Technical Report 2

University Hospitals
Case Medical Center
Cancer Hospital

11100 Euclid Avenue Cleveland, Ohio

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Structural Option
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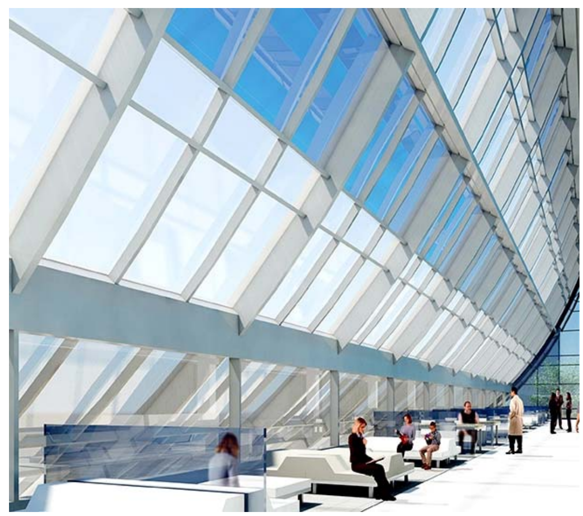
http://www.engr.psu.edu/ae/thesis/portfolios/2009/dcm234

Executive Summary

The University Hospitals Case Medical Center Cancer Hospital is a 9 story research and patient care facility located in Cleveland, Ohio. Its infrastructure consists of steel and steel composite members which have been carefully arranged in order to conform to the modular architectural design system known as the *Universal Grid*, allowing full optimization of available space for varying use. Sloped curtain walls envelope the Cancer Hospital, consisting of exterior glazing and curved steel. The new Cancer Hospital will serve as an addition to the adjacent Case Medical Center which will integrate medical services once spread through 7 different buildings.

The purpose of this report is to evaluate and determine the feasibility of 4 distinct structural slab systems in relation to the Cancer Hospital. The slab systems selected include the existing Composite Slab in conjunction with a Non-Composite Slab, a Precast Hollow Core Slab, A Two-Way Concrete Flat Slab w/ Drop Panels, and a Two-Way Post-Tensioned Flat Slab. A typical bay adhering to the 31’-6” by 31’-6” dimensions of the *Universal Grid* has been selected for analysis with each of the slab systems.



Each system has been designed in accordance with applicable codes and analyzed for cost, weight, deflection, depth, change in building frame, constructability, fireproofing, and feasibility. As instructed general vibration characteristics have been however, in depth calculations will be performed in a later report.

Low weight and the use of the compressive strength of concrete make the existing composite deck economical and effective however disadvantages include a high floor depth, bulky steel supporting members, and a need for fire proofing. The non-composite slab system exhibits similar advantageous characteristics although, due to even more sizeable obstructing members requiring higher floor depths as well as cost, this slab system seems inefficient when compared to the composite system. The precast hollow core system has shown to be the most constructible and economical but it will cause the highest amount of architectural change. Providing a ceiling free of supporting beams and girders, the two-way flat slab system with drop panels is a commonly used, cost efficient system which will add verticality to the typical bay however it has proven to be the heaviest and possibly the most labor intensive of all the systems. The post-tension two-way slab system will provide all the same advantages of the typical two-way concrete system although with a lower slab thickness, allowing for less mass weight and an increase in useable vertical space.

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Introduction

The University Hospitals Case Medical Center Cancer Hospital will integrate patient care and cancer research in a new and innovative way. Architecturally, the Cancer Hospital will reflect this cutting edge link by joining adjacent buildings together while serving as a primary gateway to the UHCMC campus located in Cleveland, Ohio.

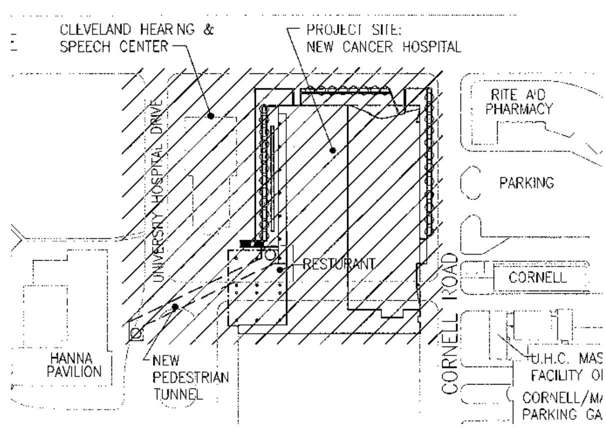
The Cancer Hospital design fulfills the wishes of former facility cancer patients in creating an appealing and comfortable environment as opposed to the sterile feel of the past. This is accomplished through use of strong architectural accents including the Cancer Hospital's most dominating feature, its curved facade. A universal grid system consisting of 31'-6" modular bays has been incorporated into design to optimize floor space for varying uses. Clinical pods have been designed for treatment of specific patient populations.

Medical services which were previously distributed among seven facilities will now be performed under one roof to optimize cancer research, education, and patient care while providing an architecturally appealing exterior as well as a warm and inviting natural interior.

Existing Structural Systems

Foundation

The Cancer Hospital consists of drilled piers transferring load to caissons for the gravity columns with the combined use of grade beams for the lateral force resisting frames. The drilled gravity piers/caissons range 30" to 60" in diameter depending on location. The drilled piers/caissons receiving lateral load are typically 66" in diameter. Along the south side, 36" thick spread footings, typically 48" by 72", have been used to carry gravity load along the existing adjacent Case Medical Center Hospital. The grade beams which carry the lateral load to the drilled piers/caissons are typically 24" by 24" and consist of Grade 60, #7 reinforcement bars. All foundations are made from concrete having a compressive strength of 4000psi with



the exception of the caissons and spread footings, which have a strength of 3000psi.

The soil on site has been classified as hard shale. Thus, giving the caissons used in the foundation an end bearing capacity of 50kpf with a skin friction capacity of 10psi below the first 5' of shale. The typical minimum penetration depth for the gravity piers/caissons is 3'-0" and for the lateral, 16'-6".

Floor System

Being a primarily steel structure, the Cancer Hospital has a fairly typical composite steel beam and girder framing system. The typical composite floor slab is 5-1/4" thick using 3000psi lightweight composite concrete, an 18 gauge 2" galvanized steel deck, and 3-1/2" metal studs. This composite floor slab is used on all but the 2nd and 8th floors. The second floor requiring a thicker slab with normal weight concrete due the vibration requirements of the surgery and imaging rooms and the 8th due to the increased load from the mechanical system. The slab used on these floors consists of 6-1/2" thick 3000psi normal weight concrete, an 18 gauge 2" galvanized steel deck, and 3-1/2" metal studs. Both decks are reinforced with 6x6 Welded Wire Fabric; W4.5xW4.5 for the first floor, W3.5xW3.5 for the second and eighth floors, and W2.1x2.1 for the remaining floors.

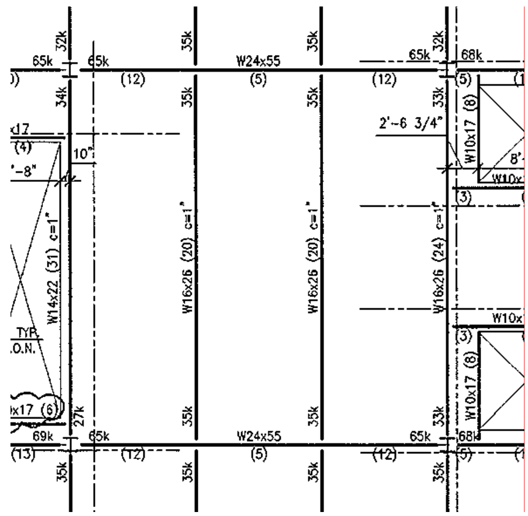


Figure 1

Framing

Bay sizes conform to the universal grid, having a typical size of 31'-6" by 31'-6". Infill beams are typically W16x26 around the interior and W14x22 around the exterior framing into W24x68 girders (see Figure 1). For the larger breaks in the slab, such as the elevator shafts, HSS 8x4x1/4 tubes have been used. On the 4th and roof level, moment connections are utilized in conjunction with cantilevered beams in order to support the curved exterior façade. Smaller breaks used for mechanical, plumbing, etc., consist typically of W10x17. Columns consist of a typical W14 member decreasing in size with elevation and spliced every other floor starting with the second. All steel members conform to ASTM A-992, Grade 50 unless otherwise noted.

Ground Level

At the ground level, a 6" thick slab-on-grade is used with Grade 60 #5 reinforcement bars spaced @ 18"oc EW. The slab rests on a 10 mils min. vapor barrier on compacted granular material over a 2000psi mud slab. In the northeastern and southeastern section of the building special research equipment has been placed requiring a 12" thick slab-on-grade with Grade 60 #5 reinforcement bars placed @ 12"oc EW.

Machine Room

A 31'-0" by 63'-0" machine room is located on the 8th floor. Framing is similar to the rest of the structure however with shorter spans and larger members to account for the additional weight. Beams range from W21 beams to W40 beams depending on specific equipment.

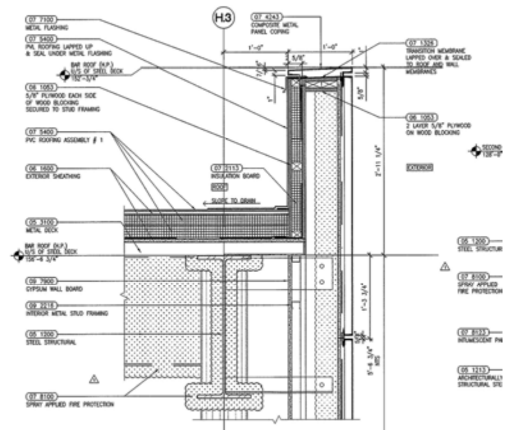


Figure 2

Roof System

The roof of the Cancer Center is a sloped deck with a 63'-0' by 63'-0" elevator penthouse perched at the southern corner. The roof slopes downward along the east and west sides of the building and allows drainage to the center third. The roof system consists of a 3"x20ga type 'N' galvanized steel deck. The roof deck rests typically rests on W14x22 beams framing into W21x44 girders with W18x35 beams being used to support mechanical equipment spaced uniformly across the building's center. Roof decks lower than the top of the 8th level consist of 1.5"x20ga. type 'B' galvanized steel deck (see Figure 2).

Lateral System

Lateral forces are resisted by a series of concentrically braced frames located at the center of the building near the main elevator core and along isolated points of the exterior bays (see Figure 4). This system consists of four chevron braces and single diagonal brace, which are used both in the north/south direction as well as the east/ west direction. Each brace typically consists of a 31'-6" wide W24 beam, a 15'-0' tall W14 column, and two HSS8 size diagonal members (see Figure 3). Structural brace members beyond the 8th floor increase in size due to increased lateral loads.

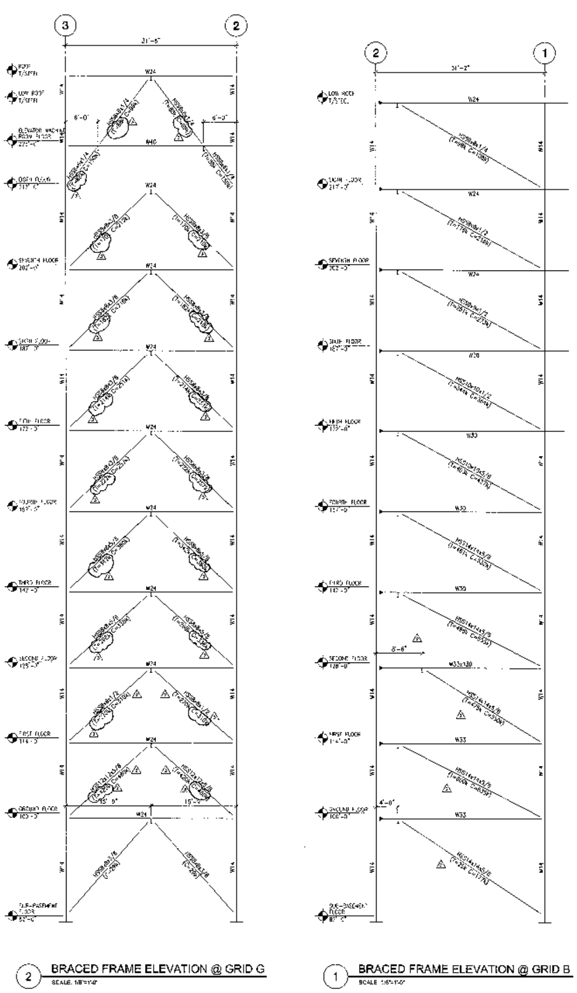


Figure 3

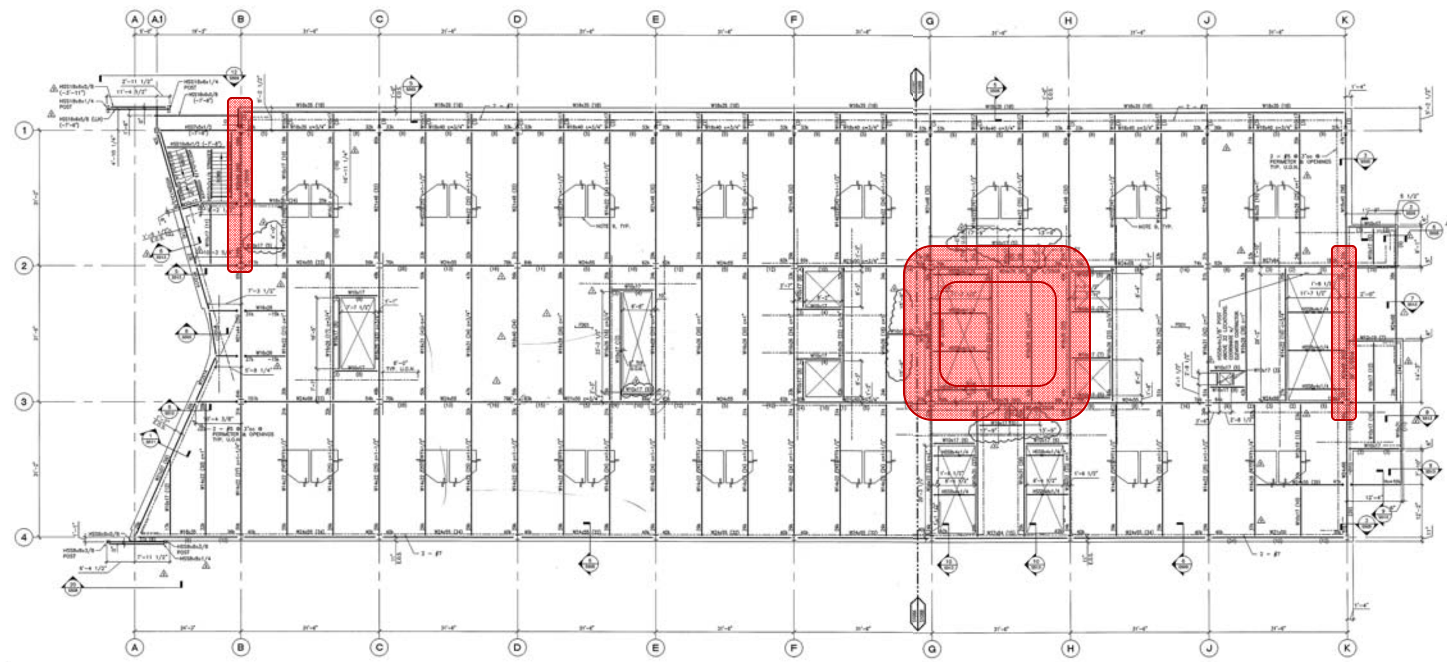


Figure 4

Slab System Overview

Slab Selection and Loading

The selection for slab comparison has been taken from the 6th floor along column lines E through F and 2 through 3 (see Figure 5). It is a typical 31’-6” by 31’-6” bay which conforms to the *Universal Grid* and has no penetrations or lateral resisting members (see Appendix E). No imaging or operating rooms are located on the 6th floor making its slab not as susceptible to the stringent vibration requirements of other locations in the building. The bay takes load from an assembly area with fixed seats and part of a corridor. Both comply with a live loading of 60 psf according to ASCE 7-05. This load is reducible however no lessening of load is obtained from the reduction. This is consistent with the Load Key Diagram provided by the structural engineer (see Appendix E). Dead loads of 5 psf for ceiling partitions and 10 psf for suspended mechanical equipment have also been used in the following calculation. Deflections have been evaluated based on a limit of L/360 for live loads and L/240 for total loads in accordance with ASCE7-05.

