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Southwest Housing, Arizona State University

Technical Report #2
Floor Systems

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Executive Summary

In the report that follows, the existing floor system was analyzed alongside 3 potential alternative floor systems to investigate the advantages and disadvantages of each design. Each floor system was evaluated with regards to a typical bay in the building, which stretches from the end of one core to the start of the next. The typical bay dimensions are 62'-6"x52', with 5 bays of beams for the girders, spaced at 12'-6". The primary factors affecting the floor system sizing were the need for a 2-hr fire rating on each floor, and the deflection requirements.

The floor systems studied in this report are as follows:

- The existing system [3", 20 gauge composite deck with 3.25" lightweight concrete topping on structural steel frame]
- A non-composite system [3", 16 gauge form deck with 3" normal weight concrete topping on structural steel frame]
- A long-span deck system [6", 14 gauge form deck with 3.25" lightweight concrete topping on structural steel frame, 4 bays of beams instead of the existing 5 bays]
- A post-tensioned 2-way concrete slab system [7" slab with 14" wide shallow beams running in the long direction; (30) ½" ϕ 7-wire, 270 ksi tendons running in the wide shallow beams, banded in 3 bundles of 10 tendons, (64) if the same 7-wire 270 ksi tendons running in the short direction, distributed over the 62'-6" span]

A cost analysis is included, as well as a comparison of self-weight, constructability, architectural impact, foundation impact and lateral system impact.

Ultimately, the existing system was deemed the most appropriate for the construction style of this building. The existing system weighs 59 lb/square foot, is 2' deep, and costs about $19.00/square foot. The non-composite system weighs 14 lb/square foot more for the same height, and costs about $1.00/square foot more. The long-span deck system weighs almost the same as the existing system, but costs about $8.00/square foot more for the same floor depth. Potential for decreasing the cost and optimizing this system is presented in the conclusions. The post-tensioned concrete system weighs the most of the systems investigated, at 99 lb/square foot. This system has the smallest depth, reducing floor thickness by 10" from the existing system, and provides the overall lowest cost at about $9.30/square foot. Of all of the systems, the likelihood for the foundation and lateral systems to be impacted results the most from the post-tensioned system, due to its high self-weight.

Introduction

The Southwest (SW) Student Housing building is a 20-story high-rise for students attending Arizona State University. The building site is located in a downtown area, at 1000 Apache Blvd. East in Tempe, Arizona (see Figure 1, the site is highlighted in red[1]). The building plans are designed to accommodate 528 beds in 268 units, with an emphasis on modularity for ease and economy of construction. There is additional potential to include an automated parking facility on the first level, which can be accounted for in the initial building design. A rendering of the potential building design can be observed on the front cover of this report.

Figure 1: Site Location, 1000 Apache Blvd. East, Tempe, AZ

This particular building has a unique structure designed for easy assembly on site to enable extremely fast and efficient construction. The building's gravity and lateral systems are one and the same: a series of three 8-inch thick concrete cores, 25' wide and 25' long. These cores are constructed first using slip-forms to within a 1/8" tolerance. The roof of the building is then assembled on the ground around the cores in two parts and lifted into place using six 75-ton strand jacks. Each subsequent floor is then assembled on the ground, half the floor area at a time (with the joint located at the precise halfway point of the floor plan, as indicated in Figure 2), and lifted into place. The building is essentially constructed from the top, down.

The floors are constructed using metal deck with lightweight concrete and structural steel beams. Each floor has a similar and regular floor plan (and thus, loading), with residential areas for 23' on each side of a 6'-wide corridor running through the center of the building, lengthwise (see Figure 2 below).

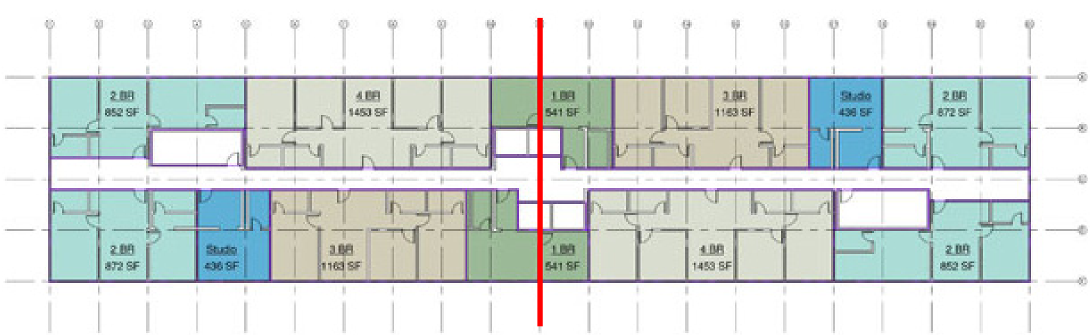

Figure 2: Typical Building Floor Plan

[1] Taken from http://maps.google.com

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Structural Systems

Foundation

The SW Student Housing building will exert significant loads to the foundation elements, according to the geotechnical report for the area. As a result, this building will require a deep foundation system that penetrates through to the second layer of soil on the site to limit settlement. The first layer of the site is Silty Sand and Poorly Graded Sand for a depth range from 10' to 35'. The second layer of soil on the site is Sand Gravel Cobble, from a depth of 35' to 100'.

The geotech report recommends drilled piers, with no pier shaft sized to a diameter of less than 12". Each pier should penetrate at least twice the shaft diameter into the second layer of soil. The predicted settlement for this pier configuration is less than one inch for an isolated pier shaft with a diameter of less than 60".

Floor System

The floor system is the same on all floors. This system consists of 3-1/4" lightweight concrete on 3" metal deck, with a minimum gauge of 20. The composite deck is supported by a structural steel frame, with wide-flange sizes ranging from W14x22 infill beams to W24x176 interior girders, as prescribed by the typical framing plan shown in Figure 3, and reiterated in the notes included in Appendix A. All four girders span the length of the building (250'), and all typical beams span the width of the building (52'). Infill beams span either 12'-6" or 24', depending on their location within the building. The typical members are labeled in Figure 3. Every structural steel element in the typical frame is cambered. Some members are cambered up to 4 inches at the cantilevered ends (See Appendix A for the project structural engineer's camber diagrams).

