Technical Report II

Timothy Bailiff | Structural Option

SUSQUEHANNA CENTER EXPANSION AND RENOVATION, HARFORD COMMUNITY COLLEGE

- Picture taken from Turner website

Executive Summary

The purpose of Technical Report II is to study and analyze the pros and cons of alternative floor systems for the Susquehanna Center Renovation and Expansion. The existing floor system will be analyzed along with three other floor systems and then compared.

These systems were analyzed at a typical bay of 26' x26'

- Two-way slab with beams
- Composite deck on wide flange beams
- One-way slab with beams
- Hollow-core plank on steel beam

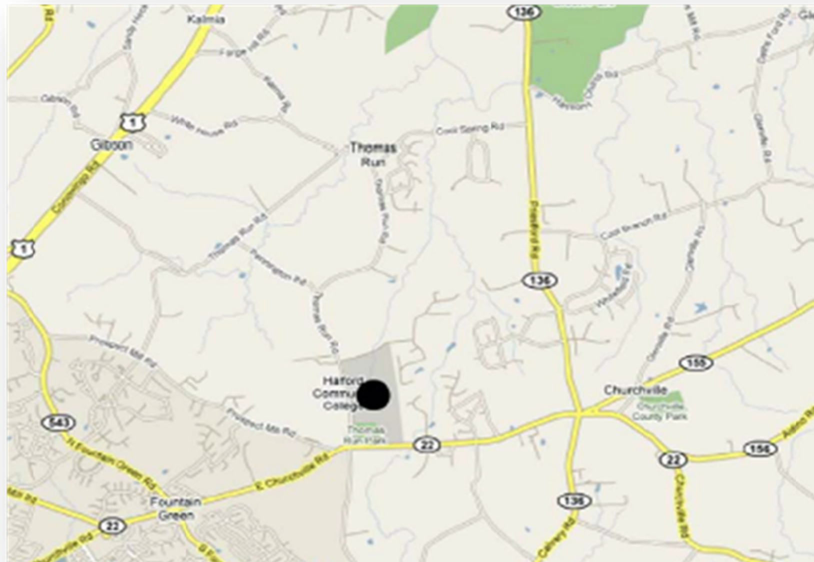
The composite deck system was designed using the Vulcraft Manuel and the AISC Steel Construction Manuel. The system consisted of a 2VLI20 deck and a W12X14 beam. The one-way slab with beams was designed using the ACI 318-08 reference code. The slab was 8" with #4 bars spaced @ 8" O.C. and the beam was sized to a 22"x16" beam with ($) #9's at the bottom. The hollow-core plank system was designed using the PCI handbook. The plank was sized for a 4HC8 68-S strand with no topping with a W21X62 to support it.

Each of these systems will be analyzed on a set of criteria which involves costs, height, weight, fire-rating, and structural and non-structural advantages and disadvantages.

Drawing, hand calculations and floor plans that are necessary will be provided in the appendices of the report.

Building Introduction:

The Susquehanna Center Renovation and Expansion at Harford Community College is located on 401 Thomas Run Road in Bel Air, MD. The project will be constructed in August 2010 in collaboration with hord | coplan | macht as the architect, Site Resources, Inc. as the civil engineer, CMJ Structural Engineering, Inc. as the structural engineer, Burdette, Koehler, Murphy & Associates, Inc. as the mechanical electrical engineer and Counsilman Hunsaker as the natatorium consultant.

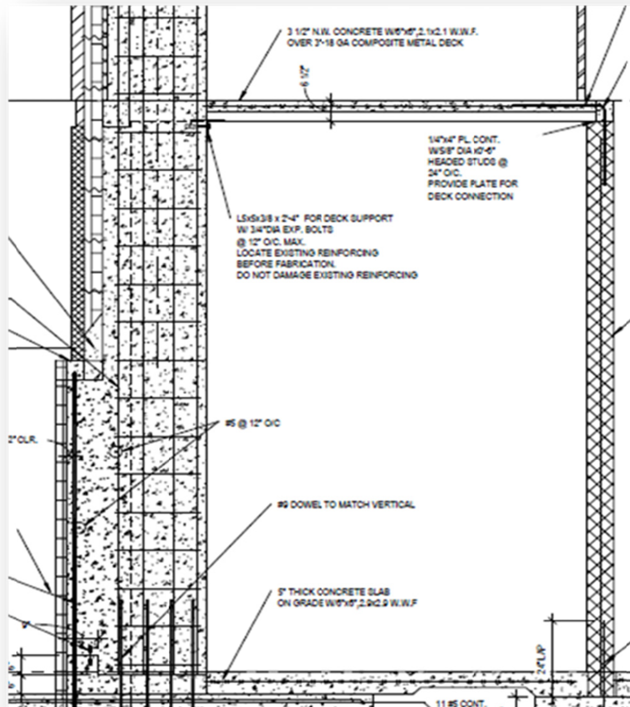


The Susquehanna Center consists of a renovated arena, pool and a fitness center. The center is 49,150 SF which will be totally interiorly renovated and the expansion will include a new 37,460 SF arena, which will expand the total area of the building to 86,610 SF. The project will also include a new parking lot of 160 spaces, a new loop around the building and realigning of the entrances at the entrance drive.

Structural Systems

Floor Systems

A typical floor in the expansion consists of 3 ½” N.W. concrete with 6”x6”, 2.1x2.1 W.W.F. over 3”-18 gage composite metal deck. The arena floor consists of a 5” thick slab on grade.



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Framing Systems

All of the structural columns in the expansion are 28” x 28” cast-in-place concrete columns which extend from the foundation to the full height of the building.

Lateral System

The lateral system contains concrete moment frames consisting of concrete wall beams and interior beams.

Roof Systems

The roof system in the expansion was erected using 96SLHSP joists spaced at 8’-0” o.c. and span the length of the arena.

Building Materials Used

The following tables provided will list the materials used in construction of the building, which were located in the structural drawings and the specifications.

Concrete		
Usage	Weight	Strength (PSI)
Spread Footing Foundations	Normal	4000
Retaining Walls	Normal	4000
Slab on Grade	Normal	4000
Elevated Slab	Normal	4000

Table 1: Concrete Materials

Steel		
Usage	Standard	Grade
W-Shaped Structural Steel	ASTM 992 A	50
Steel Pipe	ASTM A 501	B
Steel Tube	ASTM A 500	B
Steel Deck	ASTM A 611/ASTM A 446	N/A
Bolts, Nuts, and Washers	ASTM A 325/ASTM F 1852	N/A
Welded Wire Fabric	ASTM A 615	65
Reinforcing Bars	ASTM A 615/A 615M	60

Table 2: Steel Materials

Design Codes

All of the structural design and construction of the Susquehanna Center Renovation and Expansion shall comply with the all of the articles and sections of the following codes in compliances with all Federal, State, County, and Local ordinances and regulations:

- 2006 International Building Code (IBC)
- National Electrical Code (NEC)
- Uniform Plumbing Code (UPC)
- National Sanitation Foundation (NSF)
- Building Code Requirements for Reinforced Concrete (ACI 318-08)
- American Society of Civil Engineers (ASCE 7-10)

Gravity Loads

This report includes calculated dead, live and snow loads. These calculations were compared to the actual calculations in the structural drawing and general notes.

Dead and Live Loads

Superimposed Dead Loads	
Description	Loads
Roof	
Insulation	3 PSF
Structural Framing	15 PSF
Ceiling	2 PSF
MEP	15 PSF
Miscellaneous	15 PSF
Total	50 PSF
Floor	
Structural Framing	66 PSF
Ceiling	2 PSF
MEP	5 PSF
Miscellaneous	5 PSF
Total	78 PSF
Snow	30 PSF

Table 3: Design Dead Loads

Description	Quantity (SF)
Main Level	78670
Arena Level	39760
Roof	78670

Table 4: Typical Floor Area

Design Live Loads	
Description	Design Loads
Roof	30 PSF
Floor	100 PSF

Table 5: Design Live Loads

Lateral Loads

In this report, wind and seismic loads will be partially analyzed to create a more accurate sense of how the lateral resisting system (moment frames) reacts under these loads. A complete and more detailed analysis will be composed in Tech II and III.

Wind Loads

To accurately portray the transfer of lateral loads in the ground, E-W wind pressures will be applied to the building and an analysis will be performed.

