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Technical Report #2: Pro-Con Structural Study of Alternate Floor Systems
Submittal Date: 31 October 2005

Executive Summary

Technical Report #2 is a basic overview of the main floor system of the Erie Convention Center and Sheraton Hotel, along with a comparison of this existing system to four other alternative floor systems. The proposed floor system is a steel framing structure with 8" hollow-core precast concrete plank. Through the use of RAM Structural System Design, the CRSI Design Manual (2002), and hand calculations, I have analyzed and designed the members for the following four floor systems:

- Composite steel beams with composite steel deck
- Non-composite steel beams with form deck
- Open web steel joists with form deck
- One-way concrete pan joist

With each system, I compared the floor sandwich depth, weight, vibrations, time, and cost concerns with each other and with the existing system. From this analysis, I found that the existing system has the quickest erection time due to the use of precast concrete. The 8" plank will minimize the vibrations greatly, meeting serviceability requirements. Even though the floor sandwich is very large in comparison to the other systems, the difference in time and cost outweighs the benefits offered by this factor.

Other viable options are the composite system, the non-composite system, and the steel joist system. These structures are much lighter than the existing system, however vibrations for the non-composite and joist systems must be taken into consideration because of the thin slabs. The one-way concrete system is not a feasible option because of the on-site time for forming, pouring, finishing, and curing the concrete, as well as the greatly increased weight on the foundation and large girders needed.

Introduction:

The proposed Erie Convention Center and Sheraton Hotel is a 132,000 sq. ft., eleven story hotel and conference center, located on the Presque Isle Bay in Erie, Pennsylvania. The framing system of the hotel is comprised of a steel structure with a hollow core precast concrete plank system to resist the gravity loads. Laterally, the structure is fully restrained in the East/West (E/W) direction, and partially restrained by cross bracing and knee braces in the North/South (N/S) direction. The foundation is comprised of caissons drilled 3 feet into the bedrock, supporting grade beams and an 8" structural concrete slab.

The loading used to design and analyze the structural system is as follows:

Dead Loads: (Assumed)

- Framing members = 10 psf
- 8" Hollow core precast concrete plank = 56 psf
  (weight given by engineer)
- Metal Stud Walls with 5/8" gypsum wall board = 10 psf
- MEP = 10 psf
- Carpet = 1 psf
- Ceiling Finishing = 1 psf

Total 88 psf

Live Loads: (IBC 2003)

- Public Rooms and Corridors =100 psf
- Private Rooms and Corridors = 40 psf
- Mechanical Spaces =150 psf
- Stairs =100 psf
- Ground Snow Load = 30 psf
  p_f=16.8 psf

Live loads are reducible

Existing Structure:

The existing floor system for the Erie Convention Center and Sheraton Hotel is composed of steel framing members that support 8-inch hollow core precast concrete plank panels without a topping. The steel framing members are A992 Grade 50, and the hollow core precast plank is composed of 5000 psi concrete.

A typical floor in the high-rise portion of the building is shown in Figure 2.1, with the bays that I will be analyzing highlighted in red.

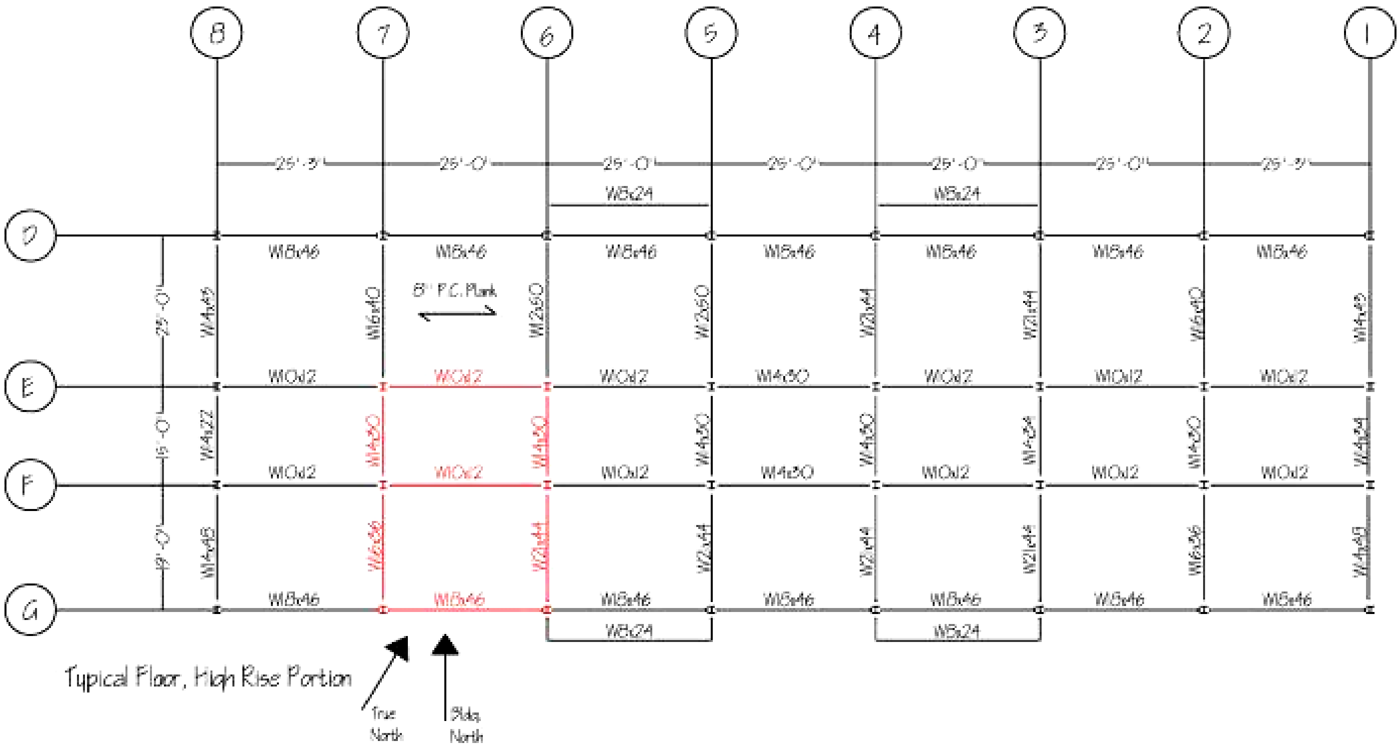


Figure 2.1
This is the simplified version of a typical floor for the high rise portion of the Erie Convention Center and Sheraton Hotel. I have chosen to analyze one interior 25’×15’ bay with an adjacent exterior 25’×19’ bay. W14×30s in the N/S direction support the gravity loads from the hollow core pre-cast concrete plank which runs in the E/W direction. Spanning in the E/W direction are W10×12s for column stability.

After examining the details given in the structural drawings, I found that the plank is attached directly to the top of the steel floor beams. The deepest floor beam in these two bays that I am analyzing is 21 inches, thus with the 8 inch concrete plank, a total depth of 29 inches will be reached.

A steel structure with hollow core concrete plank is a common framing system for hotels. First of all, hollow core precast concrete plank is much lighter than a solid concrete slab. The need for steel beams only between columns shortens the steel erection time, and since the majority of the system is precast, the construction time is shortened further. Hollow core concrete plank is easy to install and lighter than a full concrete slab and deck system. Also, fireproofing is only required for the steel members, and vibrations are reduced because of the thickness of the concrete. The current trend in hotel design is the use of hollow core precast concrete plank without a topping because of the savings in cost as well as weight. For example, the pre-cast in this project is estimated at $400,000. This value would be greatly increased if topping were added. In addition a topping would increase the weight of the plank from 56 psf to 81 psf.