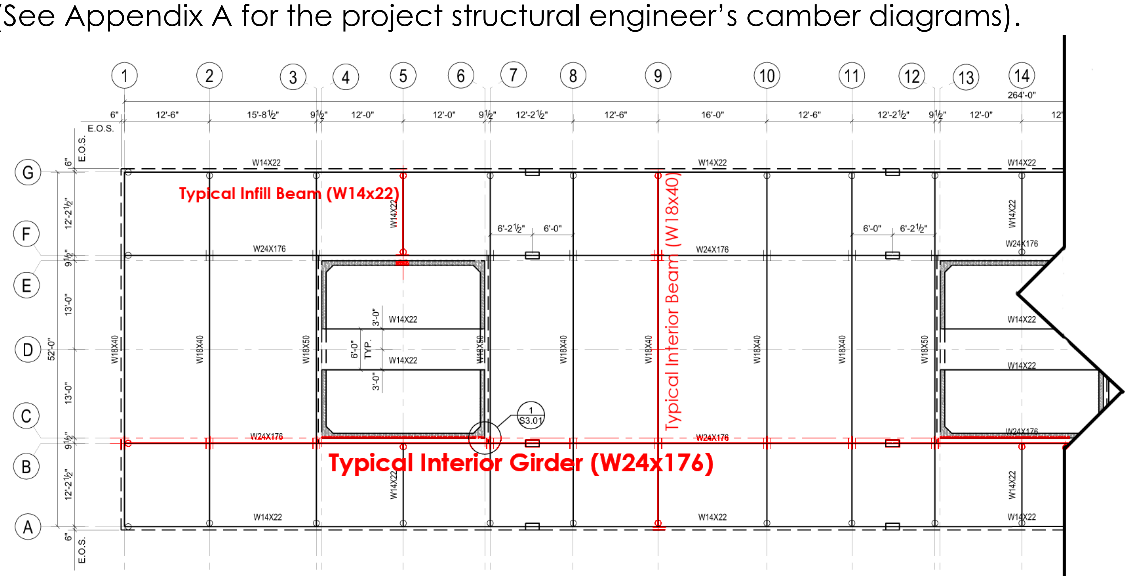
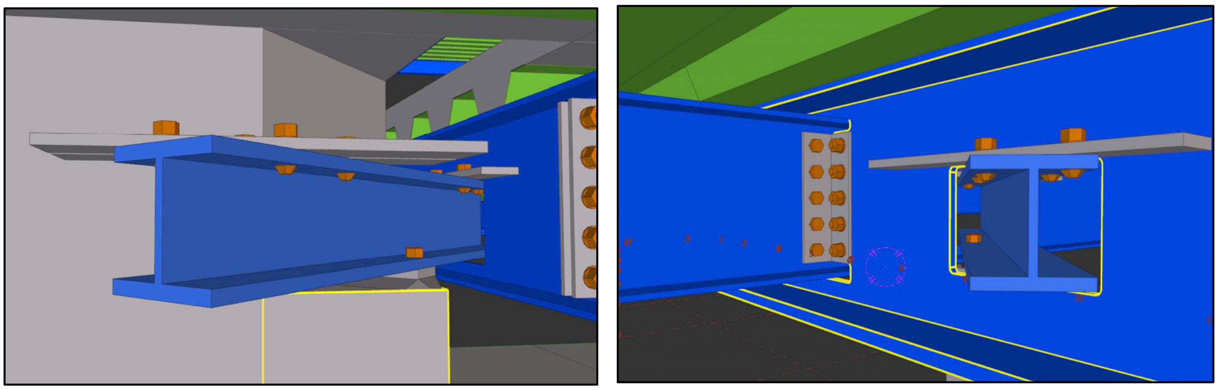


Figure 3: Typical Framing Plan (building is symmetric about line 14)

Gravity and Lateral System

Unlike some conventional construction, this building has no columns. The three 8-inch thick, 25'x25' (at the centerline) concrete cores carry all of the gravity weight of each floor. As a result, the floors are cantilevered off of the cores (spaced at 62'-6" clear span), which support the structural steel floor framing via a wide-flange beam inserted through each of the four corners in every core, as illustrated in Figure 4. During construction, half of a floor is lifted via the 75-ton strand jacks and then fitted into place using the aforementioned corner details. The cores are designed as walls using ACI 318-05. As a result, each core has a minimal amount of reinforcement through the center (one layer of the smallest permitted rebar size by code).



Figures 4.1 and 4.2: Corner detail at every floor, framing into the interior girder to support each level

The concrete cores are also the building's sole lateral system, and provide lateral bracing in both directions in the form of shear walls. For clarity, the cores are highlighted in the typical building floor plan below in Figure 5, with boundaries at openings selected. It can be observed in Figure 6 on the next page that the openings are only present for a minimal height on each floor so that the shear walls can be reunited via large coupling beams for added rigidity and support. The coupling beams are approximately 2' high, and the floor-to-floor height is 10'.

Figure 5: Typical Building Floor Plan (Core areas are highlighted in red, core walls are highlighted in green)

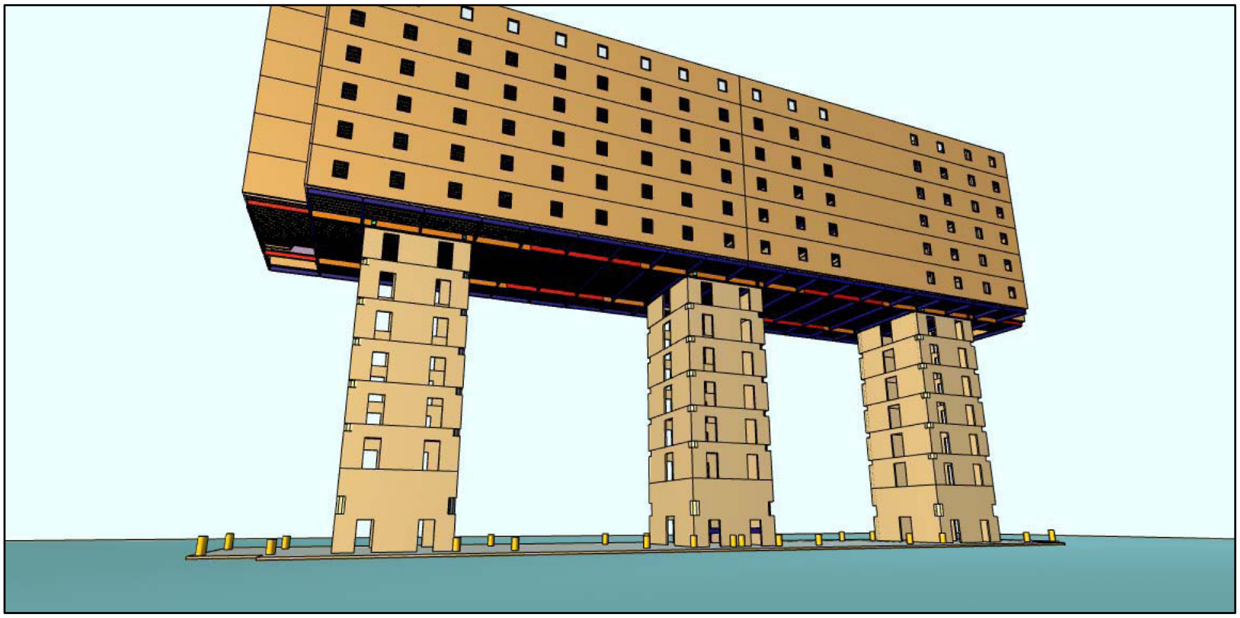


Figure 6: Rendering of visible openings in concrete cores

The theory behind this building design seems to be simplicity: a single set of structural elements to resist all loading. The sizing of these elements was carried out using a combination of hand calculations employing ASD, and computer modeling for more precise answers. ASD hand calculations were found to be generally with 10% of the computer modeling outputs, which used the LRFD method of design.