Wind Design Criteria		
Design Wind Speed (V)	90 MPH	ASCE 7-10, Fig. 6-1
Directional Factor (Kd)	0.85	ASCE 7-10, Table 6-4
Importance Factor (I)	1.10	ASCE 7-10, Table 6-1
Exposure Category	C	ASCE 7-10, 6.5.6.2 and 6.5.6.3
Topography Factor (Kzt)	1.00	ASCE 7-10
Internal Pressure (GCpi)	0.18	ASCE 7-10

Table 6: Wind Design Values

External Pressure Coeff. (Cp)		
Description	N/S Wind	E/W Wind
L/B	0.531	1.88
Windward Wall	0.80	0.8
Leeward Wall	-0.50	-0.324
h/L	0.169	0.089
Roof Windward	-0.3	-0.3
Roof Leeward	-0.18	-0.18

Table 7: External Pressure Coefficient

Velocity Pressure Coefficient and Velocity Pressure			
Level	Elevation	Kz	Qz
Arena	0'-0"	0.85	16.5
Main	15'-0"	0.85	16.5
Roof	34'-91/2"	1.04	20.2

Table 8: Velocity Pressure Coefficient and Velocity Pressure

Seismic Loads

A seismic ground motion was calculated in this report per ASCE 7-10 and the force equaled 901 k. The design values that were calculated are located in the table.

Seismic values	
Ss	0.20g
S1	0.053g
Sms	0.24
Sm1	0.0901
SDS	0.16
SD1	0.06
Ie	1.25
R	3
CT	0.016
x	0.9
T	0.39
Cs	0.066
k	1.00
w	13647 k
V	901 k

Table 9: Seismic Design Values

Floor System Comparison Analysis

A spot check of the existing floor system consisting of a two-way slab with beams can be located in Appendix A. In addition three other floor systems will be designed, including a one-way slab with beams, composite deck on a beam, and hollow-core planks of an adjusted bay for easiness of calculation. These systems were designed using ACI 318-08, PCI design handbook, Vulcraft Manuel, and AISC thirteenth edition design references.

A cost assessment analysis will also be performed using RS Means: Square Foot Cost-2011 to compare each floor system through the most economical means.

Existing Floor System: Two-Way Slab with Beams

The existing floor system in the building was a two-way slab with beams. For easiness of calculation the bay was made uniform to a 26'x26' bay. The slab is 8" deep with #4 bars @ 12" O.C. There were two different sizes of beams in the bay that was chose to analyze. Two beams were 24"x32" with (5) #7's on the bottom and (4) #6's on the top. The other beam in the bay was 18"x28" with (5) #8's on the bottom and (4) #6's on the top.

Advantages:

An advantage to this system is that it can eliminate the vibration concerns which can occur with steel systems. It also doesn't need additional fire proofing which will raise the cost. Also the materials used in the system are easily accessible for construction.

Disadvantages:

A disadvantage to this system is that it is a very deep system decreasing the floor to floor height which can cause problems with MEP.

Proposed Floor System: Composite Deck on Steel Beams

This floor system will consist of a 2"-20 gauge deck with a 4.5" thick slab referenced through the Vulcraft Manuel and wide flange steel beams and girders.

Decking:

The composite deck spans a distance of 6'-6" and takes a total load of 160 psf. A 2VLI20 deck was selected for design because at a 3 span condition its construction span equals 10'-7" which is adequate for an unshored condition at a span of 6'-6". The given strength of the deck is 275 psf being approximately 50% more capacity than the total load. Also, the unprotected 2VLI20 deck with a 4.5" slab achieves a 2 Hr. firing rating according to the Vulcraft Manuel.

Composite Beam:

A W12X14 was selected and proven to work for the required loads proved by the floor system. With the flexural strength of the beam being ΦM_n=160 ft-k which exceeds M_u=128 ft-k, the compact section criteria was satisfied. The beam was adequate for live load deflection, wet concrete deflection and unshored strength. The values were as follows; live load deflection was = 0.73in < 0.87in, wet concrete deflection was = 1.22in < 1.3in, and unshored strength (ΦM_p=65.3 ft-k > M_u=48.7 ft-k).

Advantages:

A critical advantage that this system provides is that decreases the total depth of the system to 16 inches. At this depth mechanical equipment and lighting fixtures will have more than reasonable space to be installed and ensure an increase of floor to floor height. Also another advantage would be the decrease in overall weight compared to the existing concrete system.

Disadvantages:

The critical disadvantage of this floor system would include the installation of fireproofing that would be required for the steel beams and girders, which could increase the cost of the system.

Proposed Floor System: One-Way Slab with Beams

The span of the beams in the one-way slab was made uniform for easiness of calculation. The slab thickness was determined to be 6" according to Table 9.5(a) for min slab thickness (ACI 318-08). The floor system was designed to support a live load of 100 psf, superimposed dead load of 15 psf, and the weight of the framing members themselves.

The self weight of the slab was determined to be 75 psf which is added to the superimposed and live loads that the beams were designed to support. The beam was sized to be a 22"x16" with (4) #9's located at the bottom of the beam, which satisfied the flexural capacity (ΦM_n=283 ft-k > M_u=282 ft-k). The slab was designed to be a 6" slab with #4 bars @ 8" O.C. which meet the flexural requirements (ΦM_n=6.45 ft-k > M_u=5.66 ft-k).

Advantages:

An advantage for a floor system of this choice is that the materials used are very easily accessible to the contractors for construction. The on-way slab with beams can also decrease the floor to floor height and span longer distances.

Disadvantages:

a disadvantage for this system is that is a heavy system which can cause concerns for the foundation when changing the system.

Proposed Floor System: Hollow-Core Planks on Beam

The bay size of this floor system was reduced to 26’x24’ for the planks to fit uniformly throughout the 24’ span. The floor system consists of 4’x8” (4HC8) hollow-core planks with no topping and W21X62 beams supporting a total load of 171 psf. For a span of 24’ a 68-S strand was selected and satisfies the load requirements according to PCI handbook (180 psf > 171 psf). The W21X62 satisfied the flexural requirements (ΦMn=540 ft-k > 497 ft-k). Also the beam was adequate for live load deflection = 0.503” < 0.8”, and total load deflection = 0.872” < 1.2”.

Advantages:

An advantage of the hollow-core plank system is that it will decrease the overall floor to floor height which makes MEP installation more convenient. Since the planks are precast, it will decrease the time for erection saving money and storage space in the process.

Disadvantages:

The critical disadvantage of this system would be that the supporting beams would have to be fire proofed which will increase costs and labor.

Floor System Comparison

Floor system Comparison					
		Floor Systems			
		Existing Two-Way Slab with Beams	**Composite Deck on Wide Flange Beam**	**One-Way Slab with Beams**	Hollow-Core Plank
Limitations	System Weight (psf)	**135**	58.7	275	165
	Slab depth (in)	**8**	4.5	6	8
	Total Depth (in)	32	16.5	28	29
Cost and Safety	Fire rating	2	2	2	2
	Extra fire proofing required	No	Yes	No	Yes
	Total Cost ($/SF)	17.8	17.75	18.7	10.39
Impact	Foundation impact	N/A	Yes	Yes	Yes
	Architectural Impact	N/A	Yes	Yes	Yes
	Constructability	Moderate	Easy	Moderate	Easy
Consideration	Vibration Concerns	Minimal	Some	Minimal	Minimal
	Additional Study	N/A	Yes	Yes	Yes

Conclusion

There were three alternative floor systems that were studied and designed in accordance with the existing system which was a one-way slab, composite deck on steel beams and hollow-core plank on steel beams. All of these floors system were analyzed using a modified bay size of 26 feet by 26 feet.

The one-way slab was designed using the ACI 318-08 code. The slab was checked to pass and sized at 6 inches and having #4 reinforcing bars placed at 8 inches on center. The beam was designed to the flexural requirements and was sized to be a 22 inch by 16 inch beam with (4) #9 reinforcing bars in the bottom. The composite deck system consisted of a 2VLI 20 gauge deck with a 4.5 inch slab. The beam supporting the deck was sized to W12X14, which passed for both live load wet concrete deflections. The hollow-core plank was sized to be an 8 inch plank with 68-S strand (4HC8) spanning a length of 26 feet. This system was designed using the PCI handbook.

After studying and comparing these systems, two of the system didn't seem likeable for this building, the hollow-core plank and composite deck. The hollow core plank decreased the floor to floor height but ordering material of this magnitude can be very expensive. The composite deck system also decreases the floor height but it is very susceptible to vibration concerns especially in an arena area like this building. The logical alternative would be the one-way slab; it increases the weight slightly but also decreases the floor to floor height.