Alternative Floor Systems:

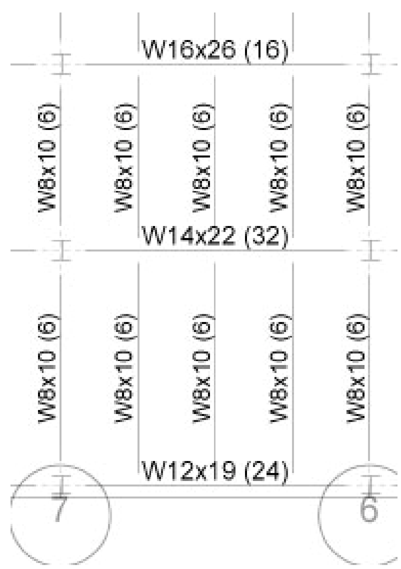
Using the two interior bays highlighted above, I will analyze four different alternative floor systems. These systems include:

1. Composite Steel Beams with Composite Deck
2. Non-composite Steel Beams with Deck
3. Steel Joists with Non-Composite Steel Deck
4. One-way concrete system

Alternative #1- Composite Steel Beams with Composite Deck:

The first option that I chose to analyze is a composite steel beam and deck system. I spaced beams at 6’-4” on center in the N/S direction with composite steel deck spanning in the E/W direction. To support the gravity loads, I used USD 22 Gage 2” Lok-Floor Composite Deck with a 4 ½” total slab depth. The deck has a material strength of 33 ksi, while the concrete has a strength of 3 ksi. The welded wire fabric needed was found to be 6×6-W4.0×W4.0 with a strength of 60 ksi. The member sizes, found using RAM, can be seen in Figure 2.2. Deflections are accounted for in RAM to meet L/360 (live load), and L/240 (total load).

Figure 2.2
The member sizes are shown here
for a composite beam and deck system,
found using RAM Structural System Design.



The depth of the system is the 16 inch floor beam plus the 4.5” total slab depth totaling 20.5 inches. Because there are more members to support the loads in the composite system than the existing system, the members can be smaller, therefore making the floor sandwich smaller. This thinner slab could cause for more vibrations, however, the steel and concrete working together in a composite action allows for the larger loads to be carried with fewer vibrations. Another advantage of composite systems is that the steel deck is very strong and can span between beams, acting as a working surface during construction. The shape of the deck also allows for conduit and pipes to be run through, without causing a major increase in thickness. Fire proofing must be sprayed on steel members and deck.

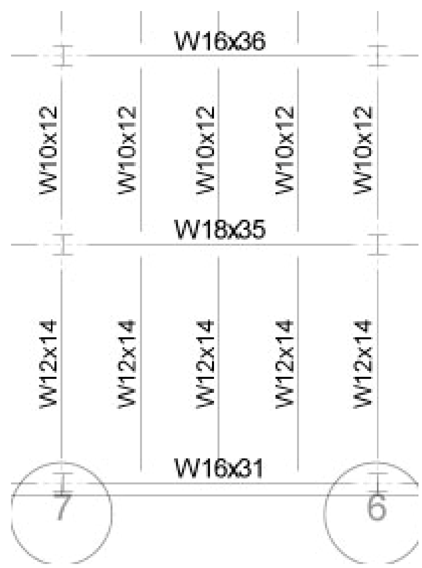
Alternatively, composite systems take a longer time to erect because of the larger amount of steel than the existing system, as well as the curing time of the concrete slab. In a project such as a hotel, it is favorable to produce the final project as quickly as possible and as inexpensive as possible, while keeping the strength and serviceability required.

Please see Appendix 2.1 for additional calculations.

Alternative #2- Non-Composite Steel Beams with Deck

The second analysis was completed using the same beam layout as Alternative #1 but with non-composite beams and deck instead of composite beams and deck. Using the LRFD criteria found in the USD Design Manual and Product Catalog, I found that 24 Gage, UF2X form deck will hold the total factored load. A 4.5" deep slab is poured into this form deck, with 60 ksi, 6×6-W4.0×W4.0 welded wire fabric. The concrete is assumed to have a strength of 3 ksi. The member sizes for the non-composite system, found by RAM, can be seen in Figure 2.3. Deflections in members are accounted for in RAM to meet L/360 (live load), and L/240 (total load).

Figure 2.3
The member sizes for a non-composite system are shown in the diagram to the right. Notice the larger sized members as compared to the composite system.



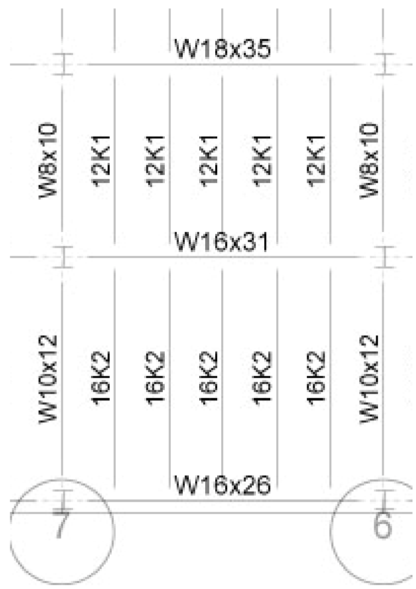
The members of the non-composite system are larger than in the composite system. This is due to the lack of shear studs working with the concrete and steel. These larger members make for a total floor sandwich depth of 22.5", which is also less than the existing system. The vibrations of the thinner slab need to be taken into account because the steel and concrete are not working together. Also, as with the composite system, there is more steel to erect than in the existing system, as well as the additional curing time for the concrete slab, increasing the time and money spent. Fire proofing must be sprayed on steel members and deck.

Please see Appendix 2.2 for additional calculations

Alternative #3-Steel Joists with Non-Composite Steel Deck

Steel joists were also analyzed and were spaced at 4'-2" on center, spanning in the N/S direction. The deck was chosen from the USD design manual: 26 gage UF1X form deck, with a 3 ksi, 3.5" concrete slab, reinforced with 60 ksi, 6×6-W4.0×4.0 welded wire fabric. The joists, designed using RAM, were found to be 12K1 for the 25'×15' bay, and 16K2 for the 25'×19' bay. These joists have a material strength of 30 ksi. See Figure 2.4, for additional member sizes. RAM limits the deflection to a live load of L/360, and a total load of L/240.

Figure 2.4
The loads are carried from the 12K1 and 16K2 joists to W18×25, W16×31 and W16×26 girders.



Because of the large girders needed to support the weight of the larger bays, the total floor sandwich is 21.5" thick. The lighter members of the joists, and the fact that they are not solid steel, may provide for greater vibrations. In addition, the thinner slab of 3.5" will not hinder vibrations as much as the larger slabs would. Fire proofing must be sprayed on steel members and deck, which could be a difficult task because of the smaller components that the joists are made of. As with the other alternate systems, the large amount of steel needed to be erected, along with the curing time of the concrete increases the time and money spent on the floor system. The joists, however, would not obstruct the placement of ductwork or electrical wiring.

Please see Appendix 2.3 for additional calculations

Alternative #4- One-way Concrete System

Finally, I analyzed the 2 bays using a one-way concrete system, or a pan-joist system. A one-way system is one where the joists, slab, and girders are poured monolithically.

Using the CRSI 2002 Design Handbook, I considered 20 inch forms with 5 inch ribs spaced at 25” on center. These ribs, when 10 inches deep with a 3 inch top slab, totaling a depth of 13”, are enough to carry my design loads. A concrete strength of 4,000 psi and a steel strength of 60,000 psi were assumed. I carried out a design for the 19 foot end span to find the worst case. The same size pans and reinforcement for all bays will be easier for construction. I assumed the clear span to be one foot less than the beam span. The reinforcement I found is as follows:

19’ span:

l_n= 18’-0”

Top reinforcement- #4 at 10 in. on center

Bottom reinforcement- (2) #4

Based on the chart, deflection is not a factor

Figures 2.5 and 2.6 show a plan view and section of this pan joist concrete system.

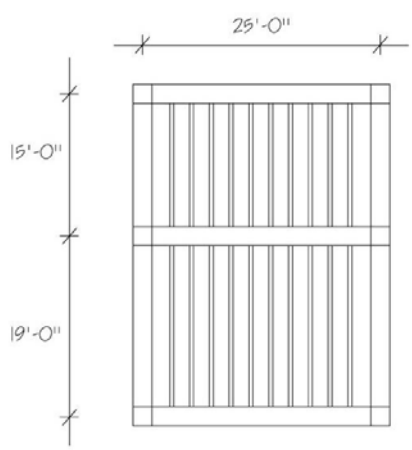


Figure 2.5

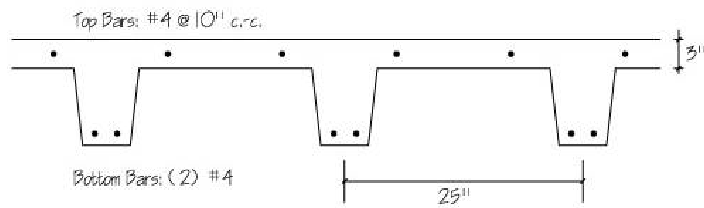


Figure 2.6

The surrounding girders needed were found to be 34” × 13”. The width of this girder is impractical and adds a large weight to the system. I originally calculated this system using a depth of 11”, however the girders were even wider and failed deflection criteria. The sandwich depth needed to be increased to increase the moment of inertia, in turn decreasing the deflection.