Roof System

The roof system is a simple, long-lasting construction of the typical floor framing (3-1/4" lightweight concrete with 3" metal deck, minimum 20 gauge), 3" of rigid insulation and an Ethylene Propylene Diene Terpolymer (EPDM) membrane on top. There is no mechanical equipment on the roof- the major mechanical elements will be located on the ground floor, and will serve each unit in the building via a 2-pipe system.

Codes, References and Standards

Building Design Codes:

Model Code:

International Building Code, 2006 Edition, as amended by the city of Tempe, AZ

Design Codes:

American Institute of Steel Construction "Specifications for Structural Steel Buildings", AISC 360-05

American Concrete Institute "Building Code Requirements for Structural Concrete", ACI 318-05

Structural Standards:

American Society of Civil Engineers "Minimum Design Loads for Buildings and other Structures", ASCE7-05

Thesis Codes:

Model Code:

International Building Code, 2006 Edition

Design Codes:

American Institute of Steel Construction "Specifications for Structural Steel Buildings", AISC 360-05 (13^{th} ed.) and AISC 360-10 (14^{th} ed.)

American Concrete Institute "Building Code Requirements for Structural Concrete", ACI 318-05

Structural Standards:

American Society of Civil Engineers "Minimum Design Loads for Buildings and other Structures", ASCE7-05

Deflection Criteria:

Limit Unfactored Live Load deflections to L/360 or less
Limit Total (Service) Load deflections to L/240 or less
Limit building drift to h/400 or less

Fire Safety:

Floor systems must have a minimum 2-hour fire rating

Materials

Structural Steel:

- All Rolled Shapes – ASTM A992 Grade 50
- All Plates and Connection Material – ASTM A36
- All Tubular Sections – ASTM A500 Grade B
- All Pipe Sections – ASTM A53 Grade B
- Anchor Rods – ASTM F1554

Cast-in-Place Concrete:

- Foundations – 4000 psi normal weight
- Slab on Grade – 4000 psi normal weight
- Structural Slab on Grade – 5000 psi normal weight
- Lightweight Concrete – 4000 psi
- Walls (core) – 4000 – 5000 psi

Reinforcement:

- Deformed Bars – ASTM A615 Grade 60 typ.; Grade 70 for #9, #10, #11
- Welded Wire Fabric – ASTM A195

Welding Electrodes:

- E70xx Low Hydrogen

Bolting Materials:

- ASTM 325 or A490

Load Calculations

Gravity Loads

See Appendix B for all calculations, including confirmation of structural steel allowance from typical framing plan and citations for calculating snow load.

Construction Dead Load:

3" Metal Deck (20 gage)	2.14	psf
3-1/4" Lightweight Concrete (110 PCF)	46	psf
Structural Steel Allowance	11	psf
Sum (CDL)	**59.14**	**psf**

Superimposed Dead Load:

Assumed, according to structural engineers	15	psf
Sum (SDL)	**15**	**psf**

Live Loads:

Building uses

Residential	40	psf
Parking	40	psf
Corridors	80	psf
Live Load (LL)	**80**	**psf**

Wall Loads:

Curtain Wall	15	psf
Sum	**15**	**psf**

Snow Loads:

Ground snow load for region	0	psf
Sum	**0**	**psf**

Floor System Analysis

The floor system analyses in this report were carried out for a typical bay in the building plan, as highlighted in blue and outlined with a dashed line in Figure 7. The girders and wide shallow beams of the existing and alternative floor systems run in the East-West direction and span 62'-6". The typical beams and distributed post-tensioning in the existing and alternate systems run in the North-South direction (the short direction). The largest unsupported span is 26', and there is a 13' cantilever off of each support. One of the items that governed each design was the goal fire rating of 2 hours. Every floor system attained at least that rating.

The lateral system in this building is independent of the floor system, and thus was minimally considered in the analysis. Ultimately, the most important feature of the floor systems (with regard to the lateral system) was the floor weight, which is also discussed in the following sections.

An approximate cost evaluation was carried out for each floor system, the documentation of which can be found in Appendix G. Cost information was found from the online CostWorks RS Means database using 2008 1st quarter estimates. Any additional information used is included in Appendix H, including prestressing tendon properties and unit reinforcing bar weight. Any of the properties used are highlighted in blue on each of the included references.

Figure 7: Typical floor plan with the typical bay considered for alternate floor systems highlighted in blue

