Sallie Mae Headquarters
Reston, VA

Frank Burke
Structural Option
Technical Assignment #2
Thesis Advisor – Thomas E. Boothby

Executive Summary

For this assignment I will be analyzing four alternative systems for my building, Sallie Mae HQ. Sallie Mae is located in Reston, Virginia and consists of an underground parking garage and a 9 story office building. The parking garage was left out for this assignment due to the fact that it will remain concrete and alternative systems seem impractical. On the other hand, I analyzed a typical floor for the office building portion of Sallie Mae. The existing system for the office building is structural steel with a high quantity of cambered beams. The controlling factor for the design of all my systems would be the 45'-55' spans of the exterior bays spanning E-W. For the purpose of this assignment I simplified a strip in the E-W direction into a set of rectangular bays.

My alternative systems are post-tensioning, steel joist, precast concrete, and steel framing. For this design I came to the conclusion that post-tensioning won't work for slabs less than 13" deep because of the long spans. Also increasing slab depth over 14" would be very uneconomical due to the self-weight of the slab. Also the long exterior span is followed by a short 25' middle span which causes the tendon profile to be critical in my design. For post-tensioning to work 2-4 strands of ½ in. diameter, 7 wire, 270ksi strands are needed per foot of width. This makes post-tensioning a very possible solution.

The next two systems ended up with solutions that required 32" of structural depth. For the one way joist system a 32LH12 LH-SERIES JOIST is needed, and 4 rows of horizontal bridging is required with 16' max spacing. Also the decking could be reduced to a total depth of 4" due to the 4' spacing between the joists. Another concrete system that seemed reasonable was precast double tees. From analyzing Nitterhouse Concrete product catalog's, I picked a 32" x 12" Double Tee with .6 in. diameter, 7 wire, 270ksi strands.

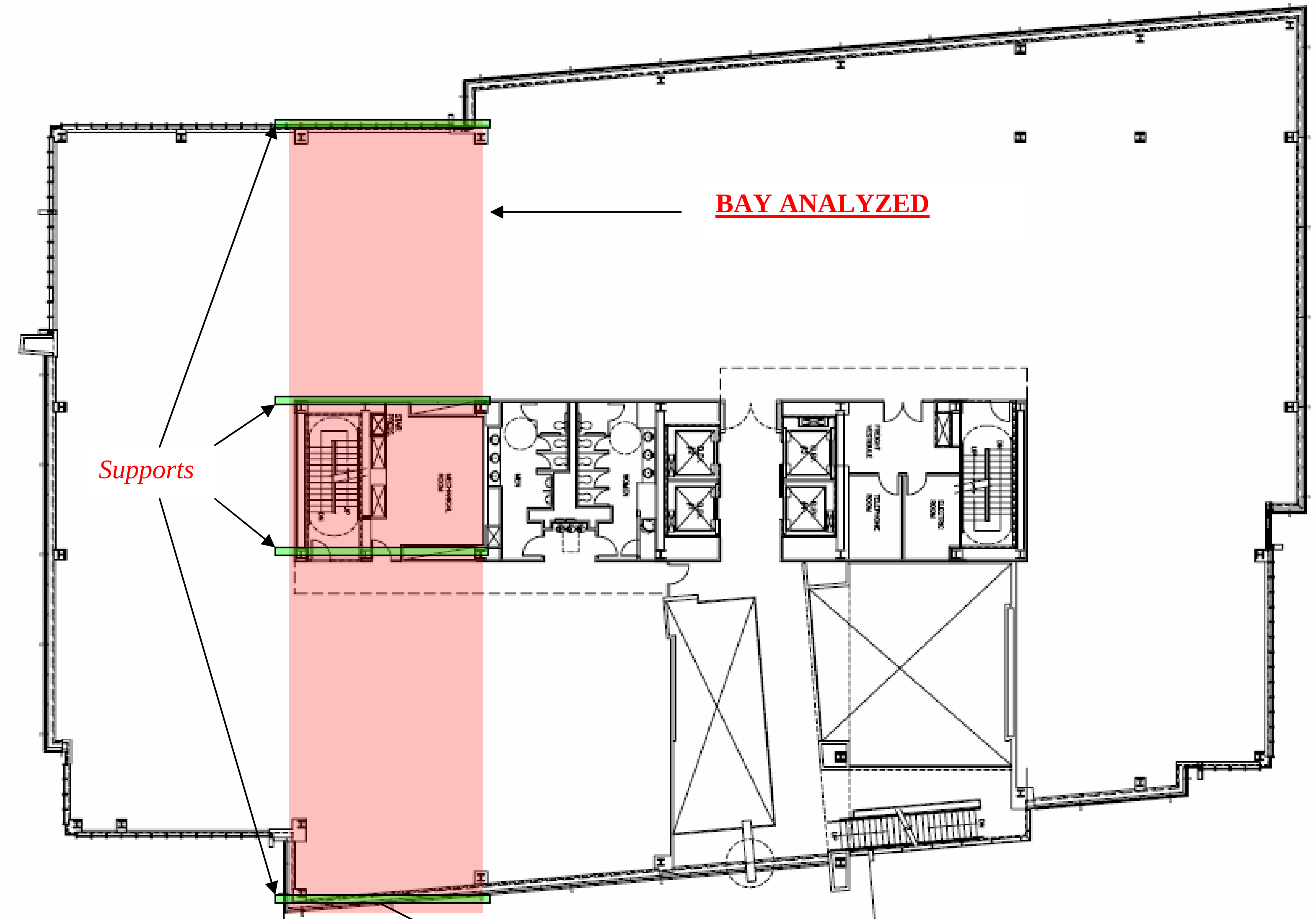
The steel beam layout of the existing system was originally spaced at 9'o.c. So I attempted to change that spacing to 7'. The beam size needed for a 53' span was a W24 x 76 due to the fact that deflection controlled the design. It seemed that a lot of strength was being wasted on deflection, and I can understand why the original design had W27 x 84's spaced at 9' and cambered to resist that deflection. This design seemed to be a failure do to the fact that an extra beam per bay for 9 stories would be very costly.

Overall an alternate steel system is not a very efficient design, the joist and double tees are still probable, but post-tensioning seems like the forerunner for the best alternative design. This is due to the fact that there is a possibility because of the small depth compared to the existing systems, that another floor can be added to the building. In the long run this extra floor can offset any additional construction cost of the post-tensioning system.

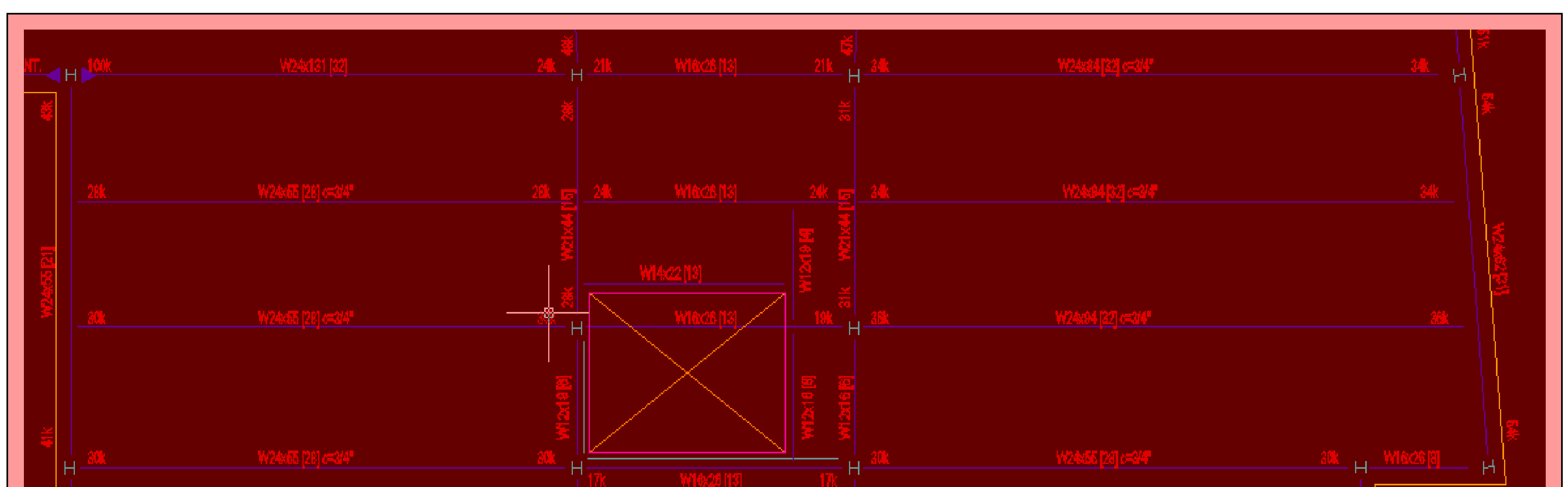
Technical Assignment 2

Sallie Mae HQ is located in Reston, Fairfax County, Virginia. Sallie Mae consists of a steel tower office building and a concrete parking garage. The office building is nine stories above grade and has a ground print of approximately 25,000sf. The parking garage reaches five stories below grade and is approximately 75,000sf. Due to the fact the parking garage must remain concrete makes it invaluable for this assignment. Instead I will analyze a typical floor of the office building.

The office part of Sallie Mae mostly consists of floors that resemble the typical floor below. There are four lines of columns along the long direction (N-S), and girders also span between those columns in the N-S direction. The beams span E-W, and reach spans up to 55 ft. These longer spans require camber due to deflection. The typical floors are made up of 3.25" lightweight concrete over 2" Deep x 18 gage galvanized composite metal deck. The connections consist of primarily simple shear connections with the exception of very few moment connections. Furthermore, there is no typical bay in the system due to the fact that the outside of the building consistently slopes. So to analyze the alternate systems of this building I assumed that the bays were rectangular.



