Handley McDonald

Technical Report 2

Claude Moore Medical Education Building

Faculty Advisor: Richard Behr

Handley McDonald

TABLE OF CONTENTS

EXECUTIVE SUMMARY

This technical report contains the results of three alternate floor systems, as compared to the existing, composite beam structure. From a wide array of possibilities, three distinct systems were chosen as potential alternatives:

- Similar deck structure, without composite beam action
- One way concrete slab
- Hollow core concrete planks on steel beams

These three systems were compared to the existing system, and to each other, by many parameters. The highest priority of these parameters was thickness of the structure. As the structural system must be as small as possible for this building, this was the most important factor. Beyond that, cost, scheduling, and necessary fireproofing were considered as they affect the construction management team. Several design factors were compared as well, including weight, deflection, fire rating, and thickness. Vibration was considered, however this particular occupancy does not require any stringent vibration guidelines, so it was only a minor concern.

The non composite deck was the main concern, and was necessary to see why composite systems were chosen over non composite. The composite system provided a large decrease in both deflection, and cost per square foot, and is the obvious choice of the two.

When it came time to analyze the concrete system, it was clear that this would not be a viable option for this structure. It is simple too large, with a beam depth of 24 inches, to properly serve this building. Beyond that, it added far too much weight on a foundation that has little room to expand.

The hollow core concrete planks were an interesting alternative. Certainly viable, however much more expensive than the cast in place decking options considered above. The inconsistency of bay size would also make this system relatively difficult to achieve, but it is certainly light enough to fit the size requirements for the building.

BUILDING INFORMATION

Claude Moore Medical Education Building

58,000 sq. ft.

Type B and A-3 mixed occupancy

6 total levels, 4 above grade

OWNER	University Of Virginia | 575 Alderman Rd Charlottesville, VA
ARCHITECT	CO Architects | 5055 Wilshire Blvd Los Angeles, CA
ASSOCIATE ARCH	Train and Partners Architects | 1218 E Market Street Charlottesville, VA
BUILDER	Barton Malow Construction | 100 Tenth Street NE #100 Charlottesville, VA
STRUCTURAL ENG	Nolen Frisa Associates | 103 Homestead Dr Forest, VA
M.E.P. ENG	Bard, Rao& Thomas | 311 Arsenal St Watertown, MA
CIVIL ENG	RMF Engineering | 217 5th St, N.E. #2 Charlottesville, VA
LANDSCAPE ARCH	Dirtworks, PC | 200 Park Avenue South New York, NY
GEOTECH ENG	Schnabel Engineering South | 2020 Avon Court, #15 Charlottesville, VA
AUDIOVISUAL	The Sextant Group | 730 River Avenue #600 Pittsburgh, PA

The Claude Moore Medical Education Building was constructed on the University of Virginia's Health System campus, where they are centralizing all of their medical facilities, both educational and practical. Completed in August of 2010, just in time for classes, the new building was to represent a huge leap forward in medical technologies, and demonstrate the new, hands on teaching facilities of the University.

The third floor Lecture hall can seat 117 students, and provides a traditional learning environment.

This new style of teaching the medical students is represented best in the Learning Center, a large, round room meant to encourage group oriented learning, as opposed to the traditional lecture hall classrooms. Below this learning center, are state of the art mock medical facilities, to provide hands on training in a controlled environment, and with trained "patients." In addition, it will also include a traditional lecture hall, administrative offices, and student lounge.

The Learning Center provides a high-tech and group oriented learning space, where students can collaborate with the teacher, as well as each other.

Exceeding the University's environmental building policy, the Claude Moore building received a LEED silver certification due to a number of environmentally friendly systems. These systems include efficient HVAC equipment, a cool roof design, and several water reduction strategies that help to reduce the amount of runoff from the building.

The entire project cost $40 million, and greatly adds to the effort of condensing the medical facilities of the University.

STRUCTURAL SYSTEM OVERVIEW

The Claude Moore Medical Education Building is a four level, composite deck system, composed of steel beams, columns, and a concrete slab on metal floor decking. This system rests on a foundation of drilled concrete piers that continue about 25'below gradeand into the bedrock. In several aspects of the design, the large circular section of the building that contains the lecture hall and Learning Center, are distinguished from the typical structural design, and is referred to as the "drum."

FOUNDATION

The foundation for the Medical Education Building is mainly made up of drilled piers. These piers are made of 4000 psi, normal weight concrete, and go 2' into the bedrock underneath the site. This decision was made based on the geotechnical report done by Schnabel Engineering South in 2006. Because of the large column loads, and limited space between this site and the adjacent buildings, a deep foundation had to be used.

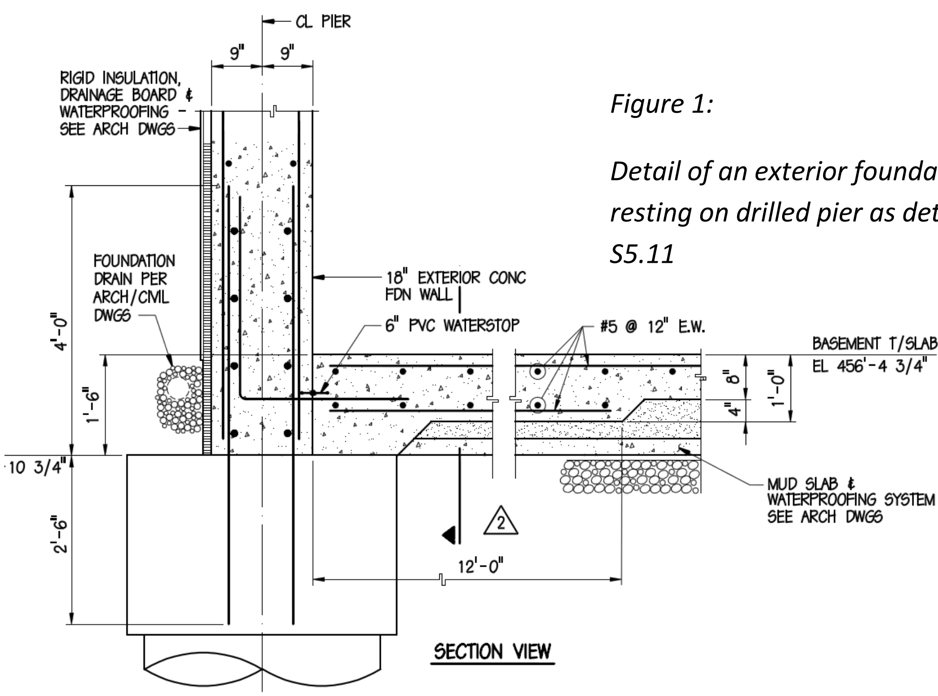


Figure 1:

Detail of an exterior foundation wall resting on drilled pier as detailed in S5.11

The basement level foundation walls are made of 18" thick cast in place concrete, reinforced with both vertical and horizontal reinforcement. These walls rest on the same centerline as the drilled piers below and connect to a 12" thick slab on grade system that includes a mud slab, and waterproofing.

FLOOR SYSTEM

The ground level is made up of an 8" thick concrete slab on grade, with reinforcing in both directions. Below this slab is a mud slab and a waterproofing system, to help stabilize and protect the slab. On each of the floors above, there is a composite metal deck with lightweight concrete, laid in thicknesses of 4.5" and 5.5" (including deck thickness). All metal decking was used in conjunction with composite steel beams, and welded shear studs. Allends were built with a minimum of 1.5" overlay, and end joints lapped at least 2". The beam and girder system here is relatively light, with most wide flanges ranging from 18" to 24" deep, and 10 to 40 pounds per linear foot. Due to the minimal amount of space, and difficulty of the structural system, there is not really any typical bay type; however the rectangular layout fits into the drum section with minimal interruption.

Figure 2: Installation of lecture hall structure

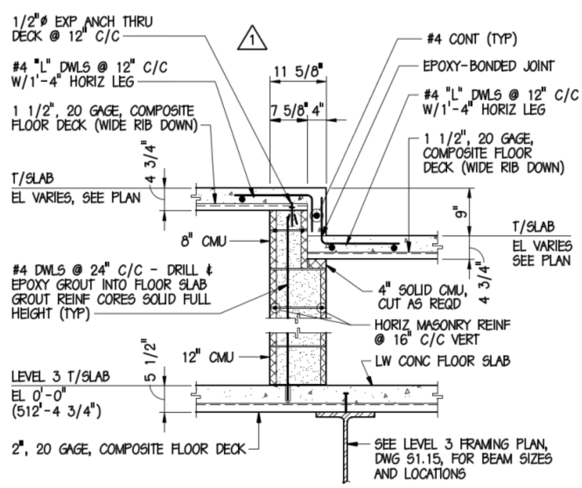


Figure 3: Detail of lecture hall floors, as noted in S5.22

For the lecture hall, 8" grout filled CMU was used to support the stepped composite floor deck. This slab is a 4.75" thick slab, and the circular CMU walls rest on a 5.5" composite floor deck. This part of the building has a much larger substructure of wide flanges, most of which are greater than 150 pounds per linear foot. There is no typical bay type for this section of the floor structure either.