Existing Floor System - Composite Deck

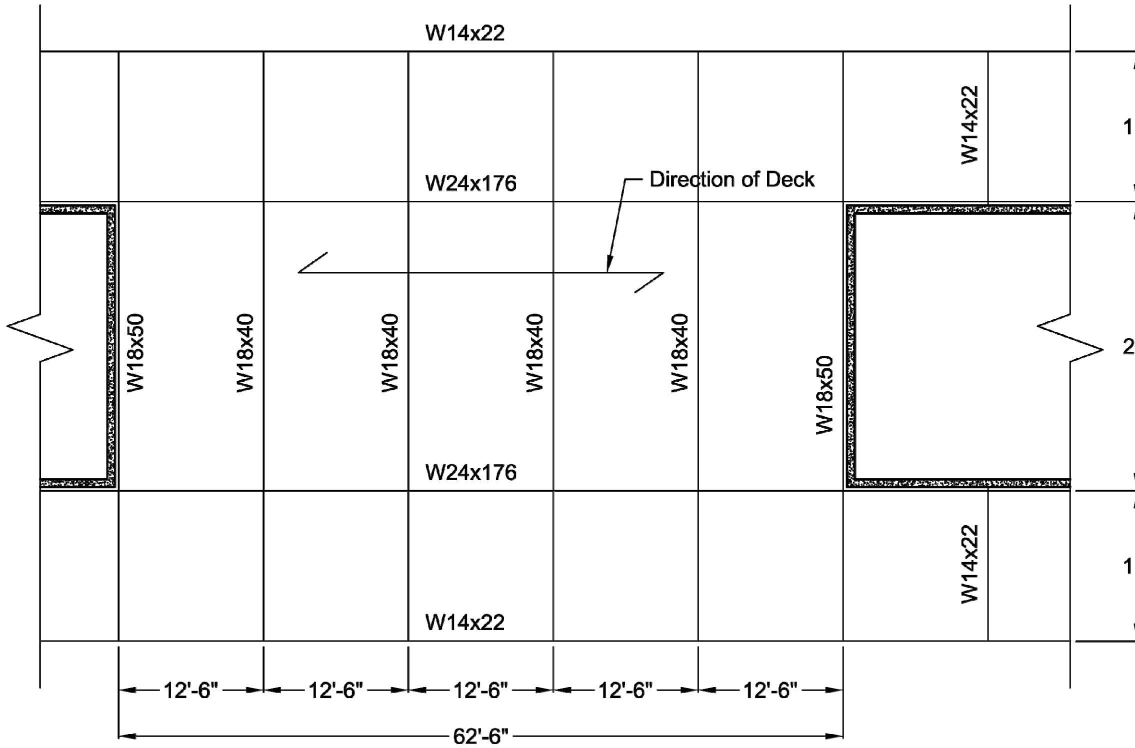


Figure 8: Existing design framing layout for the chosen typical bay

The existing floor system design consists of 3" metal deck, 20-gauge minimum, with 3.25" of lightweight concrete to meet fire safety ratings. This system sits on top of structural steel wide flanges, as previously described in the structural systems summary. The estimated floor weight is about 59 lb/square foot (as seen in Load Calculations section), making it the lightest out of the analyzed systems. The typical framing layout is featured in Figure 8, and a cross section of the floor system can be seen in Figure 9. Analysis of the existing floor system can be found in Appendix C.

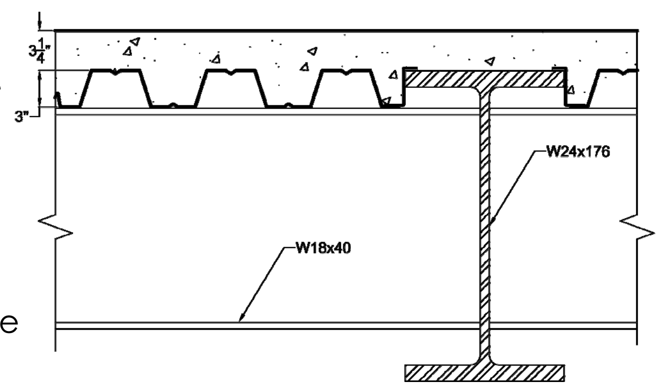


Figure 9: Cross-section of existing design floor system

The existing floor system has several advantages- the main advantage being the speed with which a floor can be erected. According to the projected schedule, half of a floor can be completed in 2 days: day 1 involves the deck placement and pouring concrete, day 2 involves fireproofing and MEP. The moment a half-floor is completed, it is elevated and fastened to the cores at its designated height. The overall cost is approximately $19.00/square foot, which is only more expensive than the alternative floor system using post-tensioned concrete. A more in-depth cost analysis can be found in Appendix G, for the floor system specified by the actual design, as well as the floor system obtained through the spot-check calculations from Technical Assignment #1 (featured in Appendix C).

This particular floor system is at a disadvantage because of the overall height of the floor assembly. 3" deck with 3.25" concrete sitting on approximately 18" tall wide flange beams leads to a floor thickness of about 2', on top of the requisite 8' of floor-to-ceiling height for habitability.

Ultimately, the existing floor system is very practical for the intended goals of this building design: low-cost construction that can be erected at high speeds.

Alternative Floor System - Non-Composite Deck

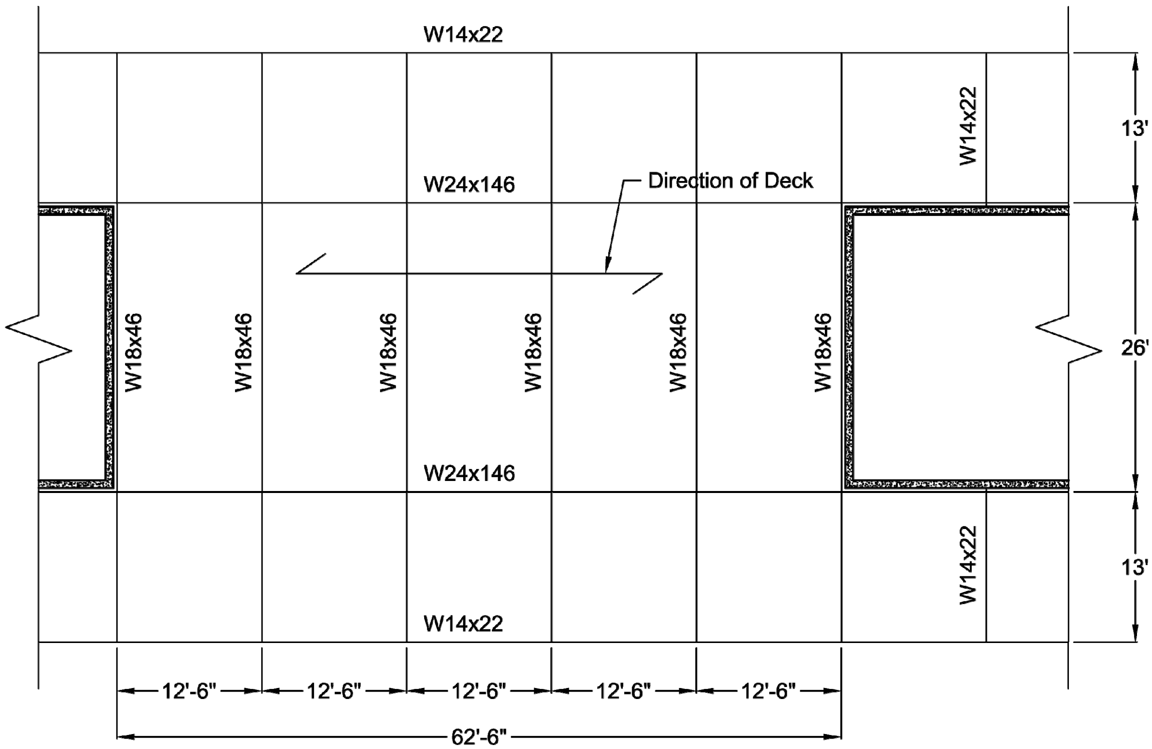


Figure 10: Non-composite (form) deck design framing layout for the chosen typical bay

The reasoning behind trying a non-composite deck as an alternative floor system was the commentary given by a member of the design team: he said that, though the existing system is a composite system, the deck does not, in reality, take advantage of the composite action. As a result of this statement, the non-composite deck analysis was carried out to compare the structural steel and metal deck sizing to the existing system.

