

ALTERNATE #2 – OPEN WEB STEEL JOIST SYSTEM

DESIGN AIDS :
- WHEELING DECK PRODUCT CATALOG
- NCJ JOIST CATALOG
- LRFD MANUAL FOR STEEL CONSTRUCTION (3RD EDITION)

FLOOR DECK – NON COMPOSITE (pg. FD-5, Wheeling Deck)

$DL = 45$ psf $\quad LL = 100$ psf

$TL_{service} = 145$ psf

SELECTED: 3" SLAB w/ 6x6 – W2.9 x W2.9

SPAN = 3'-6"

USE → TF50, S3 22GA

$W_{ALLOW} = 189$ psf $> W_u = 145$ actual

Slab wt. = 34 psf

FLOOR JOISTS – OPEN WEB STEEL JOIST (NCJ JOIST, pg. 52)

JOIST spacing = 3'-6"

$DL = 45 + 34 = 79$ psf $(3.5') = 276.5$ plf

$LL = 100$ psf $(3.5') = 350$ plf

$TL = 276.5 + 350 = 626.5$ plf

SELECTED: 20LH07 → depth = 20"

Self wt. = 17 plf

$LL_{allow} = 362$ plf $> LL_{actual} = 350$ plf

$TL_{allow} = 627$ plf $> TL_{actual} = 626.5$ plf

∴ OK

Use 3-rows of Bridging

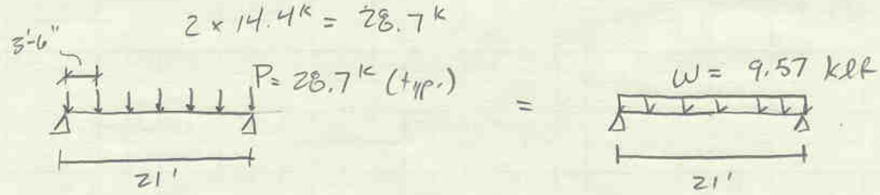
L2x2x1/8 → 10' spacing

ALTERNATE #2 (cont'd) - GIRDER - W-shape

$$W_u = 1.2(276.5 + 17) + 1.6(350) = 912.2 \text{ plf}$$

Reactions @ ends on girder $= \frac{wl}{2} = \frac{912.2(31.5')}{2} = 14.4^k$

$$2 \times 14.4^k = 28.7^k$$



Assume simple span: $M_u = \frac{wl^2}{8} = \frac{9.57(21')^2}{8} = 527.5'^k$

$$\Delta_{max} = \frac{5wl^4}{384EI} \qquad \Delta_{max} = \frac{l}{360} = \frac{21'(12)}{360} = 0.7''$$

$$\to I_{req} = \frac{5wl^4}{384E\Delta_{max}}$$

$$= \frac{5(9.57)(21)^4(12\text{ in/ft})^3}{384(29000)(0.7'')} = 2063 \text{ in}^4$$

Select: W 24×76 → $I = 2100 \text{ in}^4 > I_{req} = 2063 \text{ in}^4$

$\phi M_n = 750'^k > M_u = 527.5'^k$ ∴ ok

Non-Composite Metal Deck (Wheeling Deck Products)