Existing Bay Plan



Alternate Systems:

1. ***Post-Tensioning***
2. ***One way joist system***
3. ***Precast concrete***
4. ***Steel Framing***

Loads:

- ***Live Load = 80 PSF***
- ***Dead Load = (Varies with system)***
- ***SIDL = 20 PSF***
- ***Facade Load = 15 PSF (was not included in calcs)***

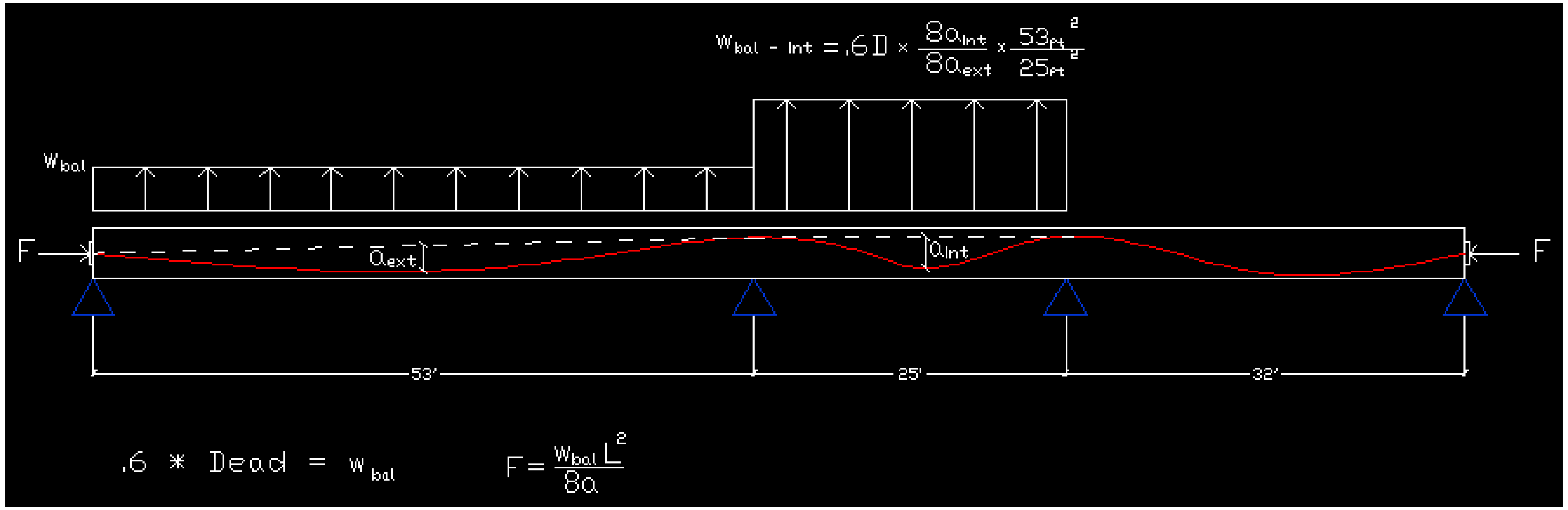
Codes and catalogs used:

- ACI 318-05
- LRFD 3rd edition
- United Steel Deck catalog
- NCJ joist catalog
- Nitterhouse Concrete Products

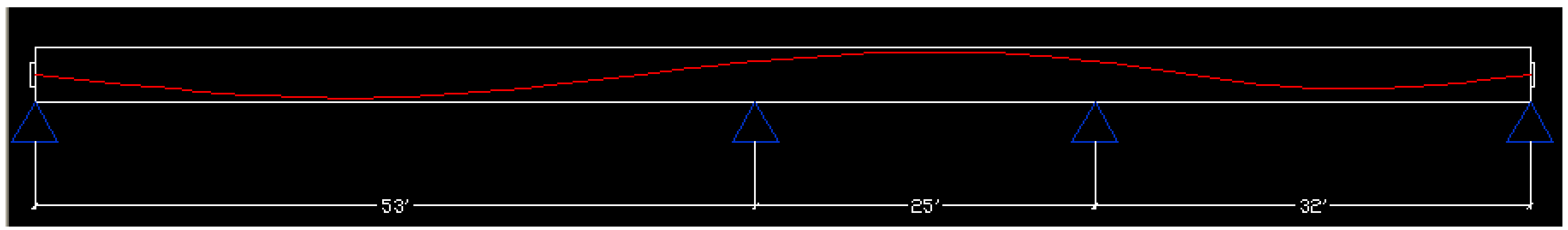
Alternate #1 Post-tensioning

Structural Details
½ in. diameter, 7 wire, 270ksi strands
2 – 4 strands need per foot of width
5000psi - 6000psi concrete required
13 – 14 inch slab thickness

Description:
When examining my structure for the consideration of post tensioning I came to the conclusion that special considerations need to be taken for my building. I began designing with a flat-plate two-way system and came to the conclusion that a 13" to 14" slab must be used to allow for a parabolic tendon profile with maximum drape for the exterior span. Designing this system seemed to be more difficult then expected due to the 53' span being directly followed by a 25' span. If continuous tendons were run across all three spans then the middle span would control the design due to uplift. (As shown below)



The most practical solution to preventing uplift for the middle span is to design a tendon profile that actually pushes down on the slab.



Benefits:
14" thickness
High weight will prevent vibration

Downfalls:
Expensive installation

Special considerations:
If the columns and slabs are the same strength concrete, construction would be safer and faster.

<u>Alternate #2 One way joist system</u>

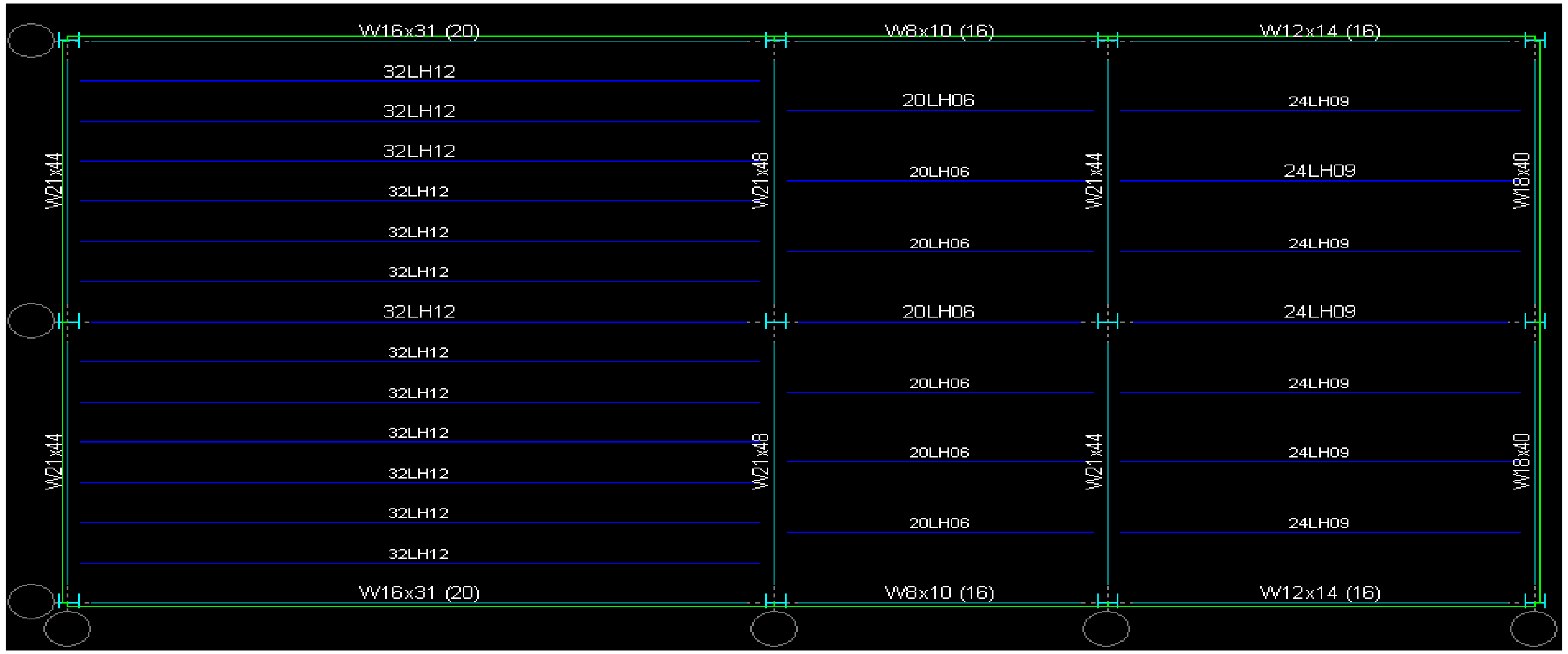
Structural Details
32LH12 LH-SERIES JOIST
4 Rows of horizontal bridging required, 16' max spacing
20 gage B-LOK deck a with lightweight slab depth of 4"

Description:
Joists are very inexpensive and seemed like a reasonable alternative due to there long span capabilities. As shown below joist on the exterior left span are spaced at 4' and the joist on the other two spans are spaced at 7'.

Benefits:
Low self weight
Cheap cost of installation

Downfalls:
32” depth

Special considerations:
Bridging is critical



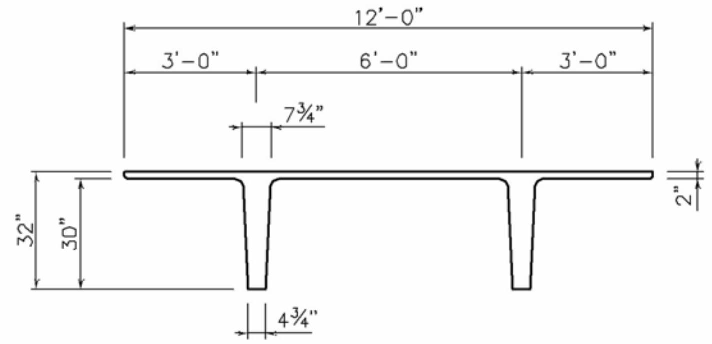
<u>Alternate #3 Precast Concrete</u>

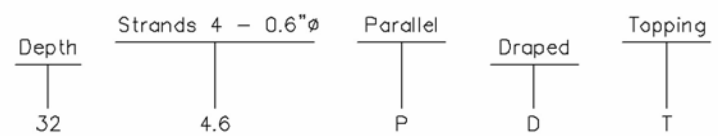
Structural Details
32” x 12” Double Tee
.6 in. diameter, 7 wire, 270ksi strands
10 parallel strands per leg
6000psi concrete

Description:
Another concrete system that can be used with the long spans of Sallie Mae is precast concrete. To decide which member to use, I simply used the NitterHouse design tables. The Double Tee shape was the only precast member that could handle long spans and referring to their charts I chose a 32” x 12” Double Tee with no topping.

Prestressed Concrete
32"x12' Double Tee
(NO TOPPING)

PHYSICAL PROPERTIES	
Precast	
A = 663 in.^{2}	S_b = 2800 in.^{3}
I = 63,361 in.^{4}	S_t = 6762 in.^{3}
Y_b = 22.63 in.	Wt. = 691 PLF
Y_t = 9.37 in.	Wt. = 58 PSF





DESIGN DATA
1. Precast strength @ RELEASE = 3000 PSI. (min.)
2. Precast strength @ 28 days = 6000 PSI.
3. Precast Density = 150 PCF
4. Strand = 0.6"ø 270k LO-relaxation.
5. Maximum bottom tensile stress is $12\sqrt{f'c}$ =930 PSI.
6. All superimposed load is treated as live load in the flexural strength analysis.
7. Flexural capacity is based on stress/strain strand relationships.
8. Maximum moment capacity is critical at midspan for parallel strands and is critical near 0.4 span for draped strands.