The typical framing plan is featured in Figure 10, and a cross section of the system is featured in Figure 11. The calculations carried out for the analysis of the non-composite floor system can be found in Appendix D.

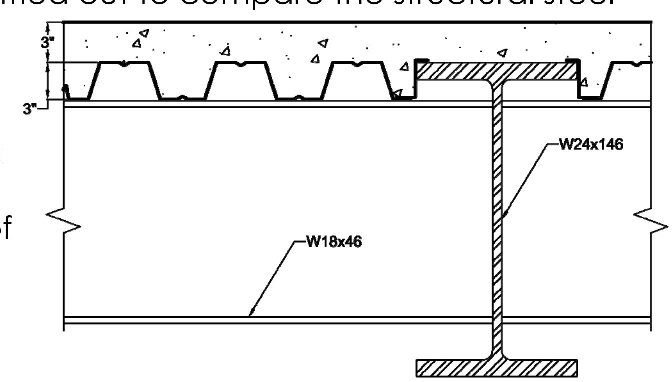


Figure 11: Cross-section of non-composite floor system

The most notable difference between the systems is the use of a much heavier gauge deck, thus resulting in a slightly more expensive system. The non-composite floor system consists of 3" form deck, 16 gauge, with approximately the same size beams (compare

W24x176 in the existing system vs. W24x146 in the non-composite system). There is a 3" normal weight concrete topping to adhere to the required fire rating, instead of the 3.25" lightweight topping used in the existing system. The overall cost is about $20.00/square foot, as compared to the existing system's cost of $19.00/square foot.

Ultimately, the analysis confirmed the statement made by the engineer on the design team for this building. As such, the non-composite system is ranked about even with the composite system in terms of its advantages and disadvantages: the non-composite system also has large floor plenums, but is quick and comparatively easy to construct. The method of construction planned for the existing system could be applied to the non-composite system as well, requiring no change of schedule. The non-composite system weighs about 73 lb/square foot, making it about 15 pounds heavier than the existing composite system. Of all of the analyzed systems, the non-composite system is only lighter than the post-tensioned slab system.

Alternative Floor System - Long-Span Deck

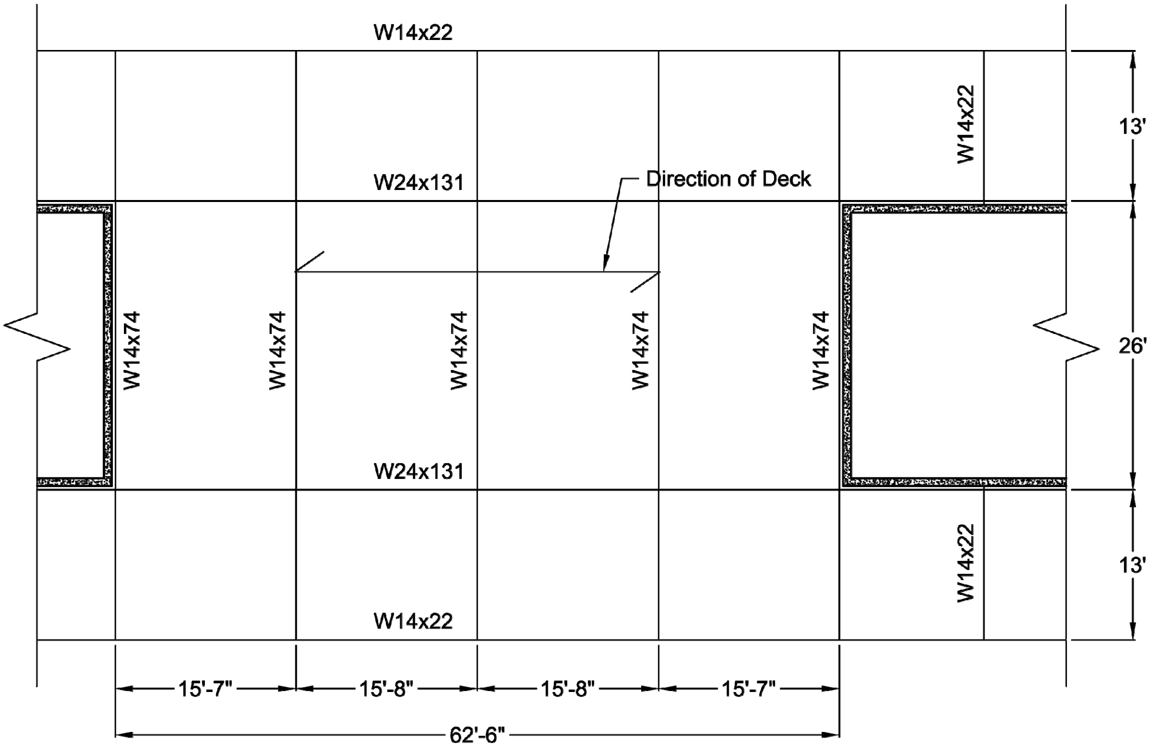


Figure 12: Long-span deck design framing layout for the chosen typical bay

The third system analyzed was a long-span deck system aimed at reducing the number of beams in the design. This goal was achieved, as can be observed in the typical framing plan shown in Figure 12. An immediate and easily observable disadvantage to long-span deck is the considerably larger deck height. In order to get rid of one of the beams in each typical 62.5' girder span (so that there are only 4 bays instead of 5), the deck had to be sized to withstand 61 lb/square foot of floor weight over 16' spans. The final deck choice was for a 6" roof and form deck, with references for allowable loads provided by Diomede Enterprises (see Appendix H for the tables). A cross-section of the deck assembly can be found in Figure 13. Additionally, calculations for the long-span deck design can be found in Appendix E.