Because of the high weight of the system, vibrations will be lessened, but the seismic loads on the building will also increase. The foundation will also need to be examined more closely because of the poor soil conditions on site. Lateral loads will need to be resisted by a different system such as shear walls. It takes time to form, pour, cure, and finish the concrete. This increase in time spent also means an increase in price. No additional fireproofing is needed for a concrete structure.

See Appendix 2.4 for additional calculations and design aids

Floor System	Depth	Weight	Cost	Time	Vibration	Fireproofing	Possible System?
Existing: Precast hollow-core plank and steel structure	29”	60 psf	·Heavier members ·Less time=less labor cost	·Fewer beams to erect ·Precast off site	·Thick plank= less vibration	·Spray on steel members	Least amount of time for construction is the most practical for a hotel
Composite steel beams with composite deck	20.5”	46 psf	·More steel to erect	·More steel to erect ·Curing of concrete	·Composite system reduces vibration	·Spray on steel members and deck	Yes: vibration and weight are low Time is an issue
Non-composite steel beams with deck	22.5”	47 psf	·More steel to erect	·More steel to erect ·Curing of concrete	·Thinner slab increases vibration	·Spray on steel members and deck	Yes: weight is low Check vibrations and time
Steel Joists with non-composite steel deck	21.5”	40 psf	·More steel to erect ·Smaller concrete slab	·More steel to erect ·Curing of concrete	·Thinner slab and open web joists increase vibration	·Spray on steel members and deck	Yes: weight is low Check vibrations and time
One-way concrete system	13”	118 psf	·Re-useable pans ·Very large girders	·Form, pour, cure, finish	·Heavy concrete reduces vibration	·No fireproofing needed for concrete	No: cast-in-place is not practical for this type of construction

Conclusions:

The composite, non-composite, and joist systems are all feasible, but would take a longer time to erect because of more members and the curing time of the concrete. Also, changes in the foundation design would need to be considered for each alternative because of the varying weight of the systems. The one-way concrete system can be ruled out because of the extremely heavy weight and the time spent for construction.

After completing an analysis and comparison of the five systems, I have found that the existing designed system of hollow core precast concrete plank supported by a steel structural system is the best option. This system requires the least amount of construction time because of the precast floor system, and fireproofing is minimized to the steel members only. Also, vibrations are reduced because of the thickness of the slab, which is favorable for serviceability. On the downside, however, this system is the thickest due to the deep exterior beam. The analysis that I completed in RAM in Alternatives #1, #2, and #3 only took into account the gravity loads. The 25’ girder spanning in the E/W direction might need to be larger in these systems due to the moment connections for lateral resistance. If these systems had deeper exterior girders, the floor sandwich depth would increase and be closer to the depth of the precast system.

Appendix 2.1

Composite Steel Beams with Steel Deck Design:

Loads:

Dead Loads-

MEP, Finishing	= 12 psf
Metal Studs with 5/8" gypsum wall board	= 10 psf
Metal Composite Deck and Concrete	= 42 psf
Total	**= 64 psf**

Structural framing member weights are included in the RAM design and calculations.

Line load for exterior wall = 0.144 klf

Live Loads-

Private Floor and their Corridors (Service) = 40 psf

Factored Loads-

1.6L = 1.6(40 psf) = 64 psf

Deck:

USD (United Steel Deck)

Composite Deck

22 Gage, 2" Lok-Floor

2.5" Normal Weight Concrete Slab, f'_c= 3 ksi

Weight (deck and concrete slab) = 42 psf (Table A2.1a)

Capacity : 365 psf , 6.5 ft. span (Table A2.1b)

Welded Wire Fabric

6×6-W1.4 × W1.4, f'_c= 60 ksi (Table A2.1a, A2.1c)

Studs

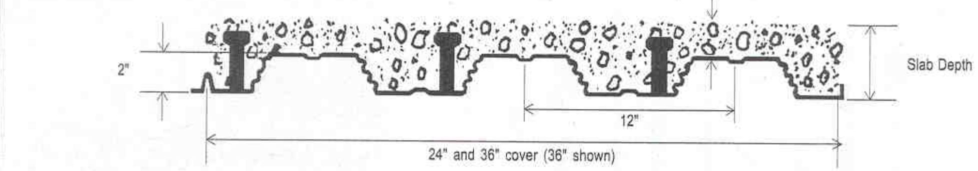
¾" Diameter

4" Long

USD United Steel Deck, Inc.

LRFD

2" LOK-FLOOR



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

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

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

COMPOSITE PROPERTIES										Max. unshored spans, ft.			
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	42	1.05	5.9	29.40	5030	5.82	7.83	7.92	0.023
	5.00	46.44	37.5	0.333	48	1.23	8.0	34.53	5480	5.54	7.47	7.56	0.027
	5.25	49.53	40.0	0.354	51	1.32	9.2	37.16	5720	5.41	7.31	7.39	0.029
	5.50	52.61	42.6	0.375	54	1.42	10.5	39.81	5960	5.30	7.16	7.24	0.032
	6.00	58.78	48.0	0.417	60	1.61	13.5	45.21	6460	5.09	6.89	6.97	0.036
	6.25	61.87	50.8	0.438	63	1.71	15.3	47.95	6720	5.03	6.76	6.84	0.038
	6.50	64.95	53.6	0.458	66	1.81	17.1	50.70	6980	4.97	6.65	6.72	0.041
	7.00	71.12	59.5	0.500	73	2.01	21.2	56.26	7530	4.85	6.43	6.51	0.045
	7.25	74.21	61.9	0.521	76	2.11	23.5	59.07	7750	4.79	6.32	6.41	0.047
	7.50	77.29	64.3	0.542	79	2.21	26.0	61.88	7970	4.74	6.22	6.31	0.050
20 gage	4.50	48.60	32.6	0.292	42	1.26	6.3	35.43	5450	6.81	8.97	9.27	0.023
	5.00	56.18	37.5	0.333	48	1.48	8.6	41.65	5900	6.47	8.55	8.83	0.027
	5.25	59.96	40.0	0.354	51	1.60	9.8	44.84	6140	6.32	8.36	8.63	0.029
	5.50	63.75	42.6	0.375	54	1.71	11.3	48.07	6380	6.18	8.18	8.45	0.032
	6.00	71.32	48.0	0.417	60	1.95	14.5	54.63	6880	5.94	7.85	8.11	0.036
	6.25	75.11	50.8	0.438	63	2.07	16.3	57.96	7140	5.86	7.70	7.95	0.038
	6.50	78.90	53.6	0.458	66	2.19	18.2	61.31	7400	5.79	7.56	7.80	0.041
	7.00	86.47	59.5	0.500	73	2.43	22.6	68.09	7950	5.65	7.29	7.53	0.045
	7.25	90.26	61.9	0.521	76	2.55	25.0	71.50	8170	5.58	7.17	7.41	0.047
	7.50	94.05	64.3	0.542	79	2.67	27.6	74.93	8390	5.52	7.05	7.28	0.050
19 gage	4.50	55.85	32.6	0.292	42	1.45	6.7	40.69	5850	7.65	9.76	10.08	0.023
	5.00	64.68	37.5	0.333	48	1.71	9.0	47.87	6300	7.26	9.30	9.61	0.027
	5.25	69.10	40.0	0.354	51	1.84	10.4	51.56	6540	7.09	9.09	9.39	0.029
	5.50	73.52	42.6	0.375	54	1.97	11.9	55.30	6780	6.93	8.90	9.19	0.032
	6.00	82.35	48.0	0.417	60	2.24	15.2	62.90	7280	6.65	8.54	8.83	0.036
	6.25	86.77	50.8	0.438	63	2.38	17.1	66.76	7540	6.56	8.38	8.66	0.038
	6.50	91.19	53.6	0.458	66	2.52	19.2	70.65	7800	6.48	8.23	8.50	0.041
	7.00	100.03	59.5	0.500	73	2.80	23.8	78.50	8350	6.32	7.94	8.20	0.045
	7.25	104.44	61.9	0.521	76	2.94	26.3	82.46	8570	6.24	7.81	8.07	0.047
	7.50	108.86	64.3	0.542	79	3.08	29.0	86.45	8790	6.17	7.68	7.94	0.050
18 gage	4.50	62.08	32.6	0.292	42	1.62	7.0	45.34	6080	8.42	10.48	10.83	0.023
	5.00	72.04	37.5	0.333	48	1.90	9.5	53.36	6670	7.98	9.99	10.32	0.027
	5.25	77.02	40.0	0.354	51	2.05	10.9	57.48	6910	7.79	9.77	10.10	0.029
	5.50	82.00	42.6	0.375	54	2.20	12.4	61.66	7150	7.61	9.56	9.88	0.032
	6.00	91.95	48.0	0.417	60	2.50	15.9	70.18	7650	7.30	9.18	9.49	0.036
	6.25	96.93	50.8	0.438	63	2.66	17.9	74.50	7910	7.20	9.01	9.31	0.038
	6.50	101.91	53.6	0.458	66	2.81	20.0	78.85	8170	7.11	8.85	9.14	0.041
	7.00	111.87	59.5	0.500	73	3.13	24.8	87.66	8720	6.93	8.54	8.82	0.045
	7.25	116.85	61.9	0.521	76	3.28	27.4	92.10	8940	6.85	8.40	8.68	0.047
	7.50	121.83	64.3	0.542	79	3.44	30.2	96.57	9160	6.77	8.26	8.54	0.050
16 gage	4.50	62.08	32.6	0.292	42	1.99	7.7	45.34	6080	9.58	11.63	12.02	0.023
	5.00	72.04	37.5	0.333	48	2.35	10.4	53.36	6980	9.08	11.10	11.47	0.027
	5.25	77.02	40.0	0.354	51	2.53	11.9	57.48	7450	8.85	10.85	11.22	0.029
	5.50	82.00	42.6	0.375	54	2.72	13.6	61.66	7940	8.65	10.63	10.98	0.032
	6.00	91.95	48.0	0.417	60	3.10	17.4	70.18	8460	8.29	10.21	10.55	0.036
	6.25	96.93	50.8	0.438	63	3.29	19.5	74.50	8720	8.17	10.02	10.35	0.038
	6.50	101.91	53.6	0.458	66	3.48	21.8	78.85	8980	8.07	9.84	10.17	0.041
	7.00	111.87	59.5	0.500	73	3.88	27.0	87.66	9530	7.86	9.50	9.82	0.045
	7.25	116.85	61.9	0.521	76	4.08	29.8	92.10	9750	7.77	9.35	9.66	0.047
	7.50	121.83	64.3	0.542	79	4.28	32.8	96.57	9970	7.67	9.20	9.50	0.050