Appendices

Appendix A: Two-Way Slab with Beams

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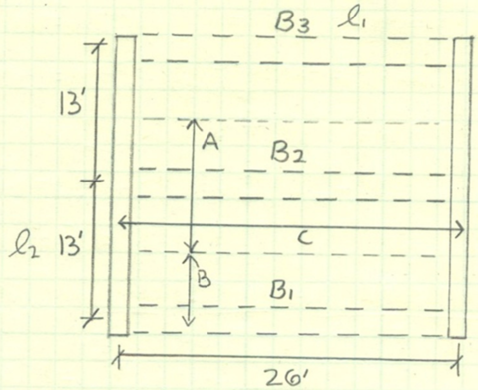
Existing Two-Way Slab with Beams



DL = 15psf + 135 psf

LL = 100psf

f'c = 4000psi

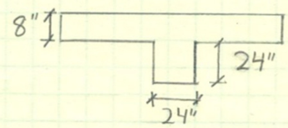
fy = 60000psi

$B_1 - B_3 \Rightarrow 24'' \times 32''$

$B_2 = 18'' \times 28''$

$$\alpha = \frac{E_{cb} I_b}{E_{cs} I_s}$$

BM B_1-B_3 : $b_E = b_w + 2h_w$



$b_E = 24 + 2(24) = 72''$

$72'' < 24'' + 8(8'') = 88'' \Rightarrow$ ok

$I_b = K \frac{b_w h^3}{12}$, $\frac{b_E}{b_w} = \frac{72}{24} = 3$, $K = 1 + 0.2(3) = 1.6$

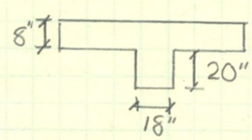
$I_b = \frac{(1.6)(24)(32)^3}{12} = 104858 \text{ in}^3$

Slab: $I_s = \frac{\ell_2 t^3}{12} = \frac{(13)(12)(8)^3}{12} = 6656 \text{ in}^3$

$\alpha = \frac{104858}{6656} = 15.8$

BM B_2 : $b_E = b_w + h_w$

$b_E = 18 + 2(20) = 58''$



$I_b = K \cdot \frac{b_w h^3}{12}$, $\frac{b_E}{b_w} = \frac{58}{18} = 3.22$

$K = 1 + 0.2(3.22) = 1.64$

$I_b = (1.64)(18)(28)^3/12 = 54002 \text{ in}^3$

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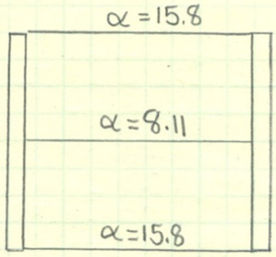
$I_s = \frac{\ell_2 t^3}{12} = \frac{(13)(12)(8'')^3}{12} = 6656 \text{ in}^3$

$\alpha = \frac{54002}{6656} = 8.11$



- Longer clear span $\ell_n \Rightarrow \ell_1 - 12''$, $26' - 1' = 25'$

- Shorter clear span $\ell_n \Rightarrow \ell_2 - \left(\frac{24+18}{2}\right)$, $= 13' - 21/12 = 11.25'$

- ratio $\beta = \frac{25'}{11.25'} = 2.22$

- Average $\alpha = (15.8 + 8.11)/2 = 11.96$

$\alpha_u = 2.0$, $M_{int} = \frac{\ell_n(0.8 + f_y/200{,}000)}{36 + 9\beta} \geq 3.5''$

$M_{int} = \frac{(25')(0.8 + 60000/200{,}000)}{36 + 9(2.22)} = 5.89'' < 6.5''$

$w_u = 1.2 w_D + 1.6 w_L$

$= 1.2(15 + 135) + 1.6(100) = 340 \text{ psf}$

Frame A: $M_o = 1/8\, w_u \ell_2 \ell_n$

$= 1/8(.34)(13)(25)^2$

$= 345.3 \text{ ft-K}$

Frame B: $M_o = 1/8(.34)(13)(25)^2$

$= 345.3 \text{ ft-K}$

Frame C: $M_o = 1/8(.34)(26')(11.25)^2$

$= 140 \text{ ft-K}$

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Frame A: Ext. Span $M^{-}ext = 0.16(345.3) = 55.2 ft\text{-}K$

$M^{+}ext = 0.57(345.3) = 197 ft\text{-}K$

$M^{-}int = 0.70(345.3) = 242 ft\text{-}K$

$M^{-} = 0.65(345.3) = 224.4 ft\text{-}K$

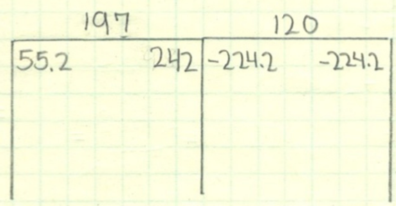
$M^{+} = 0.35(345.3) = 120 ft\text{-}K$

Frame C: Ext. Span $M^{-}ext = 0.16(140) = 22.4 ft\text{-}K$

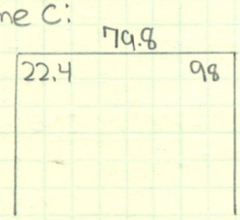
$M^{+}ext = 0.57(140) = 79.8 ft\text{-}K$

$M^{-}int = 0.70(140) = 98 ft\text{-}K$

Frame A = Frame B



Frame C:



- Transverse Distribution:

$\frac{\ell_2}{\ell_1} = \frac{13}{26} = 0.5$

$C = \Sigma(1 - 0.63\, x/y)(x^3 y/3)$

Interior Beam:

a) $\left[1 - \frac{0.63(8)}{72}\right]\left[\frac{72(8)^3}{3}\right] + \left[1 - \frac{0.63(24)}{24}\right]\left[\frac{24(24)^3}{3}\right]$

$= 11428 + 40919 = 52347 in^4 \Rightarrow$ use this

b) $\left[1 - \frac{0.63(8)}{58}\right]\left[\frac{58(8)^3}{3}\right] + \left[\frac{1 - 0.63(18)}{20}\right]\left[\frac{20(18)^3}{3}\right]$

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$= 8968 + 16835 = 25803 in^4$

Frame A: = Frame B

$\ell_2/\ell_1 = 0.5$, $\alpha \cdot \frac{\ell_2}{\ell_1} = (15.8)(0.5) = 7.9 > 1$

$C = 52347 in^4$

$\beta_t = \frac{C}{2I_s} = \frac{52347}{2(6656)} = 3.93$

$M_{ext}^- = 55.2$

-55.2 — 90% to column strip = -49.68 → 85% to beam = -42.2; 15% to slab = -7.45

-55.2 — 10% to middle strip = -5.52

$M_{ext}^+ = 197$

242 — 90% to column strip = 217.8 → 85% to beam = 185.13; 15% to beam = 32.67

242 — 10% to middle strip = 24.2

$M_{int}^- = 197$

-197 — 90% to column strip = 177.3 → 85% to beam = 150.7; 15% to beam = 26.59

-197 — 10% to middle strip = 19.7

Frame C:

$\alpha \cdot \ell_2/\ell_1 = (8.11)(0.5) = 4.06 > 1$

$C = 52347 in^4$

$\beta_t = \frac{C}{2I_s} = \frac{52347}{2(6656)} = 3.93$

$M_{ext}^- = 22.4$

-22.4 — 90% to Column strip = -20.16 → 85% to beam = -17.14; 15% to slab = -3.02

-22.4 — 10% to middle strip = -2.24

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Mext+ = 79.8

79.8
- 90% to column strip = 71.82
  - 85% to beam = 61.05
  - 15% to slab = 10.77
- 10% to middle strip = 7.98

Mint- = 98

98
- 90% to column strip = 88.2
  - 85% to the beam = 74.97
  - 15% to the slab = 13.23
- 10% to middle strip = 9.8

Frame A - Frame B

M- = 224.2 ft-k

-224.2
- 90% to column strip = -201.8
  - 85% to beam = -172
  - 15% to slab = -30.3
- 10% to middle strip = -22.5

M+ = 120 ft-k

120
- 90% to column strip = 108
  - 85% to beam = 92
  - 15% to slab = 16.2
- 10% to middle strip = 12

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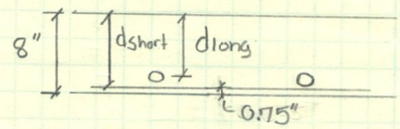
Reinforcement design and distribution

Column Strip = 2 x 1/4 x 13' = 6.5' = 78"

Subtract beam web: 78" - 18" = 60"

slab t = 8.0", Assume #5 bars



$d_{short} = 8" - 0.75 - 0.625/2 = 6.94"$

$d_{long} = 6.94 - 0.625 = 6.32"$

Column Strip Frame A:

Item No.	Description	Ext Span M^-_{int}	Ext Span M^+	Ext Span M^-_{ext}	Int Span M^-	Int Span M^+
1.	Mu (k-ft)	-26.59	32.67	-7.45	-30.3	16.2
2.	width of strip b	60"	60"	60"	60"	60"
3.	Effective depth	6.32"	6.32"	6.32"	6.32"	6.32"
4.	Mu x 12/b $\frac{k\text{-}in}{in}$	-5.31	6.53	-1.49	-6.06	3.24
5.	$M_n = M_u/\phi$	-29.5	36.3	-8.27	-33.6	18
6.	$R = \frac{M_n}{bd^2}$	148	182	41.4	152	90.1
7.	ρ	.00254	.00315	.0007	.00261	.00153
8.	$A_s = \rho bd$	0.96	1.19	0.27	0.99	0.58
9.	$A_{s,min} = .002bt$	0.96	0.96	0.96	0.96	0.96
10.	$N = \frac{> \text{of } 8 \text{ or } 9}{0.31}$	3.09 = 4	3.83 = 4	3.09 = 4	3.19 = 4	3.09 = 4
11.	$N_{min} = \frac{\text{wid of strip}}{2t}$	4	4	4	4	4