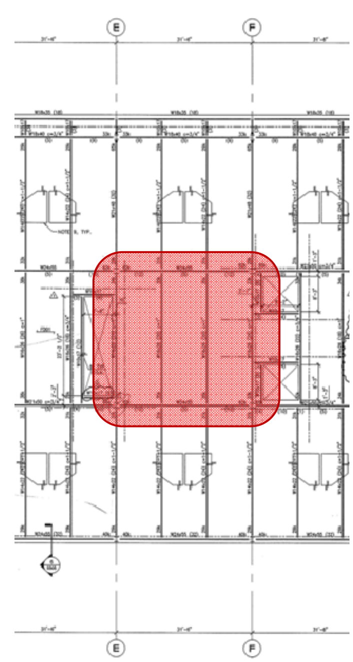


Figure 5

A summary of loads is as follows:

Dead Load:

Floor Self Weight .... Varies

Ceiling Partitions .... 5 psf

Suspended Mechanical Equipment .... 10 psf

Total: 15 psf (+SW)

Live Load:

Floor Load .... 60 psf

Total: 60 psf

Composite / Non-Composite

Composite Floor Slab

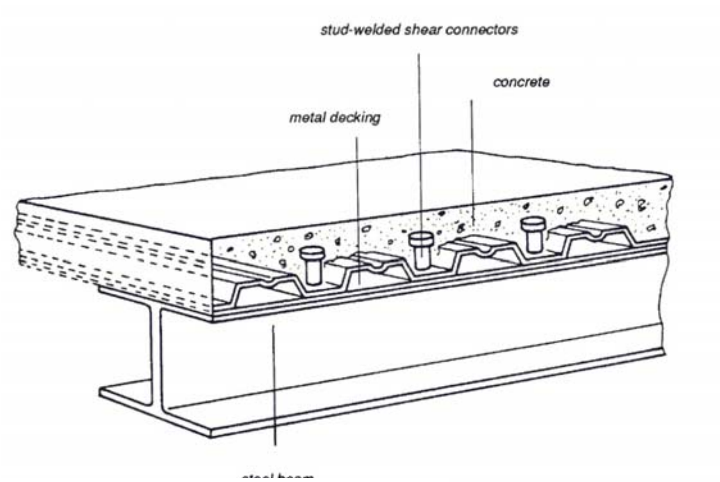


Figure 6

A composite 5-1/4" metal deck using 3000psi lightweight concrete with 3-1/2" metal studs has been selected for use in the Cancer Hospital. The metal deck is made of 18 gauge, 2" galvanized steel with a 3-1/4" concrete topping (see Figure 1). W2.1x2.1 Welded Wire Fabric has been used to provide additional strength in the slab. In order to evaluate conditions which have led to this design, a model was created in *RAM Structural Systems* using this exact design criteria (see Figure 6). In order to accurately evaluate the girder size, all adjacent bays and their loads have also been modeled. This included a reduced 40 psf hospital room live load for the exterior bays, a .225 kpf line load around open shafts on the surrounding bays, and a .3 kpf line load on the exterior side of the top and bottom end bays (see Appendix E). Under these loads, this evaluation has determined that the most efficient design consists of W16x26 beams framing into W24x55 girders. These member sizes are consistent with those of the existing design. All calculations are provided in Appendix A.

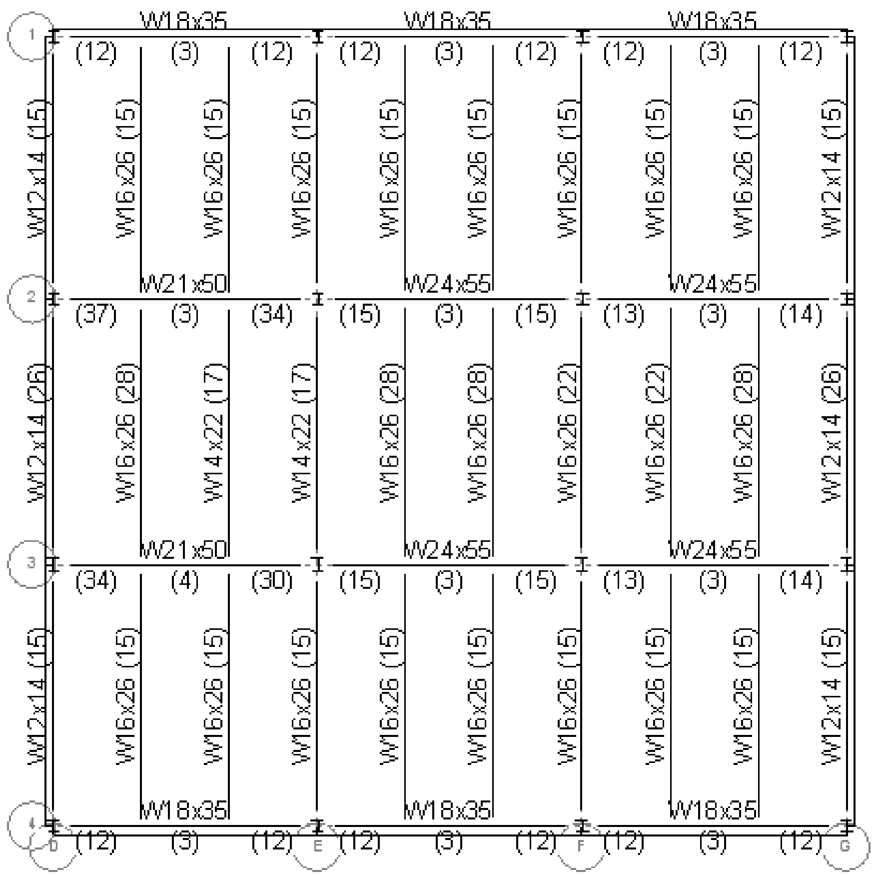


Figure 6

Non-Composite Floor Slab

An evaluation of a non-composite metal deck was also performed in RAM Structural Systems in order to allow for future computation of vibration characteristics (see Figure 7). Since no compressive forces are able to be transferred from the concrete slab to the steel beam or girder supporting it, those members will now be larger. This additional stiffness gained from the increase in size of these members may prove later to be a cost saving way of maintaining the required vibration criteria in the rest of the structure. This floor slab has been designed similarly to the composite metal deck however with the absence of shear studs. The non-composite design has yielded members consisting of W18x40 beams framing into W27x84 girders. All calculations are provided in Appendix A.

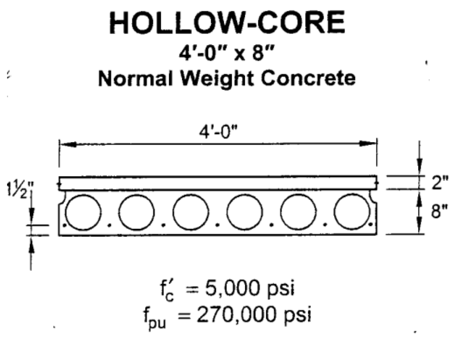


Figure 7

Precast Hollow-Core

A precast slab system has been selected for investigation due to the modularity of the Cancer Center. The large majority of bays do adhere to the 31'6" by 31'-6" dimensions *Universal Grid*, making a prefabricated system ideal for cost savings and simplicity in construction. For this evaluation, Precast Concrete Hollowcore Planks have been selected for use. Hollowcore planks are superior in acoustic performance, fire resistance, and vibration reduction. Sizing and selection of Hollowcore Planks has been accomplished using Sections 2 and 10.5 of the PCI Design Handbook/6th Edition (see Figure 7). Strength calculations are based strain compatibility with the bottom tension of a member being limited to 7.5√f'c. 8 – 4' wide, 6" thick planks (68-S) have been used in the typical bay (see Figure 8). Each plank has a compressive strength of 5,000psi and is assumed to support a 2" normal weight topping.

The Hollow-core planks will rest on the beam and girder system similarly to the existing design. Ram structural system has been used to design the supporting steel members consisting of W18x35 beams framing into W24x55 girders. See Appendix B for calculations.

4HC8 + 2

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

Strand Designation Code	Span, ft																										
	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$ *; see pages 2–7 through 2–10 for explanation.*

Figure 8

Two-Way Flat Slab w/ Drop Panels

A Two-Way Slab supported by drop panels has been evaluated in order to determine the feasibility of the use of a concrete system. The use of drop panels will allow the column locations to remain the same and adhere to the *Universal Grid*. This will result in an increase in slab thickness due to the absence of beams and girders. Concrete has great fireproofing qualities and good vibration absorption, characteristics held paramount in the design of the Cancer Hospital. Selection of this system would not be ideal in the event of an abundance of long spans in the structure however the floor plans of the hospital are held as a majority to 31'-6".

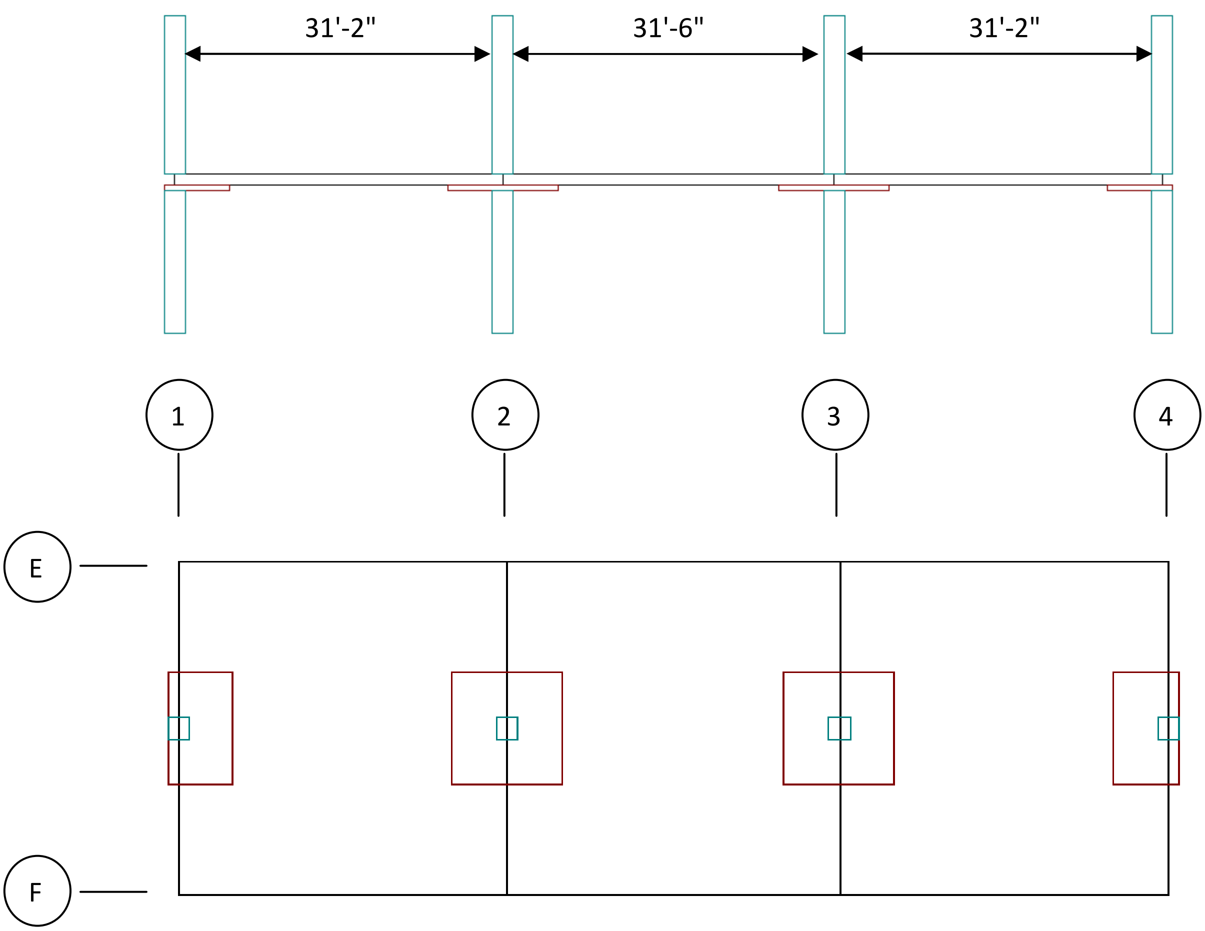


Figure 9

PCA Slab has been used to provide data for design and information obtained has been checked in accordance with ACI 318-08. For analysis, a three bay section has been selected in order to account for edge slabs and beams (see Figure 9). The Two-Way slab has been designed with 5000psi, 150 pcf normal weight concrete. A 12" slab thickness has been used in accordance with the minimum thickness provided by Table 9-5.c of ACI 318-08. An arbitrary typical column size of 24" by 24" been selected for analysis purposes with an assumption of 50% moment absorption. Drop panels have been sized at 5'-3" by 5'-3" with an 8" thickness and are adequate to sustain punching shear and loading. An efficient 60ksi rebar layout has been determined consisting of #5 bars for both the column and middle strips of the section selected (see Figure 10). All calculations are provided in Appendix C.

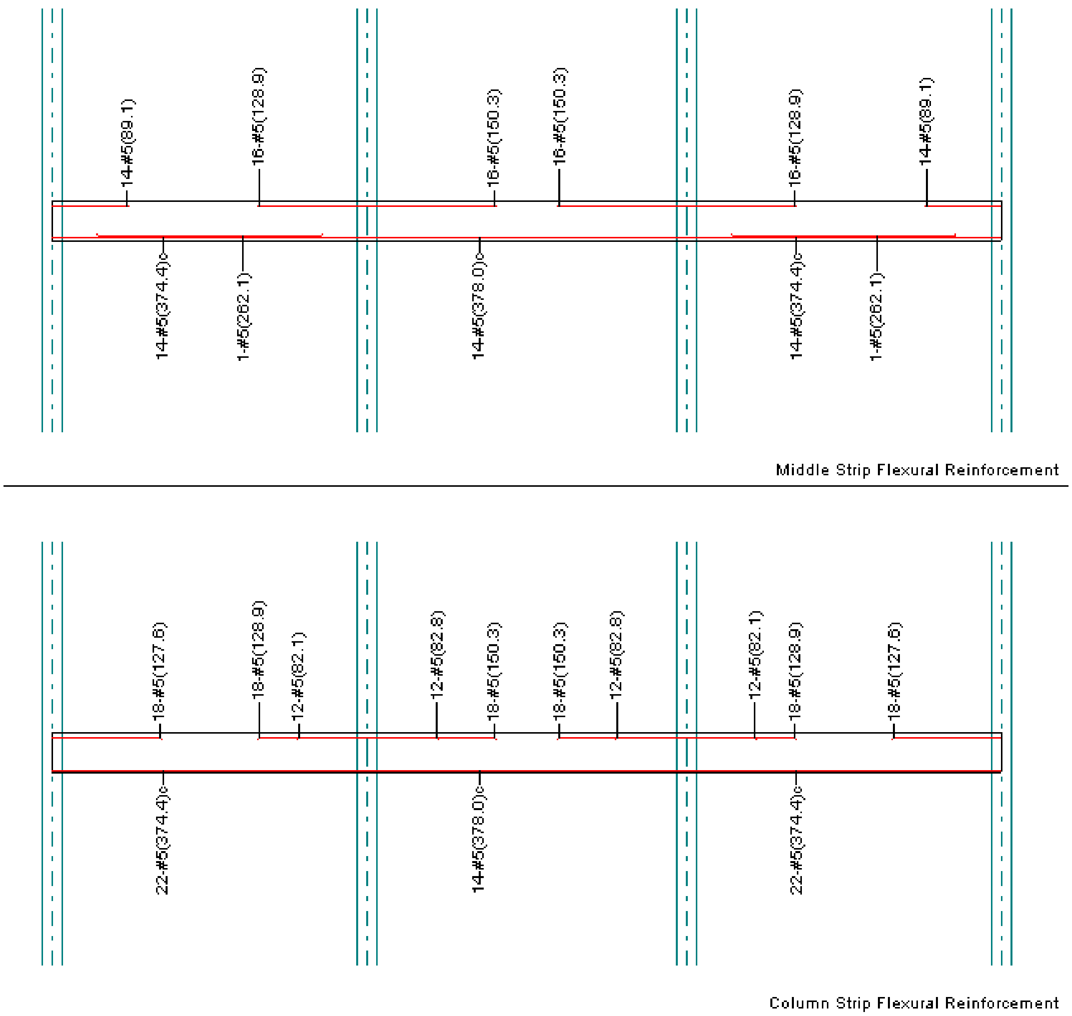


Figure 10

Two-Way Post-Tensioned Flat Plate

An evaluation has been preformed on a Two-Way Post-Tension Flat Plate Slab System to determine efficiency when compared to the typical Two-Way System (see Figure 11). Similar characteristics will be exhibited by the post-tensioned system in regard to fireproofing and vibration when compared to the two-way slab. However, due to strength added by post-tensioning, the slab thickness could be reduced from 12” to 8.5”. Constructability also increases due to the decreased amount of rebar needed for tension support.

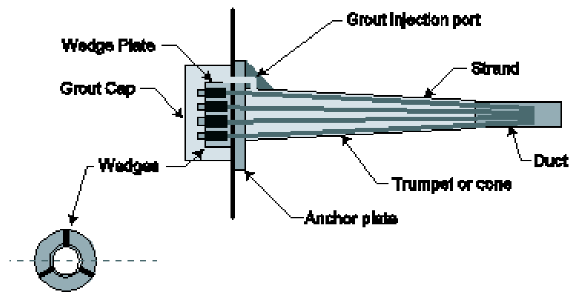


Figure 11

Similar to the standard two-way slab investigation, 5000psi normal weight concrete has been used in analysis of the post-tensioned system. A 3 by 3 bay section of the Cancer Hospital was selected for hand calculation (see Appendix D for Diagram, see Figure 12). Initial pre-stress compressive strength was assumed to be 3000psi with an estimated pre-stress loss of 15ksi. In order to simplify calculation, column stiffness was ignored. Design perimeters were found to fall under stress class “U” in accordance with ACI 318-08. Tensile and compressive loads were found at the time of a jacking and under service loads. A target load balance was established as 75% of the uniformly distributed dead load and checked in relation to stresses on interior and exterior spans. 30 tendons were determined to be required consisting of 7 strands ½” in diameter. Stresses at service and balanced loads were then used to determine rebar sizing and placement around positive and negative moment regions (see Table 1). All reinforcement consists of #5 , 60ksi bars placed in both directions. The rebar placed in the positive moment region is spaced at 6” on center. All calculations are provided in Appendix D.