As a result of the much larger deck, the beams had to be sized accordingly to maintain the current floor-to-floor height, which involves the use

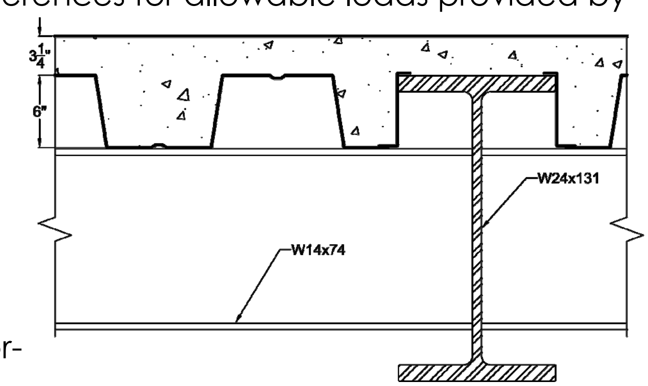


Figure 13: Cross-section of long-span deck floor system

of heavy W14's in the place of the lighter W18's used in the existing system to correlate with the height of the W24 girders. This inconvenience of sizing and necessity to invest in heavier members can be seen as a disadvantage of this system.

The long-span deck is the most expensive floor system of all of the choices analyzed, with a cost per square foot of about $27.60. This cost could be greatly reduced by increasing the floor-to-floor height to allow more room for structural steel members. With regards to constructability, this deck could be assembled much in the same way as the existing floor system, so there would be minimal (if any) hindrance to construction time and schedules. The long-span deck system is a solid floor system, but the overall cost (to maintain current floor heights) greatly cripples any advantages this system could provide, such as fewer beams (a potential, if minimal, increase in the construction speed).

Alternative Floor System - Post-Tensioned Concrete

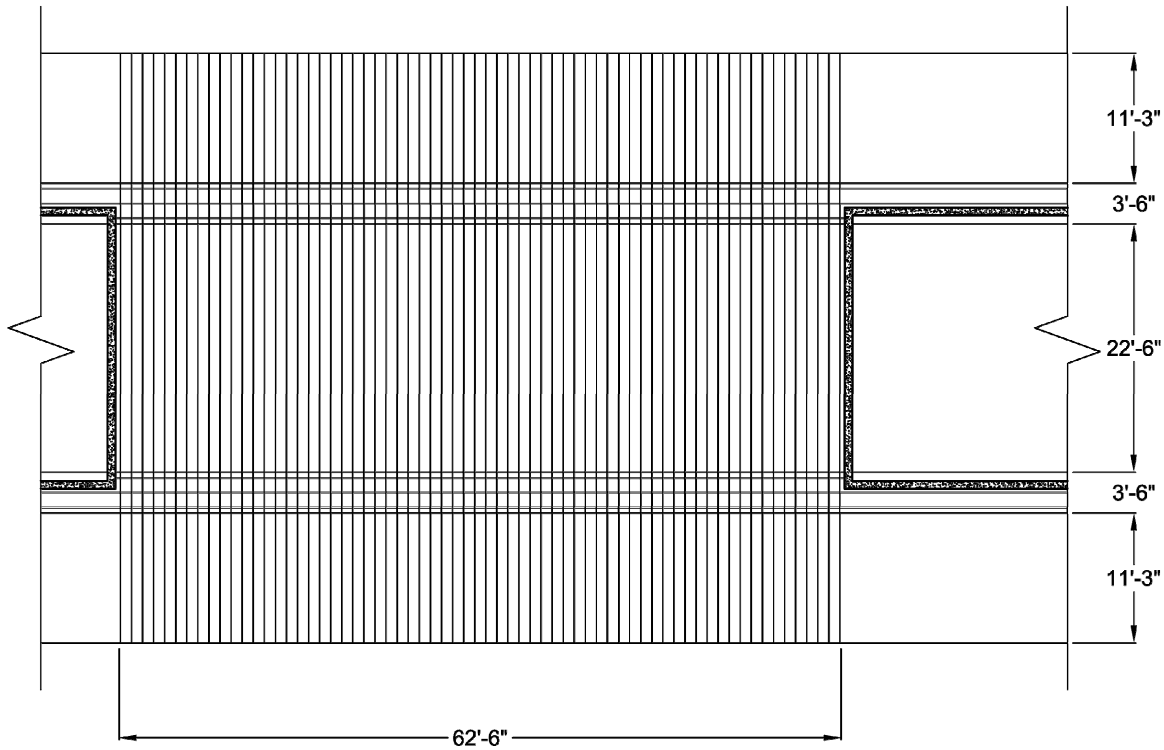


Figure 13: Post-tensioned two-way concrete slab design tendon layout for the chosen typical bay

A post-tensioned concrete system was chosen as another alternative floor system to see if an all-concrete construction would work as well as the existing system, with the potential to reduce the floor-to-floor heights or even squeeze in an extra floor for the current height. The large spans in this building required the use of wide shallow beams with banded tendons in the direction of the girders in the current design, as recommended by Dr. Lepage. In addition to these banded tendons, distributed tendons were required in the short direction (as seen in Figure 14).

Calculations for the post-tensioned concrete slab analysis can be found in Appendix F. These calculations follow an example of post-tensioned concrete design published by the Portland Cement Association, with additional information about tendon drape and wide shallow beam dimensions obtained from the Post-Tensioning Institute's Technical Note 3 and a May 2003 article from Concrete International titled "Guidelines for the Design of Post-Tensioned Floors." The formal references for these sources can be found on the "Thesis References" page of the CPEP website for this building project.

The post-tensioned concrete system was designed with the assumption that ½" diameter 7-wire, 270 ksi unbonded tendons would be used for both directions. The final post-tensioned concrete design yielded a 7" thick slab with

14" thick wide shallow beams, 42" in width. In the East-West direction (the long direction), there are (30) tendons running through the 42" wide shallow beams, banded into 3 bundles of 10 tendons. In the North-South direction (the short direction), there are 64 distributed tendons spread through the 62.5' slab width. The tendon drape profiles for both directions can also be found in Appendix F.

This particular design yielded a 14" floor thickness, which is 10" less than the existing floor system. Over the course of 20 floors, that floor thickness reduction yields about 16.5' of extra space, easily allowing for the insertion of another floor in the same building height, resulting in an increase of overall residents from 528 beds to 550 beds (an increase of 22 units). The cost of the post-tensioned system is about $9.32/square foot, making the post-tensioned concrete system the cheapest (by about $10) of the floor system choices.

Though the initial perception is that the post-tensioned system is highly economical, the costs and benefits so far mentioned do not take into account the drastically increased construction times involved in this system. The floors will no longer be lifted into the air at a rate of a floor every four days-- concrete needs to cure. The result of this would be a change in construction methodology, and a greatly extended construction schedule. The extended schedule would greatly impact any potential economic benefits of this floor system. Another thing to take into consideration when looking at this floor system is its weight- about 99 lb/square foot. The post-tensioned concrete system is by far the heaviest of the floor systems analyzed in this report, about 40 pounds heavier than the existing system, per square foot. The impact of this drastic increase in weight was not analyzed in depth in this report, but can be hypothesized to have an effect on the required foundations. The increase in the floor weight will also have an impact on the lateral system- there is potential that the current core design will be insufficient to withstand the increased design lateral forces.