Tensilform/Tensilvent 50, S3

Wheeling Form Deck

Maximum Allowable Unshored Construction Clear Spans

Slab Depth	Type	145 pcf Normal Weight Concrete				115 pcf Lightweight Concrete			
		Slab Wt. - psf	Single Span	Double Span	Triple Span	Slab Wt. - psf	Single Span	Double Span	Triple Span
2-1/2"	28	28	2'-2"	2'-10"	2'-10"	23	2'-3"	2'-11"	2'-11"
	26	28	2'-8"	3'-5"	3'-6"	23	2'-9"	3'-7"	3'-7"
	24	28	3'-6"	4'-7"	4'-7"	23	3'-8"	4'-9"	4'-10"
	22	28	4'-1"	5'-4"	5'-2"	23	4'-3"	5'-7"	5'-7"
3"	28	34	2'-1"	2'-9"	2'-9"	28	2'-2"	2'-10"	2'-10"
	26	34	2'-6"	3'-3"	3'-4"	28	2'-8"	3'-5"	3'-6"
	24	34	3'-4"	4'-4"	4'-5"	28	3'-6"	4'-7"	4'-8"
	22	34	3'-10"	5'-1"	4'-10"	28	4'-1"	5'-4"	5'-3"
3-1/2"	28	41	2'-0"	2'-7"	2'-8"	32	2'-1"	2'-9"	2'-9"
	26	41	2'-5"	3'-2"	3'-3"	32	2'-7"	3'-4"	3'-4"
	24	41	3'-2"	4'-2"	4'-3"	32	3'-4"	4'-5"	4'-6"
	22	41	3'-8"	4'-10"	4'-7"	32	3'-11"	5'-2"	4'-11"
4"	28	47	1'-11"	2'-7"	2'-7"	37	2'-1"	2'-8"	2'-8"
	26	47	2'-4"	3'-1"	3'-1"	37	2'-6"	3'-3"	3'-3"
	24	47	3'-1"	4'-0"	4'-1"	37	3'-3"	4'-3"	4'-4"
	22	47	3'-6"	4'-8"	4'-5"	37	3'-9"	5'-0"	4'-9"
4-1/2"	28	53	1'-11"	2'-6"	2'-6"	42	2'-0"	2'-7"	2'-8"
	26	53	2'-3"	3'-0"	3'-0"	42	2'-5"	3'-2"	3'-2"
	24	53	2'-11"	3'-11"	3'-11"	42	3'-2"	4'-2"	4'-2"
	22	53	3'-4"	4'-6"	4'-3"	42	3'-7"	4'-10"	4'-7"
5"	28	59	1'-10"	2'-5"	2'-5"	47	1'-11"	2'-7"	2'-7"
	26	59	2'-2"	2'-11"	2'-11"	47	2'-4"	3'-1"	3'-1"
	24	59	2'-10"	3'-9"	3'-9"	47	3'-1"	4'-1"	4'-1"
	22	59	3'-3"	4'-4"	4'-1"	47	3'-6"	4'-8"	4'-4"

Allowable Uniform Superimposed Loads for Reinforced Concrete Slabs - psf

Slab Depth	Reinforcement		Three Span Condition - Center to Center						
	W.W.R.	A_s (in²/ft)	2'-0"	2'-6"	3'-0"	3'-6"	4'-0"	4'-6"	5'-0"
2-1/2"	6x6-W1.4xW1.4	0.028*	249	151	98	66	45	31	
	6x6-W2.0xW2.0	0.040*	362	223	148	103	73	53	38
	6x6-W2.9xW2.9	0.058*	400	329	221	156	114	85	65
3"	6x6-W1.4xW1.4	0.028*	299	182	118	79	54	37	
	6x6-W2.0xW2.0	0.040*	400	269	178	124	88	64	46
	6x6-W2.9xW2.9	0.058	400	397	267	189	138	103	78
3-1/2"	6x6-W2.0xW2.0	0.040*	400	314	208	193	140	103	77
	6x6-W2.9xW2.9	0.058*	400	400	313	286	211	159	123
	6x6-W4.0xW4.0	0.080	400	400	400	392	292	224	175
4"	6x6-W2.9xW2.9	0.058*	400	400	359	356	263	200	155
	6x6-W4.0xW4.0	0.080	400	400	400	400	367	282	221
	4x4-W2.9xW2.9	0.087	400	400	400	400	400	315	248
4-1/2"	6x6-W4.0xW4.0	0.080*	400	400	400	400	400	339	267
	4x4-W2.9xW2.9	0.087	400	400	400	400	400	379	299
	4x4-W4.0xW4.0	0.120	400	400	400	400	400	400	400
5"	6x6-W4.0xW4.0	0.080*	400	400	400	400	400	397	313
	4x4-W2.9xW2.9	0.087*	400	400	400	400	400	400	349
	4x4-W4.0xW4.0	0.120	400	400	400	400	400	400	400

*A_s does not meet A.C.I. criteria for temperature and shrinkage reinforcement (0.0018Ac)

Open Web Steel Joist (NCJ Joist Catalog)

STANDARD LOAD TABLE

LONGSPAN STEEL JOISTS, LH-SERIES

Based on a Maximum Allowable Tensile Stress of 30 ksi
Adopted by the Steel Joist Institute May 25, 1983;
Revised to May 1, 2000 – Effective August 1, 2002

The black figures in the following table give the TOTAL safe uniformly distributed load-carrying capacities, in pounds per linear foot, of **LH-Series** Steel Joists. The weight of DEAD loads, including the joists, must in all cases be deducted to determine the LIVE load-carrying capacities of the joists. The approximate DEAD load of the joists may be determined from the weights per linear foot shown in the tables.

The RED figures in this load table are the LIVE loads per linear foot of joist which will produce an approximate deflection of 1/360 of the span. LIVE loads which will produce a deflection of 1/240 of the span may be obtained by multiplying the RED figures by 1.5. In no case shall the TOTAL load capacity of the joists be exceeded.

This load table applies to joists with either parallel chords or standard pitched top chords. When top chords are pitched, the carrying capacities are determined by the nominal depth of the joists at the center of the span. Standard top chord pitch is 1/8 inch per foot. If pitch exceeds this standard, the load table does not apply. Sloped parallel-chord joists shall use span as defined by the length along the slope.