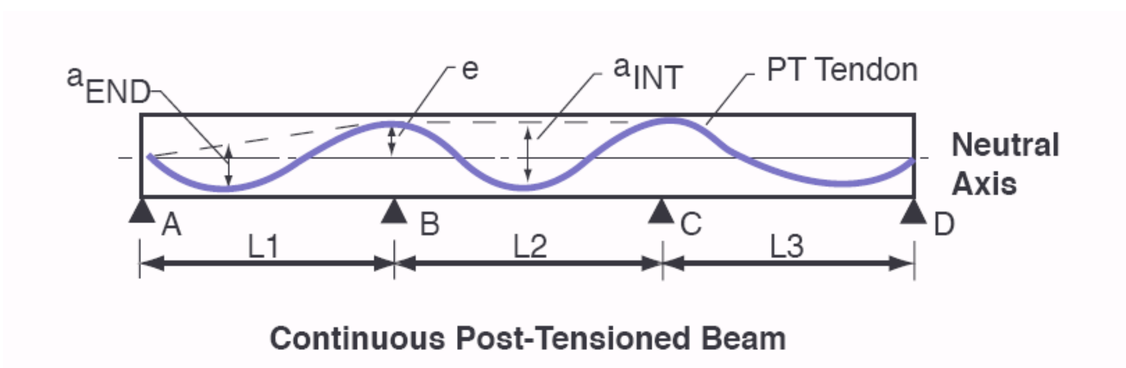


Figure 12

STRESS AFTER JACKING			
	Midspan Stresses		Support
	Interior Span	End Span	
M(dl)	94.3	302	378
M(ll)	46.8	150	187
M(bal)	79.6	254.8	318
S	4551	4551	4551
P/A	248	248	248
f(top)	-286.76	-372.46	-89.79
f(bot)	-209.24	-123.54	-406.21

STRESSES AT SERVICE LOAD			
	Midspan Stresses		Support
	Interior Span	End Span	
M(dl)	94.3	302	378
M(ll)	46.8	150	187
M(bal)	79.6	254.8	318
S	4551	4551	4551
P/A	248	248	248
f(top)	-410.16	-767.97	403.29
f(bot)	-85.84	271.97	-899.29

Table 1

Codes and Referenced Standards

Codes

ASCE-7-05 *Design Code for Minimum Design Loads*

LRFD Specifications for Structural Steel Design – Unified Version, 2005

ACI 318-08 *Building Code Requirements for Structural Concrete, 2008*

References

CRSI Handbook

United Steel Deck Manual

RS Means Assemblies Cost Data (2009)

RAM Structural Systems

PCA Slab

PCI Design Handbook/6th Edition

Cost

COMPOSITE METAL DECK				
Component	Unit Cost	Quantity	Cost	
Beams	47.8/ ft	94.5	$	4,517.10
Girders	95.87/ ft	31.5	$	3,019.91
Concrete	4.04/ sf	992.25	$	4,008.69
Shear Studs	0.42/ sf	992.25	$	416.75
Metal Deck	5.84/ sf	992.25	$	5,794.74
Member Fireproofing	7.3/ ft	94.5	$	689.85
	10.22/ ft	31.5 ft	$	321.93
Deck Fireproofing	1.95	992.25	$	1,934.89
Total			$	20,703.85

Table 2

NON-COMPOSITE METAL DECK				
Component	Unit Cost	Quantity	Cost	
Beams	71.62/ ft	94.5 ft	$	6,768.09
Girders	143.54/ ft	31.5 ft	$	4,521.51
Concrete	4.04/ sf	992.25 sf	$	4,008.69
Metal Deck	5.84/ sf	992.25 sf	$	5,794.74
Member Fireproofing	8.1/ ft	94.5 ft	$	522.45
	12.34/ ft	31.5 ft	$	388.71
Deck Fireproofing	1.95/ sf	992.25 sf	$	1,934.89
Total			$	23,939.08

Table 3

PRECAST HOLLOWCORE SLABS				
Component	Unit Cost	Quantity		Cost
Beams	71.62/ ft	94.5 ft	$	6,768.09
Girders	143.54/ ft	31.5 ft	$	4,521.51
Precast Slabs	4.04/ sf	992.25 sf	$	4,008.69
Member Fireproofing	8.1/ ft	94.5 ft	$	522.45
	10.22/ ft	31.5 ft	$	321.93
Total			$	16,142.67

Table 4

TWO-WAY FLAT SLAB W/ DROP PANELS				
Component	Unit Cost	Quantity		Cost
Concrete	21.05/ sf	992.25 sf	$	20,886.86
Total			$	20,886.86

Table 5

TWO-WAY POST-TENSIONED FLAT SLAB				
Component	Unit Cost	Quantity		Cost
Concrete	743.5/ cf	24.5 cf	$	18,215.75
Stressing Tendons	4.6/ lb	983.9 lb	$	4,525.94
Total			$	22,741.69

Table 6

Cost comparison was conducted using RS Means 2009 (see Table 2-6). The most economical system was found to be the Precast Hollowcore Slab. This is as expected due to the majority of its members having the ability to be prefabricated off site and shipped during construction. Both concrete systems were found to be close in cost however estimates for the post-tensioned slab were obtained using pricing for a typical flat plate slab. This may have led to a price increase due to the fact that the post tensioning element allows for a smaller than typical slab thickness. The two-way flat slab is within 1% of the cost of the composite deck, making concrete very feasible. The non-composite system has been found to sizably increase the cost of the existing design by 15.6%, making it the least economical system.

Weight

A slab weight comparison revealed large differences between concrete and steel systems (see Chart 1). The transition between metal deck and concrete has been to found to be at a cost of a 150% increase in weight. The 12" two-way flat slab was found to be the heaviest however normal weight concrete has been used in analysis. With the use of lightweight concrete, a more reasonable value could be found to decrease the load between 25 to 35psf. This holds true as well for the post-tension system. The low weight steel systems appear to be ideal in reducing seismic period, however due to the low seismic region in which the Cancer Hospital lies, additional weight may be beneficial to vibration control throughout the building. A more in depth study of the vibrational characteristics of the specific slabs has been found to be beneficial, and will be included in a future report.

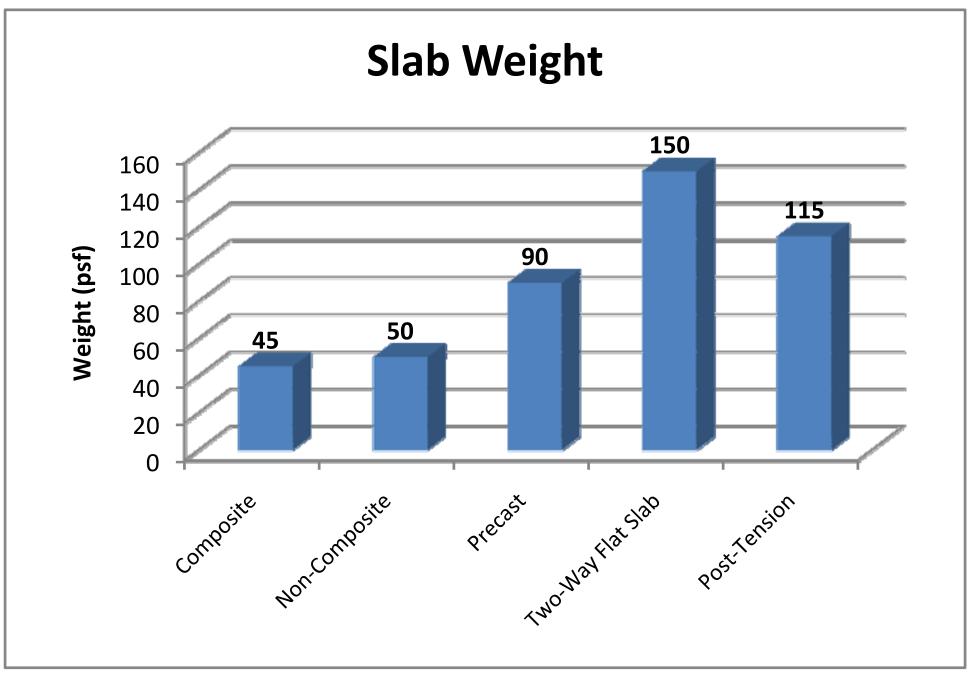


Chart 1

Deflection

Deflections have been determined and evaluated for each slab system (see Table 7). All deflections are based on the design guidelines of ASCE7-05. This limits an acceptable deflection from live load to L/360 and total load to L/240. All slab system designs have been held within these limits. Concrete slabs have been designed based on the deflection standards of ACI 318-08 Table 9.5c.

SLAB DEFLECTIONS					
	Composite	Non-Composite	Precast	Two-Way Flat Slab	Post-Tension
Live	0.549	0.786	0.952	0.064	N/A
Total	1.384	1.149	1.458	0.185	N/A

Table 7

A noticeable difference can be seen between the concrete and steel slab systems. The two-way slab design has not been controlled by deflection although its un-braced lengths are 3 times larger than that of the steel systems. This low amount of deflection may be useful in decreasing disturbances on imaging and operating room floors of the Cancer Hospital. For simplicity in calculation, deflection in the post-tension slab system has been assumed to be adequate based on data from the two-way flat slab with drop panel. Deflections in the precast planks were found to be minimal while the supporting steel member deflections controlled. These members have caused the largest amount of deflection out of all the slab systems evaluated. All calculations are provided in Appendix A-D.

Architectural Effect

All of the slab systems have been designed and evaluated to adhere to and preserve the *Universal Grid.* No column or beam relocations have been necessary in the typical bay analyzed, however non-typical bays such as those located on the ground floor may affect both the concrete slab systems as well as the precast slab system. Minor architectural changes will have to be made inside bays in regard to the addition of the 24” by 24” columns necessary for both two-way concrete slab systems. Floor systems utilizing steel as supporting members have roughly two times the floor depth of the concrete systems. The concrete slab systems, through use of flat plates, are able to remain unobstructed by beams in typical bays, adding additional verticality to each room. The concrete systems will also be able to more adequately provide room for MEP equipment without compromising strength (see Table 8).

Depth of Floor System				
Composite	Non-Composite	Precast	Two-Way Flat Slab	Post-Tension
30"	33"	34"	20"	8.5"

Table 8

Constructability

Although the steel systems may not be as architecturally friendly, they provide a large amount of ease in construction. The precast system requires the least amount of construction effort. Prefabricated slabs and an erected steel infrastructure will allow labor to be less and without specialization. The two-way slab system will require formwork and placement of rebar as opposed to the metal deck slab systems. The post-tension two-way slab system allows for less steel to be placed however specialization is required for tensioning.

Fireproofing

The concrete slab systems will have an adequate 2 hour rating without the addition of fireproofing. Both composite and non-composite metal decks will require 2 hour fire proofing on both the supporting steel members and the deck itself. The cost of spray fireproofing has been taken into account during cost analysis. Precast hollow-core planks will not require additional fireproofing however, the supporting steel members will still require it.

System Summary and Conclusions

SLAB COMPARISON SUMMARY					
	Composite	Non-Composite	Precast	Two-Way Flat Slab	Post-Tension
Cost	$20,703.85	$23,939.08	$16,142.67	$20,886.86	$22,741.69
Weight	45psf	50psf	90psf	150psf	115psf
Deflection	1.384"	1.149"	1.458"	.185"	N/A
Depth	30"	33"	34"	20"	8.5"
Frame Change	No	Small	No	Small	Small
Constructability	Better	Better	Best	Good	Good
Fireproofing	No	No	No	Yes	Yes
Feasible	Yes	No	Pending	Yes	Yes

System Summary

Composite Slab

The composite slab shows to be an economical and effective system. Its low weight and lack of required formwork while utilizing the compressive strength of concrete make it ideal for the Cancer Hospital. Due to its light weight, the composite slab deck is to be checked to meet the stringent vibrational criteria of the Cancer Hospital. Disadvantages include the need for spray-on fire proofing and a high floor depth due to obstructing bulky steel supporting members.

Non-Composite Slab

Exhibiting similar characteristics to the composite slab system, the non-composite is low weight and very constructible. However, due to even more sizeable obstructing members requiring higher floor depths as well as cost, this slab system seems inefficient when compared to the composite system. Little stiffness seems to be gained in supporting members therefore any increase in vibration resistance may likely be negligible.

Precast Hollow-Core Plank Slab

The precast hollow-ore system has shown to be the most economical and constructible of all the systems. A higher weight will presumably make this system more resistant to vibration upon investigation. Disadvantages include that of the metal deck in that this system requires large supporting members which will decrease ceiling height due increase in floor depth. The precast slab will cause the most architectural change in that it has the highest floor depth. Fireproofing will be required but only on supporting members. Use in the hospital is questionable and would depend on budget information.

Two-Way Flat Slab w/ Drop Panel

Providing a ceiling free of supporting beams and girders, the two-way flat slab system with drop panels is a cost efficient system which will add verticality to the typical bay in the Cancer Hospital. This additional space will aid in placing MEP equipment without compromising strength. The two-way system requires no fireproofing and has little deflection when loaded. This system will likely provide a large amount of vibration resistance however it has the highest mass weight. Formwork, shoring, and rebar placement will slow the constructability of this slab system although its common use and low cost make this a feasible option for the Cancer Hospital.

Two-Way Post-Tensioned Flat Slab

The post-tension two-way slab system will provide all the same advantages of the typical two-way concrete system however with a lower slab thickness allowing for less mass weight. Due to the reduced amount of rebar in this system constructability increases although, specialized construction is required for tensioning. A post-tension system would be beneficial to the Cancer Hospital by creating even larger vertical spaces without the need for such a heavy slab or drop panel.

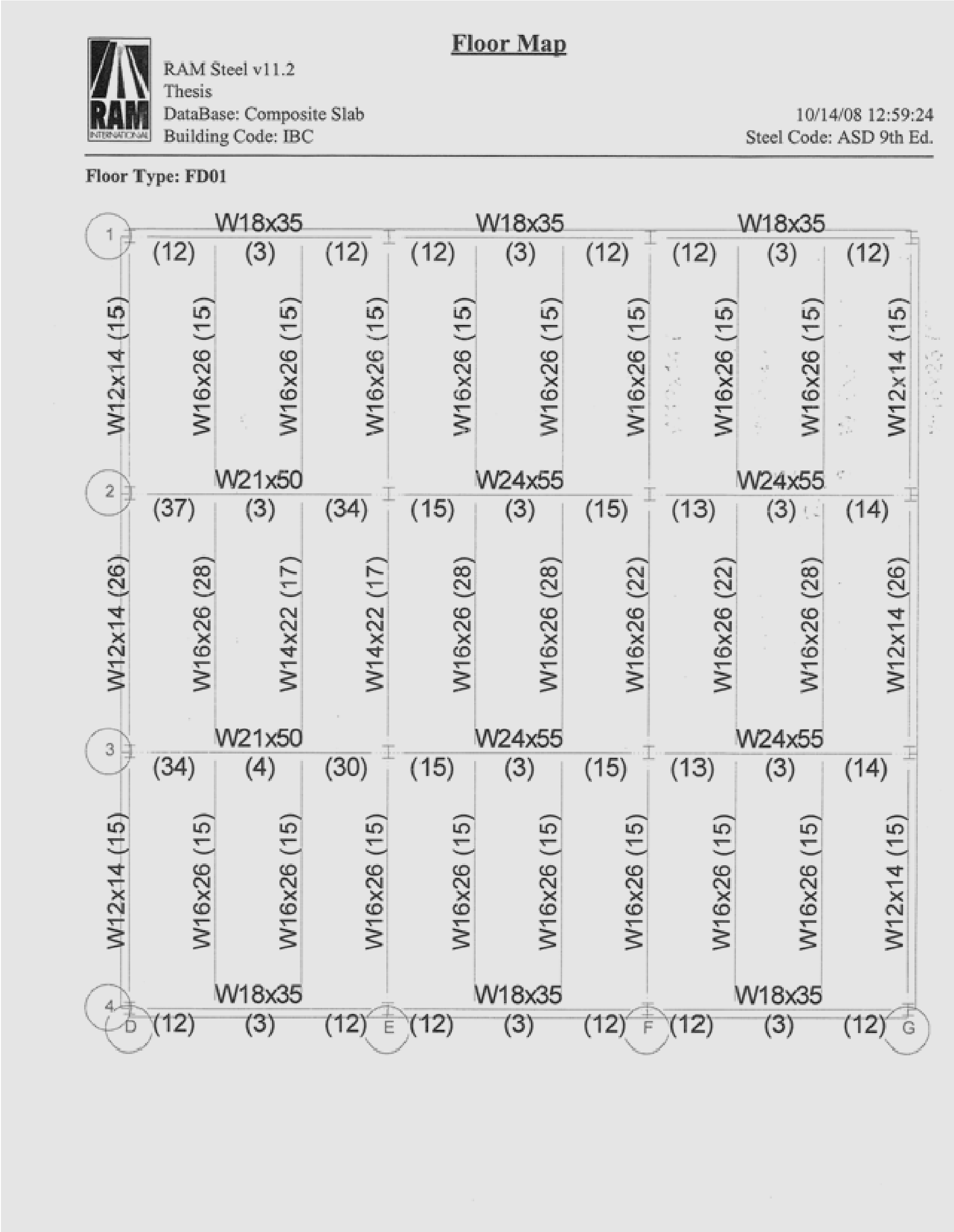
Conclusions

In conclusion, the existing composite metal deck was found to be an economical and effective slab system. Its low weight and use of the compressive strength of concrete make this system ideal. Disadvantages include a high floor depth, bulky steel supporting members, and a need for fireproofing. The non-composite slab system exhibits similar advantageous characteristics however, due to even more sizeable obstructing members requiring higher floor depths as well as cost, this slab system seems inefficient when compared to the composite system.

The precast hollow-core system has shown to be the most economical and constructible of all the systems. A higher weight will presumably make this system more resistant to vibration upon investigation. Downfalls include that of the metal deck systems although the precast slab will cause the most architectural change in that it has the highest floor depth. Providing a ceiling free of supporting beams and girders, the two-way flat slab system with drop panels is a cost efficient system which will add verticality to the typical bay in the Cancer Hospital. The two-way system requires no fireproofing and has little deflection when loaded although it weighs the most of all the systems. Formwork, shoring, and rebar placement will slow the constructability of this slab system however its common use and low cost make this a feasible option for the Cancer Hospital. The post-tension two-way slab system will provide all the same advantages of the typical two-way concrete system although with a lower slab thickness, allowing for less mass weight. A post-tension system would be beneficial to the Cancer Center by creating even larger vertical spaces without the need for such a heavy slab or drop panel.