Summary and Conclusions

Table 1: Comparison of each floor system design based on listed criteria

Property	Existing - Composite	Non-Composite	Long-Span Deck	Post-Tensioned Concrete
Self Weight (psf)	59.00	73.00	61.00	99.00
Foundation Impact	--	Some	None	Significant
Total Depth (in)	24.00	24.00	24.00	14.00
Constructability	Easy	Easy	Easy	Hard
Architectural Impact	--	None	None	Significant
Total Cost per ft^2 ($)	19.00	19.97	27.60	9.32
Lateral System Impact = none for all systems, they are independent of the lateral system				
Additional Study?	--	No	**Yes**	**Yes**

The Southwest Student Housing building was designed with a particular construction philosophy. Namely, this philosophy demands easy, modular construction that can be completed at a very high speed. The existing system works very well with that philosophy, as it allows for a whole floor to be lifted into place every 4 days. The two alternative deck systems also operate on similar principles, and as a result, the construction philosophy can be implemented with non-composite and long-span deck designs as well.

The post-tensioned concrete system, on the other hand, poses several problems. The first of these problems is the need for the concrete to cure before it can be lifted into place; this necessity would suck up valuable time and extend the construction schedule, potentially negating any economic advantage it has over the other systems (namely it's comparatively low cost per square foot). The second problem has to do with current laws and practices in the United States: lift-slab construction has been deemed dangerous due to a fatal accident that killed 28 construction works (L'Ambiance Plaza).

This second problem also has potential implications with the other systems, since they operate on the same principle of building the floor on the ground and jacking it up to its required location. Another potential pitfall of the post-tensioned system is the large self-weight, which might require an alteration to the current foundation design. The weight of the post-tensioned system is about 40% higher than the weight of the existing system, which is a significant increase in the building weight. This increase in building weight also has potential to affect the lateral system design, because base shear forces would be greater, thus causing a potential need for thicker and more heavily reinforced cores.

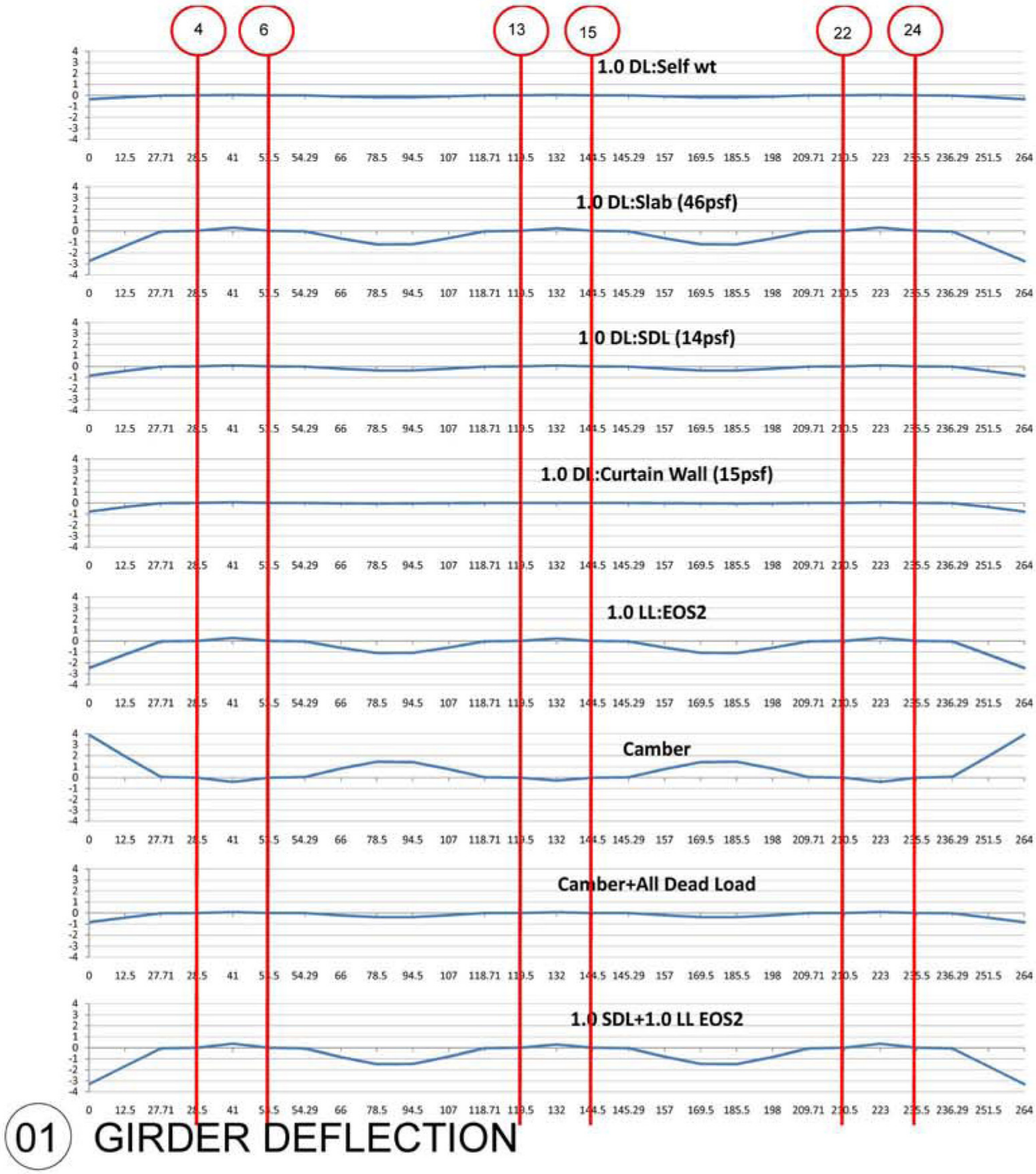
Despite the downfalls of the post-tensioned system, it does have a great economic benefit because of its small floor thickness and low cost (it could provide base construction savings of about $2.5 million over the whole building, compared to the existing system). Further study is needed to ascertain whether the economic advantages outweigh the system disadvantages.

The other floor system that is worth addition study is the long-span system. There are no regional building requirements in Tempe, Arizona for the building's zone, which means that there is no real limit on the floor-to-floor heights. There are certainly fiscal and structural reasons to limit floor-to-floor height, but it becomes reasonable to alter them if a system is deemed advantageous enough. The long-span deck system can reduce the structural steel requirements on every floor by at least 200 linear feet, which allows for the system to have an overall potential cost savings compared to the existing system, given free reign to alter floor heights.