Table of Safe Superimposed Loads (lbs. per sq. ft.)

Section	ØM$_n$	Span in Feet																				
	(in. Kips)	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88	90	92
32–4.6 P	6,075	26																				
32–6.6 P	8,806	59	51	44	38	32																
32–8.6 P	11,280	88	79	70	62	55	48	42	37	32												
32–10.6 P	13,476	115	104	93	83	75	67	60	54	48	42	38	33									
32–12.6 P	15,554	140	126	114	103	93	84	76	69	62	56	50	45	40	36	32						
32–14.6 D	20,052	194	176	161	146	134	122	112	102	94	86	78	72	66	59	54	50	45	41	37	33	30
32–16.6 D	22,484				171	156	143	132	121	111	102	93	85	78	71	65	59	53	48	44	40	37
32–18.6 D	24,653										117	107	98	90	83	76	69	63	58	53	48	43

This table is for simple spans and uniform loads. Design data for any of these span–load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths.

NITTERHOUSE
CONCRETE PRODUCTS

2655 MOLLY PITCHER HWY. SOUTH, BOX N
CHAMBERSBURG, PA 17201-0813
717-267-4505 • FAX: 717-267-4518

Benefits:
Low self weight
Cheap cost of installation

Downfalls:
Expensive connections and custom made designs
32” depth

Special considerations:
Special designs may need to be made due to the odd slope of the exterior

Alternate #4 Steel Framing

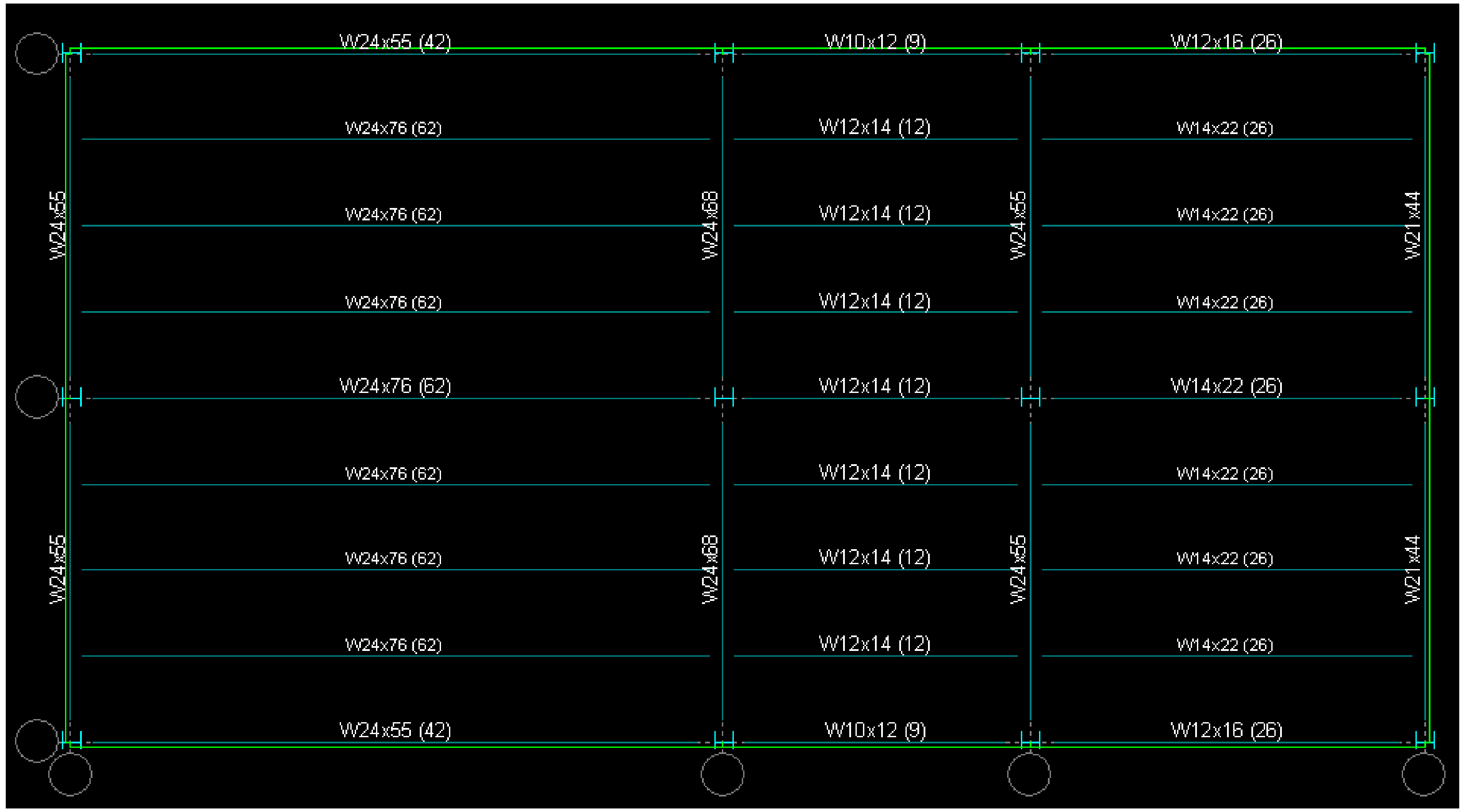
Structural Details
W24 x 76 beams
7’ spacing
20 gage B-LOK deck with a lightweight slab depth of 4”

Description:
The existing design of Sallie Mae had the cambered beams spanning E-W, spaced at 9’ on average. I decided to try spacing these beams at 7’ spacing with the aid of RAM. This reduction allowed for the decks spanning perpendicular to the beams, to be made lighter. This alternative design also allowed W27 x 76 (non-cambered) beams to be used.

Benefits:
Reduced deck size from 5.25” to 4”

Downfalls:
This system due to the increase in number of beams will be more expensive. This includes the cost of steel and the cost of labor.
A possible reduction in fire rating

Special considerations:
Vibration may be dampened by the closer spacing of the beams, but may be increased by the smaller deck size

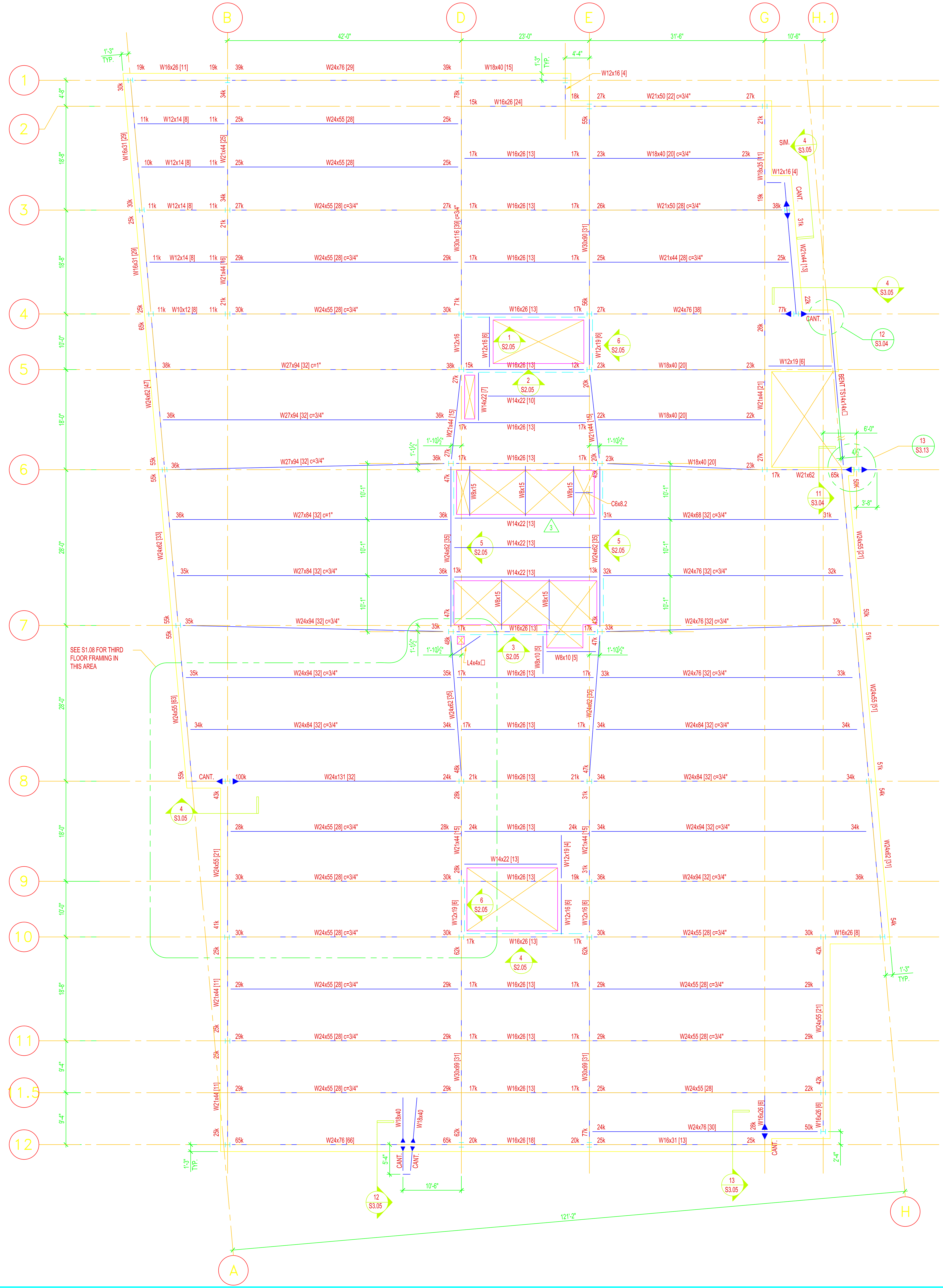


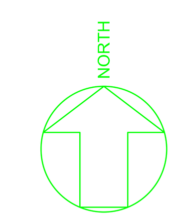
	Cost of building (1 - 5=Expensive)	Constructability	Depth of system	Fire Protection	Vibration	Continue Analysis
Existing Steel System	3	NA	27"	Required fireproofing	NA	NA
Post-Tensioning	5	Difficult procedure for building but very possible	14"	Very Good	Very Good	YES
One way joist system	1-2	Very easy installation but difficult to get to site	32"	Needs fireproofing	Not sure - Good	YES
Precast Concrete	2-3	Easy installation but difficult to get to site	32"	Good	Not Sure - Good	YES
Alternate Steel System	3	OK	24"	Need fireproofing	May be a problem due to thin decking	NO

Remarks:

Overall an alternate steel system is not a very efficient design, the joist and double tees are still probable, but post-tensioning seems like the forerunner for the best alternative design. This is due to the fact that there is a possibility because of the small depth compared to the existing systems, that another floor can be added to the building. In the long run this extra floor can offset any additional construction cost of the post-tensioning system.