2" LOK-FLOOR

Table A2.1a

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

LRFD

Gage	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
			L, Uniform Live Service Loads, psf *												
22 gage	4.50	40.27	400	365	310	265	230	200	175	155	135	120	105	95	85
22 gage	5.00	46.44	400	400	360	305	265	230	200	175	155	140	125	110	95
22 gage	5.50	52.61	400	400	400	350	300	260	230	200	175	155	140	125	110
22 gage	6.00	58.78	400	400	400	390	335	295	255	225	200	175	155	140	125
22 gage	6.50	64.95	400	400	400	400	370	325	285	250	220	195	175	155	135
22 gage	7.00	71.12	400	400	400	400	400	355	310	275	240	215	190	170	150
22 gage	7.25	74.21	400	400	400	400	400	370	325	285	250	225	200	175	155
22 gage	7.50	77.29	400	400	400	400	400	385	340	295	260	230	205	185	165
20 gage	4.50	48.60	400	400	380	325	285	245	215	190	170	150	135	120	110
20 gage	5.00	56.18	400	400	400	380	330	285	250	220	195	175	155	140	125
20 gage	5.50	63.75	400	400	400	400	375	325	285	250	225	200	175	160	140
20 gage	6.00	71.32	400	400	400	400	400	365	320	285	250	225	200	180	160
20 gage	6.50	78.90	400	400	400	400	400	400	355	315	280	245	220	195	175
20 gage	7.00	86.47	400	400	400	400	400	400	390	345	305	270	240	215	195
20 gage	7.25	90.26	400	400	400	400	400	400	400	360	320	285	255	225	205
20 gage	7.50	94.05	400	400	400	400	400	400	400	375	330	295	265	235	210
19 gage	4.50	55.85	400	400	400	380	330	290	255	225	200	180	160	145	130
19 gage	5.00	64.68	400	400	400	400	385	335	295	260	230	205	185	165	150
19 gage	5.50	73.52	400	400	400	400	400	380	335	295	265	235	210	190	170
19 gage	6.00	82.35	400	400	400	400	400	400	375	335	295	265	235	215	190
19 gage	6.50	91.19	400	400	400	400	400	400	400	370	330	295	265	235	210
19 gage	7.00	100.03	400	400	400	400	400	400	400	400	360	320	290	260	235
19 gage	7.25	104.44	400	400	400	400	400	400	400	400	375	335	300	270	245
19 gage	7.50	108.86	400	400	400	400	400	400	400	400	395	350	315	280	255
18 gage	4.50	62.08	400	400	400	400	370	325	285	255	225	200	180	160	145
18 gage	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
18 gage	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
18 gage	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
18 gage	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
18 gage	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
18 gage	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
18 gage	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
16 gage	4.50	62.08	400	400	400	400	370	325	285	255	225	200	180	160	145
16 gage	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
16 gage	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
16 gage	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
16 gage	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
16 gage	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
16 gage	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
16 gage	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
	4.50	29.40	305	255	215	185	160	135	120	105	90	80	70	60	50
	5.00	34.53	360	305	255	220	185	160	140	120	105	95	80	70	65
	5.50	39.81	400	350	295	255	215	190	165	140	125	110	95	85	75
	6.00	45.21	400	400	340	290	250	215	185	160	140	125	110	95	85
	6.50	50.70	400	400	380	325	280	240	210	185	160	140	125	110	95
	7.00	56.26	400	400	400	360	310	270	235	205	180	155	140	120	105
	7.25	59.07	400	400	400	380	325	285	245	215	190	165	145	130	115
	7.50	61.88	400	400	400	400	345	295	260	225	200	175	155	135	120
	4.50	35.43	375	315	270	230	200	170	150	130	115	100	90	80	70
	5.00	41.65	400	375	315	270	235	205	175	155	135	120	105	95	85
	5.50	48.07	400	400	365	315	270	235	205	180	160	140	125	110	95
	6.00	54.63	400	400	400	360	310	270	235	205	180	160	140	125	110
	6.50	61.31	400	400	400	400	350	300	265	230	205	180	160	140	125
	7.00	68.09	400	400	400	400	390	335	295	260	230	200	180	160	140
	7.25	71.50	400	400	400	400	400	355	310	270	240	210	190	165	150
	7.50	74.93	400	400	400	400	400	370	325	285	250	225	200	175	155
	4.50	40.69	400	370	315	270	230	200	175	155	135	120	105	95	85
	5.00	47.87	400	400	370	315	275	240	210	185	160	145	125	115	100
	5.50	55.30	400	400	400	365	320	275	240	215	190	165	150	130	120
	6.00	62.90	400	400	400	400	365	315	275	245	215	190	170	150	135
	6.50	70.65	400	400	400	400	400	355	310	275	245	215	190	170	155
	7.00	78.50	400	400	400	400	400	395	350	305	270	240	215	190	170
	7.25	82.46	400	400	400	400	400	400	365	320	285	255	225	200	180
	7.50	86.45	400	400	400	400	400	400	385	340	300	265	235	210	190
	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220
	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195

1 STUD/FT.

NO STUDS

* The **Uniform Live Loads** are based on the LRFD equation $\phi M_n = (1.6L + 1.2D)l^2/8$. Although there are other load combinations that may require investigation, this will control most of the time. The equation assumes there is no negative bending reinforcement over the beams and therefore each composite slab is a single span. Two sets of values are shown; ϕM_{nf} is used to calculate the uniform load when the full required number of studs is present; ϕM_{no} is used to calculate the load when no studs are present. A straight line interpolation can be done if the average number of studs is between zero and the required number needed to develop the "full" factored moment. The tabulated loads are checked for shear controlling (it seldom does), and also limited to a live load deflection of 1/360 of the span.