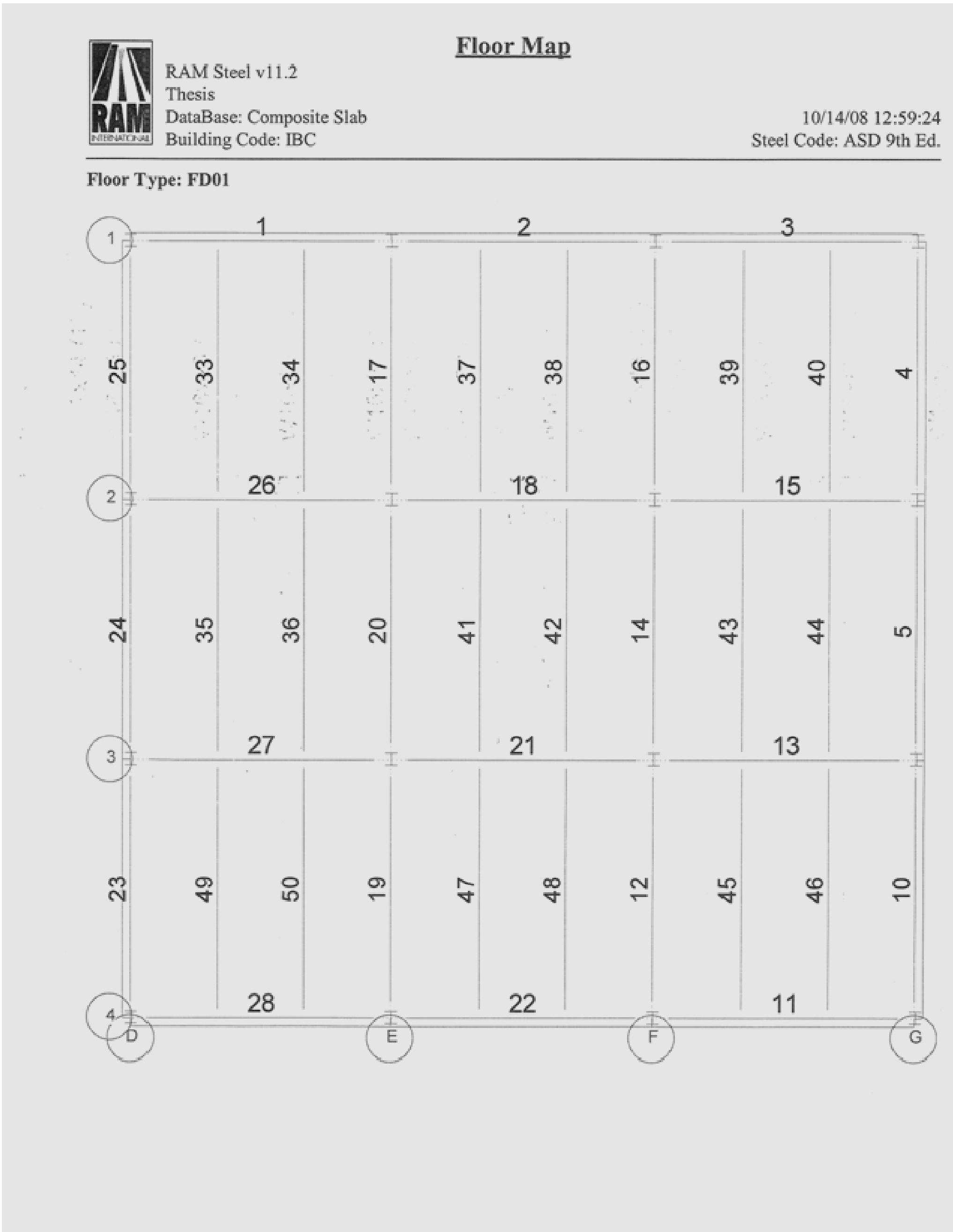
Appendix A

Composite



Appendix A

Composite



Appendix A

Composite

Beam Deflection Summary

RAM Steel v11.2
Thesis
DataBase: COMPOS~1 10/14/08 12:59:24
Building Code: IBC Steel Code: ASD 9th Ed.

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: FD01

Composite / Unshored

Bm #	Beam Size	Initial in	PostLive in	PostTotal in	NetTotal in	Camber in
23	W12X14	0.117	0.562	1.337	1.454	
28	W18X35	0.105	0.379	1.127	1.232	
24	W12X14	0.122	0.621	1.263	1.385	
27	W21X50	0.092	0.382	0.871	0.963	
25	W12X14	0.117	0.562	1.337	1.454	
26	W21X50	0.092	0.389	0.865	0.957	
1	W18X35	0.105	0.379	1.127	1.232	
49	W16X26	0.064	0.489	1.163	1.226	
35	W16X26	0.066	0.549	1.116	1.182	
33	W16X26	0.064	0.489	1.163	1.226	
50	W16X26	0.064	0.489	1.163	1.226	
36	W14X22	0.085	0.476	1.299	1.384	
34	W16X26	0.064	0.489	1.163	1.226	
19	W16X26	0.064	0.489	1.163	1.226	
22	W18X35	0.105	0.379	1.127	1.232	
20	W14X22	0.085	0.476	1.299	1.384	
21	W24X55	0.071	0.424	0.896	0.967	
17	W16X26	0.064	0.489	1.163	1.226	
18	W24X55	0.071	0.424	0.896	0.967	
2	W18X35	0.105	0.379	1.127	1.232	
47	W16X26	0.064	0.489	1.163	1.226	
41	W16X26	0.066	0.549	1.116	1.182	
37	W16X26	0.064	0.489	1.163	1.226	
48	W16X26	0.064	0.489	1.163	1.226	
42	W16X26	0.066	0.549	1.116	1.182	
38	W16X26	0.064	0.489	1.163	1.226	
12	W16X26	0.064	0.489	1.163	1.226	
11	W18X35	0.105	0.379	1.127	1.232	
14	W16X26	0.066	0.471	1.135	1.202	
13	W24X55	0.071	0.402	0.909	0.980	
16	W16X26	0.064	0.489	1.163	1.226	
15	W24X55	0.071	0.402	0.909	0.980	
3	W18X35	0.105	0.379	1.127	1.232	
45	W16X26	0.064	0.489	1.163	1.226	
43	W16X26	0.066	0.471	1.135	1.202	
39	W16X26	0.064	0.489	1.163	1.226	
46	W16X26	0.064	0.489	1.163	1.226	
44	W16X26	0.066	0.549	1.116	1.182	
40	W16X26	0.064	0.489	1.163	1.226	

Appendix A

Composite

Beam Deflection Summary

RAM Steel v11.2
Thesis
DataBase: COMPOS~1
Building Code: IBC

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10/14/08 12:59:24
Steel Code: ASD 9th Ed.

Bm #	Beam Size	Initial	PostLive	PostTotal	NetTotal	Camber
10	W12X14	0.117	0.562	1.337	1.454	
5	W12X14	0.122	0.621	1.263	1.385	
4	W12X14	0.117	0.562	1.337	1.454	

Appendix A

Composite

Beam Summary

RAM Steel v11.2
Thesis
DataBase: Composite Slab
Building Code: IBC

10/14/08 12:59:24
Steel Code: ASD 9th Ed.

STEEL BEAM DESIGN SUMMARY:

Floor Type: FD01

Bm #	Length ft	+M kip-ft	-M kip-ft	Seff in3	Fy ksi	Beam Size	Studs
23	31.17	69.6	0.0	25.9	50.0	W12X14	15
28	31.50	224.9	0.0	82.2	50.0	W18X35	12, 3, 12
24	31.50	81.2	0.0	29.9	50.0	W12X14	26
27	31.50	402.9	0.0	146.6	50.0	W21X50	34, 4, 30
25	31.17	69.6	0.0	25.9	50.0	W12X14	15
26	31.50	408.4	0.0	148.9	50.0	W21X50	37, 3, 34
1	31.50	224.9	0.0	82.2	50.0	W18X35	12, 3, 12
49	31.17	138.9	0.0	53.3	50.0	W16X26	15
35	31.50	162.1	0.0	59.5	50.0	W16X26	28
33	31.17	139.0	0.0	53.3	50.0	W16X26	15
50	31.17	138.9	0.0	53.3	50.0	W16X26	15
36	31.50	117.5	0.0	43.4	50.0	W14X22	17
34	31.17	139.0	0.0	53.3	50.0	W16X26	15
19	31.17	138.9	0.0	53.3	50.0	W16X26	15
22	31.50	224.9	0.0	82.2	50.0	W18X35	12, 3, 12
20	31.50	117.5	0.0	43.4	50.0	W14X22	17
21	31.50	421.6	0.0	155.0	50.0	W24X55	15, 3, 15
17	31.17	139.0	0.0	53.3	50.0	W16X26	15
18	31.50	421.7	0.0	155.0	50.0	W24X55	15, 3, 15
2	31.50	224.9	0.0	82.2	50.0	W18X35	12, 3, 12
47	31.17	138.9	0.0	53.3	50.0	W16X26	15
41	31.50	162.1	0.0	59.5	50.0	W16X26	28
37	31.17	139.0	0.0	53.3	50.0	W16X26	15
48	31.17	138.9	0.0	53.3	50.0	W16X26	15
42	31.50	162.1	0.0	59.5	50.0	W16X26	28
38	31.17	139.0	0.0	53.3	50.0	W16X26	15
12	31.17	138.9	0.0	53.3	50.0	W16X26	15
11	31.50	224.9	0.0	82.2	50.0	W18X35	12, 3, 12
14	31.50	154.6	0.0	57.1	50.0	W16X26	22
13	31.50	415.3	0.0	152.4	50.0	W24X55	13, 3, 14
16	31.17	139.0	0.0	53.3	50.0	W16X26	15
15	31.50	415.3	0.0	152.4	50.0	W24X55	13, 3, 14
3	31.50	224.9	0.0	82.2	50.0	W18X35	12, 3, 12
45	31.17	138.9	0.0	53.3	50.0	W16X26	15
43	31.50	154.6	0.0	57.1	50.0	W16X26	22
39	31.17	139.0	0.0	53.3	50.0	W16X26	15
46	31.17	138.9	0.0	53.3	50.0	W16X26	15
44	31.50	162.1	0.0	59.5	50.0	W16X26	28
40	31.17	139.0	0.0	53.3	50.0	W16X26	15
10	31.17	69.6	0.0	25.9	50.0	W12X14	15

Appendix A

Composite

Beam Summary

RAM Steel v11.2
Thesis
DataBase: Composite Slab
Building Code: IBC

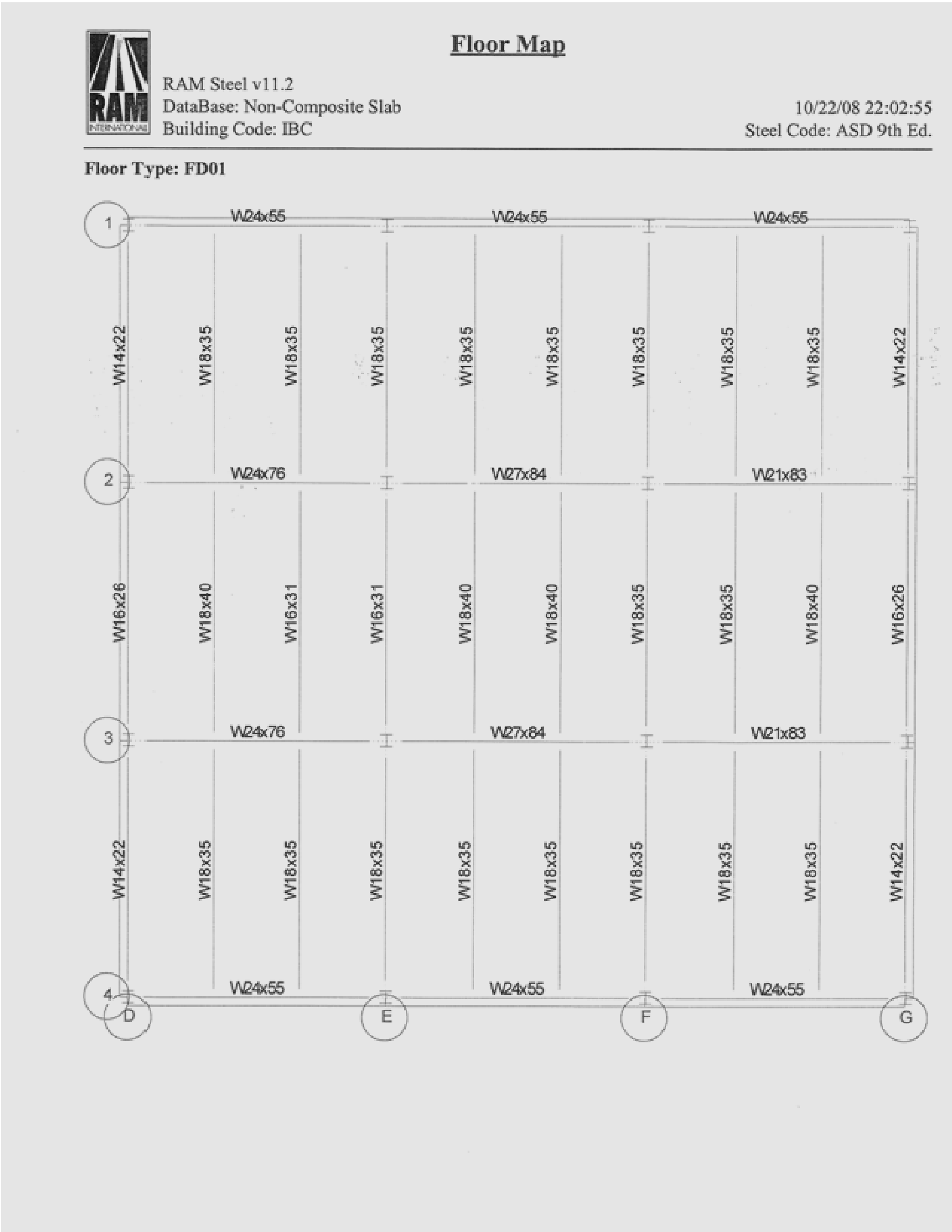
Page 2/2
10/14/08 12:59:24
Steel Code: ASD 9th Ed.

Bm #	Length	+M	-M	Seff	Fy	Beam Size	Studs
5	31.50	81.2	0.0	29.9	50.0	W12X14	26
4	31.17	69.6	0.0	25.9	50.0	W12X14	15

* after Size denotes beam failed stress/capacity criteria.
# after Size denotes beam failed deflection criteria.
u after Size denotes this size has been assigned by the User.

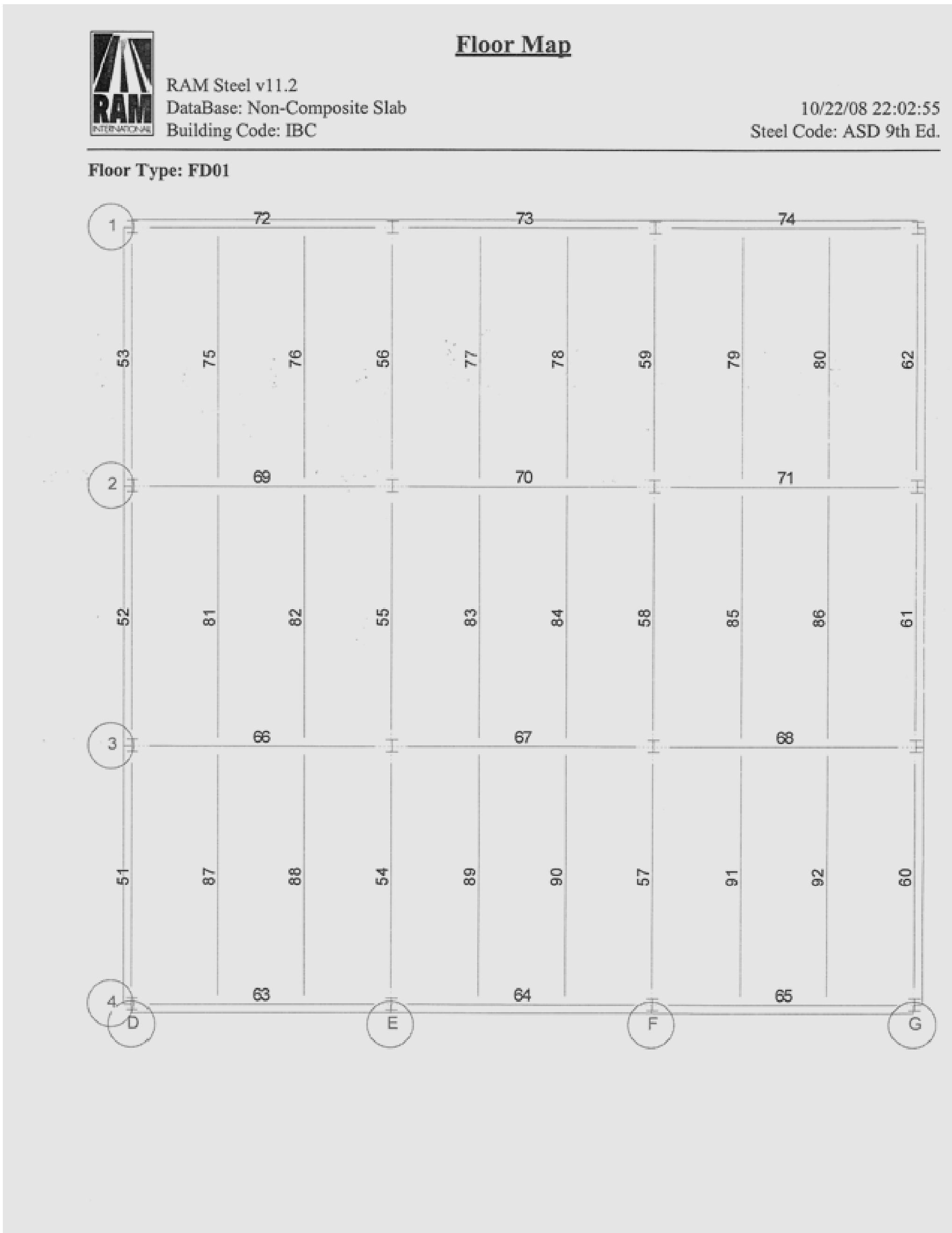
Appendix A

Non-Composite



Appendix A

Non-Composite



Appendix A

Non-Composite

Beam Deflection Summary

RAM Steel v11.2
DataBase: Non-Composite Slab
Building Code: IBC

10/14/08 13:04:16
Steel Code: ASD 9th Ed.