Appendix A – Typical Floor





TYPICAL FLOOR FRAMING PLAN (3RD THRU 6TH)

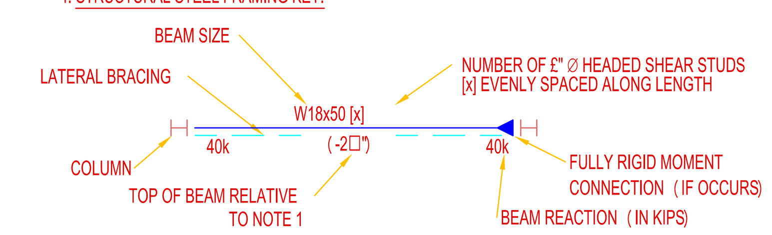
□"=1'-0"

PLAN NOTES:

1. TOP OF SLAB IS AT ELEVATION AS FOLLOWS FOR EACH FLOOR UNLESS NOTED. THIS IS TO BE REFERENCE ELEVATION FOR THIS FLOOR.

   THIRD FLOOR = 431.50'
   FOURTH FLOOR = 445.50'
   FIFTH FLOOR = 459.50'
   SIXTH FLOOR = 473.50'

2. TOP OF STEEL (BOTTOM OF METAL DECK) IS 5□" BELOW TOP OF FLOOR SLAB UNLESS NOTED.
3. STRUCTURAL SLAB TO BE 3□" LIGHTWEIGHT CONCRETE OVER 2" DEEP x 18 GAGE GALVANIZED COMPOSITE METAL DECK (TOTAL THICKNESS = 5□") REINFORCED WITH 6 x 6 - W2.0 x W2.0 WWF.
4. STRUCTURAL STEEL FRAMING KEY:



5. ALL BEAMS ARE SPACED EVENLY BETWEEN COLUMN LINES UNLESS NOTED.
6. SEE S1.00 FOR GENERAL NOTES.
7. SEE S3.05 FOR TYPICAL DETAILS.
8. COORDINATE SIZE AND LOCATION OF ALL OPENINGS AND SLEEVES WITH ARCHITECTURAL, MECHANICAL, ELECTRICAL, AND PLUMBING DRAWINGS.
9. MAXIMUM SPAN OF DECK TO BE 10'-1" UNDER A 3 SPAN CONDITION AND 9'-0" UNDER A 2 SPAN CONDITION. DECK WILL REQUIRE SHORING IF THIS SPAN IS EXCEEDED.

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PROJECT No. 2002 080.00

NO	DATE	REMARKS
	12/03/02	FOOTING & FOUNDATION PERMIT
	01/27/03	SITE WORK BID
	3/3/03	CONCRETE & STEEL BID SET
	3/3/03	ISSUE FOR BUILDING PERMIT
	3/31/03	ISSUE FOR COMPREHENSIVE BID
	4/01/03	ISSUE FOR BUILDING PERMIT
3 4	07/07/03	FOR CONSTRUCTION SET
	08/11/04	PRINTING UPDATE

REVISIONS

DRAWN L.M.R.
CHECKED A.M.P.
APPROVED D.C.
DATE 8/11/04
JOB NUMBER

TYPICAL FLOOR FRAMING PLAN (3RD THRU 6TH)
1/8"=1'-0"

SHEET NO S1.07

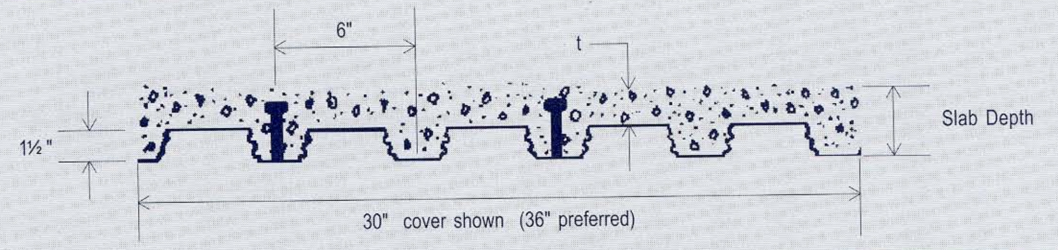
Appendix B – Chart Used

Joist Designation	Approx. Wt in Lbs. Per Linear Ft. (Joists only)	Depth in inches	SAFELOAD* in Lbs. Between		CLEAR SPAN IN FEET															
			28-32		33	34	35	36	37	38	39	40	41	42	43	44	45	46	47	48
24LH03	11	24	11500		342 235	339 226	336 218	323 204	307 188	293 175	279 162	267 152	255 141	244 132	234 124	224 116	215 109	207 102	199 96	191 90
24LH04	12	24	14100		419 288	398 265	379 246	360 227	343 210	327 195	312 182	298 169	285 158	273 148	262 138	251 130	241 122	231 114	222 107	214 101
24LH05	13	24	15100		449 308	446 297	440 285	419 264	399 244	380 226	363 210	347 196	331 182	317 171	304 160	291 150	280 141	269 132	258 124	248 117
24LH06	16	24	20300		604 411	579 382	555 356	530 331	504 306	480 284	457 263	437 245	417 228	399 211	381 197	364 184	348 172	334 161	320 152	307 142
24LH07	17	24	22300		665 452	638 421	613 393	588 367	565 343	541 320	516 297	491 276	468 257	446 239	426 223	407 208	389 195	373 182	357 171	343 161
24LH08	18	24	23800		707 480	677 447	649 416	622 388	597 362	572 338	545 314	520 292	497 272	475 254	455 238	435 222	417 208	400 196	384 184	369 173
24LH09	21	24	28000		832 562	808 530	785 501	764 460	731 424	696 393	663 363	632 337	602 313	574 292	548 272	524 254	501 238	480 223	460 209	441 196
24LH10	23	24	29600		882 596	856 559	832 528	809 500	788 474	768 439	737 406	702 378	668 351	637 326	608 304	582 285	556 266	533 249	511 234	490 220
24LH11	25	24	31200		927 624	900 588	875 555	851 525	829 498	807 472	787 449	768 418	734 388	701 361	671 337	642 315	616 294	590 276	567 259	544 243
			33-39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55	56
28LH05	13	28	14000	14000	337 219	323 205	310 192	297 180	286 169	275 159	265 150	255 142	245 133	237 126	228 119	220 113	213 107	206 102	199 97	193 92
28LH06	16	28	18600	18600	448 289	429 270	412 253	395 238	379 223	364 209	350 197	337 186	324 175	313 166	301 156	291 148	281 140	271 133	262 126	253 120
28LH07	17	28	21000	21000	505 326	484 305	464 285	445 267	427 251	410 236	394 222	379 209	365 197	352 186	339 176	327 166	316 158	305 150	295 142	285 135
28LH08	18	28	22500	22500	540 348	517 325	496 305	475 285	456 268	438 252	420 236	403 222	387 209	371 196	357 185	344 175	331 165	319 156	308 148	297 140
28LH09	21	28	27700	27700	667 428	639 400	612 375	586 351	563 329	540 309	519 291	499 274	481 258	463 243	446 228	430 216	415 204	401 193	387 183	374 173
28LH10	23	28	30300	30300	729 466	704 439	679 414	651 388	625 364	600 342	576 322	554 303	533 285	513 269	495 255	477 241	460 228	444 215	429 204	415 193
28LH11	25	28	32500	32500	780 498	762 475	736 448	711 423	682 397	655 373	629 351	605 331	582 312	561 294	540 278	521 263	502 249	485 236	468 223	453 212
28LH12	27	28	35700	35700	857 545	837 520	818 496	800 476	782 454	766 435	737 408	709 383	682 361	656 340	632 321	609 303	587 285	566 270	546 256	527 243
28LH13	30	28	37200	37200	895 569	874 543	854 518	835 495	816 472	799 452	782 433	766 415	751 396	722 373	694 352	668 332	643 314	620 297	598 281	577 266
			38-46	47-48	49	50	51	52	53	54	55	56	57	58	59	60	61	62	63	64
32LH06	14	32	16700	16700	338 211	326 199	315 189	304 179	294 169	284 161	275 153	266 145	257 138	249 131	242 125	234 119	227 114	220 108	214 104	208 99
32LH07	16	32	18800	18800	379 235	366 223	353 211	341 200	329 189	318 179	308 170	298 162	288 154	279 146	271 140	262 133	254 127	247 121	240 116	233 111
32LH08	17	32	20400	20400	411 255	397 242	383 229	369 216	357 205	345 194	333 184	322 175	312 167	302 159	293 151	284 144	275 137	267 131	259 125	252 120
32LH09	21	32	25600	25600	516 319	498 302	480 285	463 270	447 256	432 243	418 230	404 219	391 208	379 198	367 189	356 180	345 172	335 164	325 157	315 149
32LH10	21	32	28300	28300	571 352	550 332	531 315	512 297	495 282	478 267	462 254	445 240	430 228	416 217	402 206	389 196	376 186	364 178	353 169	342 162
32LH11	24	32	31000	31000	625 385	602 363	580 343	560 325	541 308	522 292	505 277	488 263	473 251	458 239	443 227	429 216	416 206	403 196	390 187	378 179
32LH12	27	32	36400	36400	734 450	712 428	688 406	664 384	641 364	619 345	598 327	578 311	559 295	541 281	524 267	508 255	492 243	477 232	463 221	449 211
32LH13	30	32	40600	40600	817 500	801 480	785 461	771 444	742 420	715 397	690 376	666 354	643 336	621 319	600 304	581 288	562 275	544 262	527 249	511 238
32LH14	33	32	41800	41800	843 515	826 495	810 476	795 458	780 440	766 417	738 395	713 374	688 355	665 337	643 321	622 304	602 290	583 276	564 264	547 251
32LH15	35	32	43200	43200	870 532	853 511	837 492	821 473	805 454	791 438	776 422	763 407	750 393	725 374	701 355	678 338	656 322	635 306	616 292	597 279
			42-46	47-56	57	58	59	60	61	62	63	64	65	66	67	68	69	70	71	72
36LH07	16	36	16800	16800	292 177	283 168	274 160	266 153	258 146	251 140	244 134	237 128	230 122	224 117	218 112	212 107	207 103	201 99	196 95	191 91
36LH08	18	36	18500	18500	321 194	311 185	302 176	293 168	284 160	276 153	268 146	260 140	253 134	246 128	239 123	233 118	227 113	221 109	215 104	209 100
36LH09	21	36	23700	23700	411 247	398 235	386 224	374 214	363 204	352 195	342 186	333 179	323 171	314 163	306 157	297 150	289 144	282 138	275 133	267 127
36LH10	21	36	26100	26100	454 273	440 260	426 248	413 236	401 225	389 215	378 206	367 197	357 188	347 180	338 173	328 165	320 159	311 152	303 146	295 140
36LH11	23	36	28500	28500	495 297	480 283	465 269	451 257	438 246	425 234	412 224	401 214	389 205	378 196	368 188	358 180	348 173	339 166	330 159	322 153
36LH12	25	36	34100	34100	593 354	575 338	557 322	540 307	523 292	508 279	493 267	478 255	464 243	450 232	437 222	424 213	412 204	400 195	389 187	378 179
36LH13	30	36	40100	40100	697 415	675 395	654 376	634 359	615 342	596 327	579 312	562 298	546 285	531 273	516 262	502 251	488 240	475 231	463 222	451 213
36LH14	36	36	44200	44200	768 456	755 434	729 412	706 392	683 373	661 356	641 339	621 323	602 309	584 295	567 283	551 270	535 259	520 247	505 237	492 228
36LH15	36	36	46600	46600	809 480	795 464	781 448	769 434	744 413	721 394	698 375	677 358	656 342	637 327	618 312	600 299	583 286	567 274	551 263	536 252