Where the joist span is in the RED SHADED area of the load table, the row of bridging nearest the midspan shall be diagonal bridging with bolted connections at chords and intersection. Hoisting cables shall not be released until this row of bolted diagonal bridging is completely installed.

Where the joist span is in the BLUE SHADED area of the load table, all rows of bridging shall be diagonal bridging with bolted connections at chords and intersection. Hoisting cables shall not be released until the two rows of bridging nearest the third points are completely installed.

The approximate moment of inertia of the joist, in inches[4] is;

$I_j = 26.767(W_{LL})(L^3)(10^{-6})$, where W_{LL} = RED figure in the Load Table, and L = (clear span + .67) in feet.

When holes are required in top or bottom chords, the carrying capacities must be reduced in proportion to the reduction of chord areas.

The top chords are considered as being stayed laterally by floor slab or roof deck.

The approximate joist weights per linear foot shown in these tables do not include accessories.

Joist Designation	Approx. Wt in Lbs. Per Linear Ft (Joists only)	Depth in inches	SAFE LOAD* in Lbs. Between	CLEAR SPAN IN FEET															
			21-24	25	26	27	28	29	30	31	32	33	34	35	36				
18LH02	10	18	12000	468	442	418	391	367	345	324	306	289	273	259	245				
				313	284	259	234	212	193	175	160	147	135	124	114				
18LH03	11	18	13300	521	493	467	438	409	382	359	337	317	299	283	267				
				348	317	289	262	236	213	194	177	161	148	136	124				
18LH04	12	18	15500	604	571	535	500	469	440	413	388	365	344	325	308				
				403	367	329	296	266	242	219	200	182	167	153	141				
18LH05	15	18	17500	684	648	614	581	543	508	476	448	421	397	375	355				
				454	414	378	345	311	282	256	233	212	195	179	164				
18LH06	15	18	20700	809	749	696	648	605	566	531	499	470	443	418	396				
				526	469	419	377	340	307	280	254	232	212	195	180				
18LH07	17	18	21500	840	809	780	726	678	635	595	559	526	496	469	444				
				553	513	476	428	386	349	317	288	264	241	222	204				
18LH08	19	18	22400	876	843	812	784	758	717	680	641	604	571	540	512				
				577	534	496	462	427	387	351	320	292	267	246	226				
18LH09	21	18	24000	936	901	868	838	810	783	759	713	671	633	598	566				
				616	571	527	491	458	418	380	346	316	289	266	245				
			22-24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
20LH02	10	20	11300	442	437	431	410	388	365	344	325	307	291	275	262	249	237	225	215
				306	303	298	274	250	228	208	190	174	160	147	136	126	117	108	101
20LH03	11	20	12000	469	463	458	452	434	414	395	372	352	333	316	299	283	269	255	243
				337	333	317	302	280	258	238	218	200	184	169	156	143	133	123	114
20LH04	12	20	14700	574	566	558	528	496	467	440	416	393	372	353	335	318	303	289	275
				428	406	386	352	320	291	265	243	223	205	189	174	161	149	139	129
20LH05	14	20	15800	616	609	602	595	571	544	513	484	458	434	411	390	371	353	336	321
				459	437	416	395	366	337	308	281	258	238	219	202	187	173	161	150
20LH06	15	20	21100	822	791	763	723	679	635	596	560	527	497	469	444	421	399	379	361
				606	561	521	477	427	386	351	320	292	267	246	226	209	192	178	165
20LH07	17	20	22500	878	845	814	786	760	711	667	627	590	556	526	497	471	447	425	404
				647	599	556	518	484	438	398	362	331	303	278	256	236	218	202	187
20LH08	19	20	23200	908	873	842	813	785	760	722	687	654	621	588	558	530	503	479	457
				669	619	575	536	500	468	428	395	365	336	309	285	262	242	225	209
20LH09	21	20	25400	990	953	918	886	856	828	802	778	755	712	673	636	603	572	544	517
				729	675	626	581	542	507	475	437	399	366	336	309	285	264	244	227
20LH10	23	20	27400	1068	1028	991	956	924	894	865	839	814	791	748	707	670	636	604	575
				786	724	673	626	585	545	510	479	448	411	377	346	320	296	274	254

Appendix E

Alternate Floor System #3
Composite Beam/Composite Slab

ALTERNATE # 3 - COMPOSITE BEAM / COMP. SLAB

DESIGN AIDS: • Wheeling Deck Product Catalog
• LRFD Manual for STEEL Construction (3rd Edition)

FLOOR DECK: COMPOSITE (pg. CD-4, Wheeling Deck)