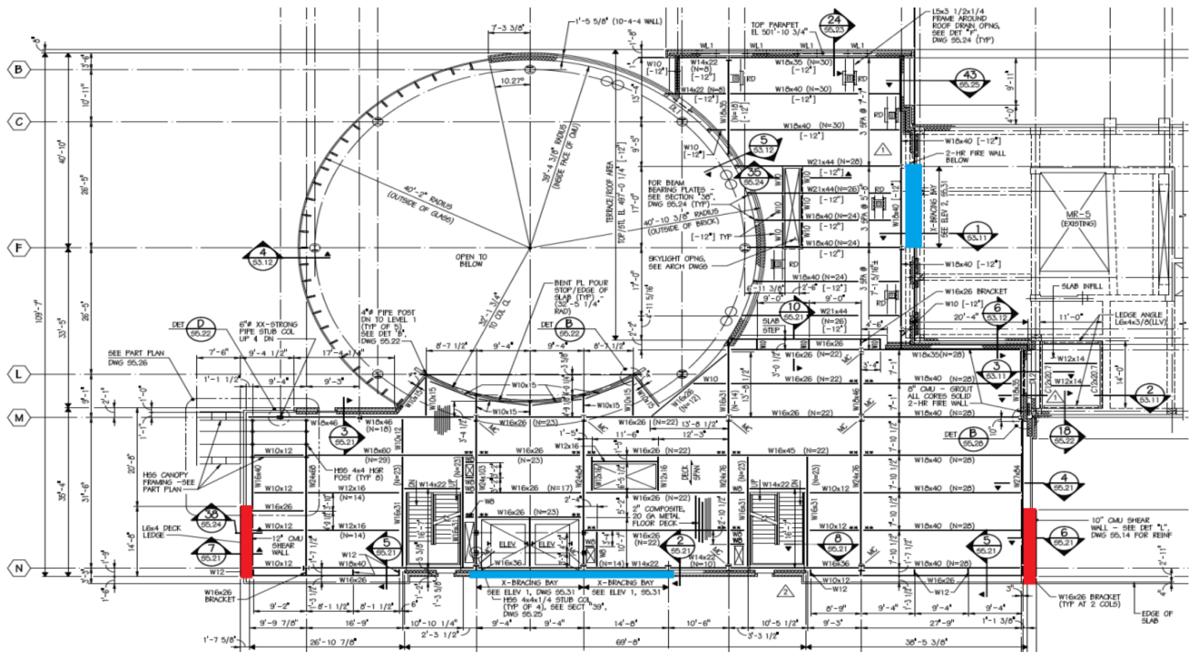
FRAMING SYSTEM

All of the framing for the Claude Moore Building was done with steel wide flanges. The beams, as previously mentioned, unfortunately do not follow much of a typical plan for size or spacing, but one should note that very minimal deviations were made as far as fitting the structure of the drum area into the rectangular structure of the rest of the building. A larger picture for reference is located in Appendix D. The columns are mostly 12" deep wide flanges; however the weights and spacings vary greatly within that. Because of the irregularity in the framing system, several transfer girders were necessary to allow for the change in structure from floor to floor. Most of these transfers happen below the first floor, and allow for the load to move from the main structure to the structure below grade.

LATERAL RESISTING SYSTEM

The lateral resisting system for this project is mostly made up of moment frames. Originally, the intent was to use only moment frames, with limited X-bracing to react with the curtain wall system. Changes were made, however, when the owner and architect modified the design, and limited the space enough that other options had to be considered. As a result, the system is a hybrid of moment frames, X-bracing, and shear walls.

The bays that include X-bracing are shown below. The east wall braces are made of HSS 4x4x3/16 sections, and the south wall employs several different sizes, but they are all HSS sections as well. The loads applied to these systems are transferred to the cast in place concrete foundation wall below, using a bolted base plate connection. In addition to these braced frames, two 14' long 12" CMU shear walls (red) were added at the plan southwest and southeast corners of the building. These walls help for shear in the north-south direction, and transfer their loads directly to the basement foundation below.The moment frame lies along column lines J and M, and is connected using welded and bolted angle plates of varying sizes to resist the moment.



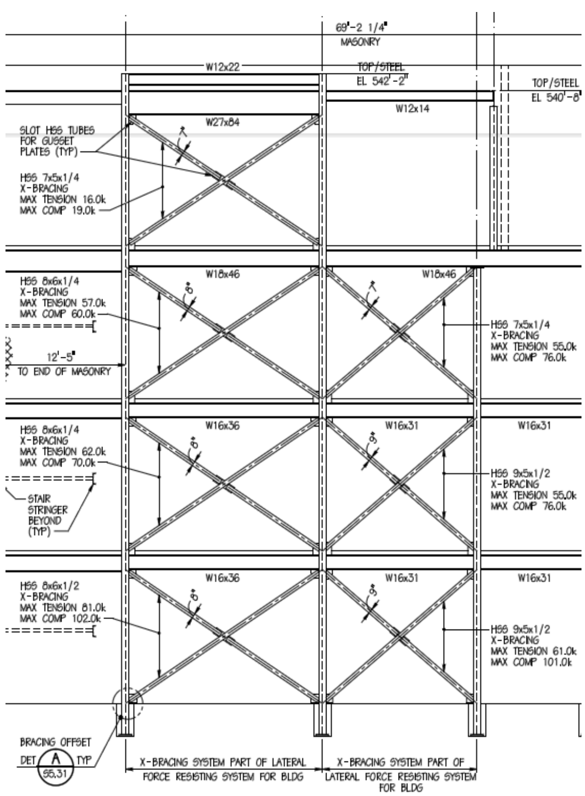


Figure 4 (above): Framing plan including highlights of non moment-frame lateral resisting elements. Detailed in S1.14.

Figure 5 (left): Elevation of X-bracing between column lines 3 and 5.9 as detailed in S5.31.

DESIGN CODES

According to sheets S0.11 and A0.02, the following major code regulations were applied to this project:

- IBC 2003 with VA amendments (Virginia Uniform Statewide Building Code)
- IFC 2003 with VA amendments (Virginia Statewide Fire Prevention Code)
- IMC 2003 International Mechanical Code
- IPC 2001 International Plumbing Code
- ANSI/ASME A17.1 Safety Code for Elevators and Escalators
- Local ordinances and amendments to all of the above codes
- ACI 318-02 Structural Concrete Building Code
- AISC Manual of Steel Construction, 9th edition
- ASCE 5-02, 6-02 Code Requirements and Specifications for Masonry Structures
- ASCE 7-02 Minimum Design Loads for Buildings

These code standards vary from the ones used in this report, and from the ones that will be used in future reports. These differences will result in variations between the report results, and the results used in the building design.

MATERIALS USED

The following is a breakdown of the structural materials used throughout the building as taken from S0.11

STEEL		
Use	**Class**	**Strength**
W Sections	ASTM A992 GR 50	50000 psi
Channels, Angles, & Plates	ASTM A36	36000 psi
Hollow Structural Sections	ASTM A500 GR B	46000 psi
Steel Pipe Section	ASTM A53 GR B Type E or S	35000 psi
Structural Bolts	ASTM A325 and A490	n/s
Welding Electrodes	--	E70xx
Anchor Bolts	ASTM F1554 GR 36	36000 psi
Headed Shear Studs for Composite Beams	ASTM A108	60000 psi Designed for 11.4k per stud