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: FD01

Noncomposite

Bm #	Beam Size	Dead in	Live in	NetTotal in	Camber in
51	W14X22	1.278	0.868	1.146	1
63	W24X55	0.719	0.336	0.555	1/2
52	W16X26	0.892	0.799	1.192	1/2
66	W24X76	0.704	0.500	0.705	1/2
53	W14X22	1.279	0.868	1.147	1
69	W24X76	0.703	0.524	0.728	1/2
72	W24X55	0.719	0.336	0.555	1/2
87	W18X35	0.985	0.677	0.912	3/4
81	W18X40	0.863	0.786	1.149	1/2
75	W18X35	0.985	0.678	0.912	3/4
88	W18X35	0.985	0.677	0.912	3/4
82	W16X31	1.269	0.696	0.965	1
76	W18X35	0.985	0.678	0.912	3/4
54	W18X35	0.985	0.677	0.912	3/4
64	W24X55	0.719	0.336	0.555	1/2
55	W16X31	1.269	0.696	0.965	1
67	W27X84	0.523	0.424	0.947	
56	W18X35	0.985	0.678	0.912	3/4
70	W27X84	0.523	0.424	0.947	
73	W24X55	0.719	0.336	0.555	1/2
89	W18X35	0.985	0.677	0.912	3/4
83	W18X40	0.863	0.786	1.149	1/2
77	W18X35	0.985	0.678	0.912	3/4
90	W18X35	0.985	0.677	0.912	3/4
84	W18X40	0.863	0.786	1.149	1/2
78	W18X35	0.985	0.678	0.912	3/4
57	W18X35	0.985	0.677	0.912	3/4
65	W24X55	0.719	0.336	0.555	1/2
58	W18X35	1.109	0.750	1.109	3/4
68	W21X83	0.840	0.606	0.946	1/2
59	W18X35	0.985	0.678	0.912	3/4
71	W21X83	0.840	0.607	0.946	1/2
74	W24X55	0.719	0.336	0.555	1/2
91	W18X35	0.985	0.677	0.912	3/4
85	W18X35	1.109	0.750	1.109	3/4
79	W18X35	0.985	0.678	0.912	3/4
92	W18X35	0.985	0.677	0.912	3/4
86	W18X40	0.863	0.786	1.149	1/2
80	W18X35	0.985	0.678	0.912	3/4

Appendix A

Non-Composite

Beam Deflection Summary

RAM Steel v11.2
DataBase: Non-Composite Slab
Building Code: IBC

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10/14/08 13:04:16
Steel Code: ASD 9th Ed.

Bm #	Beam Size	Dead	Live	NetTotal	Camber
60	W14X22	1.278	0.868	1.146	1
61	W16X26	0.892	0.799	1.192	1/2
62	W14X22	1.279	0.868	1.147	1

Appendix A

Non-Composite

Beam Summary

RAM Steel v11.2
DataBase: Non-Composite Slab
Building Code: IBC

10/22/08 22:02:55
Steel Code: ASD 9th Ed.

STEEL BEAM DESIGN SUMMARY:

Floor Type: FD01

Bm #	Length ft	+M kip-ft	-M kip-ft	Seff in3	Fy ksi	Beam Size	Studs
51	31.17	70.6	0.0	29.0	50.0	W14X22	
63	31.50	228.9	0.0	115.0	50.0	W24X55	
52	31.50	82.7	0.0	38.4	50.0	W16X26	
66	31.50	409.3	0.0	176.0	50.0	W24X76	
53	31.17	70.6	0.0	29.0	50.0	W14X22	
69	31.50	414.8	0.0	176.0	50.0	W24X76	
72	31.50	228.9	0.0	115.0	50.0	W24X55	
87	31.17	140.0	0.0	57.6	50.0	W18X35	
81	31.50	163.9	0.0	68.4	50.0	W18X40	
75	31.17	140.0	0.0	57.6	50.0	W18X35	
88	31.17	140.0	0.0	57.6	50.0	W18X35	
82	31.50	118.6	0.0	47.2	50.0	W16X31	
76	31.17	140.0	0.0	57.6	50.0	W18X35	
54	31.17	140.0	0.0	57.6	50.0	W18X35	
64	31.50	228.9	0.0	115.0	50.0	W24X55	
55	31.50	118.6	0.0	47.2	50.0	W16X31	
67	31.50	429.0	0.0	213.0	50.0	W27X84	
56	31.17	140.0	0.0	57.6	50.0	W18X35	
70	31.50	429.0	0.0	213.0	50.0	W27X84	
73	31.50	228.9	0.0	115.0	50.0	W24X55	
89	31.17	140.0	0.0	57.6	50.0	W18X35	
83	31.50	163.9	0.0	68.4	50.0	W18X40	
77	31.17	140.0	0.0	57.6	50.0	W18X35	
90	31.17	140.0	0.0	57.6	50.0	W18X35	
84	31.50	163.9	0.0	68.4	50.0	W18X40	
78	31.17	140.0	0.0	57.6	50.0	W18X35	
57	31.17	140.0	0.0	57.6	50.0	W18X35	
65	31.50	228.9	0.0	115.0	50.0	W24X55	
58	31.50	155.7	0.0	57.6	50.0	W18X35	
68	31.50	421.8	0.0	171.0	50.0	W21X83	
59	31.17	140.0	0.0	57.6	50.0	W18X35	
71	31.50	421.8	0.0	171.0	50.0	W21X83	
74	31.50	228.9	0.0	115.0	50.0	W24X55	
91	31.17	140.0	0.0	57.6	50.0	W18X35	
85	31.50	155.7	0.0	57.6	50.0	W18X35	
79	31.17	140.0	0.0	57.6	50.0	W18X35	
92	31.17	140.0	0.0	57.6	50.0	W18X35	
86	31.50	163.9	0.0	68.4	50.0	W18X40	
80	31.17	140.0	0.0	57.6	50.0	W18X35	
60	31.17	70.6	0.0	29.0	50.0	W14X22	

Appendix A

Non-Composite

Beam Summary

RAM Steel v11.2
DataBase: Non-Composite Slab
Building Code: IBC

Page 2/2
10/22/08 22:02:55
Steel Code: ASD 9th Ed.

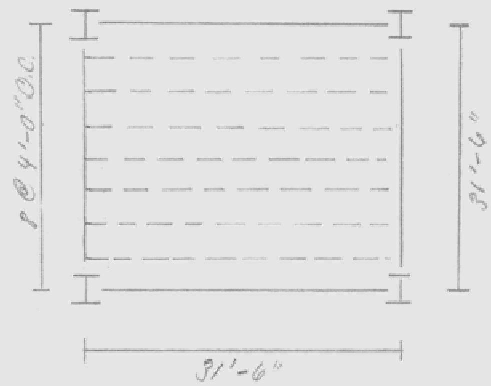
Bm #	Length	+M	-M	Seff	Fy	Beam Size	Studs
61	31.50	82.7	0.0	38.4	50.0	W16X26	
62	31.17	70.6	0.0	29.0	50.0	W14X22	

* after Size denotes beam failed stress/capacity criteria.
\# after Size denotes beam failed deflection criteria.
u after Size denotes this size has been assigned by the User.

Appendix B

Precast Hollow-Core Plank

PRECAST HOLLOW-CORE SLAB



• Loads:

Dead
Floor Load — Included
Ceiling Partition — 5psf
Suspended Mech. Equipment — 10psf
— 15psf

Live
Floor Load — 60 (reducible)

Live Load Reduction:

$L = L_o\left(.25 + \frac{15}{\sqrt{1.0(331)}}\right)$

$= 64.4\,psf$

$\neq 60\,psf$ (no reduction)

Appendix B

Precast Hollow-Core Plank

- From PCI Handbook section 2

Select 68-S 4'-0" x 8" Hollow-Core Plank

Available Service Load = 103.5 psf
97 psf > 75 psf ✓okay

Span = 32' > 31.5' ✓okay

- Deflection Check

$w_{s.w} = 224\ plf$
$w_{D+L} = (75\ psf)(4) = 300\ plf$
$624\ plf$

$$\Delta_{TOT} = \frac{5\left(\frac{624}{1000}\right)(31.5)^4(1728)}{384(763)(57000\sqrt{5})} = .142 < \frac{(31.5)(12)}{240} = 1.6''$$

✓okay

$$\Delta_{LL} = \frac{5\left(\frac{420}{1000}\right)(31.5)^4(1728)}{384(763)(57000\sqrt{5})} = .11 < \frac{(31.5)(12)}{360} = 1.1''$$

✓okay

Appendix B

Precast Hollow-Core Plank

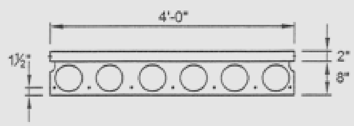
Strand Pattern Designation
76-S
S = straight
Diameter of strand in 16ths
No. of Strand (7)

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

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



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

Section Properties

	Untopped	Topped
A =	215 in.^{2}	311 in.^{2}
I =	1,666 in.^{4}	3,071 in.^{4}
y_b =	4.00 in.	5.29 in.
y_t =	4.00 in.	4.71 in.
S_b =	417 in.^{3}	581 in.^{3}
S_t =	417 in.^{3}	652 in.^{3}
wt =	224 plf	324 plf
DL =	56 psf	81 psf
V/S =	1.92 in.	

4HC8

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

Strand Designation Code	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	458	415	378	346	311	269	234	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	–0.1	–0.2	–0.3	–0.5	–0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.4						
76-S	470	424	387	355	326	303	276	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.6	–0.8	–1.1	–1.4	–1.7	–2.0			
58-S	464	421	384	352	323	300	280	260	244	229	211	194	177	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.4	–0.3	–0.5	–0.7	–0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	–0.2	–0.5	–0.8	–1.1	–1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	–0.3	–0.7

4HC8 + 2

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

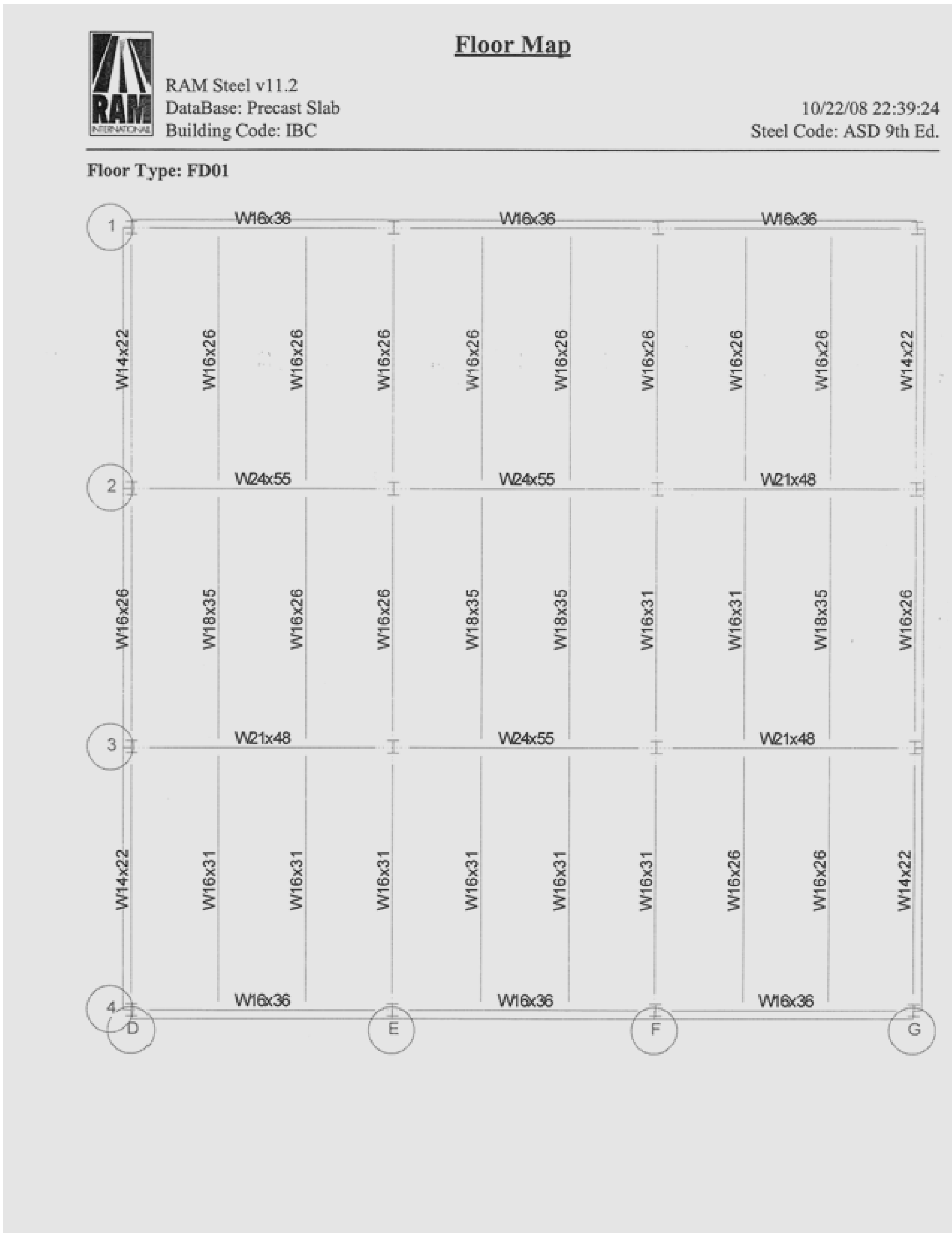
Strand Designation Code	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	489	445	394	340	294	256	224	197	173	153	135	119	105	93	82	68	56	45	36	26								
	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	–0.0	–0.1	–0.2	–0.3								
	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	–0.1	–0.2	–0.3	–0.4	–0.6	–0.7	–0.9	–1.2	–1.4								
76-S	498	457	420	387	347	304	267	235	208	184	164	146	130	116	103	88	74	62	51	41	31							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	–0.0	–0.1	–0.2							
	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9	–1.2	–1.4							
58-S	492	451	414	384	357	333	310	293	274	245	219	196	177	159	143	126	110	95	82	70	59	49	40	32				
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.1	0.3	0.2	0.1	0.0	–0.1				
	0.3	0.3	0.4	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.4	0.3	0.3	0.3	0.2	0.1	–0.1	–0.2	–0.4	–0.6	–0.9	–1.2	–1.5	–1.8				
68-S		463	426	393	366	342	319	299	282	267	251	239	216	195	177	158	140	124	110	97	84	73	62	53	44	36	28	
		0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	
		0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.6	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4	
78-S		472	435	402	375	348	325	305	288	273	257	245	232	220	207	186	167	149	133	119	106	94	83	73	64	55	46	38
		0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.7	0.6	0.5	0.3
		0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9	0.9	0.9	0.8	0.8	0.7	0.7	0.6	0.4	0.3	0.1	–0.1	–0.3	–0.6	–0.9	–1.3	–1.7	–2.2

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 2–7 through 2–10 for explanation.

2–32 PCI Design Handbook/Sixth Edition

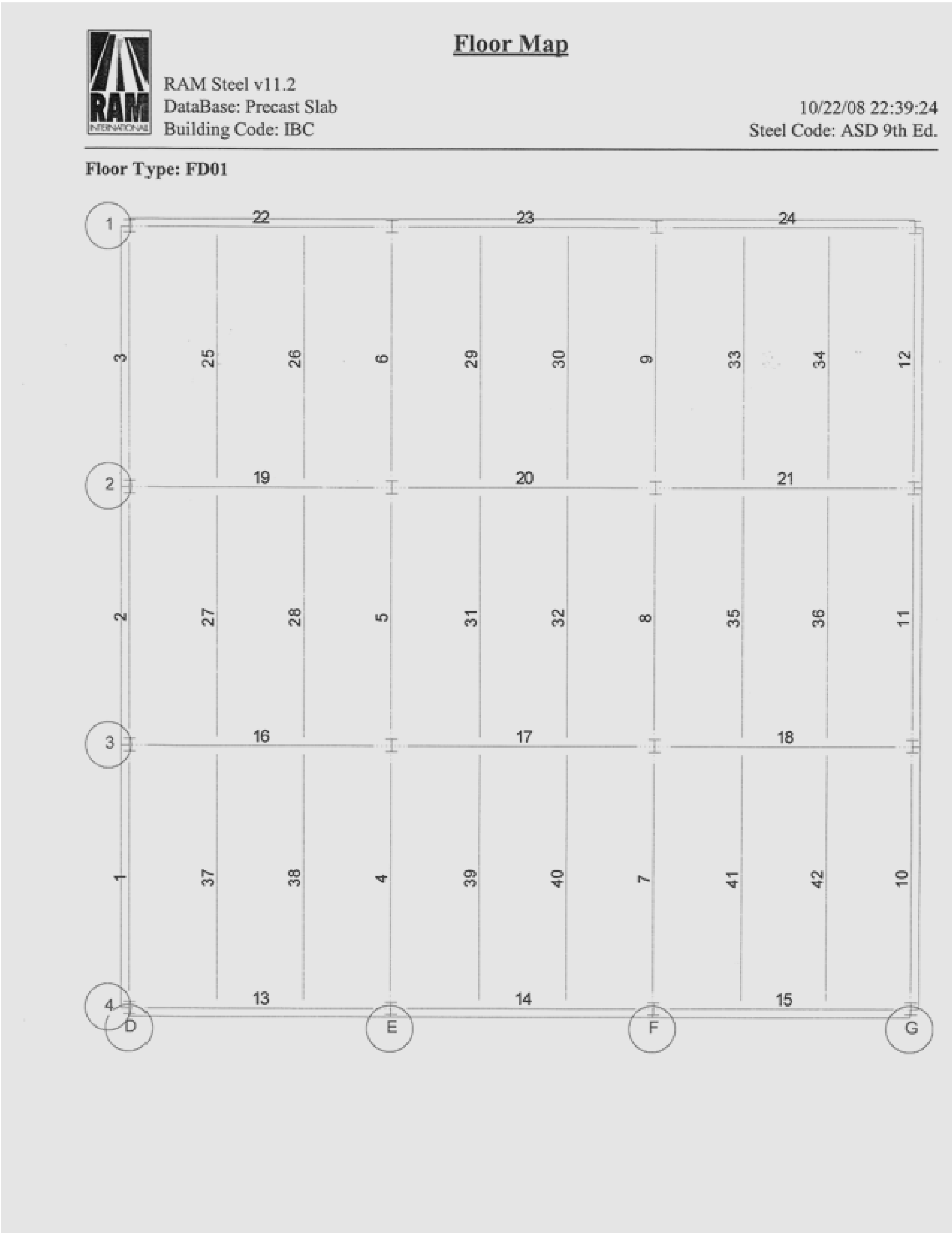
Appendix B

Precast Hollow-Core Plank



Appendix B

Precast Hollow-Core Plank



Appendix B

Precast Hollow-Core Plank

Beam Deflection Summary

RAM Steel v11.2
DataBase: Precast Slab 2 — 10/22/08 22:39:24
Building Code: IBC — Steel Code: ASD 9th Ed.