$DL = 45$ psf $LL = 100$ psf

$TL = 1.2(45) + 1.6(100)$
$= 214$ psf

SELECTED: 4" SLAB w/ 6×6 − W1.4×W1.4 (pg. CD-13)
SPAN = 7'-0"

USE → 1.5 SB NORMAL WT.
$W_{ALLOW} = 292$ psf $> W_{actual} = 214$ psf ∴ OK

Slab wt. = 36.3 psf
$A_c = 20.6 \text{ in}^2$

COMPOSITE BEAM

$W_u = 1.2(45 + 36.3) + 1.6(100) = 257.6 \text{ psf}(7') = 1.803 \text{ klf}$

$M_u = \frac{wl^2}{8} = \frac{1.803(31.5^2)}{8} = 223.6 ^{\prime k}$

$b_{eff} = \begin{cases} L/4 = \frac{31.5(12)}{4} = 94.5" \\ \text{spacing} = 7' = 84" \leftarrow \text{controls} \end{cases}$

$d_c = 4"$. Assume $a = 2"$ $Y_2 = 4 - \frac{2}{2} = 3.0"$

(Table 5-14, LRFD) TRY W16×26 ⇒ $A_s = 7.68 \text{ in}^2$
$\phi M_n = 285 ^{\prime k} > M_u = 223.6$

$C_{conc} = 0.85 f'_c bd = 0.85(4)(84)(2.5) = 714^k$ $T_{steel} = A_s F_y = 7.68(50) = 384^k \leftarrow$ controls

∴ p.n.a. is in the concrete

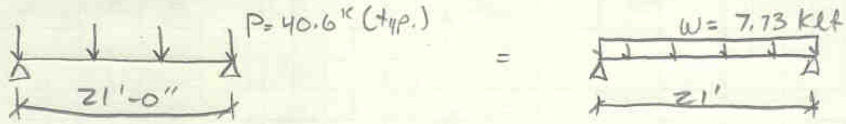
$a = \frac{A_s F_y}{0.85 f'_c b} = \frac{7.68(50)}{0.85(4)(84)} = 1.34" < 4"$ ∴ p.n.a is in concrete

$\phi M_n = 0.9(384(\frac{15.7}{2}) + 384(4 - \frac{1.34}{2})) = 289.8 ^{\prime k} > M_u = 223.6 ^{\prime k}$ ∴ OK

$\Sigma Q_n = 337$, $Q_n = 26.1$ → # studs $= \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{337}{26.1} \times 2 =$ 26 studs

ALTERNATE #3 (cont'd) - GIRDER - W-SHAPE

Reaction from beams = $\frac{wl}{2} = \frac{1.803(31.5)}{2} + \frac{1.803(13.5)}{2} = 40.6^k$



$M_u = \frac{wl^2}{8} = \frac{7.73(21)^2}{8} = 426.1'^k$ $\qquad b_{eff} = \frac{l}{4} = \frac{21(12)}{4} = 63''$

$d_c = 4''$; assume $a = 2''$

$y_2 = 4 - \frac{2}{2} = 3.0''$

(Table 5-14, LRFD) Try W18x40 $\rightarrow A = 11.8\ in^2$

$b_f = 6.02$

$\phi M_n = 482'^k > M_u = 426.1'^k$ $\qquad t_w = 0.315$, $t_f = 0.525$

$d = 17.9''$

$C_{conc.} = 0.85 f'_c bd = 0.85(4)(63)(2.5)$
$= 535.5$ ← controls

$T_{steel} = A_s F_y = 11.8(50) = 590^k$ ∴ p.n.a. is in steel, assume it's in the flange

$\frac{535.5^k}{50\ ksi} = 10.71\ in^2$ $\qquad 10.71 - 6.02(0.525) = 7.54$

$0.315(17.9 - [2(0.525)]) = 5.31$

$7.54 - 5.31 = 2.23\ in^2 \rightarrow \frac{2.23\ in^2}{6.02''} = 0.37''$

∴ p.n.a. is in steel flange

$P_u = 590 - 535.5 = 54.5$

$M_p = 535.5(4 - \frac{2.5}{2}) + 590(\frac{17.9}{2}) + 54.5(\frac{0.37}{2}) = 561.9'^k$

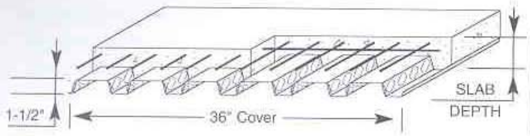
$\phi M_n = 0.9(561.9) = 505'^k > M_u = 426.1'^k$ ∴ ok

$\Sigma Q_n = 511 \Rightarrow$ # studs $= \frac{511}{26.1}(2) = 40$ shear studs

Composite Metal Deck (Wheeling Deck Products)