$\rho_{max} = 0.0181$

$M_u = \phi M_n = \phi \rho f_y b d^2 (1 - 0.59 \rho f_y / f'_c)$

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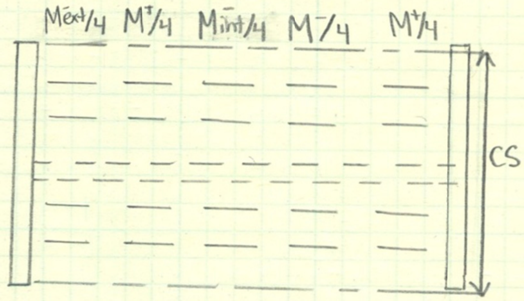
$$d^2 s = \frac{M_u}{0.9(0.0181)(60000)(12'')(1 - 0.59 \times .0181 \times 60/4)}$$

$$\approx \frac{M_u}{9850}$$

$$d_{min} = \sqrt{\frac{6.53 \times 12000}{9850}} = 2.82'' < 6.32'' \quad \text{OK} \checkmark$$

Reinforcement Placement



Check shear in slab:

$w_L = 100\,psf \quad w_D = 15\,psf + \left(\frac{8}{12}\right)(150) = 100\,psf$

$w_u = 1.2(100) + 1.6(100) = 280\,psf$

$V_u = (.280)(1')\left(10' - \frac{24/12}{2} - 6.94''/12\right)$

$V_u = 2.08K$

$\phi V_c = \phi 2 \sqrt{f'c}\, bd$

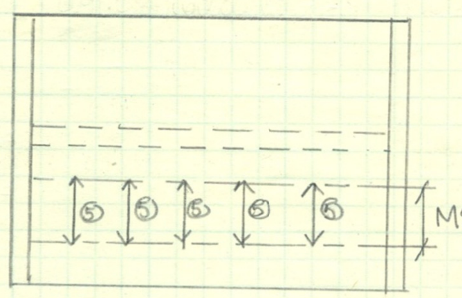
$= 0.75(2)\sqrt{4000}(12'')(6.94)/1000$

$= 7.9k > V_u = 2.08k \quad \text{ok} \checkmark$

Design of Slab Reinforcement in Middle Strip Frame A

Item No.	Description	Ext. Span M^-_{ext}	Ext. Span M^+	Ext. Span M^-_{int}	Int. Span M^-	Int. Span M^+
1.	Mu (k-ft)	-5.52	24.2	19.7	-22.5	12
2.	wid. of M.S. (b)	78	78	78	78	78
3.	Effective depth	6.32	6.32	6.32	6.32	6.32
4.	Mux12/b	-.85	3.72	3.03	-3.46	1.85
5.	$M_n = M_u/\phi$	-6.13	26.8	21.8	-25	13.3
6.	$R = \frac{M_n}{bd^2}$	23.6	103.2	84	96.3	51.2
7.	ρ	0.00004	.00172	.0014	.0016	.00085
8.	$A_s = \rho bd$	0.0197	.845	.69	.79	.419
9.	$A_{smin} = .002bt$	1.25	1.25	1.25	1.25	1.25
10.	N	4.03=5	5	5	5	5
11.	N_{min}	4.88=5	5	5	5	5

Reinforcement Placement:



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Beam Design:

$DL = 15psf + (8/12)(150) = 115psf$

$LL = 100psf$

$w_u = 1.2D + 1.6L$

$= 1.2(115) + 1.6(100) = 298 psf \times 13'$

$= 3.87 KLF$

$$M_u = \frac{w_u \ell n^2}{8} = \frac{(3.87)(26' - 24/12)^2}{8} \times 1.1 = 307 ft\text{-}K$$

- Estimate size:

$bd^2 = 20 M_u$

let $b = 16''$

$(16'')d^2 = 20(307)$

$d = 19.6 = 20''$

$h = d + 2.5 = 20 + 2.5'' = 22.5''$

use $h = 24''$ and $b = 16''$

$bd^2 = (16)(20)^2 = 6400 in^3$

- self weight

$$W_{sw} = \frac{(16)(24)(150pcf)}{144} = 400 plf$$

$W_u = 3.87 KLF + 1.2(.4) = 4.35 KLF$

$$M_u = \frac{(4.35)(24')^2}{8} = 313.2 ft\text{-}k$$

$20 \times 313.2 = 6264 < 6400 in^3$ ok ✓

- Required Steel

$$A_s = \frac{M_u}{4d} = \frac{313.2}{4(20)} = 3.92 in^2$$

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Try (4) #9's = 4in²

Nominal Moment:

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(4)(60)}{.85(4)(16)} = 4.41$

$c = a/B_1 = 4.41/0.85 = 5.19$

$d = 24'' - 3/4'' - 1.128/2 = 22.7''$

$\varepsilon_s = \varepsilon_u\left(\frac{d-c}{c}\right) = .003\left(\frac{22.7-5.19}{5.19}\right) = 0.01 > 0.005$ ok ✓

use $\phi = 0.9$

$M_n = A_s f_y (d - a/2)$

$= (4)(60)(22.7 - 4.41/2)$

$= 4919$ in-K

$= 410$ ft-K

$\phi M_n = 0.9(410) = 369$ ft-K $> M_u = 313.2$ ft-K ok ✓

Min Steel Area:

$A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{60000}(16)(22.7) = 1.21 in^2$

$A_s = 4 in^2 > A_{s\,min} = 1.21 in^2$ ok ✓

Check Reinforcement Ratio

$\rho_{max} = 0.85 B_1 \left(\frac{f'_c}{f_y}\right)\left(\frac{\varepsilon_u}{\varepsilon_u + 0.005}\right)$

$= 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.008}\right) =$

$= 0.0181$

$\rho = \frac{A_s}{bd} = \frac{4}{(16)(22.7'')} = 0.011 < \rho_{max}$ OK ✓

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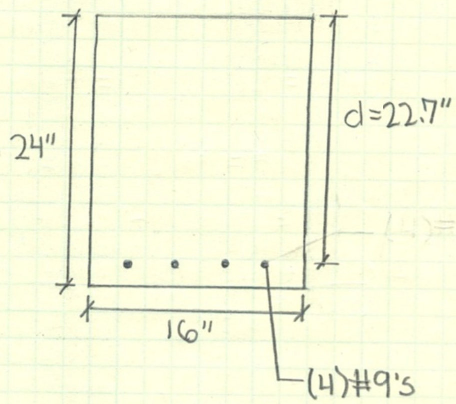

Appendix B: Composite Deck on Steel Beams

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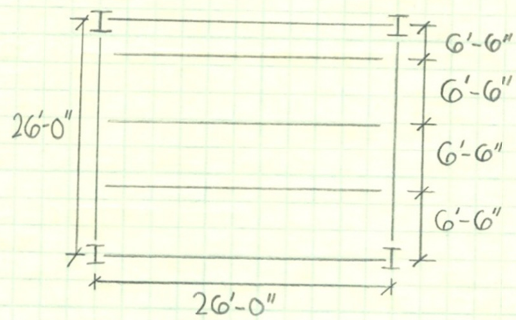
Composite Steel Deck



- Loads

Live Loads = 100 PSF

Superimposed Dead Loads = 15 PSF

Dead Loads = 45PSF (Slab Self Weight)

- Vulcraft Deck Used

Slab Depth = 4.5"

Topping = 2.5"

Normal Weight Concrete (145 PCF)

3 Span Condition

f'c = 4000 PSI

$F_{y,STEEL}$ = 50,000 PSI

Total Load = 100+15+45 = 160 PSF

use: 2VLI20 Deck, 3 Span Condition

Span = 6'-6", 20 Gauge

Superimposed LL Max. Capacity = 275 PSF > 160 PSF

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Composite Steel Deck (Cont.)