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: FD01

Noncomposite

Bm #	Beam Size	Dead in	Live in	NetTotal in	Camber in
1	W14X22	0.142	0.806	0.948	
13	W16X36	0.671	0.976	1.147	1/2
2	W16X26	0.099	0.760	0.859	
16	W21X48	0.308	0.982	1.289	
3	W14X22	0.138	0.779	0.916	
19	W24X55	0.144	0.743	0.886	
22	W16X36	0.659	0.968	1.126	1/2
37	W16X31	0.063	0.856	0.919	
27	W18X35	0.050	0.897	0.946	
25	W16X26	0.064	1.030	1.094	
38	W16X31	0.063	0.856	0.919	
28	W16X26	0.736	0.952	1.188	1/2
26	W16X26	0.064	1.030	1.094	
4	W16X31	0.063	0.856	0.919	
14	W16X36	0.671	0.976	1.147	1/2
5	W16X26	0.736	0.952	1.188	1/2
17	W24X55	0.081	0.798	0.879	
6	W16X26	0.064	1.030	1.094	
20	W24X55	0.077	0.795	0.872	
23	W16X36	0.659	0.968	1.126	1/2
39	W16X31	0.063	0.856	0.919	
31	W18X35	0.050	0.897	0.946	
29	W16X26	0.064	1.030	1.094	
40	W16X31	0.063	0.856	0.919	
32	W18X35	0.050	0.897	0.946	
30	W16X26	0.064	1.030	1.094	
7	W16X31	0.063	0.846	0.910	
15	W16X36	0.606	0.896	1.502	
8	W16X31	0.553	0.905	1.458	
18	W21X48	0.280	0.928	1.207	
9	W16X26	0.064	1.019	1.083	
21	W21X48	0.280	0.925	1.204	
24	W16X36	0.606	0.888	1.494	
41	W16X26	0.066	1.043	1.110	
35	W16X31	0.557	0.879	1.436	
33	W16X26	0.064	1.008	1.072	
42	W16X26	0.066	1.043	1.110	
36	W18X35	0.053	0.877	0.931	
34	W16X26	0.064	1.008	1.072	

Appendix B

Precast Hollow-Core Plank

Beam Deflection Summary

RAM Steel v11.2
DataBase: Precast Slab 2
Building Code: IBC

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10/22/08 22:39:24
Steel Code: ASD 9th Ed.

Bm #	Beam Size	Dead	Live	NetTotal	Camber
10	W14X22	0.142	0.789	0.931	
11	W16X26	0.099	0.743	0.843	
12	W14X22	0.138	0.762	0.900	

Appendix B

Precast Hollow-Core Plank

Beam Summary

RAM Steel v11.2
DataBase: Precast Slab
Building Code: IBC
10/22/08 22:39:24
Steel Code: ASD 9th Ed.

STEEL BEAM DESIGN SUMMARY:

Floor Type: FD01

Bm #	Length ft	+M kip-ft	-M kip-ft	Seff in3	Fy ksi	Beam Size	Studs
1	31.50	30.6	0.0	29.0	50.0	W14X22	
13	31.50	118.1	0.0	56.5	50.0	W16X36	
2	31.10	43.1	0.0	38.4	50.0	W16X26	
16	31.50	197.5	0.0	93.0	50.0	W21X48	
3	31.23	30.1	0.0	29.0	50.0	W14X22	
19	31.50	191.9	0.0	115.0	50.0	W24X55	
22	31.50	116.7	0.0	56.5	50.0	W16X36	
37	31.50	55.9	0.0	47.2	50.0	W16X31	
27	31.10	80.4	0.0	57.6	50.0	W18X35	
25	31.23	54.4	0.0	38.4	50.0	W16X26	
38	31.50	55.9	0.0	47.2	50.0	W16X31	
28	31.10	84.6	0.0	38.4	50.0	W16X26	
26	31.23	54.4	0.0	38.4	50.0	W16X26	
4	31.50	55.9	0.0	47.2	50.0	W16X31	
14	31.50	118.1	0.0	56.5	50.0	W16X36	
5	31.10	84.6	0.0	38.4	50.0	W16X26	
17	31.50	190.1	0.0	115.0	50.0	W24X55	
6	31.23	54.4	0.0	38.4	50.0	W16X26	
20	31.50	188.6	0.0	115.0	50.0	W24X55	
23	31.50	116.7	0.0	56.5	50.0	W16X36	
39	31.50	55.9	0.0	47.2	50.0	W16X31	
31	31.10	80.4	0.0	57.6	50.0	W18X35	
29	31.23	54.4	0.0	38.4	50.0	W16X26	
40	31.50	55.9	0.0	47.2	50.0	W16X31	
32	31.10	80.4	0.0	57.6	50.0	W18X35	
30	31.23	54.4	0.0	38.4	50.0	W16X26	
7	31.50	55.4	0.0	47.2	50.0	W16X31	
15	30.83	112.5	0.0	56.5	50.0	W16X36	
8	31.10	90.9	0.0	47.2	50.0	W16X31	
18	30.83	194.5	0.0	93.0	50.0	W21X48	
9	31.23	53.8	0.0	38.4	50.0	W16X26	
21	30.83	193.9	0.0	93.0	50.0	W21X48	
24	30.83	111.9	0.0	56.5	50.0	W16X36	
41	31.50	54.2	0.0	38.4	50.0	W16X26	
35	31.10	89.6	0.0	47.2	50.0	W16X31	
33	31.23	53.3	0.0	38.4	50.0	W16X26	
42	31.50	54.2	0.0	38.4	50.0	W16X26	
36	31.10	79.0	0.0	57.6	50.0	W18X35	
34	31.23	53.3	0.0	38.4	50.0	W16X26	
10	31.50	30.1	0.0	29.0	50.0	W14X22	

Appendix B

Precast Hollow-Core Plank

Beam Summary

RAM Steel v11.2
DataBase: Precast Slab
Building Code: IBC

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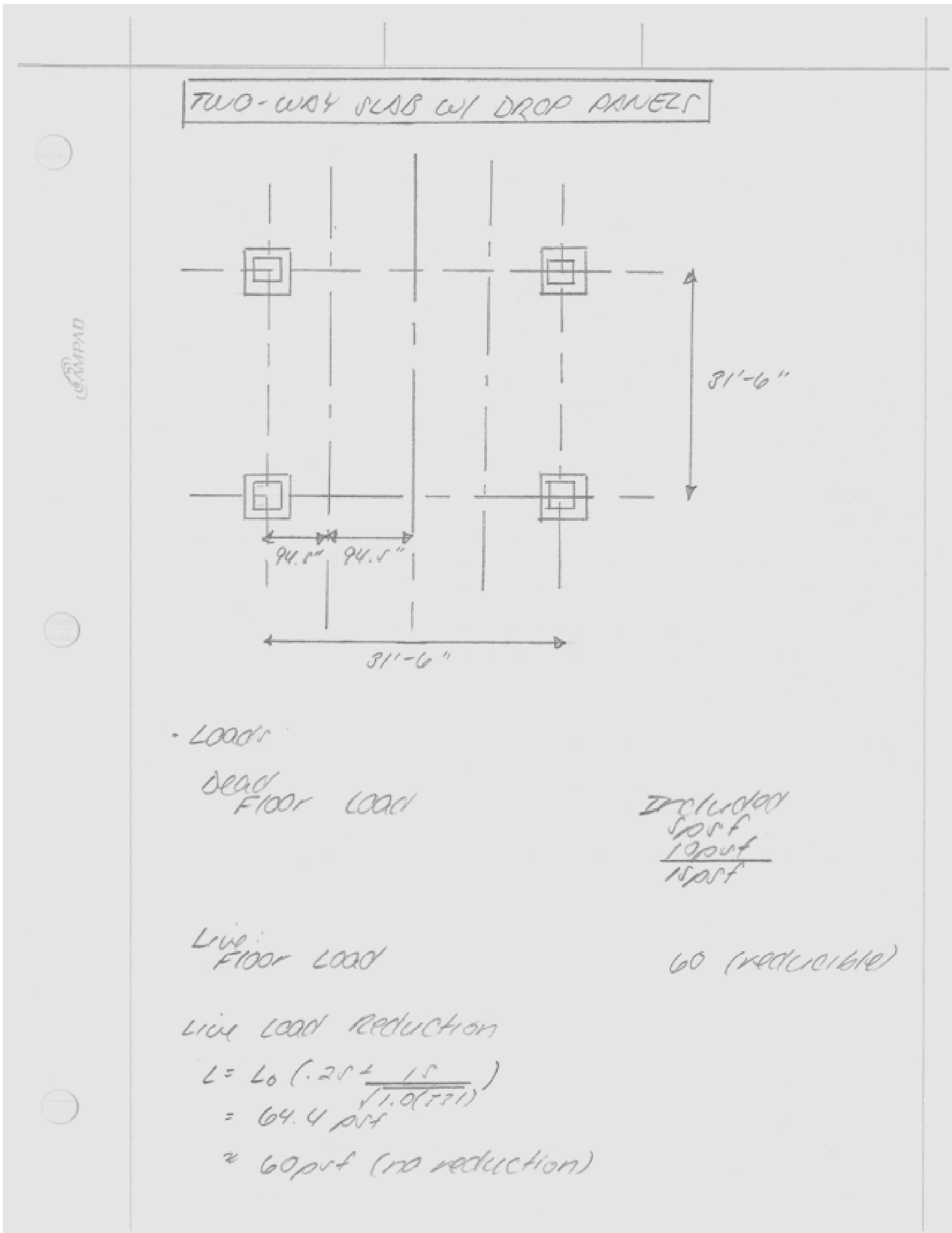
10/22/08 22:39:24
Steel Code: ASD 9th Ed.

Bm #	Length	+M	-M	Seff	Fy	Beam Size	Studs
11	31.10	42.2	0.0	38.4	50.0	W16X26	
12	31.23	29.6	0.0	29.0	50.0	W14X22	

* after Size denotes beam failed stress/capacity criteria.
# after Size denotes beam failed deflection criteria.
u after Size denotes this size has been assigned by the User.

Appendix C

Two-Way Flat Slab w/ Drop Panels



Appendix C

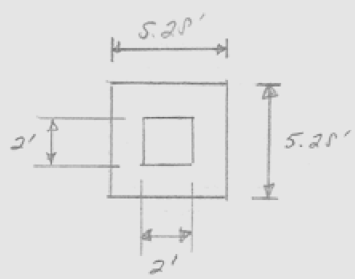
Two-Way Flat Slab w/ Drop Panels

- Dimensions

  CS width $= \frac{31.5(12)}{4} = 94.5''$

  MS width $= 94.5''$

  Column + drop panel



  drop panel thickness = 6"

- min slab thickness

  $t_{min} = \frac{l_n}{36} = \frac{(31.5)(12)}{36} = 10.5''$ (ACI 318-08 Table 9-5.c)

  $t_{ACT} = 12'' > 10.5''$ ✓ okay

Appendix C

Two-Way Flat Slab w/ Drop Panels

```
pcaSlab v1.51 © Portland Cement Association                                    10-23-2008, 10:15:06 PM
Licensed to: Penn State University, License ID: 52416-1010277-4-22545-28F4D
P:\Thesis\pcaSlab1.slb                                                                           Page 1

      ==============================================================================
                                 pcaSlab v1.51 (TM)
                 A Computer Program Analysis, Design, and Investigation of
                   Reinforced Concrete Slab and Continuous Beam Systems
      ==============================================================================
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        the pcaSlab program.

==============================================================================================
[1] INPUT ECHO
==============================================================================================

General Information:
====================
   File name: P:\Thesis\pcaSlab1.slb
   Project: UHCMC
   Frame: E-F                                         Engineer: DCM
   Code: ACI 318-02       Mode: Design                Reinforcement Database: ASTM A615
   Number of supports = 4
   Floor System: Two-Way

   Live load pattern ratio = 75%
   Minimum free edge for punching shear = 10 times slab thickness
   Deflections are based on cracked section properties.
   In negative moment regions, Ig and Mcr DO NOT include flange/slab contribution (if available)
   Compression reinforcement calculations NOT selected.

Material Properties:
====================
              Slabs|Beams       Columns
             ------------   ------------
   wc    =           150            150 lb/ft3
   f'c   =             5              5 ksi
   Ec    =        4286.8         4286.8 ksi
   fr    =       0.53033        0.53033 ksi

   fy    =            60 ksi, Bars are not epoxy-coated
   fyv   =            60 ksi
   Es    =         29000 ksi

Reinforcement Database:
===============
   Units: Db (in), Ab (in^2), Wb (lb/ft)
   Size       Db       Ab       Wb    Size       Db       Ab       Wb
   ---- -------- -------- --------    ---- -------- -------- --------
     #3     0.38     0.11     0.38      #4     0.50     0.20     0.67
     #5     0.63     0.31     1.04      #6     0.75     0.44     1.50
     #7     0.88     0.60     2.04      #8     1.00     0.79     2.67
```

Appendix C

Two-Way Flat Slab w/ Drop Panels

```
pcaSlab v1.51 © Portland Cement Association                                        10-23-2008, 10:15:06 PM
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P:\Thesis\pcaSlab1.slb                                                                              Page 2

     #9     1.13     1.00     3.40     #10     1.27     1.27     4.30
    #11     1.41     1.56     5.31     #14     1.69     2.25     7.65
    #18     2.26     4.00    13.60

Span Data:
==========
   Slabs: L1, wL, wR (ft); t, Hmin (in)
   Span Loc        L1        t       wL       wR     Hmin
   ---- ---- -------- -------- -------- -------- --------
      1 ExtL   31.200    12.00   15.750   15.750    10.62
      2 Int    31.500    12.00   15.750   15.750     9.83
      3 ExtR   31.200    12.00   15.750   15.750    10.62

Support Data:
=============
   Columns: c1a, c2a, c1b, c2b (in); Ha, Hb (ft)
   Supp      c1a      c2a       Ha        c1b      c2b       Hb  Red%
   ---- -------- -------- --------   -------- -------- --------  ----
      1    24.00    24.00   15.000      24.00    24.00   15.000    50 *
      2    24.00    24.00   15.000      24.00    24.00   15.000    50
      3    24.00    24.00   15.000      24.00    24.00   15.000    50
      4    24.00    24.00   15.000      24.00    24.00   15.000    50 *
   * Do not check punching shear around this column.

   Drop Panels: h (in); L1, L2, W1, W2 (ft)
   Supp        h       L1       L2       W1       W2
   ---- -------- -------- -------- -------- --------
      1     6.00    1.000    5.200    5.250    5.250 *a b d
      2     6.00    5.200    5.250    5.250    5.250 *b
      3     6.00    5.250    5.200    5.250    5.250 *b
      4     6.00    5.200    1.000    5.250    5.250 *a b d
   *a- Do not check punching shear around this drop panel.
   *b- Standard drop.
   *d- Excessive drop thickness will not be used for flexural design.

   Boundary Conditions: Kz (kip/in); Kry (kip-in/rad)
   Supp    Spring Kz   Spring Kry Far End A Far End B
   ---- ------------ ------------ --------- ---------
      1            0            0     Fixed     Fixed
      2            0            0     Fixed     Fixed
      3            0            0     Fixed     Fixed
      4            0            0     Fixed     Fixed

Load Data:
==========
   Load Cases and Combinations:
   Case      SELF     Dead     Live     Wind       EQ
   Type      DEAD     DEAD     LIVE  LATERAL  LATERAL
   U1       1.400    1.400    0.000    0.000    0.000
   U2       1.200    1.200    1.600    0.000    0.000
   U3       1.200    1.200    1.600    0.800    0.000
   U4       1.200    1.200    1.600   -0.800    0.000
   U5       1.200    1.200    1.000    1.600    0.000
   U6       1.200    1.200    1.000   -1.600    0.000
   U7       0.900    0.900    0.000    1.600    0.000
   U8       0.900    0.900    0.000   -1.600    0.000
   U9       1.200    1.200    1.000    0.000    1.000
   U10      1.200    1.200    1.000    0.000   -1.000
   U11      0.900    0.900    0.000    0.000    1.000
   U12      0.900    0.900    0.000    0.000   -1.000

   Span Loads:
   Span Case               Wa
   ---- -------- ------------
   Area Loads - Wa (lb/ft2):
      1 Dead               15
      2 Dead               15
      3 Dead               15
      1 Live               40
      3 Live               40
      2 Live               60

   Support Loads:   --- NONE ---

   Support Displacements:   --- NONE ---

   Lateral Load Effects - M (k-ft):
   Span Case              Mleft       Mright
   ---- -------- ------------ ------------
      1 EQ                  0            0
      2 EQ                  0            0
      3 EQ                  0            0
      4 EQ                  0            0
      1 Wind                0            0
      2 Wind                0            0
```