1.5 SB Normal Weight

Wheeling Composite Deck



Section Properties (per ft. of width)

Gage	t in	Wd psf	Sp in^3	Sn in^3	Ip in^4	In in^4	As in^2	Fy ksi
22	0.0295	1.7	0.172	0.180	0.146	0.182	0.478	50
20	0.0358	2.0	0.218	0.229	0.190	0.221	0.581	50
18	0.0474	2.7	0.301	0.311	0.284	0.294	0.769	40
16	0.0600	3.4	0.388	0.394	0.374	0.373	0.973	40

145 pcf Normal Weight Concrete

Total Slab Depth D / Wt. Conc. / Area Conc.	Gage	Maximum Unshored Clear Spans: Single Span	Double Span	Triple Span	Composite Properties: Iavg in^4/ft	Sc in^3/ft	Superimposed Live Loads - psf: No Studs — Span - Feet and Inches: 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
4" 36.3 psf 20.6 in^2	22	5'-10"	7'-9"	7'-11"	3.573	0.887	400	343	292	251	217	189	166	146	129	114	101	90
	20	6'-9"	9'-0"	9'-2"	3.854	1.052	400	400	352	303	262	229	201	178	158	140	125	111
	18	7'-2"	9'-5"	9'-8"	4.333	1.345	400	400	360	310	269	235	206	182	161	142	128	115
	16	8'-4"	10'-6"	10'-11"	4.782	1.638	400	400	360	310	269	235	206	182	161	142	128	115
4-1/2" 42.4 psf 24.8 in^2	22	5'-6"	7'-5"	7'-6"	5.107	1.087	400	400	360	309	268	233	205	180	160	142	126	113
	20	6'-4"	8'-7"	8'-8"	5.496	1.291	400	400	400	373	324	283	249	220	195	174	156	140
	18	6'-9"	8'-11"	9'-3"	6.160	1.653	400	400	400	383	332	290	255	226	200	179	160	143
	16	7'-10"	10'-0"	10'-4"	6.789	2.018	400	400	400	383	332	290	255	226	200	179	160	143
5" 48.4 psf 29.3 in^2	22	5'-3"	7'-1"	7'-2"	7.022	1.293	400	400	400	370	320	279	245	216	191	170	152	136
	20	6'-1"	8'-2"	8'-4"	7.544	1.538	400	400	400	400	388	339	298	264	235	209	187	168
	18	6'-5"	8'-6"	8'-9"	8.431	1.972	400	400	400	400	398	348	307	271	241	215	193	173
	16	7'-6"	9'-6"	9'-10"	9.280	2.415	400	400	400	400	398	348	307	271	241	215	193	173
5-1/2" 54.4 psf 34.1 in^2	22	5'-0"	6'-9"	6'-10"	9.360	1.503	400	400	400	400	374	326	287	253	224	199	178	159
	20	5'-10"	7'-10"	7'-11"	10.036	1.791	400	400	400	400	400	397	349	309	275	245	220	197
	18	6'-2"	8'-2"	8'-5"	11.187	2.301	400	400	400	400	400	400	360	318	283	253	227	204
	16	7'-2"	9'-2"	9'-5"	12.298	2.824	400	400	400	400	400	400	360	318	283	253	227	204
6" 60.5 psf 39.4 in^2	22	4'-10"	6'-6"	6'-7"	12.157	1.717	400	400	400	400	400	374	329	290	258	229	205	183
	20	5'-7"	7'-6"	7'-8"	13.012	2.048	400	400	400	400	400	400	400	355	316	282	253	227
	18	5'-11"	7'-10"	8'-1"	14.468	2.636	400	400	400	400	400	400	400	366	326	291	261	235
	16	6'-10"	8'-9"	9'-1"	15.883	3.242	400	400	400	400	400	400	400	366	326	291	261	235