An upper limit of 400 psf has been applied to the tabulated loads. This has been done to guard against equating large concentrated to uniform loads. Concentrated loads may require special analysis and design to take care of servicibility requirements not covered by simply using a uniform load value. On the other hand, for any load combination the values provided by the composite properties can be used in the calculations.

Welded wire fabric in the required amount is assumed for the table values. If welded wire fabric is not present, deduct 10% from the listed loads.

Refer to the example problems for the use of the tables.

Table A2.1b

The Welded Wire Fabric is found using the required area of steel (A_{wwf}), found in Table A2.1a.

reinforcement properties

Welded Wire Fabric used in this manual					
Conventional (USA)	Metric** (INTERNATIONAL)	Wire Area		Wire Diameter	
		in²/ft	mm²/m	inches	mm
6 x 6 W 1.4 x 1.4	152 x 152 MW 9.1 x 9.1	0.028	59.3	0.135	3.43
6 x 6 W 2.0 x 2.0	152 x 152 MW 12.9 x 12.9	0.040	84.6	0.162	4.11
6 x 6 W 2.9 x 2.9	152 x 152 MW 18.7 x 18.7	0.058	122.7	0.192	4.88
6 x 6 W 4.0 x 4.0	152 x 152 MW 25.8 x 25.8	0.080	169.2	0.225	5.72
4 x 4 W 1.4 x 1.4	102 x 102 MW 9.1 x 9.1	0.042	88.9	0.135	3.43
4 x 4 W 2.0 x 2.0	102 x 102 MW 12.9 x 12.9	0.060	126.9	0.162	4.11
4 x 4 W 2.9 x 2.9	102 x 102 MW 18.7 x 18.7	0.087	184.1	0.192	4.88
4 x 4 W 4.0 x 4.0	102 x 102 MW 25.8 x 25.8	0.120	253.8	0.225	5.72

**This table is a soft metric conversion of wire sizes normally used. As of this printing the Wire Reinforcement Institute has not formalized the metric designations.

Table A2.1c

Appendix 2.2

Non-Composite Steel Beams with Form Deck and Slab:

Loads:

Dead Loads-

MEP, Finishing	= 12 psf
Metal Studs with 5/8" gypsum wall board	= 10 psf
Metal Form Deck with 4.5" Slab	= 42 psf
Total	**= 64 psf**

Structural framing member weights are included in the RAM design and calculations.

Line load for exterior wall = 0.144 klf

Live Loads-

Private Floor and their Corridors = 40 psf

Factored Loads-

1.4 D + 1.7 L = 1.4(64psf) + 1.7(40psf) = 157.6 psf

Deck:

USD (United Steel Deck)

Non-Composite Form Deck

24 Gage, UF2X

4.5" Normal Weight Concrete Slab, $f'_c = 3$ ksi

Weight (deck and concrete slab) = 42 psf (Table A2.2b)

Capacity (for a 6'-6" span): (Table A2.2c)

162 psf (total load)

6×6-W4.0×W4.0 @ 6'-6", $f'_c = 60$ ksi (Table A2.2a)

185 psf capacity

concrete slabs on UF2X form deck - UNIFORM LOADS, PSF

Slab	Mesh	+d	-d	+M	-M	Spans, feet 4'6"	5'0"	5'6"	6'0"	6'6"	7'0"	7'6"	8'0"	8'6"	9'0"
4.0"	66 - W2.0 x 2.0* 66 - W2.9 x 2.9	1.919 1.904	3.007 2.962	4.060 5.785	6.326 8.921	157 224	127 181	105 150	88 126	75 107	65 93	57 81	50 71	44 63	56
4.5"	66 - W4.0 x 4.0 44 - W2.9 x 2.9 44 - W4.0 x 4.0	2.387 2.404 2.387	3.412 3.462 3.412	9.975 10.893 14.708	14.062 15.463 20.585	386 ### ###	313 342 ###	259 282 381	217 237 320	185 202 273	160 174 235	139 152 205	122 133 180	108 118 160	97 105 142
5.0"	66 - W4.0 x 4.0* 44 - W2.9 x 2.9 44 - W4.0 x 4.0	2.887 2.904 2.887	3.912 3.962 3.912	12.135 13.242 17.948	16.222 17.812 23.825	### ### ###	381 ### ###	315 343 ###	264 289 389	225 246 332	194 212 286	169 185 249	149 162 219	132 144 194	117 128 173
5.5"	44 - W2.9 x 2.9* 44 - W4.0 x 4.0	3.404 3.387	4.462 4.412	15.591 21.188	20.161 27.065	### ###	### ###	392 ###	329 ###	281 377	242 325	211 283	185 249	164 220	146 197
6.0"	44 - W4.0 x 4.0	3.887	4.912	24.428	30.305	###	###	###	###	###	364	317	279	247	220
6.5"	44 - W4.0 x 4.0	4.387	5.412	27.668	33.545	###	###	###	###	###	###	351	308	273	244
7.0"	44 - W4.0 x 4.0	4.887	5.912	30.908	36.785	###	###	###	###	###	###	385	338	299	267
						24 gage						22 gage		20 gage	

Table A2.2a

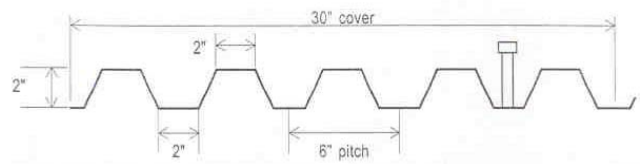
FORM DECKS weights and volumes

Total Slab Depth		UFS C_v = .0234	UF1X C_v = .0417	UFX C_v = .0547	INV. B C_v = .0781	UF2X C_v = .0833
2.5"	Wt Vol.	27 0.185				
3.0"	Wt Vol.	33 0.226	30 0.208	28 0.195		
3.5"	Wt Vol.	39 0.268	36 0.250	34 0.237	36 0.245	
4.0"	Wt Vol.	45 0.310	42 0.292	40 0.279	41 0.286	36 0.250
4.5"	Wt Vol.	51 0.352	48 0.333	46 0.320	48 0.328	42 0.292
5.0"	Wt Vol.	57 0.393	54 0.375	52 0.362	54 0.370	48 0.333
5.5"	Wt Vol.	63 0.435	60 0.417	59 0.404	60 0.411	54 0.375
6.0"	Wt Vol.	69 0.476	66 0.458	65 0.445	66 0.453	60 0.417
6.5"	Wt Vol.	75 0.518	73 0.500	71 0.487	72 0.495	67 0.459
7.0"	Wt Vol.	81 0.560	79 0.542	77 0.528	78 0.536	73 0.500

Table A2.2b

SECTION PROPERTIES							ASD		LRFD		
Metal Thickness		Wt.	I_p	S_p	S_n	V	R_1	R_2	ϕV	ϕR_1	ϕR_2
Gage	Inches	(psf)	(in.^{4})	(in.^{3})	(in.3)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
24	0.0239	1.50	0.232	0.192	0.200	2360	360	836	3223	532	1156
22	0.0295	2.00	0.300	0.252	0.263	4205	528	1484	5477	736	1992
20	0.0358	2.00	0.379	0.325	0.339	6062	728	2224	8067	1004	3064
18	0.0474	3.00	0.523	0.468	0.485	8796	1204	3948	11182	1648	5388

UF2X



The bottom flange can accept a ¾" shear stud.

approx. scale: 1½" = 1'0"