- Beam

Factored Load = 1.2D + 1.6L

1.2(15+45) + 1.6(100) = 232 PSF = 0.232 KSF

Trib. Width = 6'-6"

$w_u = 6.5'(0.232) = 1.51$ KLF

$V_u = \frac{wl}{2} = \frac{(1.51)(26)}{2} = 19.6K$

$M_u = \frac{wl^2}{8} = \frac{(1.51)(26)^2}{8} = 127.6$ ft-k $= 1531.2$ in-k

b_{eff} = min of:
- $2\left(\frac{Span}{8}\right) = 2\left[\frac{(26')(12')}{8}\right] = 39''$ ⇒ controls
- $2(1/2)$ Spacing $= 6.5'(12) = 78''$

$b_{eff} = 2(39'') = 78''$

Assume $a = 2''$, $Y_2 = 4.5'' - 2''/2 = 3.5''$

$A_s = \frac{M_u}{\phi F_y(d/2 + t - a/2)} = \frac{127.6(12)}{0.9(50)(12/2 + (4.5'' - 2/2))} = 3.58 in^2$

- Try 12x14: $A_s = 4.16 in^2$ $d = 11.9''$ $t_w = 0.2''$
$t_f = 0.225''$ $I_x = 88.6 in^4$ $S_x = 14.9 in^3$

- Assume concrete is ignored in the rib

$V'c = 0.85 f'c\, b_{eff} = (0.85)(4)(78)(2.5'') = 663$ K

$V's = A_s F_y = (4.16)(50) = 208K$

$V'c > V's$ ⇒ PNA is in the slab

$a = \frac{A_s F_y}{0.85 f'c\, b_{eff}} = \frac{(4.16)(50)}{0.85(4)(78)} = 0.78'' < 2.0''$ OK ✓

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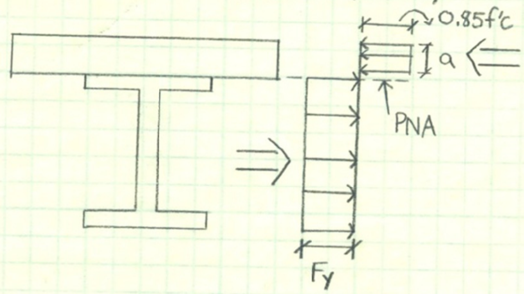
Composite Steel Deck (Cont.)



$\phi M_n = \phi(A_s F_y)[(d/2) + h_r + t - (a/2)]$

$= 0.9(4.16 \times 50)[(11.9/2) + 2 + 2.5 - (0.78/2)] = 1883 \text{ in-K}$

$\phi M_n = 156.9 \text{ ft-K} > M_u = 127.6 \text{ ft-K}$ ok ✓

$\phi V_n = \phi 0.6 F_y A_w = (0.9)(0.6)(50)(11.9)(0.2) = 64.3K$

$\phi V_n = 64.3K > V_u = 19.6K$ ok ✓

$\phi M_p = 65.3 \text{ ft-K}$

$Y_2 = 4.5'' - 0.78''/2 = 4.11'' > 3.5'' \therefore$ conservative

- Assume 3/4" ϕ studs, deck running perpendicular

Weak 1 stud/rib = 17.2

$\Sigma Q_n = \min \begin{cases} V'_c = 663K \\ V'_s = 208K \Rightarrow \text{controls} \end{cases}$

Shear Studs $\Rightarrow \frac{\Sigma Q_n}{Q_n} = \frac{208}{17.2} = 13$ Studs/half = 26 studs

- Check Live Load Deflection

$W_{LL} = (100 \text{ PSF})(6.5') = 650 \text{ PLF} = 0.65 \text{ KLF}$

Using $Y_2 = 4.5'' \therefore$ conservative, $I_{LB} = 316 in^4$

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(0.65)(26')^4(1728)}{384(29000)(316)} = 0.73''$

$l/360 = (26)(12)/360 = 0.87'' > 0.73''$ ok ✓

Check Wet Concrete Deflection:

$W_{wc} = 45\,psf\,(6.5') + 12\,Plf = 304.5\,PLF = 0.305\,KLF$
↳ wt. of Beam

$I_e = 88.6\,in^4$

$\Delta_{wc} = \dfrac{5wl^4}{384EI} = \dfrac{5(.305)(26')(1728)}{384(29000)(88.6)}$

$\Delta_{wc} = 1.22''$

$\Delta_{wc,max} = l/240 = (26)(12)/240 = 1.3'' > 1.22''$ ∴ OK ✓

Check Unshored Strength:

$C_{LL} = 20\,psf\,(6.5') = 130\,Plf = 0.13\,KLF$

$W_{LL} = C_{LL} = 0.13\,KLF$

$W_{DL} = (45\,psf)(6.5) + 14\,plf = 306.5\,Plf = 0.307\,KLF$

$W_u = 1.2w_D + 1.6w_L = 1.2(0.307) + 1.6(0.13) = 0.576\,KLF$

$M_u = \dfrac{W_u l^2}{8} = \dfrac{(0.576)(26')^2}{8} = 48.7\,f\text{-}K$

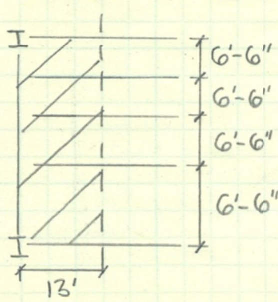
$M_u < \phi M_p = 65.3\,ft\text{-}K$ OK ✓

use W12x14 (5)

Girders:

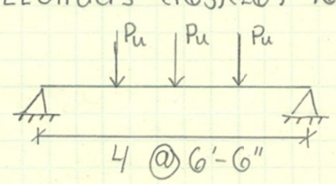
Total Dead Load $= (15+45)(6.5) = 390\,plf = .39\,KLF$

Total Live Load $= (100)(6.5') = 650\,plf = 0.65\,KLF$



DL Girders $= (.39)(26') = 10.14\,K$

LL Girders $= (.65)(26') = 16.9\,K$



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$P_u = 1.2D + 1.6L = 1.2(10.14) + 1.6(16.9) = 39.2\,K$

$V_u = P_u = 39.2K$

$M_u = (39.2)(6.5) = 254.8\,ft\text{-}K$

$b_{eff} = \min \begin{cases} 2\left(\frac{span}{8}\right) = 2\left(\frac{26(12)}{8}\right) = 39'' \Rightarrow \text{controls} \\ 2(1/2)\,\text{spacing} = (13)(12) = 156'' \end{cases}$

$b_{eff} = 78''$

Assume $a = 2''$

$A_s = \dfrac{M_u}{\phi F_y(d/2 + t - a/2)} = \dfrac{254.8(12)}{0.9(50)(14/2 + (4.5'' - 2/2)}$

$= 6.47\,in^2$

Try W14x22, $A_s = 6.49\,in^2$, $d = 13.7''$, $t_w = 0.23''$, $t_f = 0.335''$

$h/t_w = 53.3$, $I_x = 199\,in^4$, $S_x = 29\,in^3$, $Z_x = 33.2\,in^3$

- Assume concrete is ignored in the rib

$V'c = 0.85f'c\,b_{eff} = 0.85(4)(78'')(2.5'') = 663\,K$

$V's = A_sF_y = (6.49)(50) = 324.5\,K$

$V'c > V's$, PNA is in the slab

$a = \dfrac{A_sF_y}{0.85f'c\,b_{eff}} = \dfrac{(6.49)(50)}{0.85(4)(78'')} = 1.22'' < 2.0''$ ok ✓

$\phi M_n = \phi(A_sF_y)[(d/2) + h_r + t - (a/2)]$

$= 0.9(6.49 \times 50)[(13.7/2) + 2 + 2.5 - (1.22/2)]$

$= 3136.6\,in\text{-}K > M_u = 3057.6\,in\text{-}K$ ok ✓

$\phi V_n = \phi 0.6F_y A_{web} = (0.9)(0.6)(50)(13.7)(0.23)$

$= 85K > V_u = 39.2K$ ok ✓

$\phi M_n = 261.3\,ft\text{-}K$, $\phi M_p = 125\,ft\text{-}K$

$Y_2 = 4.5 - 1.22''/2 = 3.89''$, $Y_1 = 0$

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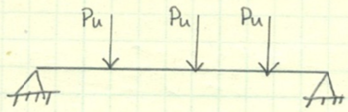
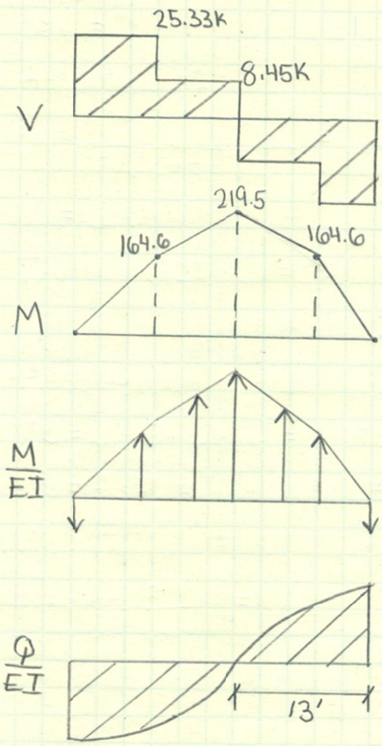
$Q_n = 17.2 \Rightarrow 3/4'' \phi$ studs, deck running ⊥

weak 1 stud/rib

$\Sigma Q_n = 324.5/17.2 = 18.8 = 19$ studs/half $= 38$ studs



$P_{LL} = 16.9K$, $I_{LB} = 583 in^4$



$$\frac{164.6(6.5)}{2} + \frac{(219.5-164.6)(6.5)}{2} + 164.6(6.5) = QA$$

$$\frac{QA}{EI} = \frac{1783.3}{EI}$$

$$\Delta_{LL} = \frac{(1783.3)(13)(2/3)(1728)}{(29000)(583)}$$

$$\Delta_{LL} = 1.58''$$

$$\Delta_{LL,max} = l/360 = \frac{(26)(12)}{360} = 0.86''$$

∴ No Good

$$I_{Req} = \frac{(1783.3)(13)(2/3)(1728)}{(29000)(0.86'')} = 1071 in^4$$

use W16x77 $I_x = 1110 in^4$

$\phi M_p = 563$ ft-K

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Check Unshored Strength:

$w_u = 1.2D + 1.6L = 1.2(0.077) = 0.0924$ KLF

$C_{LL} = 20$ psf $(6.5')(13') = 1.69$K

$C_{DC} = [45(6.5) + 77](13') = 4.8$K

$P_u = 1.2D + 1.6L = 1.2(4.8) + 1.6(1.69) = 8.47$K

$V_u = P_u = 8.47$k

$$M_u = \frac{w_u \ell^2}{8} + \frac{P_u \ell}{4} = \frac{(.0924)(26)^2}{8} + \frac{(8.47)(26)}{4}$$

$= 62.8$ ft-K

$\phi M_P = 563$ ft-K > 62.8 ft-K ok ✓

Appendix C: One-way Slab with Beams

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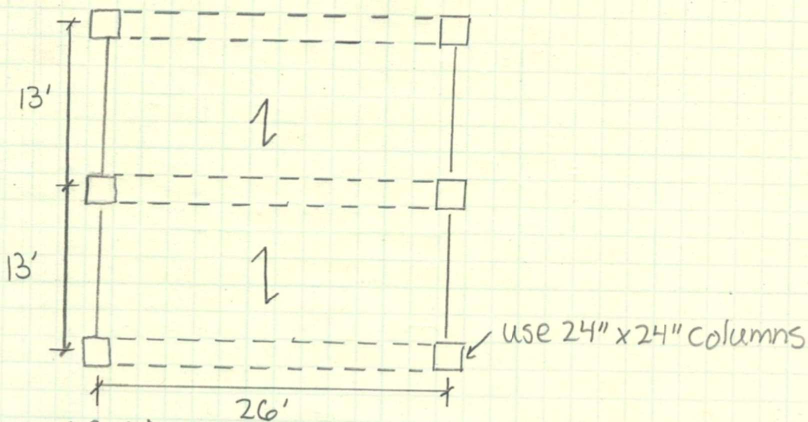
Proposed One-Way Slab with Beams



TB 9.5(a)

$h_{min} = \ell/28 = (13')(12)/28 = 5.57'' = 6''$

Try 6" slab > 5" ok for 2hr Fire Rating

LL = 100 psf

SDL = 15 psf

S.W. slab = (150 pcf)(6"/12) = 75 psf

Assume #4 bars

$d = h - cc - d_b/2 = 6'' - 3/4'' - 0.5/2 = 5''$

Beam Design:

DL = 15 psf + 75 psf = 90 psf

LL = 100 psf

$w_u = 1.2D + 1.6L$

$= 1.2(90) + 1.6(100)$

$= 268 \text{ psf} \times 13'$

$= 3.48$ KLF

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$M_u = \frac{w_u l_n^2}{8} = \frac{(3.48)(26' - 24/12)^2}{8} \times 1.1 = 276\text{K-ft}$

↳ Estimated BM S.W.

- Estimate Size

$bd^2 = 20M_u$

Let $b = 16''$

$(16)d^2 = 20(276)$

$d = 18.57 = 19''$

$h = d + 2.5 = 19 + 2.5 = 21.5''$

use $h = 22''$ and $b = 16''$

$bd^2 = (16)(19'')^2 = 5776 in^3$

- Self weight

$W_{sw} = \frac{(16)(22)(150\text{ pcf})}{144} = 367\text{ plf}$

$W_u = 3.48\text{ KLF} + 1.2(.367) = 3.92\text{ KLF}$

$M_u = \frac{(3.92)(24')^2}{8} = 282.2\text{k-ft}$

$20 \times 282.2 = 5644 in^3 < 5776 in^3$ ok ✓

- Required Steel

$A_s = \frac{M_u}{4d} = \frac{282.2}{4(19)} = 3.71 in^2$

Try (4) #9's = $4 in^2$

Nominal Moment

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(4)(60)}{.85(4)(16)} = 4.41$

$c = a/\beta_1 = 4.41/0.85 = 5.19$

$d = 22'' - .75'' - 1.128/2 = 20.7''$

$\varepsilon_s = \varepsilon_u\left(\frac{d-c}{c}\right) = .003\left(\frac{20.7 - 5.19}{5.19}\right) = 0.0089 > .005$ ok ✓

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use $\phi = 0.9$

$M_n = A_s f_y (d - a/2)$

$= (4)(60)(20.7 - 4.41/2)$

$= 4439$ k-in

$\doteq 370$ k-ft

$\phi M_n = 0.9(370) = 333$ ft-k $> M_u = 282.2$ ft-k ok ✓

Min Steel Area:

$A_{s\,min} = \frac{200}{f_y} bd = \frac{200}{60000}(16)(20.7) = 1.1\,in^2$

$A_s = 4\,in^2 > A_{s\,min} = 1.1\,in^2$ ok ✓

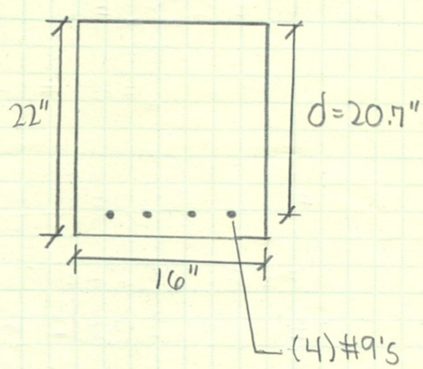
check max Reinf. Ratio:

$\rho_{max} = 0.85\beta_1 \left(\frac{f'_c}{f_y}\right)\left(\frac{\varepsilon_u}{\varepsilon_u + .005}\right)$

$= 0.85(0.85)\left(\frac{4}{60}\right)\left(\frac{.003}{.008}\right)$

$= 0.0181$

$\rho = \frac{A_s}{bd} = \frac{4}{(16)(20.7)} = 0.012 < \rho_{max}$ ok ✓



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Slab Design

$t = 6.0''$

Assume #4 bars

$A_{s min} = .002bh$

For a 1' Section

$A_{s min} = .002(12'')(6'') = 0.144 in^2/ft$ of Slab

Try #4 bars spaced @ 10" o.c. (Table A-4)

$A_s = 0.24 in^2$ per foot

$d = 6'' - d_b/2 = 6'' - 1'' = 5''$

$\rho = \frac{A_s}{bd} = \frac{0.24}{(12)(5)} = .004 \Rightarrow \phi = 0.9$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(.24)(60)}{0.85(4)(12)} = 0.35$

$\phi M_n = \phi A_s f_y (d - a/2)$

$= 0.9(24)(60)(5 - .35/2)$

$= 62.5$ k-in

$= 5.21$ k-ft

$W_u = 1.2D + 1.6L$

$= 1.2(15+75) + 1.6(100)$

$= 268$ KLF / 1ft Section

$M_u = \frac{W_u l^2}{8} = \frac{(.268)(13')^2}{8} = 5.66$ k-ft

$\phi M_n < M_u \Rightarrow$ NG

Try #4's @ 8" o.c., $A_s = 0.3 in^2/ft$ of Slab

$d = 6'' - 3/4'' - .25'' = 5''$

$\rho = \frac{A_s}{bd} = \frac{0.3}{(12)(5')} = 0.005 \Rightarrow \phi = 0.9$

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$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.3)(60)}{0.85(4)(12)} = 0.441$

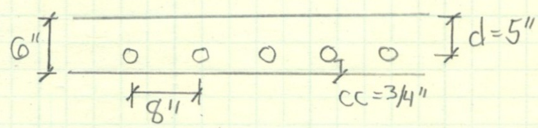
$\phi M_n = \phi A_s f_y (d - a/2)$

$= 0.9(0.3)(60)(5 - .441/2)$

$= 77.4 \text{ k-in}$

$= 6.45 \text{ k-ft} > M_u$ ok ✓

∴ use a 6" slab w/ #4 bars @ 8" o.c.