D, Wc, Ac	Gage	Single Span	Double Span	Triple Span	Stud Factors: 2' o.c.	3' o.c.	Superimposed Live Loads - psf: Studs @ 1'-0" O.C. — Span - Feet and Inches: 6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"
4" 36.3 psf 20.6 in^2	22	5'-10"	7'-9"	7'-11"	0.91	0.84	400	400	400	371	305	255	215	182	156	135	118	103
	20	6'-9"	9'-1"	9'-2"	0.88	0.82	400	400	400	400	329	275	231	197	169	146	127	111
	18	7'-2"	9'-5"	9'-5"	0.86	0.80	400	400	400	400	370	309	260	221	190	164	142	125
	16	8'-4"	10'-6"	10'-11"	0.83	0.78	400	400	400	400	400	341	287	244	209	181	157	138
4-1/2" 42.4 psf 24.8 in^2	22	5'-6"	7'-5"	7'-6"	0.92	0.85	400	400	400	400	387	339	299	261	224	193	168	147
	20	6'-4"	8'-7"	8'-8"	0.89	0.83	400	400	400	400	400	392	330	281	241	208	181	158
	18	6'-9"	8'-11"	9'-3"	0.87	0.81	400	400	400	400	400	400	370	314	270	233	203	177
	16	7'-10"	10'-0"	10'-4"	0.84	0.79	400	400	400	400	400	400	400	347	297	257	223	195
5" 48.4 psf 29.3 in^2	22	5'-3"	7'-1"	7'-2"	0.93	0.86	400	400	400	400	400	393	347	307	274	245	220	198
	20	6'-1"	8'-2"	8'-4"	0.89	0.84	400	400	400	400	400	400	400	371	330	285	248	217
	18	6'-5"	8'-6"	8'-9"	0.88	0.82	400	400	400	400	400	400	400	392	350	314	277	243
	16	7'-6"	9'-6"	9'-10"	0.85	0.81	400	400	400	400	400	400	400	400	400	351	305	267
5-1/2" 54.4 psf 34.1 in^2	22	5'-0"	6'-9"	6'-10"	0.93	0.87	400	400	400	400	400	400	395	350	312	279	250	225
	20	5'-10"	7'-10"	7'-11"	0.90	0.85	400	400	400	400	400	400	400	400	378	339	305	276
	18	6'-2"	8'-2"	8'-5"	0.88	0.83	400	400	400	400	400	400	400	400	400	359	323	292
	16	7'-2"	9'-2"	9'-5"	0.85	0.82	400	400	400	400	400	400	400	400	400	400	400	354
6" 60.5 psf 39.4 in^2	22	4'-10"	6'-6"	6'-7"	0.94	0.88	400	400	400	400	400	400	400	392	349	313	281	253
	20	5'-7"	7'-6"	7'-8"	0.91	0.86	400	400	400	400	400	400	400	400	400	381	343	310
	18	5'-11"	7'-10"	8'-1"	0.89	0.84	400	400	400	400	400	400	400	400	400	400	363	328
	16	6'-10"	8'-9"	9'-1"	0.86	0.83	400	400	400	400	400	400	400	400	400	400	400	400

1) Refer to the Design Notes, Note 7 for information on live load limits for fire-rated construction. See Page CD-3.
2) If stud spacing exceeds 1'-0" o.c., reduce live load by applicable stud factor listed above for actual stud spacing.
3) If welded wire fabric is not used, the live loads should be reduced by 10%.

Appendix F

Alternate Floor System #4
Double Tee Beam System

ALTERNATE #4 - DOUBLE TEE BEAMS

DESIGN AIDS: PCI MANUAL

$W_u = 45 + 100 = 145$ psf (service)

SPAN = 31'-5" → use 32'

↳ SELECTED: 8LDT24 + 2

$f'_c = 5000$ psi
$f_{pu} = 270$ ksi

DOUBLE TEE

8' x 24" Lt. wt concrete
w/ 2" Topping

self wt. = 40 psf = 320 plf

Section: 68-S ; $e_o = e_c = 11.15$"

$W_{allow} = 157 > W_{service} = 145$ psf ∴ ok

estimated camber @ erection = 0.9"
estimated long term camber = 0.9"

GIRDER

$W_u = 45 + 40 + 100 = 185 \text{ psf} \left(\frac{31.5}{2} + \frac{13.5}{2}\right) = 4162.5$ plf
= 4.163 klf

SPAN = 21' → use 22'

(PCI, pg. 2-44) use 28IT24 Inverted Tee Beam

$W_{allow} = 4925$ plf > $W_{actual} = 4162.5$ plf

(11) - ½" diameter low-lax strands

$e = 6.77$"

self wt. = 500 plf   depth = 24"

Double Tee Beam Load Table (PCI Design Handbook)

Strand Pattern Designation

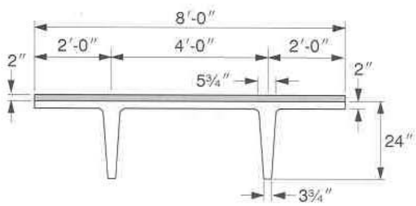
108-D1
- 10 — No. of strand (10)
- 8 — Diameter of strand in 16ths
- D — S = straight D = depressed
- 1 — No. of depression points

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Key
- 118 — Safe superimposed service load, psf
- 1.1 — Estimated camber at erection, in.
- 1.4 — Estimated long-time camber, in.