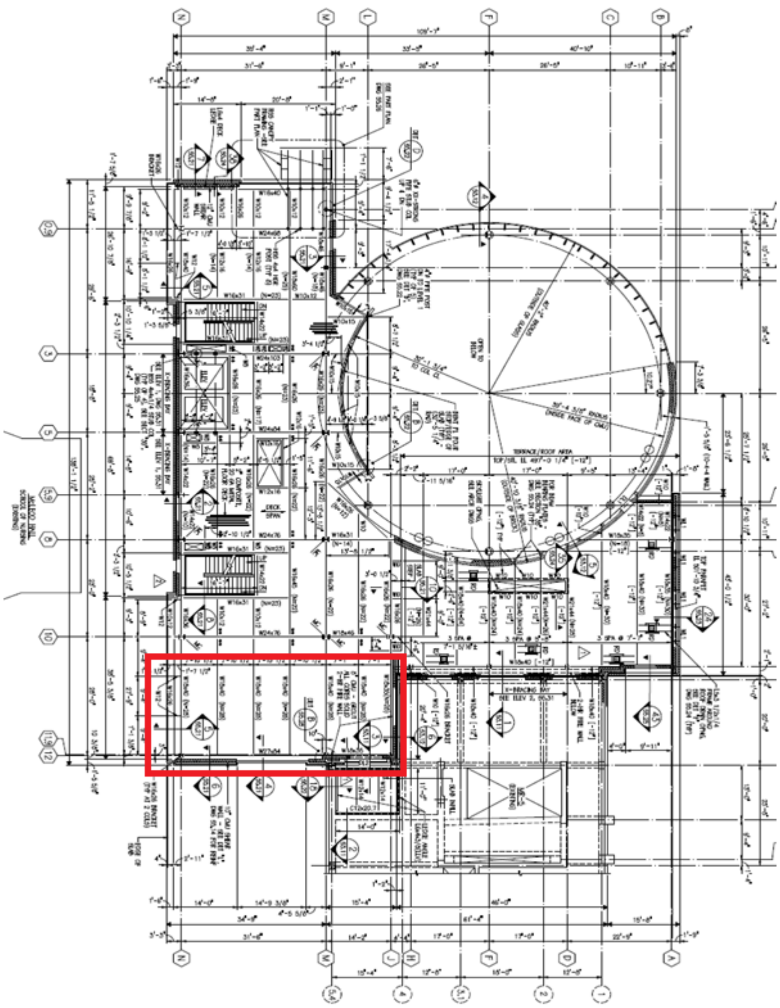
CONCRETE		
Use	**Class**	**Strength**
Slab on grade, cast in place walls & foundations	Normal Weight (Assume 150 lb/ft^{3})	4000 psi
Elevated Floor Slabs	Light Weight (Assume 100 lb/ft^{3})	4000 psi
Reinforcing Steel	ASTM A615 GR 60	Fy=60000 psi
Welded Wire Fabric	ASTM A185	Fy=60000 psi

MASONRY		
Use	**Class**	**Strength**
Lightweight CMU	ASTM C90 GR N-1	f'm=1500 psi
Mortar for CMU	ASTM C270 Type S	f'c=1800 psi
Structural Grout	ASTM C476	f'c=2500 psi
Vertical Reinforcement	ASTM A615 GR 60	fy=60000 psi
Horizontal Joint Reinforcement	ASTM A82 w/ galvanizing per ASTM A 153 class B-2	n/s

	SOILS
Use	**Strength**
Bearing Capacity	3000 psf standard bearing case
Bedrock Bearing	50 ksf for drilled piers
Disintegrated Rock Bearing	25 ksf for drilled piers
Side Friction	2 ksf for elevation below 450' above sea level

LOAD CONSIDERATIONS

This report only involves gravity load calculations, as it is a simple floor analysis. Lateral loads will be considered in Technical Report 3. All calculations were based on the single bay shown in the picture below, as this bay was relatively large in size, and offered a decently typical spacing. The dead loads considered involved the self weight of the given structure, plus a 30psf superimposed dead load to account for any finished and permanent fixtures. A live load was taken at 100psf as specified in drawing S0.11.



FOOR SYSTEM ANALYSIS

This section of the report consists of a comparison between the current floor design, and three proposed alternatives for floor construction. The first is the existing metal deck on composite steel beam system, and is done more as an exercise in checking the strength, and providing a fixed idea to compare to. The second is a similar construction; however the deck is non composite in this case. The third design is a concrete one way slab construction with infill beams, and the final design was done using hollow concrete planks. Initially, a two way flat plate concrete slab system was considered for the third alternative, but ultimately discarded due to the fact that there were already space concerns with the current steel frame, and a flat plate would have only made those issues worse.

The different systems were given a rough design sketch, and then compared on bases of several factors, the results of which are tabulated below. While there is no "typical" bay size, the analysis was done over one bay in the plan southeast corner of the building, on the second floor, where offices are located. This bay provided a decent size span, and a simple layout that allowed for simple calculations.

EXISTING SYSTEM: COMPOSITE BEAM STRUCTURE

As mentioned previously, the existing floor structure is made of 4.5" thick decking with composite beams. As the SDI catalog was unavailable, the Vulcraft decking manual was used, with a 2VLI20 deck representing the existing deck. The properties and dimensions were both similar enough to be used. This deck is filled with 4000psi lightweight concrete, and bolted to the beams below with 3/4" headed shear studs, 28 per beam.

The analysis for this structure was very similar to the spot check done in the first technical report. The only difference here is that several beams in the same bay were considered, and the supporting girders were checked as well. The deck proved to be adequate to not only handle the load, but handle the construction load without shoring as well. The span of 7-10.5" was well within the 11'7" tolerance for unshored clear span. Both the flexural and deflection criteria of this system appear to be grossly overdesigned, possibly due to a factor that was not taken into account for this rough exercise. The beams and girders were also checked for flexural strength and deflection, and had no issues as well. A composite W18X40 framing into a W27X84 brings a flexural capacity of 409 foot-kips and 915 foot-kips, respectively. A sample model of this bay and deflection analysis results are shown in Appendix A.

ADVANTAGES

The composite beam structure is great for this type of building, due to its high efficiency. With little room to spare, the two materials work together to achieve one of the lightest structures possible.

As for the cost, a large reduction is included when the need for shoring disappears. With a relatively small bay width, they were able to accomplish this, and save a lot of money, spending only $22.75 per square foot. The time it takes to build the floors is also relatively quick since the framework can go up quickly, and the deck can be laid and poured almost directly after the steel is set.

As lightweight concrete was used, the weight of the floors is greatly reduced, which allowed for smaller support beams, and more room for the building occupants. Since this was an issue, it seems if concrete is going to be used, it must be lightweight concrete.

DISADVANTAGES

The first disadvantage is the fact that, with shear studs, comes welding, and with that, specialized workers and inspection. These will add cost and time for construction, but it was worth it compared to costs associated with other types of flooring. Additional fireproofing was also needed to reach a 2 hour fire rating.

Alternative: Non composite deck

A comparison to a non composite metal deck was done mainly to see what the difference in floor thickness would be, if the floor were not designed to resist flexural load. Such an advantage is quite useful, and is the preferred method in today's building industry, but it is a helpful exercise to note the advantages and disadvantages to both.

Since it is so similar to the existing structure, the current bay sizes were used, and calculations were made treating the deck as pure weight, with no structural significance beyond its ability to span two beams. The deck, beams, and supporting girders were checked for strength much like the existing structure, and the design resulted in a deck system with very similar dimensions to the composite structure. A 2" thick deck with 2.5" thick lightweight concrete seemed to work quite well for both. The beams below showed a minor difference in weight, but the depth was exactly the same. The girder, however, showed a drastic reduction in depth and weight, which can only be attributed to an unforeseen factor that was overlooked. Even with the smaller design, the girder showed a flexural capacity of 496 foot kips, and deflected less than an inch under the load.

ADVANTAGES

Many of the properties of this floor are similar to the properties of the existing system, however the fact that this is not a composite deck, means that installation will be less stringent, and allow for a slight bit faster construction time.

DISADVANTAGES

This deck will have slightly larger deflections than the existing system. While it is not enough to be a problem, less deflection is always better, especially when there is no adjustment in the thickness of the floor. As far as cost is concerned, this floor will cost $24.18 per square foot, compared to $22.75, and when estimated over the 58,000 sq ft of the building, adds almost $83,000 to the total cost of the project.

The vibration issue will be just as prevalent with this floor, however in this occupancy, it is not a serious problem. For comparison purposes, this parameter will generally be ignored. The fireproofing, however, is an issue, and will require just the same additional spray fireproofing as the existing composite system.

ALTERNATE: ONE WAY SLAB

A one way concrete slab system was considered to possibly bring an entirely concrete system into the equation. As concrete usually brings more headache with construction, it is much cheaper, by weight, than steel, and in some areas it is used quite widely. The main issue, however, was space. Since the spatial requirements were so stringent, it would be difficult to make a full concrete system work with this building.

The same bay was considered, and after a rough design sketch, it turns out that the concrete system would, in fact, be too large to accommodate the space. A 24" infill beam is simply too much, especially when compared to the current 18" deep steel beam.

ADVANTAGES

Concrete systems will be far cheaper than steel systems, with this particular case costing just $20.10 per square foot of area. This is by a fair margin, the cheapest flooring system analyzed in this study.

The thicker, denser concrete system provides a large reduction in vibration, even considering the minimal importance of this factor. The dense concrete also allows for zero additional fireproofing, which can speed up construction time, and save money.