Appendix D: Hollow-Core Planks on Beams

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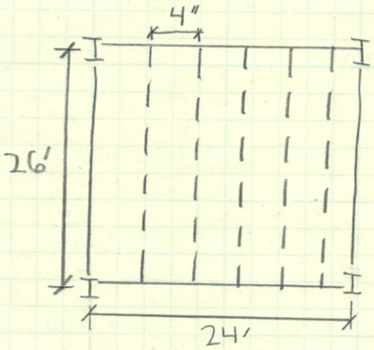
Proposed Hollow-core Plank on Beams:

*Ref PCI handbook



Try 4'x8" planks → no topping

DL = 56 Psf (↳ S.W. of plank) + 15 psf (↳ SD)

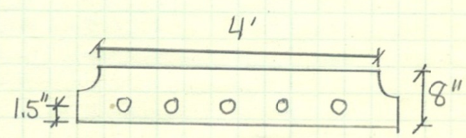
LL = 100 Psf

TL = (56+15)+100 = 171 psf

- for a span of 26', 68-S strand is selected

180 psf > 171 psf OK ✓

use 4'x8" (4HC8) @ 26' span w/ 68-S strand hollow-core plank



- Beam Design

$W_u = 1.2(71) + 1.6(100)$

$= 245.2 \text{ psf} = .245 \text{ ksf}$

$M_u = \frac{(.245.2)(24')(26')^2}{8} = 497.3 \text{ ft-k}$

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Try W21x62

$\phi M_n = 540\text{k-ft} > 497\text{ft-k}$

-check deflection

$\Delta_{LL} = l/360 = \frac{24'(12)}{360} = 0.8''$, $\Delta_{TL} = \frac{24(12)}{240} = 1.2''$

$1.2 = \frac{5wl^4}{384EI}$, $1.2 = \frac{5(56+15+100)(26')(24')^4(1728)}{384(29{,}000{,}000)I_x}$

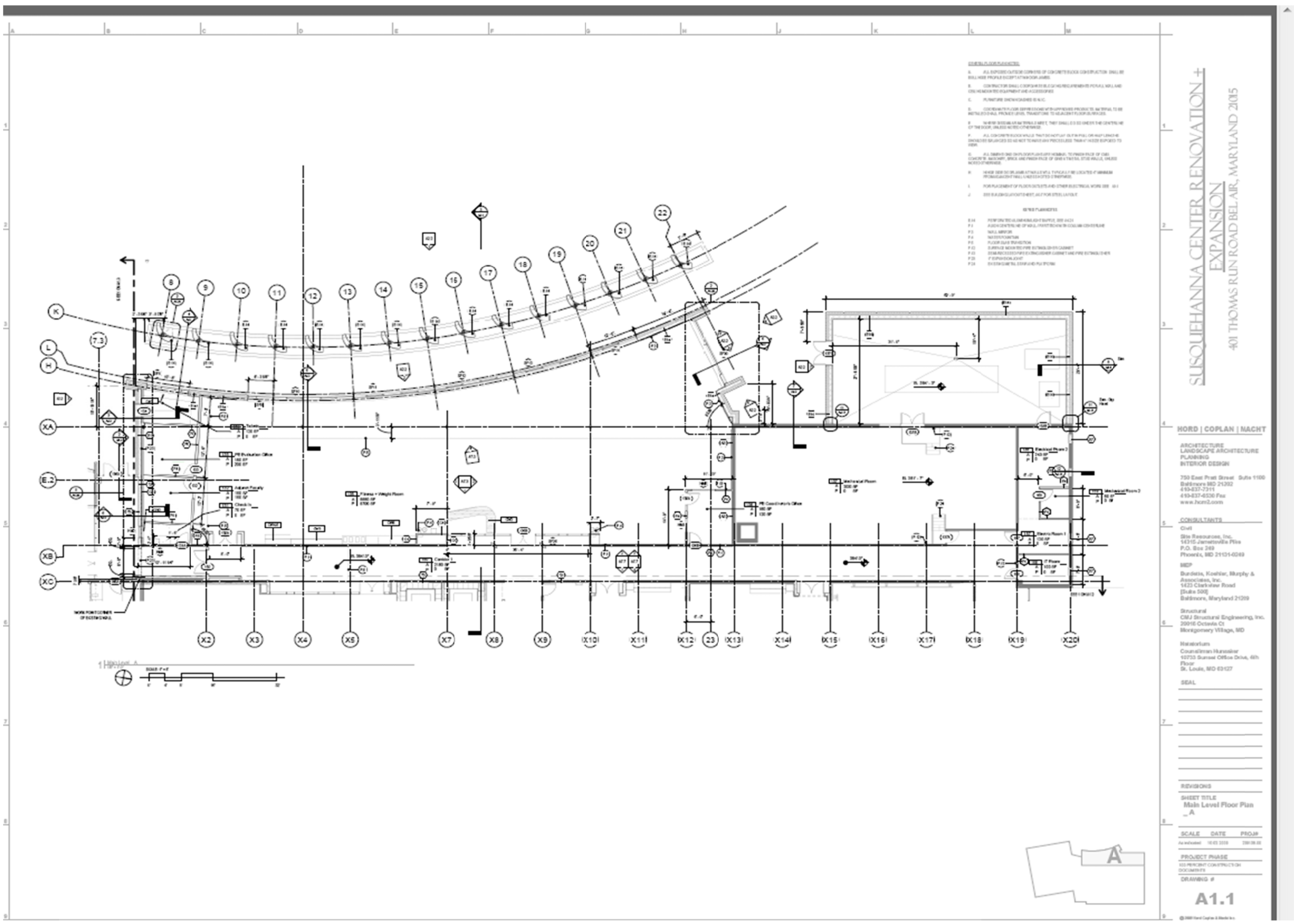
$I_x = 954\text{in}^4 < 1330\text{in}^4$ ok ✓

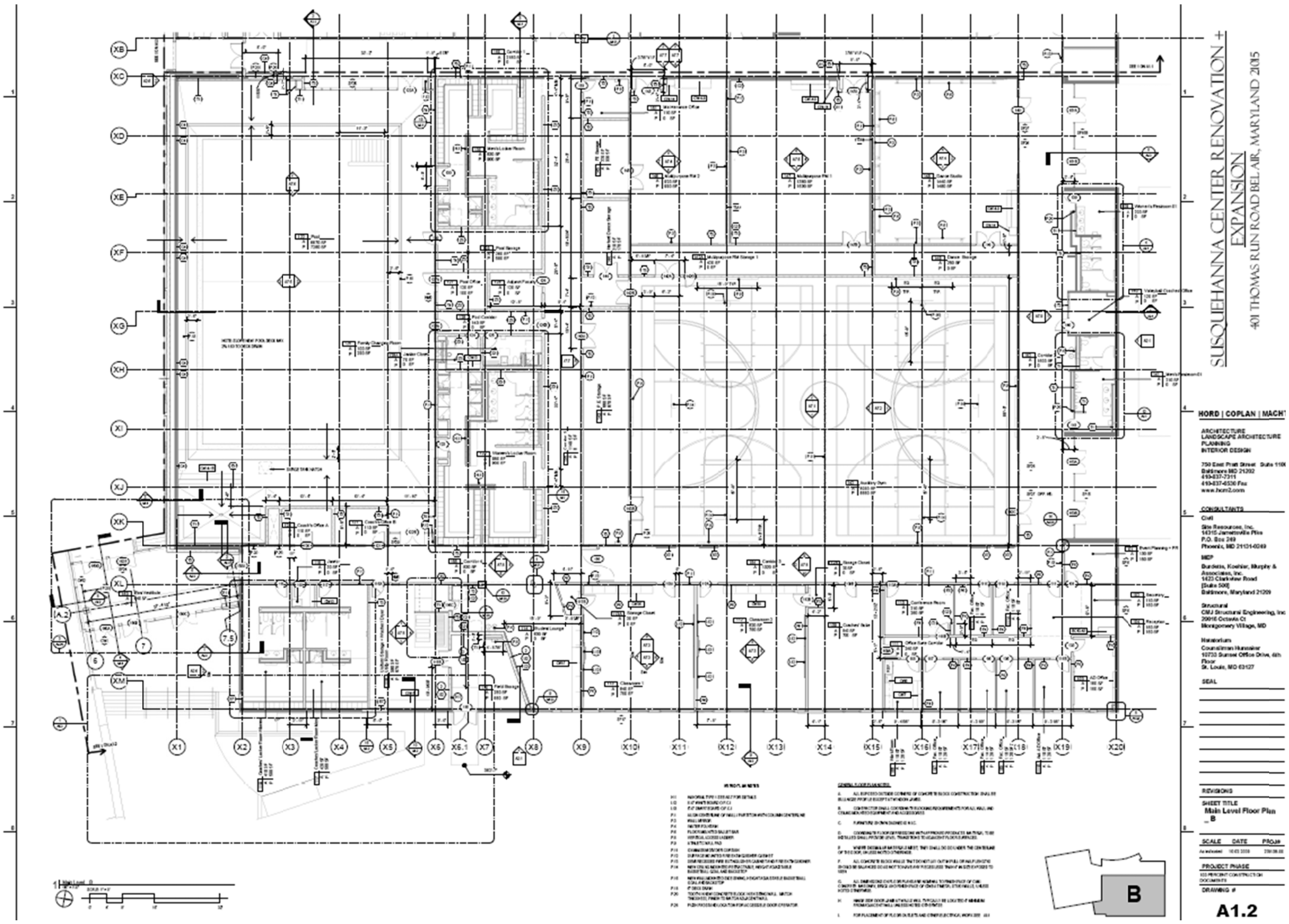
$\Delta_{LL} = \frac{5(100)(26')(24')^4(1728)}{384(29{,}000{,}000)(1330)} = 0.503'' < 0.8''$ ok ✓