DOUBLE TEE

8′-0″ x 24″
Lightweight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi

Section Properties

		Untopped		Topped	
A	=	401	in²	—	
I	=	20,985	in⁴	29,857	in⁴
y_b	=	17.15	in.	19.94	in.
y_t	=	6.85	in.	6.06	in.
S_b	=	1,224	in³	1,497	in³
S_t	=	3,063	in³	4,926	in³
wt	=	320	plf	520	plf
		40	psf	65	psf
V/S	=	1.41	in.		

8LDT24

Table of safe superimposed service load (psf) and cambers (in.) — No Topping

Strand Pattern	e_e, in. e_c, in.	Span, ft 36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80
68-S	11.15 11.15	118 1.1 1.4	103 1.2 1.4	89 1.2 1.5	78 1.3 1.5	69 1.3 1.5	60 1.4 1.5	53 1.4 1.4	46 1.4 1.4	40 1.3 1.2	35 1.3 1.0	30 1.2 0.8												
88-S	9.15 9.15	144 1.2 1.6	126 1.3 1.6	110 1.4 1.7	97 1.5 1.8	86 1.6 1.8	76 1.6 1.8	67 1.6 1.8	59 1.6 1.7	53 1.6 1.6	47 1.6 1.5	41 1.6 1.3	36 1.5 1.0	32 1.4 0.7										
88-D1	9.15 14.40		176 1.9 2.4	156 2.1 2.6	139 2.2 2.8	124 2.4 2.9	111 2.5 3.0	99 2.7 3.1	89 2.8 3.2	80 2.9 3.2	72 3.0 3.2	64 3.0 3.1	58 3.1 2.9	52 3.1 2.8	47 3.0 2.6	42 3.0 2.4	38 2.9 2.1	34 2.8 1.8						
108-D1	7.15 14.15								113 3.3 3.9	102 3.4 4.0	93 3.6 4.1	84 3.7 4.1	76 3.8 4.1	69 3.9 4.0	63 4.0 3.9	57 4.1 3.8	52 4.1 3.5	47 4.0 3.2	43 3.9 2.8	38 3.8 2.4	35 3.6 1.9	31 3.4 1.3		
128-D1	5.48 13.9																65 4.8 4.7	59 4.9 4.5	54 4.9 4.2	50 4.9 3.8	45 4.8 3.4	41 4.7 2.8	38 4.5 2.2	34 4.3 1.5
148-D1	4.29 13.65																						46 5.6 3.8	42 5.5 3.2

8LDT24+2

Table of safe superimposed service load (psf) and cambers (in.) — 2″ Normal Weight Topping

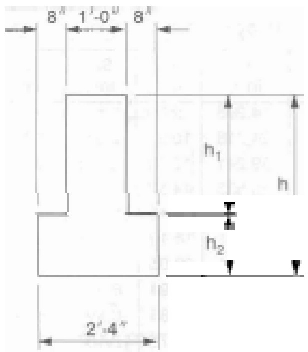
Strand Pattern	e_e, in. e_c, in.	Span, ft 26	28	30	32	34	36	38	40	42	44	46	48	50	52	54	56	58	60	62	64	66	68
48-S	14.15 14.15	193 0.6 0.6	159 0.6 0.6	132 0.7 0.7	110 0.8 0.7	92 0.8 0.7	76 0.9 0.6	63 0.9 0.6	52 1.0 0.5	43 1.0 0.4	34 1.0 0.2												
68-S	11.15 11.15			185 0.9 0.9	157 0.9 0.9	133 1.0 0.9	113 1.1 0.9	96 1.2 0.9	82 1.2 0.9	70 1.3 0.8	59 1.3 0.7	50 1.4 0.5	41 1.4 0.3	34 1.4 0.0									
68-D1	11.15 14.65					166 1.2 1.2	143 1.4 1.3	123 1.5 1.3	106 1.6 1.3	91 1.7 1.3	79 1.8 1.2	68 1.9 1.1	58 1.9 0.9	49 2.0 0.7	42 2.0 0.5	35 2.0 0.2							
88-D1	9.15 14.40						200 1.8 1.8	175 1.9 1.9	153 2.1 1.9	134 2.1 1.8	117 2.4 2.0	103 2.5 1.9	90 2.7 1.9	79 2.8 1.7	70 2.9 1.6	61 3.0 1.3	53 3.0 1.0	46 3.1 0.6	40 3.1 0.2				
108-D1	7.15 14.15													106 3.3 2.4	94 3.4 2.3	84 3.6 2.1	74 3.7 1.9	66 3.8 1.6	58 3.9 1.3	51 4.0 0.8	45 4.1 0.3		
128-D1	5.48 13.90																					53 4.8 0.7	46 4.9 0.1

Strength based on strain compatibility; bottom tension limited to $12\sqrt{f'_c}$; see pages 2-2–2-6 for explanation.
Shaded values require release strengths higher than 3500 psi.