DISADVANTAGES

There are many disadvantages to possibly using an entirely concrete system. The first of which is that the structure itself will be too large. This project was revised several times, each time requiring a thinner structure, and the massive concrete structure in this design will simply not suffice. It would require a 6" thicker floor structure than the steel system, and is not very forgiving when it comes to mechanical punch through.

The next major disadvantage is the sheer weight of the structure. As the building foundation is made of drilled piers, for space considerations, it is illogical to consider expanding the foundation to carry the load of the concrete. Just in this one bay, for one beam, it would be about 150 pounds per linear foot heavier, even with the lightweight concrete.

Finally, the added labor of formwork and shoring adds countless hours to this project. Where steel beams can be hoisted and bolted rather quickly, concrete requires far more labor, and a waiting period that halts construction in certain parts of the site.

ALTERNATE: HOLLOW CONCRETE PLANKS

Hollow concrete planks were considered as an alternative to the concrete structure, since a two way flat plate system would surely break the space requirements. Concrete planks are not used very often in current construction, due to the degree of specialty required for the installer, compared to a slab, however it did offer an interesting alternative.

Proprietary information was sought online, and found at Oldcastle Precast Building Systems, who offered several quick charts to check against the strength of their various products. A copy of this table is given in Appendix D. A 6" thickness with no topping was well within the ability to carry the load at the slightly larger spacing that was provided. While it is thicker than the other systems, the fact that it is hollow means that it is a fair bit lighter than a dedicated concrete slab.

ADVANTAGES

The biggest advantage to a concrete plank system, is the fact that it is precast, so there is no need to wait for the concrete to dry. This means construction can begin on other items sooner, and the overall construction time is decreased. Along the same line, the need for formwork and shoring disappears as well, adding even more to the amount of time saved during construction.

DISADVANTAGES

One major drawback to the idea of hollow core concrete plank, is the fact that it costs so much. This was the most expensive alternative considered, at $28.52 per square foot. It is unclear exactly how much time it may save during construction, but it may the cheaper alternative to pour on site. Another factor adding to cost is the relative amount of experience required to install the planks. Evidently, it takes a skilled worker to place them properly, and finding one may require a small amount of extra money.

Vibrations here will not be any more of an issue than the steel systems. However, the thickness is something to consider. The floor itself is thicker by 1.5 inches, and the supporting beams are thicker by 3 inches. While steel can be punched through to allow for MEP, it is an extra headache during fabrication, and may cost more as a result.

CHART COMPARISON

Parameter	System Considered			
	Existing Composite Beam	Alternative Non Composite Beam	One Way Concrete Slab	Hollow Core Plank
Cost ($/sq ft)	22.75	24.18	20.1	28.52
Fire Rating	2 hr	2hr	2hr	2hr
Vibration control	decent	decent	excellent	decent
Foundation impact	n\a	no impact	foundation needs to be expanded	no impact
Weight (psf)	34	39	58	250
Thickness (slab/beam) (in)	4.5/18	4.5/18	6/24	6/21
Deflection (in)	0.5	0.8	0.5	1.5
Fireproofing Req'd	spray bms/decking	spray bms/decking	no	spray beams
Schedule impact	n/a	none	may increase cconstruction time	may decrease construction time
constructability	easy	easy	intermediate	difficult
Feasability	n/a	YES	NO	YES

CONCLUSION

This technical report compared three different alternative flooring systems to the current composite beam system, and judged whether each system is feasible or not. This comparison was done over a single sample bay, and calculations as well as simple layouts can be found in appendices A-D. Comparisons were based on structural issues, constructability, and architectural concerns that are prevalent for this design.

The current system appears to be the most optimal, with a 4.5" slab on composite beams. These beams reach a maximum depth of 27 inches, however steel beams are able to be punched and allow for MEP to pass through if needed. This was the cheapest feasible system as well, costing 22.75 per sq ft to construct.

The alternative steel deck system, while a good exercise in comparison, simply could not match up to the existing system. Costing about 2 dollars per sq ft more to construct, it would add a large fee into the construction cost. It also allowed for 50 percent more deflection, and while it does not break serviceability requirements, does pose problems when considering other future problems.

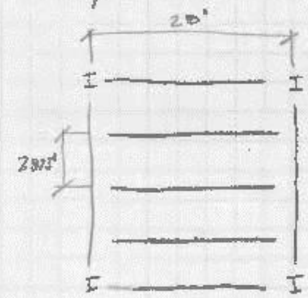
The one way concrete slab system was simply too large and too heavy to be considered a viable option. To add that much weight to the foundation would require a redesign, which is not optimal considering it is on such a small site. While it is considerably cheaper, the added cost and time of formwork and shoring necessary would counteract that discount, which leads to the ultimate conclusion that this system would not work.

Lastly the hollow core plank would be an option to be considered, however they are not quite as efficient as the composite system. They are much more expensive, as they are a proprietary product, and add a little to the thickness of the floor structure. If concrete planks were a regional standard, than maybe this system would trump the existing one, however no evidence has been found that would give reason to believe so.

In conclusion, the existing system is the best choice among these four, with all things considered. It is the most efficient for the cost, easiest to build, and lightest structure considered in this report.

APPENDIX A

HANDLEY McDONALD | TECH 2 | EXISTING CONDITIONS | 1



SYSTEM OF COMPOSITE BMS RESTING ON GIRDERS.

* SLIGHT DISCREPANCIES EXPECTED DUE TO DIFFERENCES IN DECK MANUALS. VULCRAFT DECKING MANUAL USED HERE

BM SPACING 7'10½" → ASSUME 8' FOR VULCRAFT TABLES

TAKE SDL AS 30 psf

UNFACTORED LOAD = 30 + 100 = 130 psf

SELECT 2VLI 20 (SIMILAR PROPERTIES TO DECK USED)
t = 2.5"

MAX UNSHORED SPAN = 11'7", 3 span OK

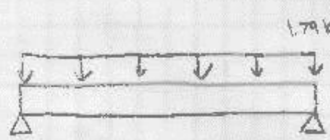
MAX LOAD, 8' SPAN = 196 psf > 130 psf OK

DECK OK

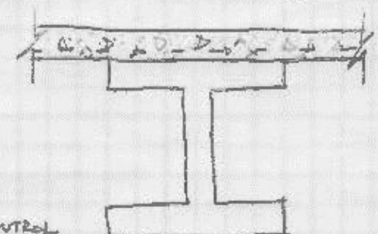
COMPOSITE BM CHECK

$$w_u^* = 1.2(\underset{SDL}{30} + \underset{DW}{1.97} + \underset{CW}{34}) + 1.6(100) = 227\text{ psf}$$

$$w_u = 227(7.875) = 1.79\text{ KLF}$$



$$V_u = \frac{1.79(28)}{2} = 25^K \quad M_u = \frac{1.79(28)^2}{8} = 175'K$$



W18x40 COMPOSITE
N = 20 ¾"⌀ x ¾" LG HEADED STUDS
2" 20VLI DECK

b_{eff} = MIN
- SPAN/8 = 28/8 = 3.5' → CONTROL
- ½ DIST TO BM = .5(7.875) = 3.94'
- DIST TO EDGE = N/A

Handley McDonald | Tech 2 | Existing Conditions | 2

$b_{eff} = 42"$

$V'_c = .85 f'_c b_{eff} = .85(4000)(42)(2.5) = 357^k$

$V'_s = A_s f_y = 11.8(50) = 590^k$

$V'_q = 357^k$

$a = \frac{V'_s}{.85 f'_c b_{eff}} = \frac{357}{.85(4)(42)} = 4.13"$

$A_{s-c} = \frac{T_s - C_c}{2 f_y} = \frac{590 - 357}{2(50)} = 2.33 \text{ in}^2$

$X = \frac{A_{s-c}}{b_f} = \frac{2.33}{6.02} = .39" \rightarrow$ PNA IN FLANGE

$C_c = .85(4)(2.5)(42) = 357^k$ $\quad C_s = \frac{11.8(50) - 357}{2} = 116.5$

$M_n = 590(17.9/2) + 357(2.5 - 4.13/2) - 116.5(.39/2) = 454'^k$

$\phi M_n = .9(454) = 409'^k$

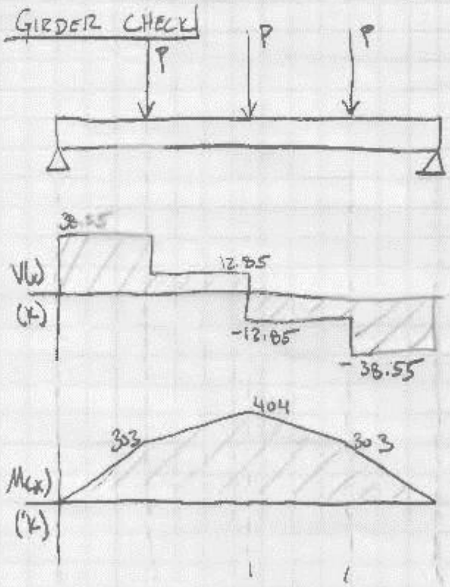
*ADD SELF WEIGHT

$W_u = 1.79 + 1.2(.04) = 1.84$ KLF

$V_u = \frac{1.84(28)}{2} = 25.7^k$ $\quad M_u = \frac{1.84(28)^2}{8} = 180.3'^k$