USD United Steel Deck, Inc.

LRFD

B-LOK



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; $\mathbf{S_p}$ and $\mathbf{S_n}$ are the section moduli for positive and negative bending (in.³); $\mathbf{R_b}$ and $\phi \mathbf{V_n}$ 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, $\phi \mathbf{M_{nf}}$.

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. $\phi \mathbf{M_{nf}}$ 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). $\mathbf{A_c}$ 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. $\mathbf{S_c}$ is the section modulus of the "cracked" concrete composite slab; in.³ per foot of width. $\mathbf{I_{av}}$ is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.⁴ per foot of width. The $\mathbf{I_{av}}$ transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. $\phi \mathbf{M_{no}}$ 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). $\phi \mathbf{V_{nt}}$ 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. $\mathbf{A_{wwf}}$ is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

DECK PROPERTIES

Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.6	0.470	0.165	0.195	0.206	1320	2620	0.52
20	0.0358	1.9	0.570	0.212	0.247	0.260	1880	3170	0.63
19	0.0418	2.3	0.670	0.260	0.292	0.304	2500	3680	0.73
18	0.0474	2.6	0.760	0.308	0.337	0.349	3200	4160	0.83
16	0.0598	3.3	0.960	0.400	0.434	0.439	4750	5210	1.05

COMPOSITE PROPERTIES

	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.00	38.19	21.3	0.255	29	0.91	3.1	25.66	2980	5.14	6.82	6.91	0.023
	4.50	44.78	24.8	0.297	34	1.11	4.3	31.13	3460	4.90	6.53	6.61	0.027
	4.75	48.07	26.5	0.318	37	1.21	5.1	33.93	3700	4.80	6.39	6.47	0.029
	5.00	51.37	28.3	0.339	39	1.31	5.9	36.77	3960	4.70	6.27	6.34	0.032
	5.50	57.96	32.1	0.380	44	1.52	7.9	42.53	4480	4.52	6.04	6.11	0.036
	5.75	61.26	34.0	0.401	46	1.62	8.9	45.45	4750	4.43	5.94	6.01	0.038
	6.00	64.55	36.0	0.422	49	1.73	10.1	48.39	5030	4.36	5.84	5.91	0.041
	6.50	71.15	40.1	0.464	53	1.94	12.8	54.31	5420	4.21	5.66	5.73	0.045
	6.75	74.44	42.2	0.484	56	2.04	14.3	57.30	5570	4.15	5.58	5.64	0.047
	7.00	77.74	44.3	0.505	58	2.15	15.9	60.30	5720	4.09	5.50	5.56	0.050
20 gage	4.00	45.45	21.3	0.255	29	1.09	3.3	30.45	2980	6.02	8.01	8.11	0.023
	4.50	53.44	24.8	0.297	34	1.32	4.7	36.98	3460	5.72	7.64	7.74	0.027
	4.75	57.44	26.5	0.318	37	1.44	5.5	40.33	3700	5.59	7.48	7.57	0.029
	5.00	61.44	28.3	0.339	39	1.56	6.4	43.73	3960	5.47	7.33	7.41	0.032
	5.50	69.43	32.1	0.380	44	1.81	8.5	50.64	4480	5.25	7.05	7.13	0.036
	5.75	73.43	34.0	0.401	46	1.93	9.7	54.14	4750	5.15	6.92	7.00	0.038
	6.00	77.43	36.0	0.422	49	2.06	11.0	57.67	5030	5.06	6.80	6.88	0.041
	6.50	85.42	40.1	0.464	53	2.31	13.8	64.79	5600	4.89	6.58	6.66	0.045
	6.75	89.42	42.2	0.484	56	2.44	15.4	68.38	5890	4.81	6.48	6.56	0.047
	7.00	93.42	44.3	0.505	58	2.57	17.1	71.98	6190	4.73	6.39	6.46	0.050
19 gage	4.00	52.41	21.3	0.255	29	1.25	3.6	35.09	2980	6.70	8.94	9.05	0.023
	4.50	61.81	24.8	0.297	34	1.52	5.1	42.65	3460	6.36	8.52	8.62	0.027
	4.75	66.51	26.5	0.318	37	1.66	6.0	46.54	3700	6.21	8.33	8.43	0.029
	5.00	71.20	28.3	0.339	39	1.80	6.9	50.49	3960	6.08	8.16	8.25	0.032
	5.50	80.60	32.1	0.380	44	2.09	9.1	58.52	4480	5.83	7.84	7.93	0.036
	5.75	85.30	34.0	0.401	46	2.23	10.4	62.60	4750	5.71	7.69	7.78	0.038
	6.00	90.00	36.0	0.422	49	2.38	11.7	66.71	5030	5.61	7.56	7.65	0.041
	6.50	99.39	40.1	0.464	53	2.67	14.8	75.02	5600	5.41	7.31	7.39	0.045
	6.75	104.09	42.2	0.484	56	2.82	16.5	79.20	5890	5.32	7.19	7.28	0.047
	7.00	108.79	44.3	0.505	58	2.97	18.3	83.41	6190	5.24	7.08	7.17	0.050
18 gage	4.00	58.42	21.3	0.255	29	1.40	3.8	39.20	2980	7.34	9.64	9.93	0.023
	4.50	69.07	24.8	0.297	34	1.70	5.4	47.67	3460	6.96	9.19	9.44	0.027
	4.75	74.40	26.5	0.318	37	1.85	6.3	52.03	3700	6.79	8.99	9.23	0.029
	5.00	79.73	28.3	0.339	39	2.01	7.3	56.46	3960	6.64	8.80	9.03	0.032
	5.50	90.39	32.1	0.380	44	2.33	9.6	65.49	4480	6.36	8.46	8.67	0.036
	5.75	95.72	34.0	0.401	46	2.50	11.0	70.08	4750	6.24	8.30	8.51	0.038
	6.00	101.05	36.0	0.422	49	2.66	12.4	74.71	5030	6.12	8.15	8.35	0.041
	6.50	111.71	40.1	0.464	53	3.00	15.6	84.06	5600	5.90	7.88	8.07	0.045
	6.75	117.04	42.2	0.484	56	3.16	17.4	88.78	5890	5.80	7.75	7.94	0.047
	7.00	122.37	44.3	0.505	58	3.33	19.3	93.52	6190	5.71	7.63	7.82	0.050
16 gage	4.00	58.42	21.3	0.255	29	1.70	4.3	39.20	2980	8.58	10.76	11.12	0.023
	4.50	69.07	24.8	0.297	34	2.08	6.0	47.67	3460	8.12	10.27	10.61	0.027
	4.75	74.40	26.5	0.318	37	2.27	7.0	52.03	3700	7.92	10.05	10.38	0.029
	5.00	79.73	28.3	0.339	39	2.46	8.1	56.46	3960	7.73	9.84	10.17	0.032
	5.50	90.39	32.1	0.380	44	2.86	10.7	65.49	4480	7.40	9.46	9.78	0.036
	5.75	95.72	34.0	0.401	46	3.07	12.1	70.08	4750	7.25	9.28	9.60	0.038
	6.00	101.05	36.0	0.422	49	3.27	13.7	74.71	5030	7.11	9.12	9.43	0.041
	6.50	111.71	40.1	0.464	53	3.69	17.2	84.06	5600	6.85	8.81	9.11	0.045
	6.75	117.04	42.2	0.484	56	3.90	19.2	88.78	5890	6.74	8.67	8.96	0.047
	7.00	122.37	44.3	0.505	58	4.11	21.3	93.52	6190	6.62	8.54	8.82	0.050

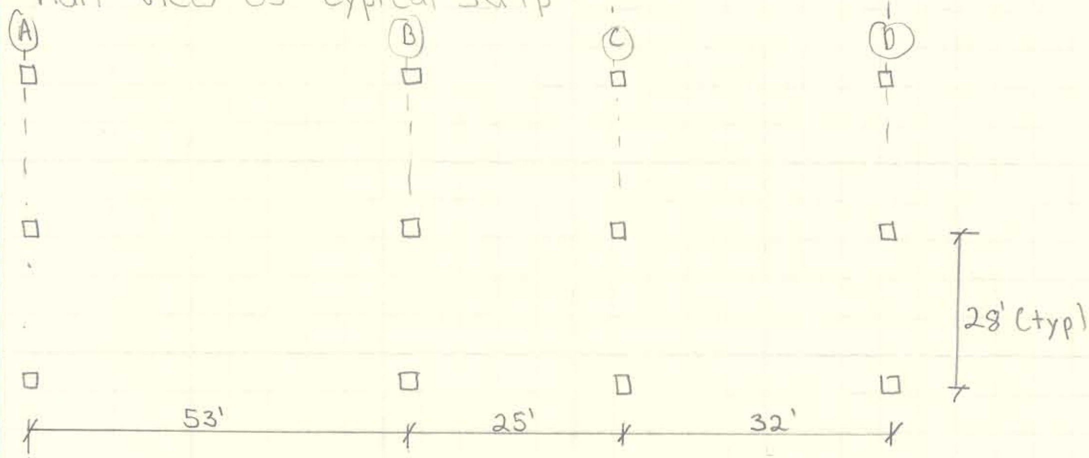
Light Weight

Appendix C – Calculations

Frank Burke
Oct 27 2005

- Sallie Mae Alternate Design #1 Post-Tensioning

• Plan View of typical Strip



Materials:

Concrete: Normal weight 150 pcf
$f'_c = 5000$ psi
$f'_{ci} = 3000$ psi

Rebar: $f_y = 60{,}000$ psi

PT: Unbonded tendons
½" Ø, 7 wire strands, A = .153
$f_{pu} = 270$ ksi
Estimated prestress losses = 15 ksi (ACI 18.6)
$f_{se} = .7(270\text{ksi}) - 15\text{ksi} = 174$ ksi (ACI 18.5.1)
$P_{eff} = A * f_{se} = (.153)(174\text{ksi}) = 26.6$ Kips/tendon

Determine Preliminary Slab Thickness
Start with $L/h = 50$
$h = 53 \times 12/50 \approx$ try 12" slab

Loading
DL = 150 psf
SIDL = 20 psf
LL_o = 80 psf

IBC 1607.9.1 allows for LL reduction

$L = 80\text{psf}\left(.25 + 15/\sqrt{1 \times (25' \times 28')}\right) = .82 \times 80\text{psf} = 65.6 \rightarrow 66\text{psf}$

Design of E-W Frame

Use Equivalent Frame method, ACI 13.7 (excluding sections 13.7.7.4-5)
Total bay width between centerlines = 28ft
No pattern loading required, since $LL/DL < 3/4$ (ACI 13.7.6)

Calculate Section properties

Two-way slab must be designed as Class U (ACI 18.3.3), Gross cross-sectional properties allowed (ACI 18.3.4)

$$A = b \cdot h = 28' \times 12'' \times 12''/ft = 4032 in^2$$

$$S = bh^2/6 = 28' \times 12''/ft \times 12^2/6 = 8064 in^3$$

Set design parameters

Allowable stresses: Class U (ACI 18.3.3)

At time of jacking (ACI 18.4.1)
$f'_{ci} = 3{,}000 psi$
Compression $= .60 f'_{ci} = .6(3{,}000 psi) = 1{,}800 psi$
Tension $= 3\sqrt{f'_{ci}} = 3\sqrt{3000} = 164 psi$

At service loads (ACI 18.4.2(a) and 18.3.3)
$f'_c = 5000 psi$
Compression $= .45 f'_c = .45(5{,}000 psi) = 2{,}250 psi$
Tension $= 6\sqrt{f'_c} = 6\sqrt{5000} = 424 psi$

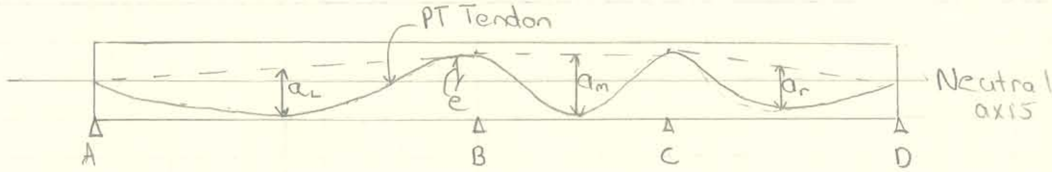
Average precompression limits
$P/A = 125$ psi min (ACI 18.12.4)
$= 300$ psi max

Target Load Balances
Use 80% of DL (selfweight) for slabs
$0.8 \times 150 psf = 120 psf$

Cover requirements
IBC2003 - Use 3/4" for 2 hour fire rating (assume carbonate aggregate)

Tendon profile

E-W direction



Tendon ordinate	Tendon (CG) Location*
Exterior support - anchor	6"
Interior support - top	11"
Interior span - bottom	2"
End span - bottom	LE - 1" RE - 2"

(CG) = center of gravity
* measure from bottom of slab

$a_{INT} = 11" - 2" = 9"$
$a_{LEFT\ END} = (6" + 11")/2 - 1" = 7.5"$
$a_{RIGHT\ END} = (6" + 11")/2 - 2" = 6.5"$

eccentricity, e, is the distance from the center of tendon to the neutral axis; varies along the span

End Span will govern maximum required post-tensioning force

$W_b = 120\text{psf} \times 28' = 3{,}360 \text{ plf} = 3.36 \text{ klf}$

$P = 3.36\text{klf} \times 53^2/8 \times (7.5"/12') = 1887\text{k}$

Check Precompression Allowance

$\#\text{ tendons} = 1887\text{k}/(26.6\text{k/tendon}) = 71 \text{ tendons}$

$P_{actual} = 71 \text{ tendons} \times 26.6\text{k/tendon} = 1888.6\text{k}$

$W_b \text{ revised} = (1888.6\text{k}/1887\text{k})(3.36\text{klf}) = 3.36\text{klf}$ no adjustment

Determine actual Precompression stress

$P_{actual}/A = 1888.6\text{k} \times 1000 / 28 \times 12 \times 12 = 468 \text{ psi}$

Check Interior Span Force

$P = (3.36\,k/ft)(25ft)^2 / [8(9''/12)] = 350k < 1887k$
- Less force required in center bay

Check Right end span

$P = (3.36\,k/ft)(32ft)^2 / [8(6.5/12)] = 794k$
- Less force required in right end span

How much force will be balanced on right end span with 1888k

$w_b = 1888 \times 8 \times (6.5/12)/32^2 = 7.98\,k/ft > .15ksf \times 28' = 4.2\,k/ft$
- there would be uplift on slab

Check int span

$w_b = 1888 \times 8 \times (9/12)/25^2 = 18.12\,k/ft > 4.2k/ft$

Try changing tendon profile

	(CG) location	
Exterior support anchor	6''	$a_{INT} = \frac{(11+9)}{2} - 4 = 6''$
Interior support top	11'' LE 9'' RE	$a_{LE} = 7.5''$
Interior span - bottom	4''	
End span - bottom	LE - 1'' RE - 3''	$a_{RE} = \left(\frac{9+6}{2}\right) - 3'' = 4.5''$

USE .7 DL for wb

$w_b = 150\,psf \times .7 \times 28' = 2940\,plf = 2.94\,klf$
$P = 2.94\,klf \times 53^2 / 8(7.5/12) = 1651\,k$

Check precompression allowance

$\#\ tendons = 1651K/(26.6k/tendon) = 63\ tendons$

$P_{actual} = 63\ tendons \times 26.6k/_{tendon} = 1676\,k$

$w_b\ revised = (1676k/1651k)(2.94klf) = 2.98klf$

Determine actual precompression stress

$P_{actual}/A = 1676K \times 1000 / 28 \times 144 = 415\,psi$

How much force will be balanced on right end span with 1676k

$w_b = 1676k \times 8 \times$

no good - try 13" slab

new $DL = \frac{13}{12} \times 150\,PCF = 162.5\,PSF$

new $A = 28' \times 12"/ft \times 13 = 4368\,in^2$

$S = 28' \times 12"/ft \times 13^2/6 = 9464\,in^3$

revise tendon profile

	(CG) location	
Exterior support anchor	6"	$a_{INT} = \left(\frac{12+7}{2}\right) - 7 = 2.5"$
Interior support top	12" LE 7" RE	$a_{LE} = \left(\frac{6+12}{2}\right) - 1 = 8"$
Interior span - bottom	7"	
End span - bottom	LE = 1" RE = 4"	$a_{RE} = \left(\frac{7+6}{2}\right) - 4 = 2.5"$

USE $.6DL = 97.5\,psf$

$w_b = 97.5\,psf \times 28' = 2730\,plf = 2.73\,klf$

$P = 2.73\,klf \times 53^2/8(8/12) = 1437K$

Check precompression allowance

$\#\ tendons = 1437k / 26.6k/tendon = 55\ tendons$

$P_{actual} = 55 \times 26.6k/tendon = 1463\,k$

$w_b = (1463k/1437k)(2.73\,klf) = 2.78\,klf$

Determine actual precompression stress

$P_{actual}/A = 1463K \times 1000 / 28 \times 12 \times 13 = 335\,psi$

Force on right end span with 1463k

$w_b = 1463k \times 8 \times (2.5/12)/32^2 = 2.38\,klf < .1625\,KSF \times 28 = 4.55\,klf$

$2.38/4.55 = 52\%$ % of DL is resisted

INT SPAN

$w_b = 1463k \times 8 \times (2.5/12)/25^2 = 3.9\,klf < 4.55\,klf$

$= 85\%$ of DL

E-W Interior Frame $P_{eff} = 1463K$

Equivalent Frame Characteristic (E-W)

- $K_c = \frac{4 E_c I_c}{L_n - 2h}$ (Assume 30" x 30" Columns - 12' tall)

- $I_c = 30^4/12 = 67500 \text{ in}^4$

- Total $K_c = \frac{4 \times 1 \times 67500 \text{ in}^4}{(12 \times 12) - 2(13)} \times 2 = 4575$ in-lb/rad/E_{cc}

- Torsional constant

$$C = \sum \left(1 - .63 \frac{x}{y}\right) \frac{x^3 y}{3} = \left(1 - .63 \times \frac{13}{12}\right) 13^3 \times 12/3 = 2790$$

- Torsional stiffness of the slab at the column line

$$K_t = \sum \frac{9 E_{cs} C}{L_2 (1 - c_2/L_2)^3} = \frac{9 \times 1 \times 2790}{12 \times 28 \left(1 - \frac{30}{12 \times 28}\right)^3} \times 2 = 198 \text{ in} \cdot \text{lb/rad}/E_{cc}$$

- Equivalent column stiffness

$$K_{ec} = \left(\frac{1}{K_c} + \frac{1}{K_t}\right)^{-1} = \left(\frac{1}{4575} + \frac{1}{198}\right)^{-1} = 190 \text{ in} \cdot \text{lb/rad}/E_{cc}$$

Slab stiffness

- $I_s = 1/12 \times 12 \times 13^3 = 2197 \text{ in}^4/\text{ft} \times 28' = 61516 \text{ in}^4$ $S = 12 \times 13^2 / 6 = 338 \text{ in}^3$

- $k_s = \frac{4 E_{cs} I_s}{L_n - c_1/2}$

- $k_{s(A,BL)} = \frac{4 \times 1 \times 61516 \text{ in}^4}{12 \times 53 - 30/2} = 396$ in-lb/rad/E_{cs}

- $k_{s(BR,CL)} = \frac{4 \times 1 \times 61516 \text{ in}^4}{12 \times 25 - 30/2} = 863$ in-lb/rad/E_{cs}

- $k_{s(CR,D)} = \frac{4 \times 1 \times 61516 \text{ in}^4}{12 \times 32 - 30/2} = 666$ in-lb/rad/E_{cs}