Inverted Tee Beam – Girder Design (PCI Design Handbook)

INVERTED TEE BEAMS

Normal Weight Concrete



f'_c = 5,000 psi

f_{pu} = 270,000 psi

½ in. diameter

low-relaxation strand

Section Properties								
Designation	h in.	h_1/h_2 in.	A in²	I in⁴	y_b in.	S_b in³	S_t in³	wt plf
28IT20	20	12/8	368	11,688	7.91	1,478	967	383
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	500
28IT28	28	16/12	528	32,076	11.09	2,892	1,897	550
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	600
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	650
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	767
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	817
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	867
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,017

1. Check local area for availability of other sizes.

2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore additional top reinforcement is required.

3. Safe loads can be significantly increased by use of structural composite topping.

Key
6,929 — Safe superimposed service load, plf
0.3 — Estimated camber at erection, in.
0.1 — Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers

Desig-nation	No. Strand	e	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	9	5.82	6929 0.3 0.1	5402 0.3 0.1	4310 0.4 0.1	3502 0.4 0.1	2887 0.5 0.1	2409 0.6 0.1	2029 0.6 0.0	1723 0.7 0.0	1473 0.7 0.0	1265 0.8 −0.1	1091 0.8 −0.1							
28IT24	11	6.77	9714 0.2 0.1	7580 0.3 0.1	6054 0.3 0.1	4925 0.4 0.1	4066 0.4 0.1	3398 0.5 0.1	2868 0.6 0.1	2440 0.6 0.1	2090 0.7 0.1	1799 0.7 0.0	1556 0.7 0.0	1351 0.8 0.0	1175 0.8 −0.1	1024 0.8 −0.2				
28IT28	13	8.44			8505 0.3 0.1	6951 0.4 0.1	5768 0.5 0.1	4848 0.5 0.2	4118 0.6 0.2	3529 0.7 0.2	3047 0.7 0.2	2648 0.8 0.2	2313 0.9 0.2	2030 0.9 0.2	1788 1.0 0.2	1579 1.0 0.1	1399 1.1 0.1	1242 1.1 0.1	1103 1.1 0.0	981 1.1 −0.1
28IT32	15	9.17				9202 0.3 0.1	7646 0.4 0.1	6435 0.4 0.1	5474 0.5 0.1	4698 0.5 0.1	4064 0.6 0.1	3538 0.6 0.1	3097 0.7 0.1	2724 0.7 0.1	2406 0.8 0.1	2132 0.8 0.1	1894 0.9 0.0	1687 0.9 0.0	1505 0.9 0.0	1345 0.9 −0.1
28IT36	16	10.81						8485 0.4 0.1	7236 0.4 0.1	6227 0.5 0.1	5402 0.5 0.1	4718 0.6 0.1	4145 0.6 0.1	3660 0.7 0.1	3246 0.7 0.1	2890 0.8 0.1	2581 0.8 0.0	2311 0.9 0.0	2075 0.9 0.0	1866 0.9 −0.1
28IT40	19	11.28							8615 0.4 0.1	7415 0.4 0.1	6433 0.5 0.1	5620 0.5 0.1	4938 0.6 0.1	4361 0.6 0.1	3868 0.7 0.1	3444 0.7 0.1	3077 0.8 0.1	2756 0.8 0.1	2475 0.8 0.1	2226 0.9 0.0
28IT44	20	12.89								9308 0.4 0.1	8092 0.5 0.1	7083 0.5 0.1	6239 0.5 0.1	5524 0.6 0.1	4913 0.6 0.1	4388 0.7 0.1	3932 0.7 0.1	3535 0.8 0.1	3186 0.8 0.1	2879 0.8 0.0
28IT48	22	14.16									9741 0.4 0.1	8539 0.5 0.1	7532 0.5 0.1	6680 0.6 0.1	5952 0.6 0.1	5326 0.7 0.1	4783 0.7 0.1	4310 0.8 0.1	3894 0.8 0.1	3528 0.8 0.1
28IT52	24	15.44											8935 0.5 0.1	7934 0.5 0.1	7080 0.6 0.1	6345 0.6 0.1	5707 0.7 0.1	5151 0.7 0.1	4664 0.8 0.1	4233 0.8 0.1
28IT56	26	16.74												9284 0.5 0.1	8294 0.6 0.1	7442 0.6 0.1	6703 0.7 0.1	6059 0.7 0.1	5493 0.8 0.1	4994 0.8 0.1
28IT60	28	18.04													9590 0.6 0.1	8613 0.6 0.2	7766 0.6 0.2	7027 0.7 0.2	6379 0.7 0.2	5807 0.8 0.2