$\phi M_n \geq M_u$ $\quad$ OK

GIRDER CHECK



$P = 25.7^k$

$V_u = 38.55^k$

$M_u = 404'^k$

SELECT W27X84

$Z_x = 244$

$I_x = 2850$

Handley McDonald — Tech 2 — Existing Conditions — 3

LTB Case II:

$C_b = 1.06 \qquad L_r = 20.8'$

$\phi M_p = 915 \qquad L_p = 7.31'$

$\phi M_r = 559$

$$M_n = C_b\left(\phi M_p - (\phi M_p - \phi M_r)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right) \leq \phi M_p$$

$$M_n = 1.06\left(915 - (915 - 559)\left(\frac{7.875 - 7.31}{20.8 - 7.875}\right)\right) \leq \phi M_p$$

$$M_n = 953 \geq \phi M_p$$

$$\phi M_n = 915'k$$

$$\phi M_n \geq M_u \quad \underline{ok}$$

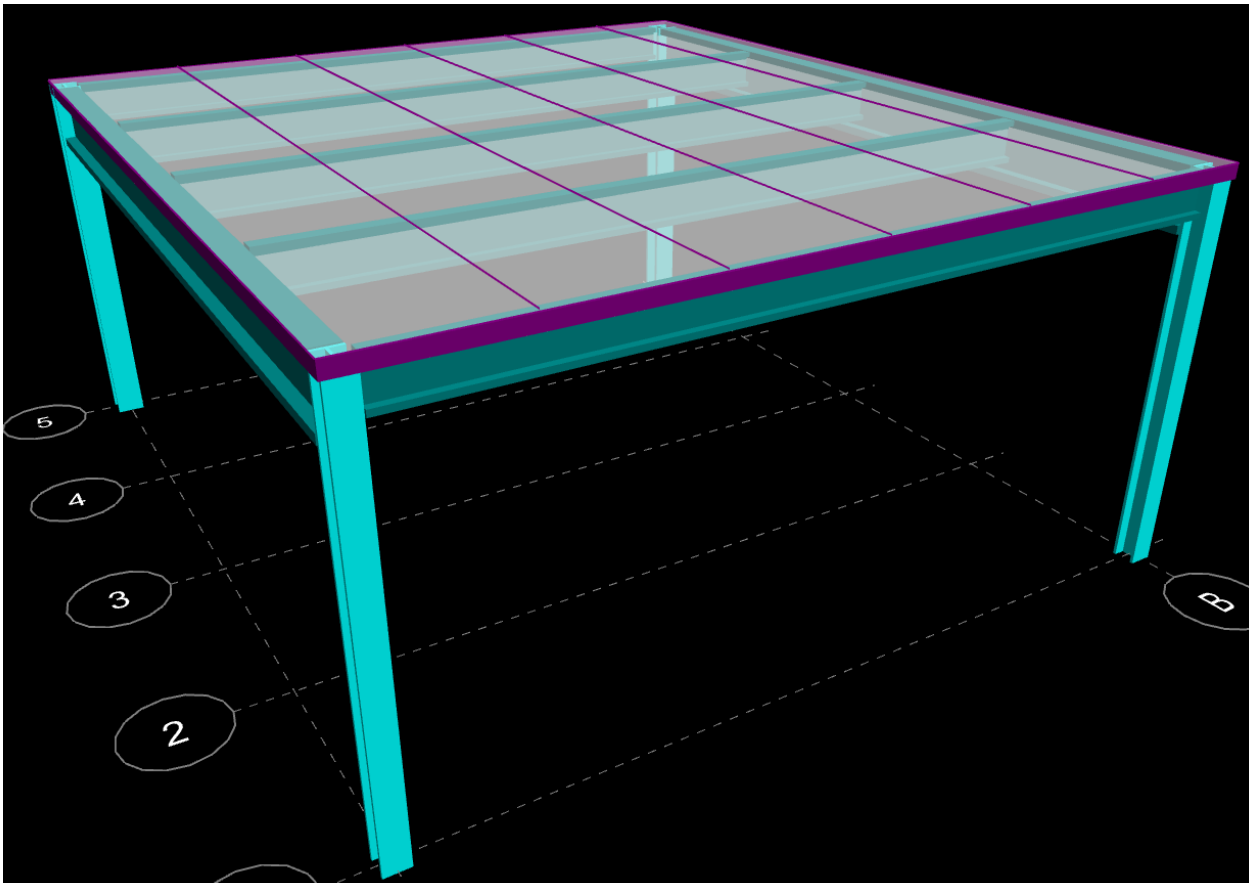
$$\Delta_{D+L} = 2\left(\frac{Pb}{24EI}\left(3L^2 - 4a^2\right)\right) + \frac{PL^3}{48EI}$$

$$\Delta_{D+L} = 2\left(\frac{27.5(7.875')}{24(29000)(2850)}\left(3(31.5')^2 - 4(7.875')^2\right)\right) + \frac{27.5(31.5')^2}{48(29000)(2850)}$$

$$\Delta_{D+L} = .374''$$

$$\Delta_{D+L\ ALL} = L/240 = \frac{31.5(12)}{240} = 1.6'' \geq .374'' \quad \underline{ok}$$

USE W27x84 GIRDER



Beam Deflection Summary

RAM Steel v14.04.07.00
DataBase: Composite
Building Code: IBC

10/15/12 11:48:26
Steel Code: AISC360-05 ASD

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: main

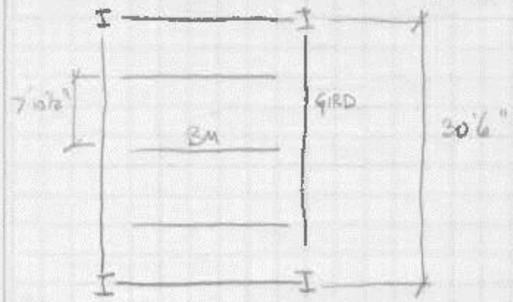
Composite / Unshored

Bm #	Beam Size	Initial in	PostLive in	PostTotal in	NetTotal in	Camber in
4	W27X84	0.176	0.265	0.344	0.520	
3	W18X40	0.158	0.202	0.262	0.420	
7	W18X40	0.255	0.320	0.415	0.671	
6	W18X40	0.255	0.320	0.415	0.671	
5	W18X40	0.255	0.320	0.415	0.671	
1	W18X40	0.158	0.202	0.262	0.420	
2	W27X84	0.176	0.265	0.344	0.520	

Appendix B

HANDLEY McDONALD TECH 2 NON COMPOSITE DECK 1

LAYOUT TO FOLLOW SAME FRAMING AS EXISTING CONDITIONS



SYSTEM WILL COMPOSE OF METAL DECK w/ CONC TOPPING, BUT NOT IN COMPOSITE ACTION. VULCRAFT DECKING MANUAL USED FOR REFERENCE

BM SPACING @ 7'10½" → ASSUME 8' FOR VULCRAFT TABLES

TAKE SDL AS 30 psf

W_o = 30 + 100 = 130 psf

SELECT 2C16 (4.5" THK)

CAPACITY FOR { 3-SPAN CONDITION, 8' SPAN, DEFL ≤ L/240 } → 170 psf

ADDING SELF-WEIGHT:

W_{DECK} = 3.29 psf
W_{CONC} = 36 psf

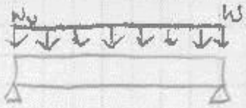
W_o = 130 + 36 + 3.29 = 169.29 ≤ 170 psf OK

USE 2C16 DECKING

Handley McDonald Tech 2 New Composite Deck 2

Beam Design

$w_u^* = 170$ psf



$W_U = 170(7.875) = 1.34$ KLF

Continuous bracing : LTB not a concern

$M_u = \frac{1.34(28)^2}{8} = 131$ 'k

$M_U = F_y Z_x$

$12(131) = 50 Z_x$

$Z_x \geq 32$ → Select W 14x30

$Z_x = 47.3 \text{ in}^3$

$I_x = 291 \text{ in}^4$

$\Delta_{DL} = \frac{5wL^4}{384EI_x} = \frac{5(1.34)(28)^4(1728)}{384(29000)(291)} = 2.6"$ NG By observation