$DF = K_s / \Sigma K \quad \Sigma K = K_{ec} + K_s(\text{left}) + K_s(\text{right})$

For A, $DF = 396/(396+190) = .676$

For B_L, $DF = 396/(396+863+190) = .273$

For B_R, $DF = 863/(863+396+190) = .595$

For C_L, $D_F = 863/(863+666+190) = .502$

For C_R, $D_F = 666/(863+666+190) = .387$

For D, $D_F = 666/(190+666) = .778$

* Fixed End Moments

	LL	DL	wbal
For AB	$66\text{psf} \times (53)^2 \times \cancel{12} / \cancel{12} = 185$ in·k	$182.5 \times 53^2 = 513$ in·k	$\frac{2.78}{28} \times 53^2 = 278$ in·k
For BC	$66 \times 25^2 = 41$ in·k	$182.5 \times 25^2 = 114$ in·k	$\frac{3.9}{28} \times 25^2 = 87$ in·k
For CD	$66 \times 32^2 = 67.5$ in·k	$182.5 \times 32^2 = 187$ in·k	$\frac{2.38}{28} \times 32^2 = 87$ in·k

- Assume carry over factor of .5

* Moment distribution of LL

	A	B	B	C	C	D
DF	.676	.273	.595	.502	.387	.778
FEM	-185	185	-41	41	-67.5	67.5
Dist	125	-39.3	-85.7	13.3	10.3	-52.5
CO	-19.6	62.5	6.6	-42.8	-26.2	5.1
Dist	13.2	-18.9	-41.1	34.6	26.7	-4
CO	-9.5	6.6	17.3	-20.5	-2	13.3
Dist	6.4	-6.5	-14.2	11.3	8.7	-10.3
CO	-3.2	3.2	5.6	-7.1	-5.1	4.3
Dist	2.1	-2.4	-5.2	6.1	4.7	-3.3
M_{LL}	-70.6	190.2	-157.7	35.9	-50.4	20.1

* Moment Distribution of DL

	A		B		C	D
DF	.676	.273	.595	.502	.387	.778
FEM	-513	513	-114	114	-187	187
Dist	346.8	-108.9	-237.4	36.6	28.3	-145.5
CO	-54.5	173.4	18.3	-118.7	-72.7	14.1
Dist	36.8	-52.3	-114.1	96.1	74.1	-11.0
CO	-26.2	18.4	48.1	-57	-5.5	37
Dist	17.7	-18.1	-39.5	31.4	24.2	-28.8
CO	-9.6	8.8	15.7	-19.8	-14.4	12.1
Dist	6.1	-6.7	-14.6	17.2	13.2	-9.4
M_{DL}	-195.3	527.5	-437.6	99.8	-139.9	55.6

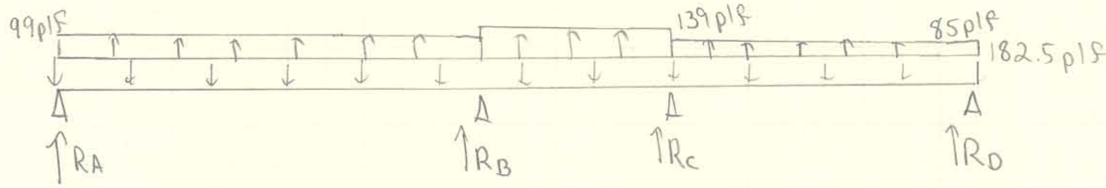
* Moment Distribution of Wbal

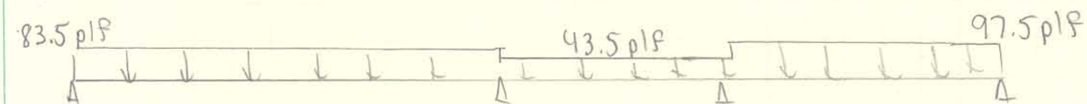
DF						
FEM	278	-278	87	-87	87	-87
Dist	-187.9	52.1	113.6	0	0	67.7
CO	26.1	-94	6	56.8	33.8	0
Dist	-17.6	25.7	55.9	-45.5	-35.1	0
CO	12.8	-8.8	-22.8	28	0	-17.5
Dist	-8.7	8.6	18.8	-14	-10.8	13.6
CO	4.3	-4.3	-7.0	9.4	6.8	-5.4
Dist	-2.9	3.1	6.8	-8.1	-6.3	4.2
M_{BAL}	104.1	-295.6	252.3	-60.5	75.5	-24.4

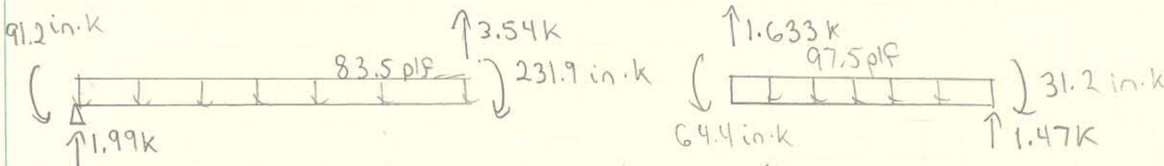
At time of jacking

DL = 182.5 psf

wbal$_{AB}$ = 99 psf wbal$_{BC}$ = 139 psf wbal$_{CD}$ = 85 psf







$\Sigma M_B = 0 \quad -F_A \times 53' - \frac{231.9''k}{12} + \frac{91.2''k}{12} + .0835 \times 53 \times 53/_2 = 0$

$\Sigma M_c = 0 \quad F_D \times 32' - \frac{31.2 \text{ in·k}}{12} + \frac{64.4 \text{ in·k}}{12} - \frac{.0975 \times 32 \times 32}{2} = 0$

$\Sigma M_c = 0 \quad -1.99k \times 78' - F_B \times 25' + 1.47 \times 32' + .0835 \times 53 \times \left(\frac{53}{2} + 25\right)$
$+ \frac{.0435 \times 25^2}{2} - \frac{.0975 \times 32^2}{2} + \frac{91.2 \text{ in·k}}{12} - \frac{31.2 \text{ in·k}}{12} = 0$

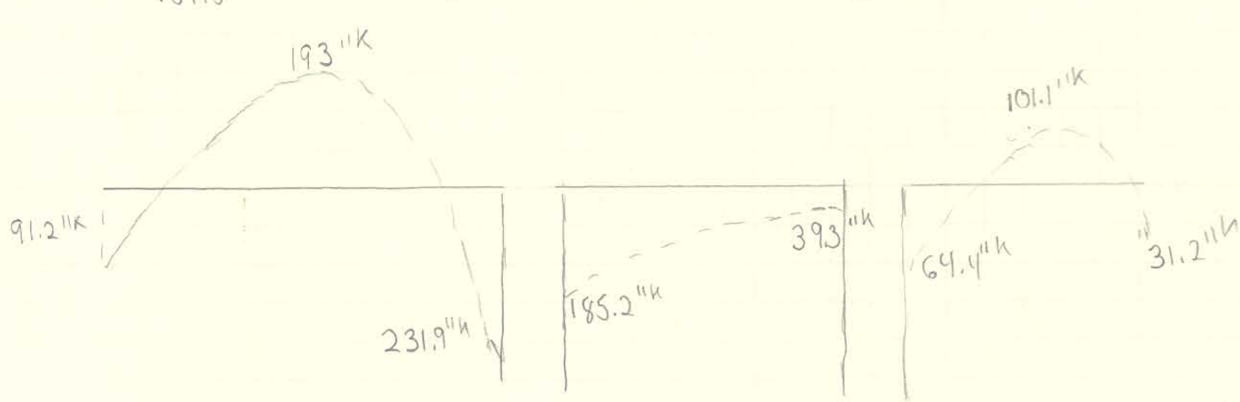
$\Sigma F_y = 0 \quad -.0835 \times 53' - .0435 \times 25 - .0975 \times 32 + 1.99 + 1.47 + 3.54 + F_c = 0$

$\Sigma M_B = 0 \quad -1.99 \times 53' + F_c \times 25' + 1.47 \times 57' + \frac{.0835 \times 53^2}{2} - \frac{.0435 \times 25^2}{2}$
$- .0975 \times 32 \times (16+25) - \frac{31.2}{12} + \frac{91.2}{12} = 0$

$1.99/_{.0835} = 23.8'$ max pos moment at left end $= -91.2 + 1.99 \times \frac{23.8}{2} \times 12 = 193''k$

$1.99 - .0835 \times 53 = 1.1045/_{.0435} = 25.39 \rightarrow -185.3 + 1.1045 \times \frac{25.39}{2} \times 12 = -17''k$ ok

$1.47/_{.0975} = 15'$ from right end $= -31.2$ink $+ 1.47 \times \frac{15}{2} \times 12 = 101.1''K$



Check stresses for jacking

At support A

$A = 12 \times 13 = 156 in^2$ $S = 12 \times 13^2/6 = 338 in^3$

$f_t = \frac{91.2''k}{338 in^3} - \frac{1463/28}{156 in^2} = -65.1$ (compression)

$f_c = 605$ psi

At midspan AB

$f_t = \frac{193000''lb}{338} - \frac{1463/28}{156} = 236$ psi $> 3\sqrt{f'_c}$ fails

$f_c = 905.9$ psi

At support B (left)

$f_t = \frac{231900''lb}{338} - \frac{1463/28}{156} = 351$ psi $> 3\sqrt{f_c}$ fails

$f_c = 1021$ psi

* This Design won't work - need to revise tendon layout

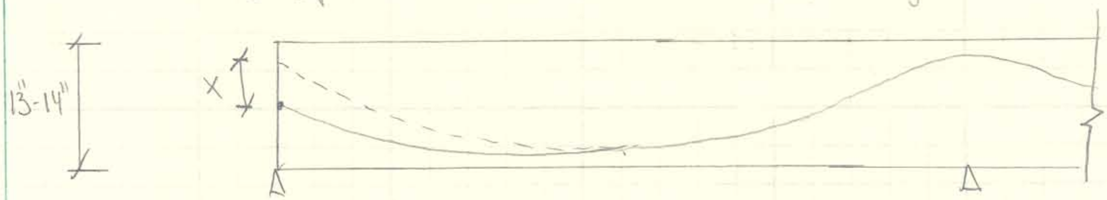
* Refering to the moment diagrams on the previous page the moments of the long span control

* When designing this slab .6DL was used as Wbal because if continous tendons are run through all the spans then the middle span which is much smaller than the exterior span will control the design. And if wbal isn't lower then .7DL for the exterior span, then when the force carries it self to the middle span → uplift will occur.