Appendix C

Two-Way Flat Slab w/ Drop Panels

```
pcaSlab v1.51 © Portland Cement Association                                 10-23-2008, 10:15:06 PM
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      3 Wind                0             0

Reinforcement Criteria:
=======================
                  _____Top bars___  ___Bottom bars__  ____Stirrups____
                    Min      Max      Min      Max      Min      Max
                  -------  -------  -------  -------  -------  -------
   Slabs and Ribs:
     Bar Size        #5       #8       #5       #8
     Bar spacing   1.00    18.00     1.00    18.00 in
     Reinf ratio   0.14     5.00     0.14     5.00 %
     Cover         1.50              1.50          in

   Beams:
     Bar Size        #5       #8       #5       #8       #3       #5
     Bar spacing   1.00    18.00     1.00    18.00     6.00    18.00 in
     Reinf ratio   0.14     5.00     0.14     5.00 %
     Cover         1.50              1.50          in
```
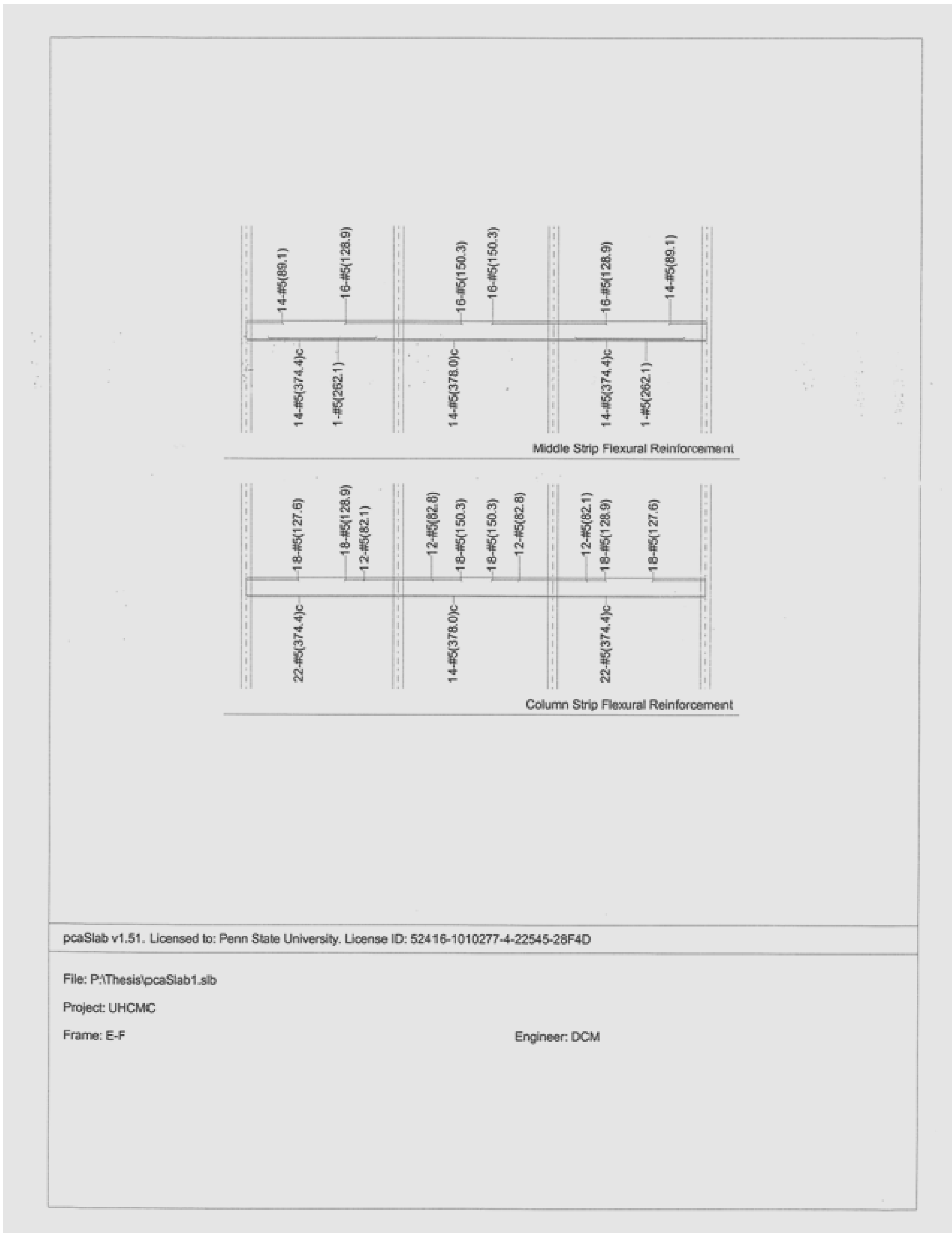
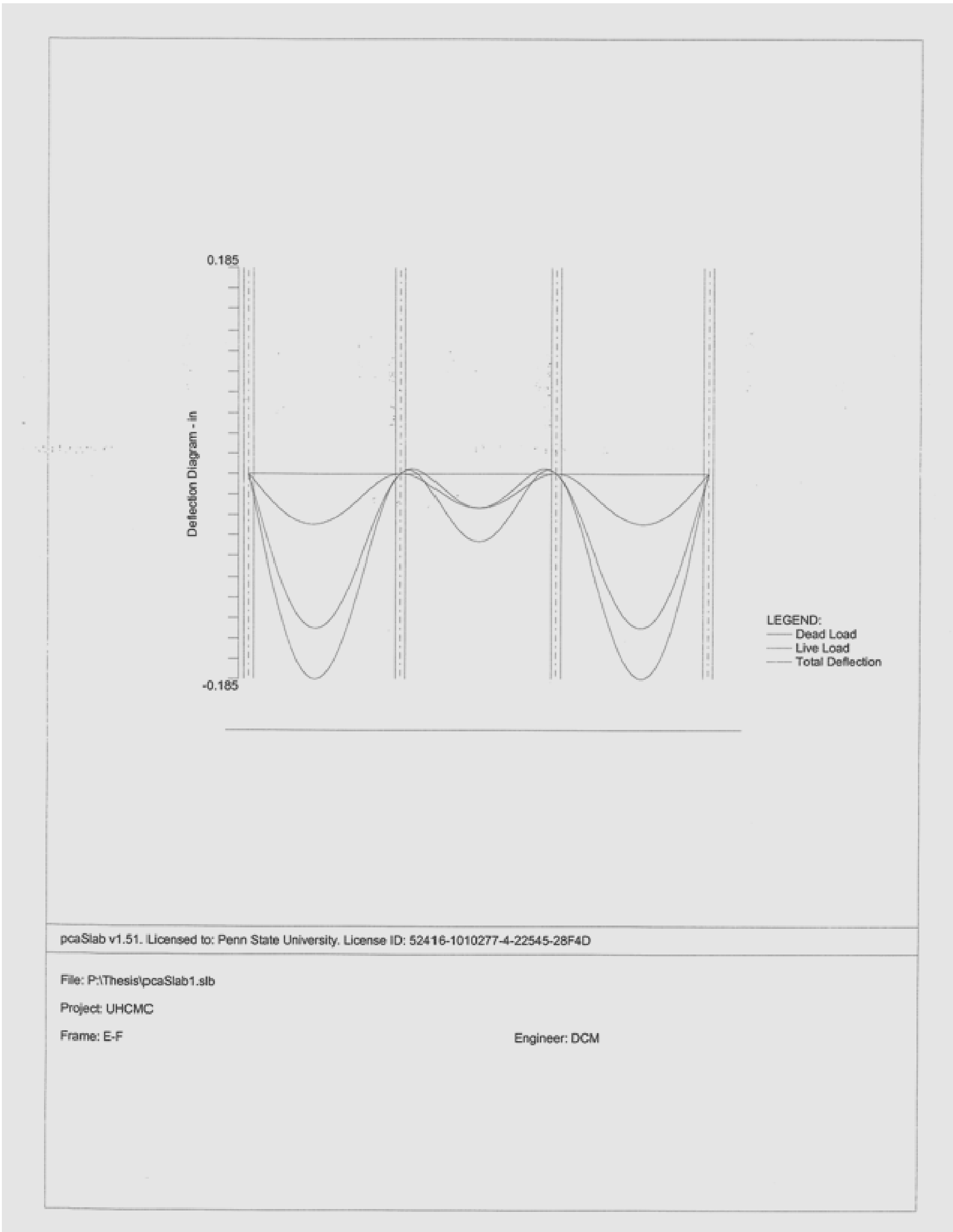

Appendix C

Two-Way Flat Slab w/ Drop Panels



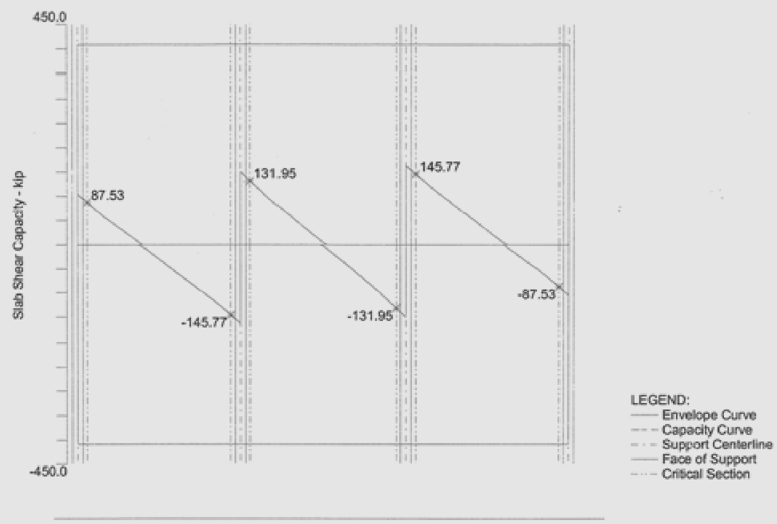
Appendix C

Two-Way Flat Slab w/ Drop Panels



Appendix C

Two-Way Flat Slab w/ Drop Panels



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File: P:\Thesis\pcaSlab1.slb

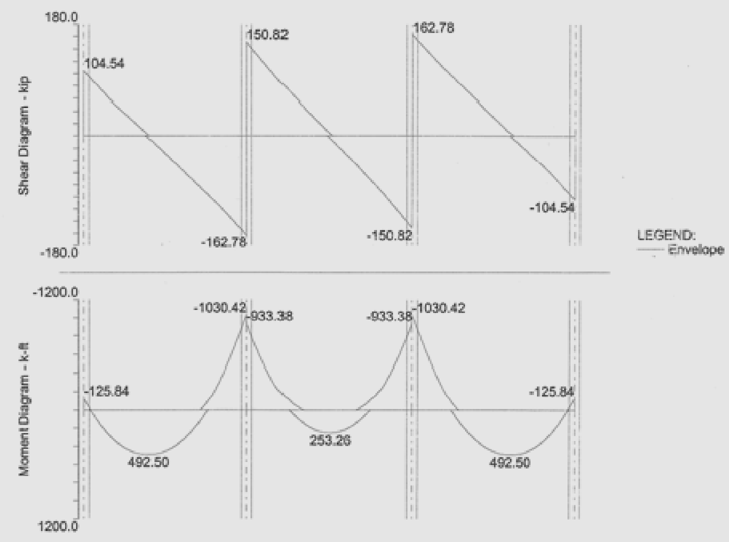
Project: UHCMC

Frame: E-F

Engineer: DCM

Appendix C

Two-Way Flat Slab w/ Drop Panels



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File: P:\Thesis\pcaSlab1.slb

Project: UHCMC

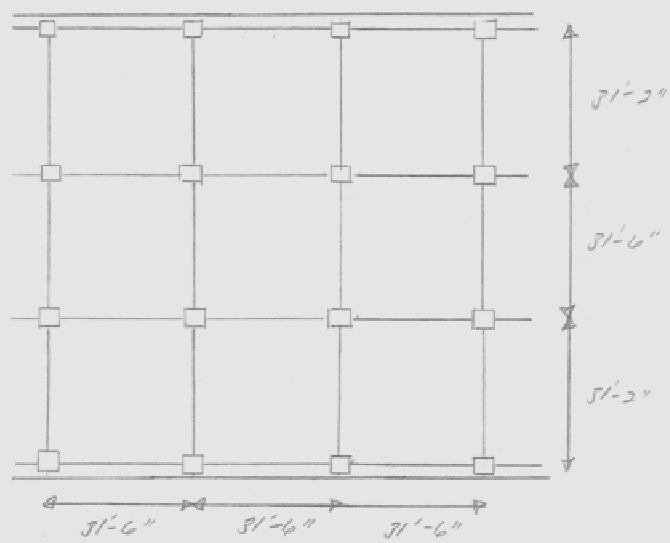
Frame: E-F

Engineer: DCM

Appendix D

Two-Way Post-Tensioned Flat Slab

Post-Tension Slab



Concrete: normal weight 150pcf

$f'_c = 5000psi$
$f'_{ci} = 3,000 psi$ (assumed)

Rebar:
$f_y = 60,000psi$

PT:
Unbonded Tendons
1/2" Ø
7 wire strands
$A = .153 in^2$
$f_{pu} = 270ksi$

Estimated prestress loss: 15ksi (ACI 318-08 18.6)

$f_{se} = .7(270ksi) - 15ksi = 174ksi$ (ACI 318-08 18.5.1)

$P_{eff} = A \cdot f_{se} = (.153)(174ksi) = 26.6$ k/tendon

Appendix D

Two-Way Post-Tensioned Flat Slab

• Determine Preliminary slab thickness

$\frac{L}{h} = 45$

$h = \frac{(31.5)(12)}{45}$
$= 8.5''$

• Loads

Dead:
Self weight — $(8.5/12)(150) = 106$ psf
Ceiling Partition — 5 psf
Suspended Mech Equip. — 10 psf
Total — 121 psf

Live:
Floor Load — 60 psf (red)

Live Load Reduction:

$L = L_o\left(.25 + \frac{15}{\sqrt{1.0(991)}}\right)$
$= 64.4$ psf
≈ 60 psf (no reduction)

• Design East-West Frame

- Ignore column stiffness for simplicity
- $\frac{LL}{DL} \leq 3/4 \Rightarrow$ no pattern loading req'd (ACI 318-08 13.7.6)

$A = bh$
$= (31.5)(12)(8.5)$
$= 3213 \text{ in}^2$

$S = \frac{bh^2}{6}$
$= \frac{(31.5)(12)(8.5)^2}{6}$
$= 4551 \text{ in}^3$

Appendix D

Two-Way Post-Tensioned Flat Slab

Set Design Parameters

Allowable Stress Class: U (ACI 318-08 18.3.3)

At time of jacking (ACI 318-08 18.4.1)

$f'_{ci} = 3{,}000\,psi$
$C = .6 f'_{ci} = 1{,}800\,psi$
$T = 3\sqrt{f'_{ci}} = 164\,psi$

At service loads (ACI 318-08 18.4.2(a))

$f'_c = 5000\,psi$
$C = .45 f'_c = 2{,}250\,psi$
$T = 6\sqrt{f'_c} = 424\,psi$

Average precompression limits (ACI 318-08 18.12.4)

$P/A = 125\,psi$ min
$= 300\,psi$ max

Target Load Balances

60% - 80% of DL for slabs

$.75 w_{DL} = .75(106)$
$= 79.5\,psf$

Cover:

$= 3/4"$ @ top (IBC 06)

Tendon Profile

$a_{INT} = 7.5" - 1.0"$
$= 6.5"$

$a_{END} = \frac{(4.0" + 7.5")}{2} - 1.75"$
$= 4"$

Appendix D

Two-Way Post-Tensioned Flat Slab

Prestress force req'd to balance 75% of DL (SW)

$w_b = .75(w_{DL}) = .75(106)(31.5) = 2.5\ ^k/ft$

Force need to counteract end bay load

$$P = \frac{w_b \cdot L^2}{8 a_{end}} = \frac{(2.5)(31.5)^2}{8(4/12)} = 930^k$$

Precompression Allowance Check

$$\#\text{ of tendons} = \frac{930^k}{26.6^k/\text{tendon}} = 34.9 \approx 35\text{ tendons}$$

$$P_{ACT} = (35)(26.6) = 931^k$$

$$w_b\,(new) = \left(\frac{930}{931}\right)(2.5) = 2.5$$

$$\sigma_{ACT} = \frac{P_{ACT}}{A} = \frac{(931)(1000)}{3213} = 289.9\text{ psi}$$

> 125 psi ✓ok
< 300 psi ✓ok

Interior Span Check

$$P = \frac{(2.5)(31.5)^2}{8(6.5/12)} = 572^k < 931^k$$

Check balanced load

$$w_b = \frac{(931)(8)(6/12)}{(31.5)^2} = 3.75\ k/ft$$

$$\frac{w_b}{w_{DL}} = \frac{3.75}{3.34} > 100\%$$ ✗ N.G.