Use I_x as control

$\frac{L}{240} \geq \frac{5wL^4}{384EI_x}$

$384EI_xL \geq 5wL^4(240)$

$I_x \geq \frac{5wL^4(240)}{384EL}$

$I_x \geq \frac{5(1.34(1/12))(28(12))^4(240)}{384(29000)(28(12))}$

$I_x \geq 456$ → Select W 18x35

$I_x = 510$

Δ_{DL}, ϕM_n OK By inspection

$w_{u,f} = 1.2(50 + 3.29 + 36)(7.875) + 1.6(100)(7.875) = 1.91$

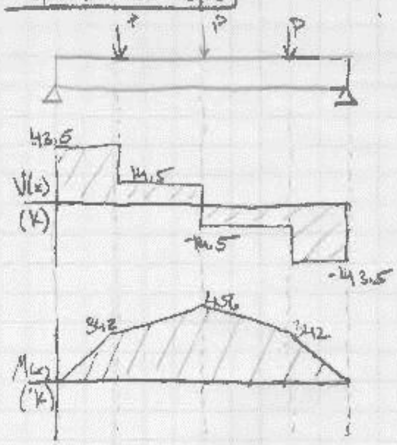
$V_u = \frac{1.91(28)}{2} = 27^k$

$\phi V_n = 159^k \geq 29^k$ OK

Use W18x35 Beam

HANDLEY McDONALD | TECH 2 | NON COMPOSITE DECK | 3

GIRDER DESIGN



$P = 28.99^k$

$V_u = 43.5^k$

$M_u = 456'k$

$M_n = F_y Z_x$

$456(12) = 50 Z_x$

$Z_x \geq 110$

SELECT W24x55

$Z_x = 134$

$I_x = 1350$

LTB CASE II: $C_b = 1.06$, $\phi M_p = 503$, $\phi M_r = 299$, $L_r = 13.9'$, $L_p = 4.73'$

$$M_n = 1.06\left(503 - (503-299)\left(\frac{7.875-4.73}{13.9-7.875}\right)\right) \leq 503'k$$

$M_n = 422'k < 456'k$ **NG**

SELECT W24x62

$Z_x = 153$

$I_x = 1550$

LTB CASE II: $C_b = 1.06$, $\phi M_p = 574$, $\phi M_r = 344$, $L_r = 14.4$, $L_p = 4.07$

$$M_n = 1.06\left(574 - (574-344)\left(\frac{7.875-4.07}{14.4-7.875}\right)\right) \leq 574'k$$

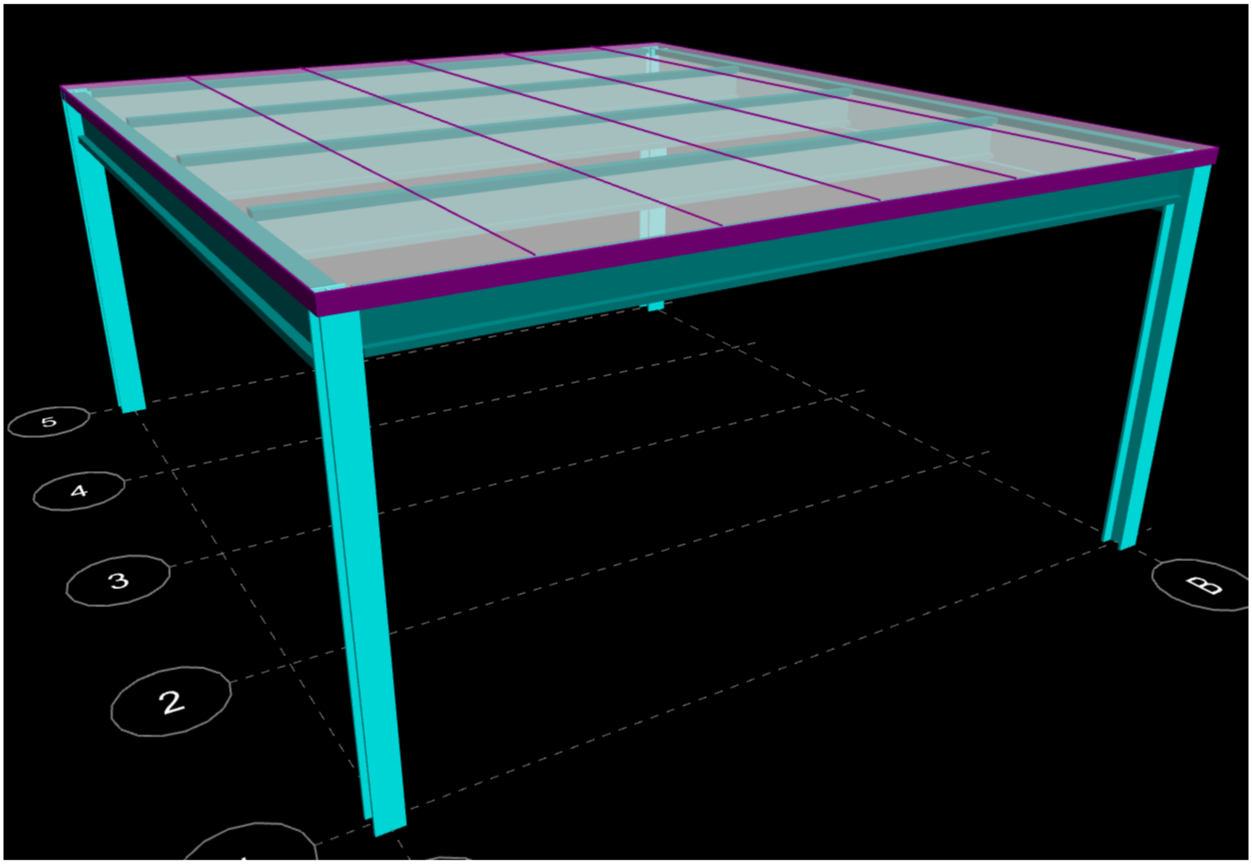
$\phi M_n = 496'k \geq 456'k$ **ok**

$$\Delta_{DL} = 2\left(\frac{28.99(7.875)}{24(29000)(1550)}\left(3(31.5)^2 - 4(7.875)^2\right)\right) + \frac{28.99(31.5)^3}{48(29000)(1550)}$$

$\Delta_{DL} = .73"$

$\Delta_{ALL} = L/360 = \frac{31.5(12)}{360} = 1.05" \geq .73"$ **ok**

USE W24X55 GIRDER



Beam Deflection Summary

RAM Steel v14.04.07.00
DataBase: NonComposite
Building Code: IBC

10/15/12 12:28:39
Steel Code: AISC360-05 ASD

STEEL BEAM DEFLECTION SUMMARY:

Floor Type: main

Noncomposite

Bm #	Beam Size	Dead in	Live in	NetTotal in	Camber in
10	W24X55	0.323	0.640	0.963	
9	W18X35	0.170	0.416	0.586	
14	W18X35	0.276	0.710	0.986	
13	W18X35	0.276	0.710	0.986	
12	W18X35	0.276	0.710	0.986	
11	W18X35	0.170	0.416	0.586	
8	W24X55	0.323	0.640	0.963	

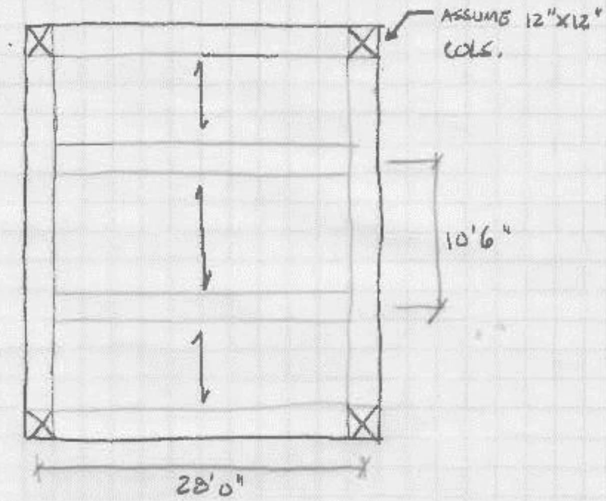
Appendix C

HANDLEY MCDONALD | TECH 2 | 1-WAY SLAB | 1

PROPOSED LAYOUT:

FOLLOWING INFO PROV'D IN SHEET S0.11:

$f'_c = 4000$ psi LWC
$f_y = 60$ ksi



By TABLE 9.5(a), MIN SLAB THK = $\frac{L}{24} = \frac{10.5(12)}{24} = 5.25"$ → USE 6"