Alternatives from the previous post-tension design

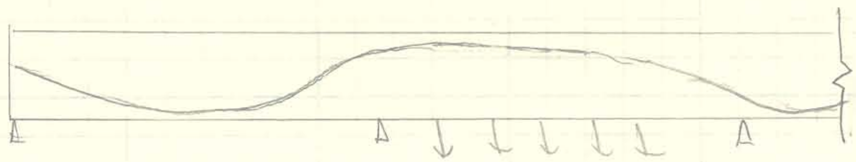
* Ways to make this work
  - need to lower moment in the exterior spans
  - need to prevent uplift in interior span

* Lowering moment while keeping this a practical design
  - a max slab thickness needs to be made at 14" or the self weight will start overrunning the design
  - the end plate can be highered to cause a higher drape which will make Wbal higher

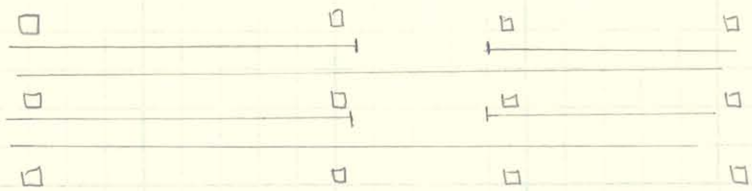


  - concrete strength can be highered to 6000 psi to allow more lee way in the design
  - a larger Pactual can be implented but this needs to be limited to 300-350 max for an effective design

* While highering Wbal ways to prevent uplift on interior span
  - one way is to revise tendon lay out to counterattack the high neg moments from the exterior span



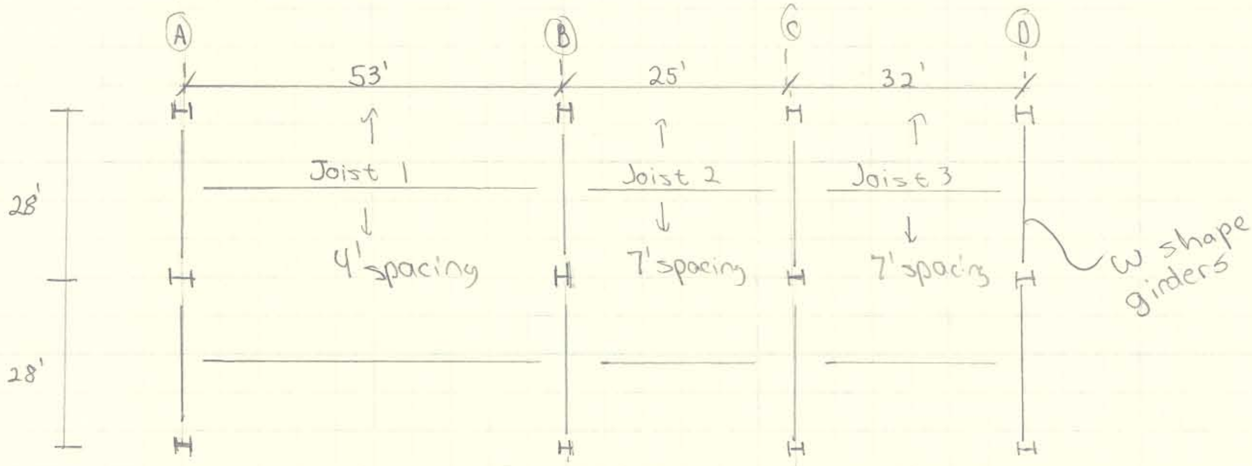
  - Another method is to only run half the tendons into the interior span



* Post-tensioning is definitely a probable design for sallie mae but further analysis needs to be made before an exact post-tensioning solution can be made but the definites about the system are:
  - 13"-14" slab
  - 5000 psi - 6000 psi concrete
  - 2-4 tendons per ft

Frank Burke
Oct 27 2005

- Sallie Mae Alternate Design #2 Steel joists



- 53' space will control design

Loading

80 PSF Live Load → = 80psf ~~(.25 + 15/√(1+(25'+28')) = 66psf~~
20 PSF SIDL / Assume 50 psf for decking and slab
Assume 50plf for joist

* Joist 1
Assume spacing of 4'

LL = 80 psf × 4' = 320 plf
DL = 70psf × 4' = 280 plf + 50 plf = 330plf
TL = 650plf

Try a 32LH12 LH-SERIES JOIST (FROM THE NCJ GUIDE)

FOR 53' of span LL for L/360 deflection = 364 plf
DL max = 641plf
W = 27plf
d = 32"

Check for decking type
(Reference page 32 of the USD guide)

Try 22 Gage B-Lok Deck with a slab depth of 4"

W = 29 psf + 1.6psf = 30.6 psf
Max unshored span - 3 span condition = 6.91' ok

* Joist 1 with known loading

- spacing = 4'

LL = 320 plf

DL = (20 + 30.6) × 4' + 27plf = 229.4 plf

TL = 549.4 plf

Stay with 32LH12 joist

- LH series joist chord and web sections are based on 36 ksi minimum yield strength
- They must extend at least 4" on to support
- Bridging is critical for LH joists
- Max. Spacing of horizontal bridging = 16' (Table 104.5.1 pg 48 NCJ)
- Required # of 4 Rows of bridging (pg XI) of NCJ)

* Stay with 22 Gage B-Lok Deck

Assume joist 3 = joist 2

* joist 3

Assume spacing of 7'

LL = 80 psf × 7 = 560 plf

DL = 70psf × 7 + 27plf = 517plf

assume 32LH12

TL = 1067 plf

Try a 32LH12 again

Safe Load = 36400 lbs / (32 + .67) = 1114 plf ok

For Live Load

$450 plf \times (49 + .67)^2 / (32 + .67)^2 = 1040 plf$ ok

Use 32LH12

Decking

Use 20 gage B Lok Deck with a slab depth of 4"

W = 31 psf

Max unshored span = 8.11' ok

can withstand up to 365 psf LL > 1.6 × 80

* For easiness of ordering and erecting use 32LH12 joist

* Use 20 gage B-Lok with depth of 4" - Lightweight

Frank Burke
Oct 27 2005

- Sallie Mae Alternate Design #3 Precast-Prestressed

Loads
SIDL = 20PSF
LL = 80PSF LL reduction (assume none)
TL = 100PSF

53' span will control design

32" depth double tee beams with no topping are satisfactory

* Using tables from Nitterhouse Concrete products

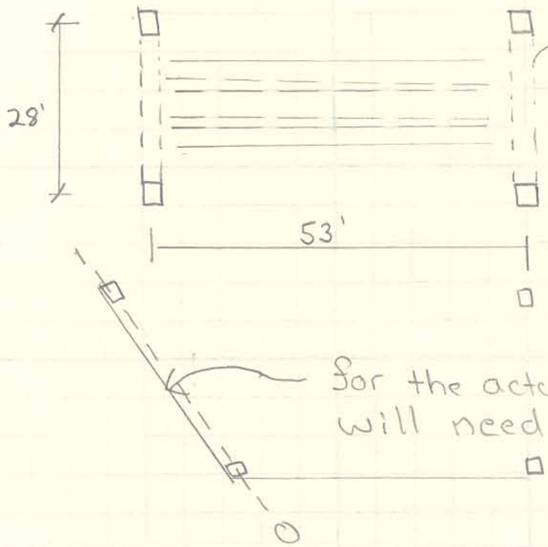
32" x 12" Double Tee (No topping)
$f'_c = 6000$ psi
f_{ci} must be 3000 psi (min) @ Release
W = 150PCF
strand = .6" Ø 270K LO-relaxation
max bottom tensile stress is $12\sqrt{f'_c} = 930$PSI

Table of Safe Superimposed Loads (lbs. per sq ft)

Section	ØMn (in. Kips)	Span 52	Span 54
32-10.6P	13,476	115	104

10 per deep section
Parallel strand
109psf capacity

Typical Bay - (rectangle for analysis)



Girders are assumed to be 32" for equal depth

For the actual design special cut precast-tees will need to be made to match column line

Frank Burke
Oct 27 2005

- Sallie Mae Alternate Design #4   Steel respaced from original design

Loads   AI = 53 × 14 = 742 ft^2

65 psf → LL $L = 80psf\left(.25 + \frac{15}{\sqrt{742}}\right) = 65psf$
20psf → SIDL
31 psf → Decking + Slab
70 plf → for beam

spacing = 7'

65 psf × 7' = 455 plf
(20psf + 31psf) × 7' + 70 plf = 427 plf
TL = 1.2 × 427 + 1.6 × 455 = 1240 plf

$M_{max} = \frac{wl^2}{8} = \frac{1.24 klf \times 53^2}{8} = 435^{lk}$

- Decking - Try 20 Gage B-Lok Deck

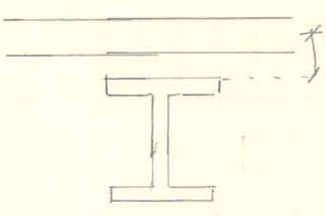
Total thickness = 4"
deck height = 1½"
LTWT concrete = 115 PCF
W = 29psf + 1.9psf = 31psf
Max unshored span = 8.11 ft

beff = smaller of (spacing = 7' × 12" = 84"
span/4 = 53 × 12/4 = 165")

beff = 84"
f'c = 3ksi
Fy = 50ksi

Try W21x44
A = 13 in^2 d = 20.7   tf = .45   bf = 6.5   tw = .35

Cc = .85 (3 ksi) (84") × 2.5" = 214 K
Ts = 50 × 13 in^2 = 650 K


2.75"

$214k \times 2.75" - 650k \times \frac{20.7}{2} + 2(t \times 6.5) \times 50 \times \frac{t}{2}$
t = 4.346

* Assume PNA is at 7
min = 455 > 435 ^{lk}
$\Sigma Q_n = \frac{163k}{21k} \times 2 = 16$ studs   ok

strength is ok

will deflection be a problem $I = 843$

$$\Delta = \frac{5wl^4 \times 1728}{384EI} = \frac{5 \times .882 \times 53^4 \times 1728}{384 \times 29000 \times 843} = 6.4''$$ major failure

Check deflection using composite section

$$I_{eff} = I_s + \sqrt{(\Sigma Q_n / C_f)}\,(I_{tr} - I_s)$$

Deflection controls this design

try to stay within $L/500 = \frac{53 \times 12}{500} = 1.272''$

$I \geq 4500\ in^4$

or camber is needed

Try 24×76 $Q_n = 13.3$? error in Ram

$C = 411$

From Ram $I_{tr} = 4135\ in^4$

$I_{eff} = 3883$

Live Load $= \frac{.455 \times 5 \times 53^4 \times 1728}{384 \times 29000 \times 3883} = .7''$ oK

USE 24×76