Appendix D

Two-Way Post-Tensioned Flat Slab

Try 30 tendons

$P_{ACT} = 30(26.6) = 798^k > 572^k$

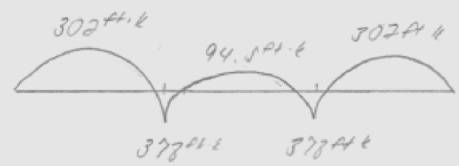
$P/A = \frac{(798)(1000)}{3213} = 248 psi$ > 125 psi ✓ok, < 300 psi ✓ok

$w_b = \frac{(798)(8)(6/12)}{(31.5)^2} = 3.21$

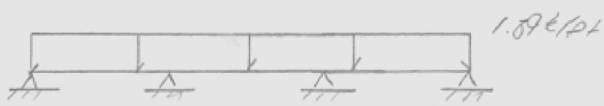
$\frac{w_b}{w_{DL}} = \frac{3.21}{3.34} = .96 < 1.0$ ✓okay

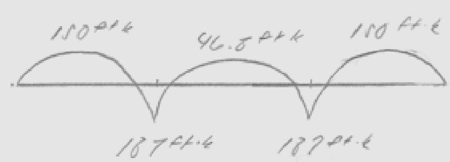
• check slab stresses

$w_{DL} = (121)(31.5)/1000 = 3.81 k/ft$



$w_{LL} = 60(31.5)/1000 = 1.89 k/ft$

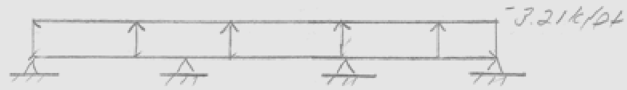


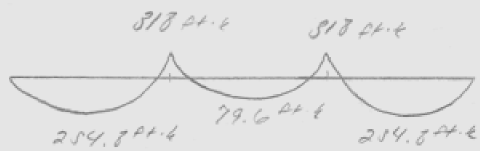


Appendix D

Two-Way Post-Tensioned Flat Slab

$w_b = -3.21\ k/ft$





- Stage 1: Stress after jacking (DL + PT)
  see spread sheet w/ calcs in accordance w/: (ACI 318-08 18.4.1)

$$f_{top} = \frac{(-M_{DL} + M_{bal})}{S} - \frac{P}{A}$$

$$f_{bot} = \frac{(+M_{DL} - M_{bal})}{S} - \frac{P}{A}$$

} reverse for support stresses

- Stage 2: Stresses at service load (DL + LL + PT)
  see spreadsheet w/ calcs in accordance w/ (ACI 318-08 18.3.3/18.4.2)

$$f_{top} = \frac{(-M_{DL} - M_{LL} + M_{bal})}{S} - \frac{P}{A}$$

$$f_{bot} = \frac{(+M_{DL} + M_{LL} - M_{bal})}{S} - \frac{P}{A}$$

- Load Checks

  Stage 1

  compression < $.6f'_{ci}$ ✓okay
  tension < $3\sqrt{f'_{ci}}$ ✓okay

Appendix D

Two-Way Post-Tensioned Flat Slab

Stage 2:

compression < $.45 f'c$ okay
tension < $6\sqrt{f'c}$ okay

- Ultimate strength

Primary moments

$M_1 = P * e$

$e = 0"$ @ ext support
$e = 3"$ @ int support (N.A.)

$M_1 = \frac{798(3)}{12} = 199.5^{k\cdot ft}$

Secondary moments

$M_{sec} = M_{bal} - M_1$
$= 318 - 199.5$
$= 118.5^{ft\cdot k}$

$M_u = 1.2 M_{DL} + 1.6 M_{LL} + 1.0 M_{sec}$

midspan:
$1.2(302) + 1.6(150) + 1.0(118.5/2) = 661.65^{ft\cdot k}$

support:
$1.2(378) + 1.6(187) + 1.0(118.5) = 871.3^{ft\cdot k}$

Appendix D

Two-Way Post-Tensioned Flat Slab

• Determine Reinforcement

Positive moment region

Int. Span

$f_t = -88.84 < 2\sqrt{f'_c}$

- No positive reinforcement req'd (ACI 318-08 18.9.3.1)

Exterior span

$f_t = 271.97 > 2\sqrt{f'_c} = 2\sqrt{5000} = 141\text{ psi}$

- min. rein. req'd

$y = \frac{f_t}{(f_t + f_c)} \cdot h$

$= \frac{271.97}{(271.9 + 767.97)} \cdot (8.5)$

$= 2.22''$

$N_c = \frac{M_{D+L}}{S}(.5)(y)(l_2)$

$= (12)\frac{(802 + 150)}{4551}(.5)(2.22)(31.5)(12)$

$= 500.17k$

$A_{s,min} = \frac{N_c}{.5 f_y}$

$= \frac{500.17}{.5(60)}$

$= 16.67\text{ in}^2$

$A_{s,min} = \frac{(16.67\text{ in}^2)}{31.5} = .52\text{ in}^2/\text{ft}$

Use #5 @ 6in oc bottom = .62 in²/ft ✓ okay

Negative moment region (Interior)

$A_{s,min} = .00075 A_{cf}$ (ACI 318-08 18.9.4.1)

$A_{cf} = (8.5)(31.5)(12)$

$= 3213\text{ in}^2$

$A_{s,min} = .00075(3213)$

$= 2.41\text{ in}^2$

Appendix D

Two-Way Post-Tensioned Flat Slab

Use (8) #5 = 8(.31) = 2.48 in² ✓ Okay

Negative Moment Region (Exterior)

$A_{s\,min} = 2.41\,in^2$

Use (8) #5 = 8(.31) = 2.48 in² ✓ Okay

• Check min. Reinforcement for Ultimate Str.

$M_n = (A_s f_y + A_{ps} f_{ps})(d - \frac{a}{2})$

$A_{ps} = .153\,in^2\,(30\ tendons)$
$= 4.59\,in^2$

$\frac{L}{h} = \frac{31.5(12)}{8.5} = 44.4 > 35$

$f_{ps} = f_{se} + 10{,}000 + \left(\frac{f'_c b d}{300 A_{ps}}\right)$ (ACI318-08 18.7.2)

$= 174{,}000 + 10{,}000 + \left(\frac{5000(31.5)(12)d}{300(4.59)}\right)$

$= 184{,}000 + 1372\,d$

At supports

$d = 8.5 - 3/4'' - 1/4'' = 7.5''$

$f_{ps} = 184{,}000 + 1372(7.5)$
$= 194{,}290$

$a = \frac{(A_s f_y + A_{ps} f_{ps})}{.85 f'_c \cdot b}$

$= \frac{(2.48)(60) + (4.59)(194{,}290)}{.85(5000)(31.5)(12)}$

$= .55''$ 7.725

$\phi M_n = \frac{.9((2.41)(60) + (4.59)(194))(7.5 - (\frac{.55}{2}))}{12}$

$= 558\ ft\text{-}k$

- Reinforcement for ultimate strength governs.

Appendix D

Two-Way Post-Tensioned Flat Slab

At midspan

$d = 8.5'' - 1\tfrac{1}{2}'' - \tfrac{1}{4}'' = 6\tfrac{3}{4}''$

$f_{ps} = 184{,}000 + 1372(6\tfrac{3}{4})$
$= 193{,}261$

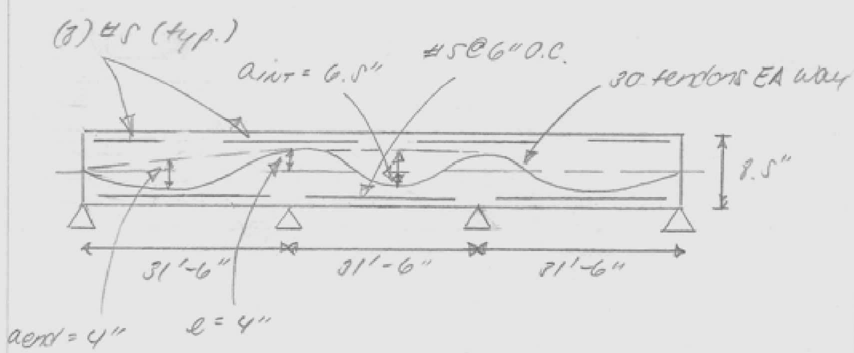
$a = \dfrac{(19.53)(60) + (4.59)(193)}{.85(5)(31.5)(12)} = 1.28''$

$\phi M_n = \dfrac{.9\left[\left((16.67)(60) + (4.59)(193)\right)\left(6.75 - \left(\frac{1.28}{2}\right)\right)\right]}{12}$

$= 864.3^{\text{ft-k}} \quad 661.65^{\text{ft-k}}$ ✓ okay

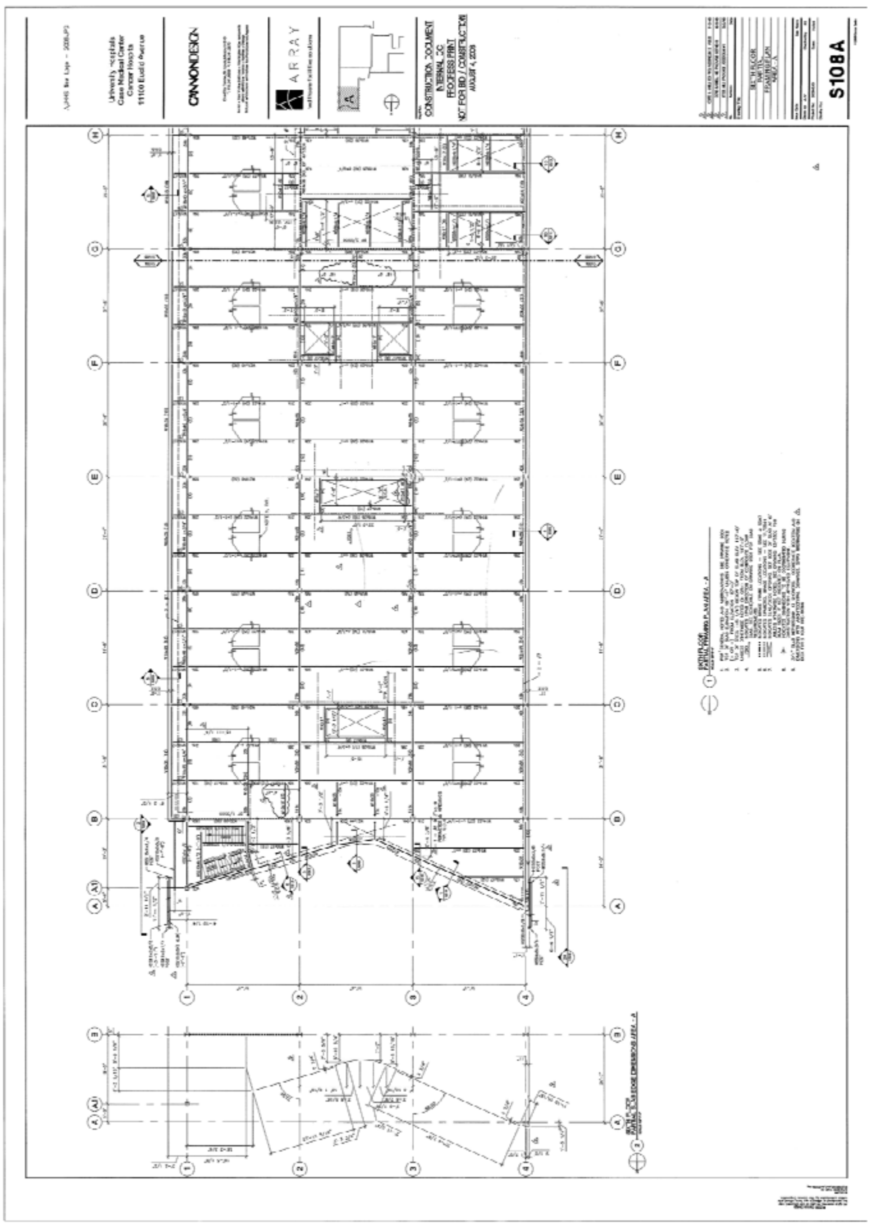
Summary:

@ Int. Support — (8) #5 (neg.)
@ Ext. Support — (8) #5 (neg.)
@ Midspan — #5 @ 6" o.c. (pos.)



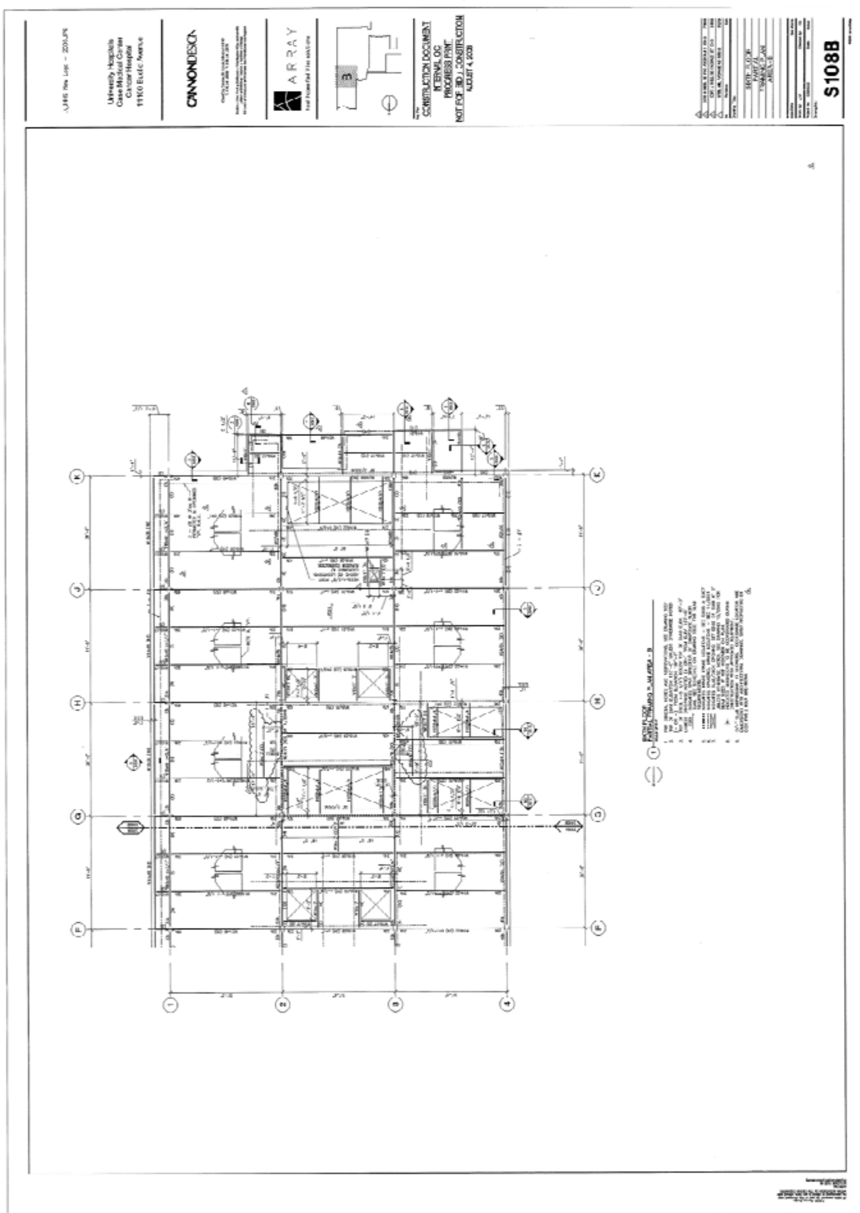
Appendix E

Level 6 Floor Plan



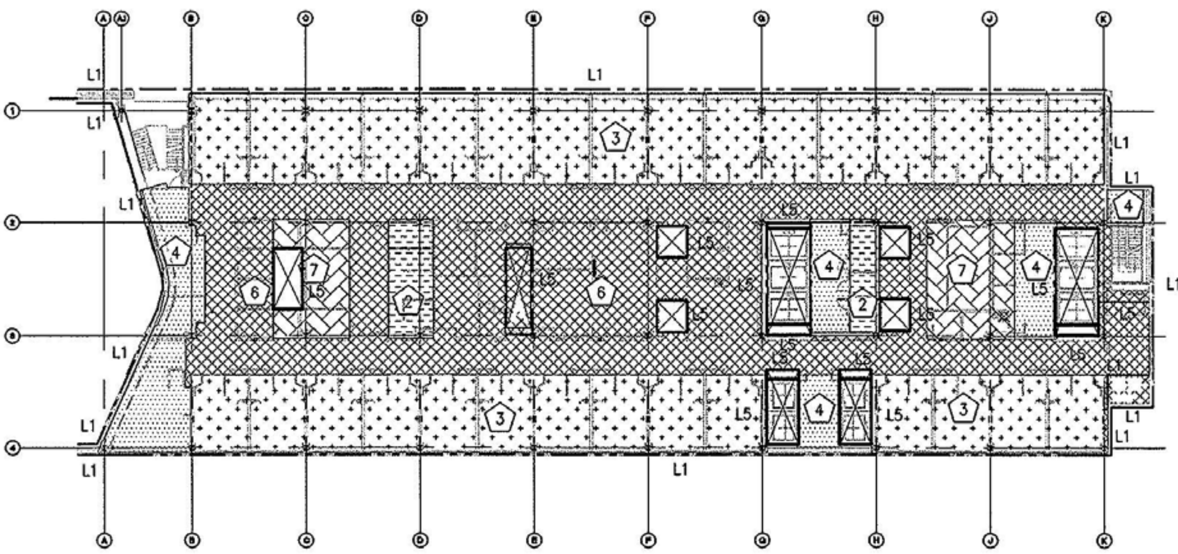
Appendix E

Level 6 Floor Plan



Appendix E

Loading Diagram



2 LEVEL 5 AND 6
LOAD KEY DIAGRAM
SCALE: 1/32=1'-0"

SUPERIMPOSED LOADS

SURFACE LOAD SCHEDULE					
LABEL	PATTERN	DL (psf)	LL (psf)	REDUCTION TYPE	MASS DL (psf)
1		47	370	Unreducible	102.5
2		47	150	Unreducible	103.2
3		47	40	Reducible	75.7
4		41	100	Reducible	41
5		47	60	Reducible	102.5
6		47	60	Reducible	75.7
7		47	125	Unreducible	124
8		25	30	Unreducible	31
9		30	150	Unreducible	175
10		340	100	Unreducible	385
11		70	175	Unreducible	181
12		30	270	Unreducible	390
13		31	53	Unreducible	53
14		150	100	Unreducible	195
15		80	150	Unreducible	215

LINE LOAD SCHEDULE			
LABEL	DL (k/ft)	MASS DL (k/ft)	LL
L1	.3	.3	
L2	.56	.42	
L3	.36	.36	
L4	.5	.5	
L5	.225	.225	
L6	.4	.4	

SNOW DRIFT LOAD SCHEDULE						
LABEL	PATTERN	DL (psf)	MASS DL (psf)	MIN SL (psf)	MAX SL (psf)	WIDTH (ft)
D1		25	31	30	58	8.2
D2		25	31	30	83	14.25
D3		25	31	30	75	12.32
D4		25	31	30	64	9.8
D5		25	31	30	50	6.2
D6		25	31	30	78	13
D7		25	31	30	108	20