DETERMINE LOADS:

TAKE SDL AS 30 psf

$W^*_{SLAB} = \frac{6}{12}(115) = 57.5$ psf

$W^*_{LL} = 100$ psf (NON REDUCED)

$W^*_u = 1.2(30 + 57.5) + 1.6(100) = 265$ psf

$W_u = 265(10.5) = 2.78$ KLF

$M_u = \frac{2.78(28)^2}{8} = 272$ 'K

ESTIMATE BM DIM:

$bd^2 \geq \frac{M_u}{\phi R}$

$\rho = \frac{\beta_1 f'_c}{4 f_y} = \frac{.85(4000)}{4(60,000)} = .014$

$\omega = \frac{\rho f_y}{f'_c} = \frac{.014(60)}{4} = .213$

$R = \omega f'_c(1 - .59\omega) = .213(4000)(1 - .59(.213)) = .745$ ksi

$bd^2 = \frac{272(12)}{.9(.745)} = 4868$ in^3

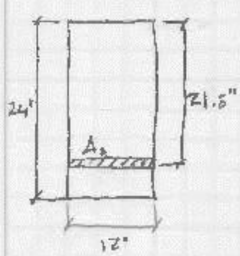
Handley McDonald | Tech 2 | 1-way slab | 2

$bd^2 = 4868$ → Take $b = 12"$

$12d^2 = 4868$

$d = 20.14$

$h = d + 2.5 = 22.64"$ → use $h = 24"$



Determine A_s

$A_s \approx \frac{M_u}{4d} = \frac{272}{4(22.5)} = 3.02 \text{ in}^2$

Use (3) #10 @ 4" → $A_s = 3(1.27) = 3.81$

Calculate M_n

$a = \frac{A_s f_y}{.85 b f'_c} = \frac{3.81(60)}{.85(12)(4)} = 5.6"$

$c = \frac{5.6}{.85} = 6.6"$

$\varepsilon_s = \frac{.003}{6.6}(21.5 - 6.6) = .0067 \geq .00207$ ok

$\varepsilon_T = .0067 \geq .005$; $\phi = .9$ ok

$\phi M_n = .9(3.81)(60)(21.5 - \frac{5.6}{2}) = 320'k \geq 272'k$ ok

Add self weight of BM

$W_{BM} = \frac{(24-6)(12)}{144}(115) = 172.5$ plf

$W_u = 2.78 + .1725 = 2.95$ klf

$M_u = \frac{2.95(28)^2}{8} = 289'k \leq 320'k$ ok

Calculate $V_{n,max}$

$V_u = \frac{2.95(28)}{2} = 41.3^k$

$V_c = 2\sqrt{f'_c}\,bd = 2\sqrt{4000}(12)(21.5) = 32.6^k$

$\phi V_c = .5(.75)(32.6) = 12.2$

$V_s = \frac{V_u}{\phi} - V_c = \frac{41.3}{.75} - 12.2 = 43^k$

$V_{s,max} = 8\sqrt{4000}(12)(21.5) = 130^k \geq 43^k$ ok

HANDLEY MCDONALD | TECH 2 | 1-WAY SLAB | 3

REINF. @ NEG. MOMENT

USING APPROX METHOD ACI 318-11 § 8.3.3

$M^-_{END} = \frac{wL^2}{10} = \frac{2.95(28)^2}{10} = 231'k$

$M^+_{CONT} = \frac{wL^2}{11} = \frac{2.95(28)^2}{11} = 210'k$

A_s REQ'D $\approx \frac{M_u}{4d} = \frac{231}{4(21.5)} = 2.7 in^2$

USE 3 #9 @ 4", $A_s = 3 in^2$

CALCULATE M_n

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{3(60)}{.85(4)(12)} = 4.4$

$c = \frac{4.4}{.85} = 5.19$

$\varepsilon_s = \frac{.003}{5.19}(21.5-5.19) = .009 \geq .00207$ OK

$\varepsilon_t = .009 \geq .005$; $\phi = .9$ OK

$\phi M_n = .9(3)(60)(21.5 - \frac{4.4}{2}) = 260'k \geq 231'k$ OK

SHEAR REINF.

SHEAR REINF NOT REQ'D WHERE $V \leq 12.2^k$

$41.3 - x(2.95) = 12.2$

$x = 9.8'$ FROM LEFT/RIGHT SUPPORTS

→ USE MIDDLE 8' STIRRUP FREE

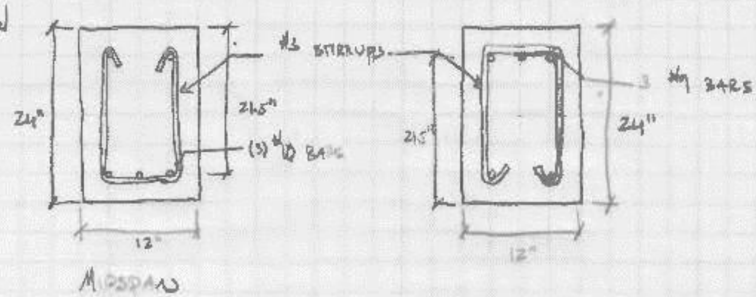
$S_{MAX} = \begin{cases} d/2 = 10.75" \rightarrow \text{CONTROL} \\ \text{MIN } 24" \end{cases}$

$A_{v,MIN} = \begin{cases} .75\sqrt{f'_c}\, b \frac{S_{MAX}}{f_{yt}} = .75\sqrt{4000}(12)\frac{10.75}{60{,}000} = .102 \\ \text{MAX } \frac{50\, b\, S_{MAX}}{f_{yt}} = \frac{50(12)(10.75)}{60{,}000} = .1075 \rightarrow \text{CONTROL} \end{cases}$

USE (2) #3 STIRRUPS, [ ], $A_v = 2(.11) = .22 in^2$

Handley McDonald — Tech 2 — 1-Way Slab — 4

BM SECTIONS|



DEFLECTIONS|

$w_{D+L} = 57.5(10.5) + 172.5 + 100(10.5) = 1.826$ KLF

$E_c = 57000\sqrt{f'_c} = 57000\sqrt{4000} = 3604$ ksi

$I_g = \frac{bh^3}{12} = \frac{12(24)^3}{12} = 13824$ in^4

$\Delta_{D+L} = \frac{5wL^4}{384EI} = \frac{5(1.826)(21.5)^4(1728)}{384(3604)(13824)} = .5"$

$\Delta_{ALL} = \frac{l}{360} = \frac{21.5(12)}{360} = .93" \geq .5"$ ok

SLAB REINF|

PLACE REINF @ 12" O.C. ; TREAT AS BM

$W_u = 1.2(30+57.5)(1') + 1.6(100)(1') = 265$ plf

$M_u = \frac{.265(10.5)^2}{8} = 3.65$ 'k

$A_{s\,REQ'D} \geq \frac{M_u}{4d}$ → TAKE d @ 4"