	Gage	Span Condition	Span 6'0"	6'6"	7'0"	7'6"	8'0"	8'6"	9'0"	9'6"	10'0"
ASD	24	Single	128 / 94	109 / 74	94 / 59	82 / 48	72 / 40	64 / 33	57 / 28	51 / 24	46 / 20
ASD	24	Double	130 / 226	111 / 178	96 / 143	84 / 116	74 / 96	66 / 80	59 / 67	53 / 57	48 / 49
ASD	24	Triple	162 / 177	138 / 139	120 / 112	105 / 91	92 / 75	82 / 62	73 / 52	66 / 45	59 / 38
ASD	22	Single	168 / 122	143 / 96	123 / 77	108 / 62	94 / 51	84 / 43	75 / 36	67 / 31	60 / 26
ASD	22	Double	173 / 293	148 / 230	128 / 184	111 / 150	98 / 123	87 / 103	78 / 87	70 / 74	63 / 63
ASD	22	Triple	215 / 229	184 / 180	159 / 144	139 / 117	122 / 97	108 / 81	97 / 68	87 / 58	78 / 49
ASD	20	Single	217 / 154	185 / 121	159 / 97	139 / 79	122 / 65	108 / 54	96 / 46	86 / 39	78 / 33
ASD	20	Double	224 / 370	191 / 291	165 / 233	144 / 189	126 / 156	112 / 130	100 / 110	90 / 93	81 / 80
ASD	20	Triple	279 / 289	238 / 228	205 / 182	179 / 148	158 / 122	140 / 102	125 / 86	112 / 73	101 / 63
ASD	18	Single	312 / 212	266 / 167	229 / 133	200 / 109	176 / 89	155 / 75	139 / 63	124 / 53	112 / 46
ASD	18	Double	320 / 510	273 / 401	236 / 321	206 / 261	181 / 215	160 / 179	143 / 151	128 / 129	116 / 110
ASD	18	Triple	399 / 399	340 / 314	294 / 252	256 / 204	226 / 168	200 / 140	179 / 118	160 / 101	145 / 86
LRFD	24	Single	177 / 94	164 / 74	149 / 59	130 / 48	114 / 40	101 / 33	90 / 28	81 / 24	73 / 20
LRFD	24	Double	154 / 226	142 / 178	132 / 143	123 / 116	116 / 96	104 / 80	93 / 67	83 / 57	75 / 49
LRFD	24	Triple	175 / 177	162 / 139	150 / 112	140 / 91	131 / 75	124 / 62	115 / 52	103 / 45	94 / 38
LRFD	22	Single	245 / 122	226 / 96	195 / 77	170 / 62	150 / 51	133 / 43	118 / 36	106 / 31	96 / 26
LRFD	22	Double	266 / 293	233 / 230	201 / 184	176 / 150	155 / 123	137 / 103	122 / 87	110 / 74	99 / 63
LRFD	22	Triple	302 / 229	279 / 180	250 / 144	218 / 117	192 / 97	171 / 81	152 / 68	137 / 58	124 / 49
LRFD	20	Single	335 / 154	292 / 121	252 / 97	220 / 79	193 / 65	171 / 54	152 / 46	137 / 39	124 / 33
LRFD	20	Double	353 / 370	301 / 291	260 / 233	227 / 189	200 / 156	177 / 130	158 / 110	142 / 93	128 / 80
LRFD	20	Triple	418 / 289	375 / 228	324 / 182	283 / 148	249 / 122	221 / 102	197 / 86	177 / 73	160 / 63
LRFD	18	Single	494 / 212	421 / 167	363 / 133	316 / 109	278 / 89	246 / 75	220 / 63	197 / 53	178 / 46
LRFD	18	Double	505 / 510	431 / 401	372 / 321	325 / 261	286 / 215	253 / 179	226 / 151	203 / 129	183 / 110
LRFD	18	Triple	627 / 399	536 / 314	463 / 252	404 / 204	356 / 168	316 / 140	282 / 118	253 / 101	229 / 86

UNIFORM TOTAL LOAD / Load that Produces l/180 Deflection, psf

Table A2.2c

Appendix 2.3

Steel Joists with Non-Composite Steel Deck:

Loads:

Dead Loads-

MEP, Finishing		= 12 psf
Metal Studs with 5/8" gypsum wall board		= 10 psf
Metal Deck and Slab		= 36 psf
Joist Weight (5.5 plf / 4.167 ft.)= 1.33 psf→		= 2 psf
(used weight of 16K2 for more conservative estimate)		
	Total	**= 60 psf**

Girder weights are included in the RAM design and calculations.

Line load for exterior wall = 0.144 klf

Live Loads-

Private Floor and their Corridors = 40 psf

Factored Loads-

1.2D + 1.6 L = 1.2(60psf) + 1.6(40psf) = 136 psf

Deck:

USD (United Steel Deck)

Non-Composite Form Deck

24 Gage, UF1X

3.5" Normal Weight Concrete Slab

Weight (deck and concrete slab) = 36 psf (Table A2.3b)

Capacity (for a 4'-6" span): (Table A2.3c)

177 psf (total load)

6×6-W4.0×W4.0 @ 4'-6" span (Table A2.3a)

230 psf capacity

concrete slabs on UF1X form deck - UNIFORM LOADS, PSF

Slab	Mesh	+d	-d	+M	-M	2'0"	2'6"	3'0"	3'6"	4'0"	4'6"	5'0"	5'6"	6'0"	6'6"
						Spans, feet									
3.0"	66 - W2.0 x 2.0* 66 - W2.9 x 2.9	1.000 1.000	1.500 1.500	2.075 2.954	3.155 4.520	### ###	260 371	181 257	133 189	102 145	80 114	65 93	54 77	45 64	
3.5"	66 - W4.0 x 4.0 44 - W2.9 x 2.9 44 - W4.0 x 4.0	2.387 2.404 2.387	1.912 1.962 1.912	9.975 10.893 14.708	7.921 8.817 11.628	### ### ###	### ### ###	### ### ###	380 ### ###	291 324 ###	230 256 338	186 207 274	154 171 226	129 144 190	110 123 162
4.0"	66 - W4.0 x 4.0 44 - W2.9 x 2.9 44 - W4.0 x 4.0	2.887 2.904 2.887	2.412 2.462 2.412	12.135 13.242 17.948	10.081 11.166 14.868	### ### ###	### ### ###	### ### ###	### ### ###	371 ### ###	293 324 ###	237 263 350	196 217 289	165 182 243	140 155 207
4.5"	44 - W2.9 x 2.9 44 - W4.0 x 4.0	3.404 3.387	2.962 2.912	15.591 21.188	13.515 18.108	### ###	### ###	### ###	### ###	### ###	393 ###	318 ###	263 352	221 296	188 252
5.0"	44 - W4.0 x 4.0	3.887	3.412	24.428	21.348	###	###	###	###	###	###	###	###	349	297
5.5"	44 - W4.0 x 4.0	4.387	3.912	27.688	24.588	###	###	###	###	###	###	###	###	###	342
6.0"	44 - W4.0 x 4.0	4.887	4.412	30.908	27.828	###	###	###	###	###	###	###	###	###	
						26 gage				24 gage		22 gage		20 gage	

Table A2.3a

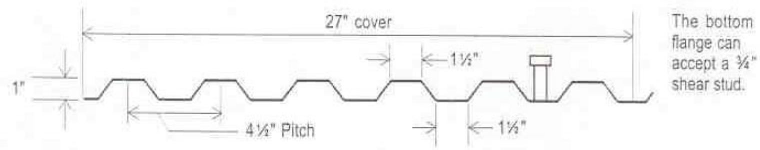
FORM DECKS weights and volumes

Total Slab Depth		UFS C_v = .0234	UF1X C_v = .0417	UFX C_v = .0547	INV. B C_v = .0781	UF2X C_v = .0833
2.5"	Wt Vol.	27 0.185				
3.0"	Wt Vol.	33 0.226	30 0.208	28 0.195		
3.5"	Wt Vol.	39 0.268	36 0.250	34 0.237	36 0.245	
4.0"	Wt Vol.	45 0.310	42 0.292	40 0.279	41 0.286	36 0.250
4.5"	Wt Vol.	51 0.352	48 0.333	46 0.320	48 0.328	42 0.292
5.0"	Wt Vol.	57 0.393	54 0.375	52 0.362	54 0.370	48 0.333
5.5"	Wt Vol.	63 0.435	60 0.417	59 0.404	60 0.411	54 0.375
6.0"	Wt Vol.	69 0.476	66 0.458	65 0.445	66 0.453	60 0.417
6.5"	Wt Vol.	75 0.518	73 0.500	71 0.487	72 0.495	67 0.459
7.0"	Wt Vol.	81 0.560	79 0.542	77 0.528	78 0.536	73 0.500