$\Delta_{TL} = \frac{5(4508)(24')^4(1728)}{384(29{,}000{,}000)(1330)} = 0.872'' < 1.2''$ ok ✓

Use W21x62

Appendix E: Floor Plans

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LANDSCAPE ARCHITECTURE
PLANNING
INTERIOR DESIGN
CONSULTANTS
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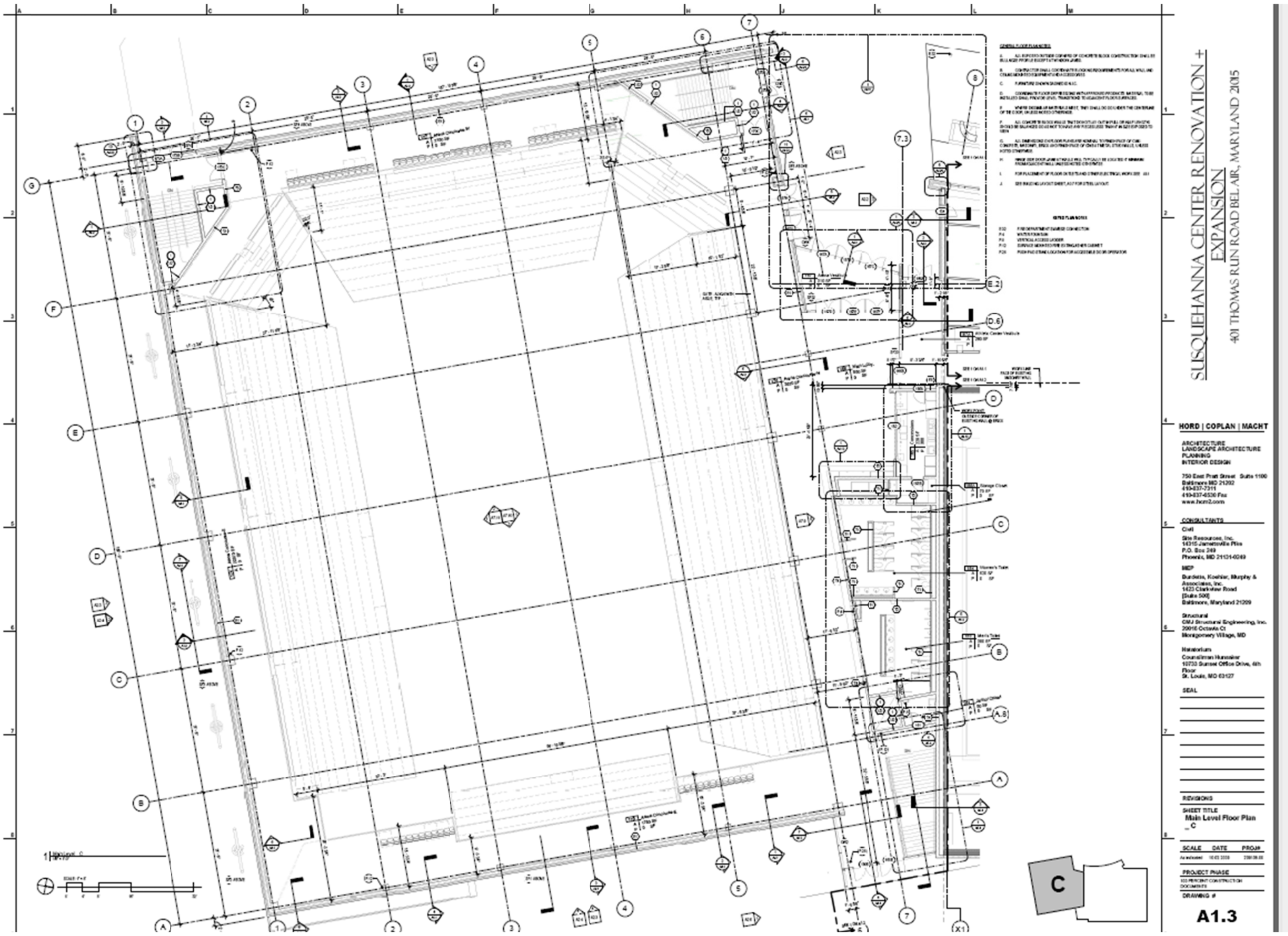
MEP
Burdette, Koehler, Murphy & Associates, Inc.
Structural
CMJ Structural Engineering, Inc
Natatorium
Counsilman Hunsaker
SEAL
REVISIONS
SHEET TITLE
Main Level Floor Plan _B
SCALE
DATE
PROJECT PHASE
DRAWING #
A1.2
B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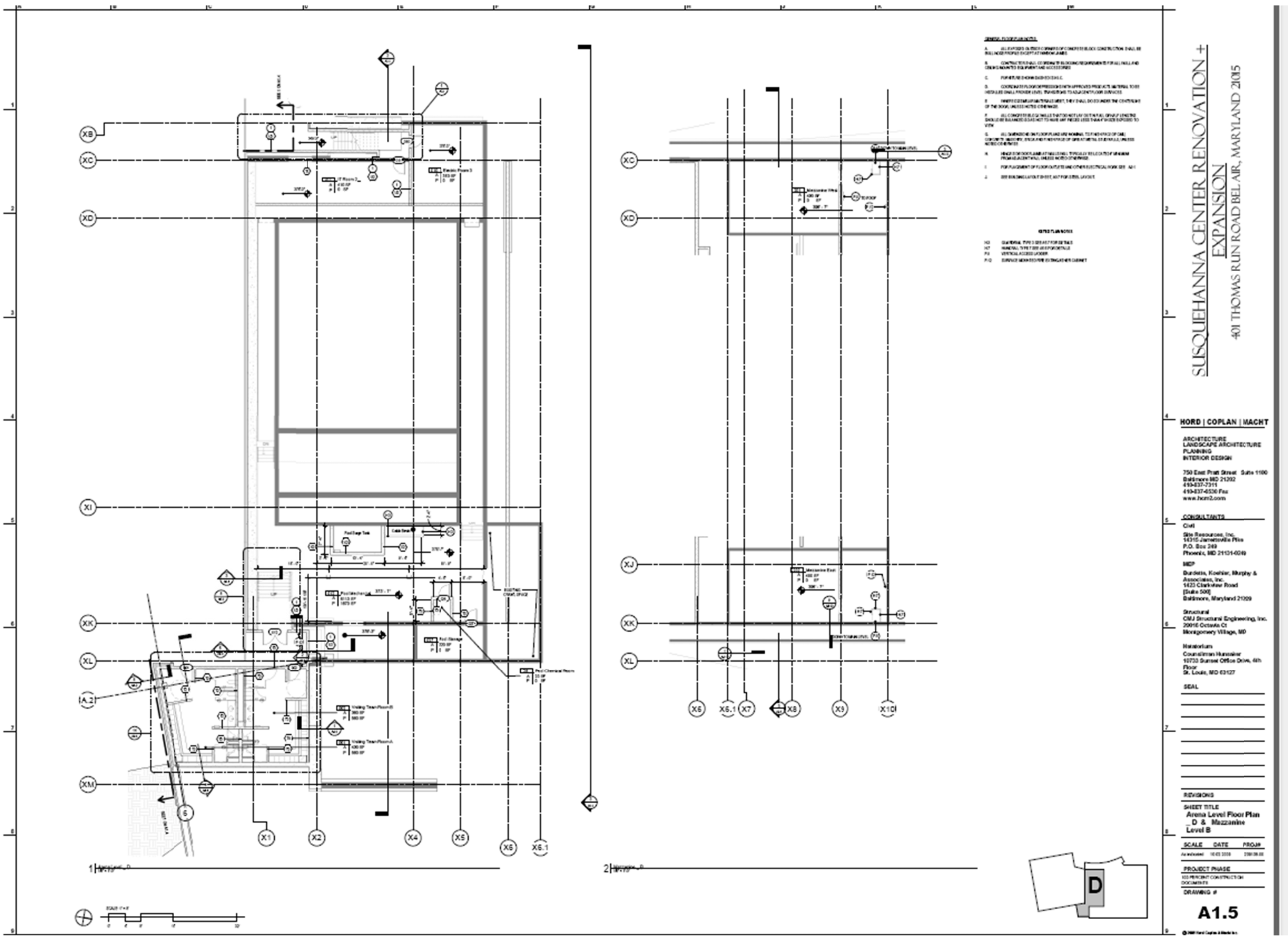
INTERIOR DESIGN
Main Level Floor Plan _ C
C
A1.3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SHEET TITLE
Arena Level Floor Plan _D & Mazzanine Level B
SCALE
DATE
PROJ#
PROJECT PHASE
DRAWING #
A1.5
D