$A_{s\,REQ'D} = \frac{3.65}{4(4)} = .228$

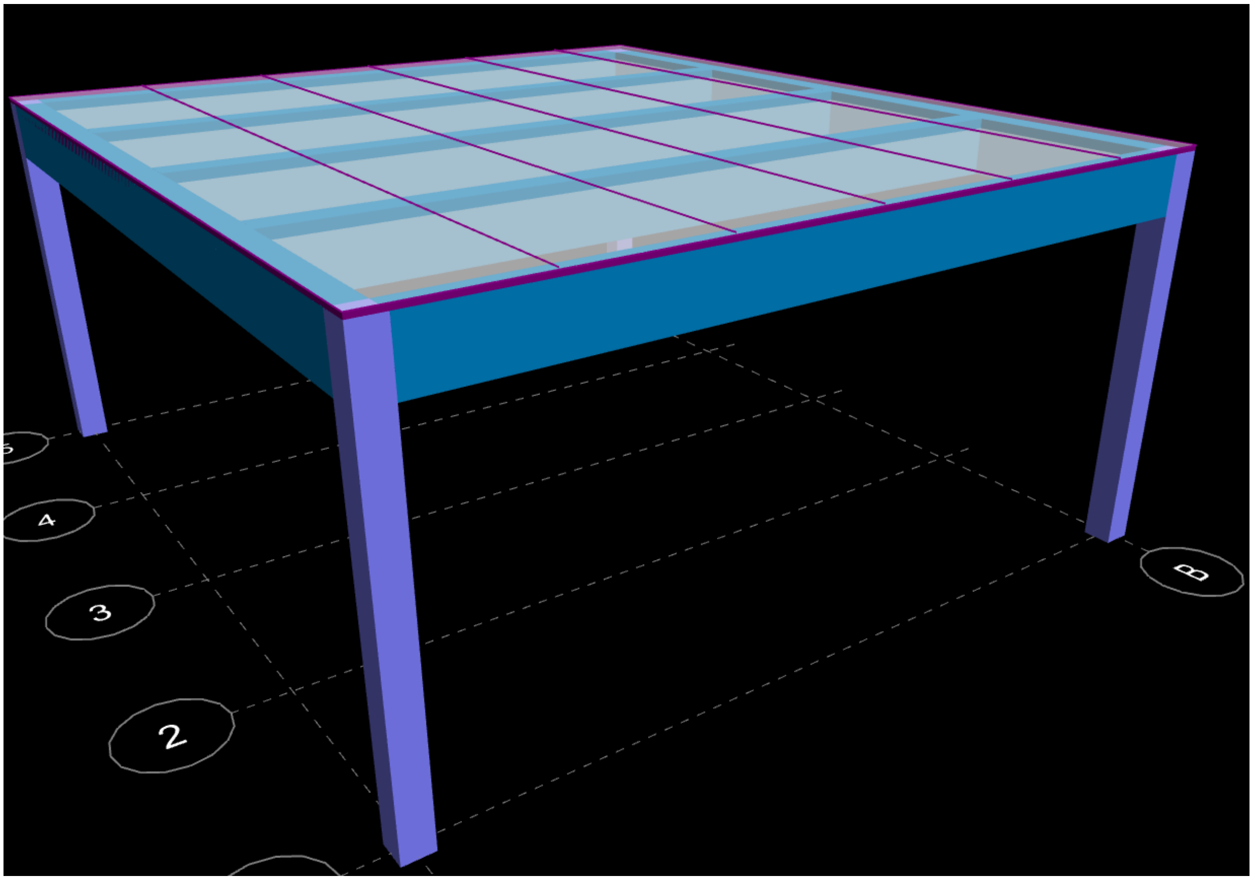
USE #5 @ 12" o.c. $A_s = .31$ in^2

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.31(60)}{.85(4)(12)} = .46$ $c = \frac{.46}{.85} = .54$

$\varepsilon_s = \frac{.003}{.54}(4 - .54) = .019 \geq .00207$ ok

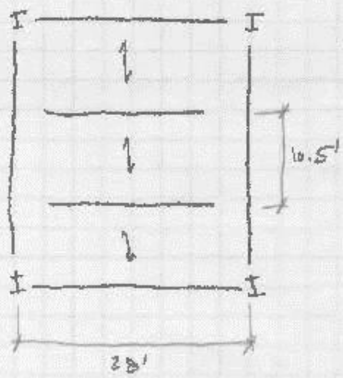
$\varepsilon_t = .019 \geq .005$, $\phi = .9$ ok

$\phi M_n = .9(.31)(60)(4 - \frac{.46}{2}) = 63$ 'k $\gg 3.65$ 'k ok


5
4
3
2
B

Appendix D

Handley McDonald | Tech 2 | Hollow Core Plank | 1



*Proprietary tables from Oldcastle Precast Building Systems, Elematic Hollow Core Plank

$W_u = 1.2(30) + 1.6(100) = 196$ psf

→ Select E6" X 48" no topping

11' span, $W_{max} = 271$ psf

*Add self weight

$W_u = 1.2(30+45) + 1.6(100) = 250$ psf

$W_{max} = 271$ psf ≥ 250 OK

Beam Design

$W_u^* = 250$ psf

$W_u = 250(10.5) = 2.63$ KLF

$M_u = \frac{2.63(28)^2}{8} = 260$ 'K

$M_u = F_y Z_x$

$12(260) = 50 Z_x$

$Z_x \geq 62$ → Deflection will probably control

$I_x \geq \frac{5 w L^4 (240)}{384 EL}$

$I_x \geq \frac{5(\frac{2.63}{12})(28(12))^4(240)}{384(29000)(28(12))}$

$I_x \geq 895$ → Select W21X48

$I_x = 959$

$Z_x = 107$ OK

$V_u = \frac{2.63(28)}{2} = 37^K$ $\phi V_n = 216^K \geq V_u$ OK

Use W21X48 Beam

HANDLEY MCDONALD | TECH 2 | HOLLOW CORE PLANK | 2

GIRDER DESIGN

$P = 37^k$

$V_u = \frac{37(2)}{2} = 37^k$

$M_u = 37(10.5) = 387'k$

$M_n = F_y Z_x$

$12(387) = 50 Z_x$

$Z_x \geq 93$ → SELECT W21X55

$Z_x = 126$

$I_x = 1140$

LTB CASE II:

$C_b = 1.06$ $\quad L_r = 17.4'$

$\phi M_p = 473'k$ $\quad L_p = 6.11'$

$\phi M_r = 289'k$

$$M_n = 1.06\left(473 - (473 - 289)\left(\frac{10.5 - 6.11}{17.4 - 10.5}\right)\right) \leq M_p$$

$M_u = 378'k$ NG → SELECT W21X62

$Z_x = 144$

$I_x = 1330$

$$M_n = 1.06\left(540 - (540 - 335)\left(\frac{10.5 - 6.25}{18.1 - 10.5}\right)\right) \leq M_p$$

$\phi M_n = 440'k \leq M_p$ OK

$$\Delta_{D+L} = \left(\frac{37(10.5')}{24(29000)(1330)}\right)\left(3(31.5')^2 - 4(10.5')^2\right) = 1.83''$$

$$\Delta_{max} = \frac{L}{240} = \frac{31.5(12)}{240} = 1.6'' \leq 1.83'' \quad NG$$

→ SELECT W21X73

$I_x = 1600$

$$\Delta_{D+L} = \left(\frac{37(10.5')}{24(29000)(1600)}\right)\left(3(31.5')^2 - 4(10.5')^2\right) = 1.5'' \leq 1.6'' \quad OK$$

USE W21X73 GIRDER

Appendix E

E6" x 48" SECTION
WITH NO TOPPING

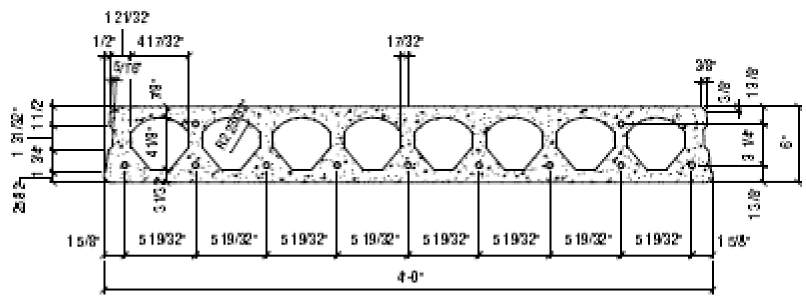
ELEMATIC® Hollowcore Plank

Oldcastle Precast®
Building Systems

UNIFORMLY DISTRIBUTED SUPERIMPOSED* LOAD IN LBS. PER SQ. FT.

Standard Designation	7-Wire 270 Lolax P/S Strand Combination	P/S Strand Area Sq. In.	Ultimate Bending Moment, φ Mn Kip-Ft. per Unit	SIMPLE SPAN IN FEET																								p Vcw in Kips per Unit
				10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	
20_06704	4-7/16"φ	0.460	39.96	304	271	245	219	191	167	147	130	116	104	91	79	68												8.58
20_06705	5-7/16"φ	0.575	48.76	314	280	253	229	210	186	164	145	129	116	104	94	84	74											8.58
20_06706	6-7/16"φ	0.690	57.04	324	289	261	237	217	193	170	151	135	121	108	98	89	79	70										8.58
20_06707	7-7/16"φ	0.805	64.48	334	298	269	244	224	206	184	163	145	130	117	106	96	87	79										8.58

* INCLUDES THE LIVE LOAD PLUS ANY DEAD LOAD THAT IS ADDITIONAL TO THE WEIGHT OF THE BARE GROUTED PLANKS IN PLACE



Grouted weight of plank is 45 lbs. per sq. ft.

f'c = 5,000 psi　f'ci = 3,000 psi　Area = 173 in^2

f'pu = 270, 000 psi　I = 719 in^4　bw = 10.0 in.

NOTES

1. Design Standard: ACI 318-2005
2. For complete and detailed calculations consult Oldcastle Precast
3. For longer spans, heavier loads, or special conditions, consult Oldcastle Precast
4. The table indicates maximum safe loads. Camber and deflection must always be investigated by the architect, and/or engineer for the contemplated loading and span so that these factors are compatible with the contiguous materials in the proposed structure.
5. Values to the left and below the heavy stepped line are controlled by shear.
6. Shaded region indicates expected camber greater than 1".