Table A2.3b

SECTION PROPERTIES							ASD		LRFD		
Metal Thickness		Wt.	I_p	S_p	S_n	V	R_1	R_2	ϕV	ϕR_1	ϕR_2
Gage	Inches	(psf)	(in.^{4})	(in.^{3})	(in.3)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)	(lbs)
26	0.0179	1.00	0.039	0.066	0.066	2009	309	396	2387	485	715
24	0.0239	1.50	0.056	0.096	0.096	2906	491	629	3310	731	875
22	0.0295	1.50	0.072	0.127	0.127	3625	715	1349	4073	992	1808
20	0.0358	2.00	0.088	0.163	0.163	4338	971	2181	4927	1339	3013

UF1X



approx. scale: 1½" = 1'0"

UNIFORM TOTAL LOAD / Load that Produces l/180 Deflection, psf			Span								
	Gage	Span Condition	3'0"	3'6"	4'0"	4'6"	5'0"	5'6"	6'0"	6'6"	7'0"
ASD	26	Single	176 / 126	129 / 80	99 / 53	78 / 37	63 / 27	52 / 21	44 / 16	37 / 12	32 / 10
		Double	174 / 304	128 / 192	98 / 128	78 / 90	63 / 66	52 / 49	44 / 38	37 / 30	32 / 24
		Triple	216 / 238	159 / 150	122 / 101	97 / 71	79 / 51	65 / 39	55 / 30	47 / 23	40 / 19
	24	Single	256 / 182	188 / 114	144 / 77	114 / 54	92 / 39	76 / 29	64 / 23	55 / 18	47 / 14
		Double	253 / 437	186 / 275	143 / 184	113 / 130	92 / 94	76 / 71	64 / 55	54 / 43	47 / 34
		Triple	314 / 342	232 / 215	178 / 144	141 / 101	114 / 74	95 / 56	80 / 43	68 / 34	59 / 27
	22	Single	339 / 233	249 / 147	191 / 98	151 / 69	122 / 50	101 / 38	85 / 29	72 / 23	62 / 18
		Double	334 / 562	246 / 354	189 / 237	150 / 167	121 / 121	100 / 91	84 / 70	72 / 55	62 / 44
		Triple	414 / 440	306 / 277	235 / 186	186 / 130	151 / 95	125 / 71	105 / 55	90 / 43	77 / 35
	20	Single	435 / 285	319 / 180	245 / 120	193 / 85	156 / 62	129 / 46	109 / 36	93 / 28	80 / 22
		Double	427 / 687	315 / 433	242 / 290	192 / 204	155 / 148	129 / 111	108 / 86	92 / 68	80 / 54
		Triple	530 / 538	392 / 339	301 / 227	239 / 159	194 / 116	160 / 87	135 / 67	115 / 53	99 / 42
LRFD	26	Single	279 / 126	205 / 80	157 / 53	124 / 37	100 / 27	83 / 21	70 / 16	59 / 12	51 / 10
		Double	191 / 304	163 / 192	143 / 128	123 / 90	99 / 66	82 / 49	69 / 38	59 / 30	51 / 24
		Triple	217 / 238	186 / 150	163 / 101	144 / 71	124 / 51	103 / 39	86 / 30	74 / 23	64 / 19
	24	Single	405 / 182	298 / 114	228 / 77	180 / 54	146 / 39	121 / 29	101 / 23	86 / 18	74 / 14
		Double	233 / 437	200 / 275	175 / 184	156 / 130	140 / 94	120 / 71	101 / 55	86 / 43	74 / 34
		Triple	265 / 342	227 / 215	199 / 144	177 / 101	159 / 74	145 / 56	125 / 43	107 / 34	92 / 27
	22	Single	536 / 233	394 / 147	302 / 98	238 / 69	193 / 50	160 / 38	134 / 29	114 / 23	98 / 18
		Double	482 / 562	385 / 354	297 / 237	235 / 167	191 / 121	158 / 91	133 / 70	113 / 55	98 / 44
		Triple	548 / 440	470 / 277	368 / 186	292 / 130	238 / 95	197 / 71	166 / 55	141 / 43	122 / 35
	20	Single	688 / 285	506 / 180	387 / 120	306 / 85	248 / 62	205 / 46	172 / 36	147 / 28	126 / 22
		Double	666 / 687	493 / 433	380 / 290	301 / 204	245 / 148	203 / 111	171 / 86	146 / 68	126 / 54
		Triple	821 / 538	610 / 339	471 / 227	374 / 159	304 / 116	252 / 87	212 / 67	181 / 53	157 / 42

NOTES:

Table A2.3c

Appendix 2.4

One-way Concrete System:

Loads:

Dead Loads (Superimposed)-

MEP, Finishing	= 12 psf
Metal Studs with 5/8" gypsum wall board	= 10 psf
Total	**= 22 psf**

Live Loads-

Private Floor and their Corridors	= 40 psf

Total Load-

w_u=1.4D + 1.7 (L) = 1.4(22)+1.7(40) = 98.8 psf

Assumed-

f'_c= 4,000 psi

f_y=60,000 psi

From Table A2.4a-

20" forms + 5" rib @ 25" c.-c.

10" deep rib + 3" top slab = 13" depth (total)

l_n=18' (End Span)

Capacity = 246 psf

Top Bars: #4 spaced at 10" o.c.

Bottom Bars: (2) #4

l_n=18' (Interior Span)

Capacity = 291 psf

Top Bars: #3 spaced at 10" o.c.

Bottom Bars: 1- #3, 1- #4

Welded Wire Fabric Reinforcement

4×12-W2.1×W1.4 (Table 2.4b)

Table A2.4a

STANDARD ONE-WAY JOISTS (1) MULTIPLE SPANS	20" Forms + 5" Rib @ 25" c.-c. (2) FACTORED USABLE SUPERIMPOSED LOAD (PSF)											f'_c = 4,000 psi f_y = 60,000 psi
	10" Deep Rib + 3.0" Top Slab = 13.0" Total Depth											
TOP BARS	Size @	# 4 10	# 4 8	# 5 10	# 6 11.5	# 6 9.5	End Span Defl. Coeff. (3)	# 3 10	# 4 12	# 4 9	# 5 10		Int. Span Defl. Coeff. (3)
BOTTOM BARS	# #	# 4 # 4	# 4 # 5	# 5 # 5	# 5 # 6	# 6 # 6		# 3 # 4	# 4 # 4	# 4 # 5	# 5 # 5		
Steel (psf)		.80	1.01	1.25	1.52	1.83		.82	1.08	1.39	1.78		
CLEAR SPAN		END SPAN						INTERIOR SPAN					
18'-0"		246 0	336 0	365* 427	375* 531	390* 569*	.878	291 0	401 0	419* 532	428* 639*		.540
19'-0"		211 0	292 0	334* 374	343* 466	356* 521*	1.090	251 0	350 0	386* 468	394* 586		.671
20'-0"		182 0	254 0	307* 328	315* 412	326* 479*	1.338	217 0	307 0	358* 413	364* 520		.823
21'-0"		156 0	222 0	284* 289	290* 365	300* 442	1.626	188 0	270 0	332* 366	338* 463		1.001
22'-0"		134 0	194 0	255 0	268* 324	277* 394	1.959	163 0	237 0	309* 325	314* 413		1.205
23'-0"		114 0	169 0	225 0	249* 288	257* 353	2.340	141 0	209 0	289* 0	293* 370		1.440
24'-0"		97 0	148 0	199 0	232* 257	239* 316	2.774	122 0	184 0	258 0	274* 332		1.707
25'-0"		82 0	129 0	176 0	216* 230	222* 284	3.266	105 0	162 0	230 0	257* 299		2.010
26'-0"		69 0	112 0	156 0	202* 205	208* 256	3.821	90 0	143 0	206 0	241* 269		2.351
27'-0"		57 0	97 0	137 0	183 0	194* 230	4.443	77 0	126 0	184 0	227* 243		2.734
28'-0"		46 0	84 0	121 0	164 0	182* 207	5.139	65 0	110 0	164 0	213* 219		3.163
29'-0"			72 0	107 0	146 0	171* 187	5.914	54 0	97 0	147 0	198 0		3.639
30'-0"			61 0	93 0	131 0	160* 169	6.772	44 0	84 0	131 0	179 0		4.168
31'-0"			51 0	82 0	116 0	141+ 152	7.722		73 0	117 0	161 0		4.752

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

PROPERTIES FOR DESIGN (CONCRETE .44 CF/SF) (4)

NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	.50	.63	.77	.96	1.16		.50	.65	.86	1.10		
STEEL % (UNIFORM)	.71	.89	1.11	1.38	1.67		.71	.92	1.23	1.59		
(TAPERED)	.43	.53	.66	.83	1.00		.43	.55	.74	.95		
EFF. DEPTH, IN.	11.8	11.8	11.7	11.6	11.6		11.8	11.7	11.7	11.6		
– ICR/IGR	.189	.224	.259	.298	.340		.189	.227	.280	.328		
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.40	.51	.62	.75	.88		.31	.40	.51	.62		
STEEL %	.14	.17	.21	.26	.30		.11	.14	.17	.21		
EFF. DEPTH, IN.	11.8	11.7	11.7	11.6	11.6		11.8	11.8	11.7	11.7		
+ICR/IGR	.198	.244	.290	.339	.389		.157	.198	.244	.290		

Top Slab Thickness (In.)	Bars Grade 60	Welded Wire Fabric***	
		One Way**	Square
2½	#3@12*	4 X 12-W2.1 X W1.4	6 X 6-W2.9 X W2.9
3	#3@15*	4 X 12-W2.1 X W1.4	6 X 6-W4 X W4
3½	#3@17	4 X 12-W2.5 X W1.4	6 X 6-W4 X W4
4	#3@15	4 X 12-W3 X W2	4 X 4-W2.9 X W2.9
4½	#3@12	4 X 12-W3.5 X W2	4 X 4-W3.5 X W3.5

* Maximum spacing permitted by ACI 7.12.2.2 (5 times slab thickness ≤ 18 in.)
** Larger diameter wires are to be placed *normal* to span of the joists.
*** Commonly available wire sizes.

Table 2.4b

Girder Design: (worst case, assume same for all beams in bay)

Live Load Reduction-

$$A_t = \frac{(15' + 19')}{2}(25') = 425 \text{ sq. ft.}$$

$$T_w = \frac{(15' + 19')}{2} = 17'$$

$K_{LL} = 2$ (ASCE 7-02, Table 4.2)

$L = L_o\,(0.25 + 15/(\sqrt{K_{LL}A_t}))$
$L = L_o\,(0.25 + 15/(\sqrt{(2)(425)}))$
$L = 0.76\,L_o > 0.4$ Use $0.76\,L_o$

$L = 0.76(40 \text{ psf}) = 30.4 \text{ psf}$

Assume column is 1'×1' ➔ $l_n = 24'$

$w_u = 1.2\,(67\text{psf} + 22 \text{ psf}) + 1.6(30.4 \text{ psf})$
$w_u = 153.04 \text{ psf}$

$W_u = 153.04 \text{ psf}\,(17') = 2.6 \text{ klf}$

$$M_u = \frac{W_u L^2}{8} = \frac{(2.6 \text{ klf})(24')^2}{8} = 187.2\text{'k}$$

$f'_c = 4 \text{ ksi}$
$f_y = 60 \text{ ksi}$
$\rho = 0.6\,\rho_{max} = 0.6(0.0206) = 0.0124$ (for a tension controlled section)
$d = 13" – 2.5" = 10.5"$

$M_u \le \Phi M_n = \Phi \rho b d^2 f_y\,(1 – 0.59\rho(f_y/f'_c))$
$187.2\text{'k} = 0.9(0.0124)(bd^2)(60\text{ksi})(1 – 0.59(0.0124)(60\text{ksi}/4\text{ksi}))\,(1'/12")$
$bd^2 = 3768.38 \text{ in}^3$
$b = 34"$
$h = 13"$

$$W_{u\ beam} = \frac{(1.2)(34")(13")(150\text{pcf})}{(144\text{in}^2/1\text{ft}^2)} = 552.5 \text{ plf} = .553 \text{ klf}$$

$$M_{u\ beam} = 187.2\text{'k} + \frac{(0.553 \text{ klf})(24')^2}{8}$$
$= 227\text{'k}$

Steel Design-

$M_u \leq \Phi A_s d\, f_y\, (1 - 0.59\rho(f_y/f'_c))$

$227\text{'k} = 0.9\, A_s\, (10.5\text{''})(60\text{ ksi})(1 - 0.59(0.0124)(60\text{ksi}/4\text{ksi}))\,(1\text{'}/12\text{''})$

$A_s = 5.4\text{ in}^2$, Use 4#11 (A_s= 6.24in^2)

Deflection Check-

$I = (1/12)(bh^3) = (1/12)(34\text{''})(13\text{''})^3$

$I = 6225\text{ in}^4$

$W_u = 2.6\text{ klf} + 0.553\text{ klf} = 3.15\text{ klf}\,(1\text{'}/12\text{''}) = 0.26\text{ k/in} = 260\text{ lb/in}$

$E = 3.6 \times 10^6\text{ psi}$

$\Delta \leq L/240$

$\leq (24\text{'})(12\text{''}/1\text{'})/240 = 1.2\text{''}$

$$\Delta = \frac{5W_uL^4}{384EI}$$

$$= \frac{5(260\text{ lb/in})(288\text{ in})^4}{(384)(3.6 \times 10^6\text{ psi})(6225\text{ in}^4)}$$

$\Delta = 1.04\text{''} \leq 1.2\text{''}$ OK

Appendix 2.5

Floor System Weights

Area= 34’ ×25’

1) Existing System:

1.0 - W18×46 (25’)
1.5 - W10×12 (25’)
0.5 - W21×44 (19’)
0.5 - W16×36 (19’)
1.0 - W14×30 (15’)

$$\frac{(46plf)(25') + (1.5)(12plf)(25') + (0.5)(44plf)(19') + (0.5)(36plf)(19') + (30plf)(15')}{(34')(25')} =$$

3.3 psf
56 psf (concrete plank weight)
Total = 60 psf

2) Composite System:

0.5 - W16×26 (25’)
1.0 - W14×22 (25’)
1.0 - W12×19 (25’)
4.0 - W8×10 (19’)
4.0 - W8×10 (15’)

$$\frac{(0.5)(26plf)(25') + (22plf)(25') + (19plf)(25') + (4)(10plf)(19') + (4)(10plf)(15')}{(34')(25')} =$$

3.19 psf
42 psf (deck and concrete slab)
Total = 46 psf

3) Non-Composite System:

0.5 – W16×36 (25’)
1.0 – W18×35 (25’)
1.0 – W16×31 (25’)
4.0 – W10×12 (15’)
4.0 – W12×14 (19’)

$$\frac{(0.5)(36plf)(25') + (35plf)(25') + (31plf)(25') + (4)(12plf)(15') + (4)(14plf)(19')}{(34')(25')} =$$

4.57 psf
42 psf (deck and concrete slab
Total = 47 psf

4) Joists

5.0 – 12K1 (15')
5.0 – 16K2 (19')
0.5 – W18×35 (25')
1.0 – W16×31 (25')
1.0 – W16×26 (25')
1.0 – W8×10 (15')
1.0 – W10×12 (19')

$$\frac{(5)(5\text{plf})(15') + (5)(5.5\text{plf})(19') + (0.5)(35\text{plf})(25') + (31\text{plf})(25') + (26\text{plf})(25') + (10\text{plf})(15') + (12\text{plf})(19')}{(35')(25')} =$$

3.69 psf
36 psf (deck and concrete slab)
Total = 40 psf

5) One-Way

From Table 8-1, CRSI
67 psf

Girders (34" × 13")

2.5 – 25'
1.0 – 15'
1.0 – 19'

150 pcf (34"/12)(13"/12) = 460.4 plf

$$\frac{(2.5)(460.4\text{ plf})(25') + (460.4\text{ plf})(15') + (460.4\text{ plf})(19')}{(35')(25')} =$$

50.78psf
Total = 118 psf