Ultimately, the existing system is a good choice for the current building design. The current design could potentially be improved by using long-span deck or post-tensioned concrete slab, but there can be no definitive answer until additional study is carried out. With current analyses, as presented in the Appendices, the existing system is the best design with regards to weight, and theoretical construction speed and ease.

Appendices

Appendix A – Building Information Notes



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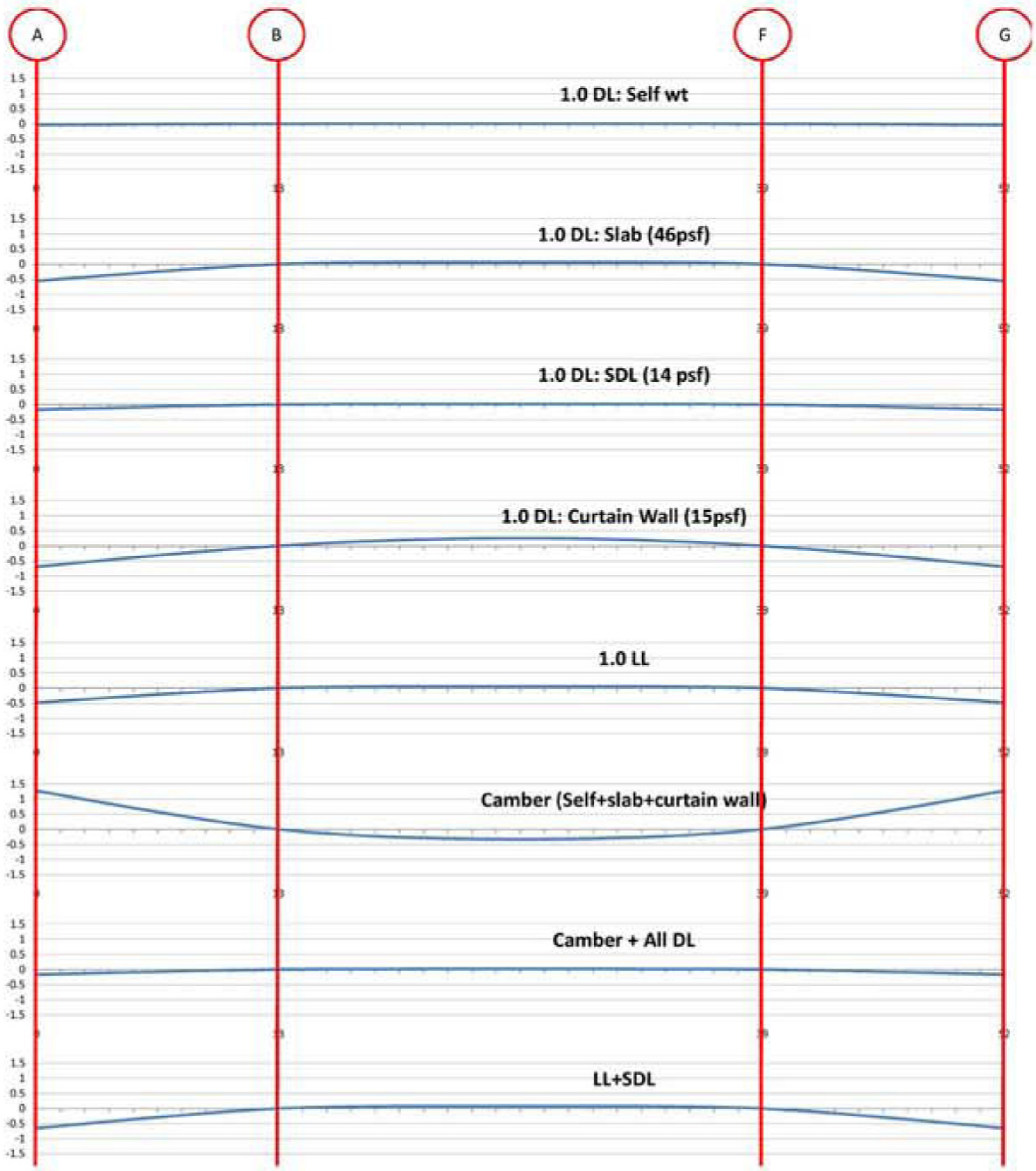

02 BEAM DEFLECTION

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TECH 1 | APPENDICES - APPENDIX A | SOUTHWEST STUDENT HOUSING | 2

BUILDING INFORMATION

- MODEL CODE: IBC 2006 AS AMENDED BY THE CITY OF TEMPE, ARIZONA
- DESIGN CODES: AISC "SPEC FOR STRUCTURAL STL BLDGS" AISC 360-05
  ACI "BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONC" ACI 318-05
- STRUCTURAL STANDARDS: $\boxed{\text{ASCE 7-05}}$

I WILL USE ASCE 7-05, AISC 13TH ED, ACI 318-05, IBC 06

- DEFLECTION CRITERIA
  - STRUCTURAL STEEL IS ALL CAMBERED TO DEAL W/ HIGH DEFLECTIONS.

OUT OF CURIOSITY - HOW DOES ONE ASSEMBLE A CAMBERED STRUCTURAL STEEL FLOOR IF THE METHOD IS TO SLIDE BEAMS THROUGH PLASMA-CUT HOLES ON HYDRAULIC ROLLERS?

CHARLIE'S NOTES: $\frac{L}{360}$ FOR LIVE $\frac{L}{240}$ FOR TOTAL $\frac{H}{400}$ FOR DRIFT

- STRUCTURAL OVERVIEW:
  - FOUNDATIONS - ~~[illegible]~~ SPREAD FOOTING ACCORDING TO CHARLIE
    ACCORDING TO SOILS REPORT RECOMMENDATIONS -
    BLDG WILL EXERT SIGNIFICANT LOADS TO FOUNDATION ELEMENTS.
    → EMPLOY DEEP FOUNDATION SYSTEM TO LIMIT SETTLEMENTS
    - DRILLED PIERS + MAT

ACCORDING TO TFG PACKAGE: (3) 350 YD3 CORE MATS, PERIMETER GRADE BEAMS, SLAB ON GRADE ON TOP

AXIAL CAPACITY = SKIN FRICTION OF SHAFT + END BEARING AT TIP

SOILS:

	DEPTH	FRICTION	END BEARING
I SILTY SAND + POORLY GRADED SAND	10-35'	0.4 KSF	—
II SAND GRAVEL COBBLE	35-100'	2.5 KSF	30 KSF

PIER SHAFTS SHOULD PENETRATE AT LEAST 2.5× PIER ϕ INTO II LAYER & NO PIER $\phi < 12"$

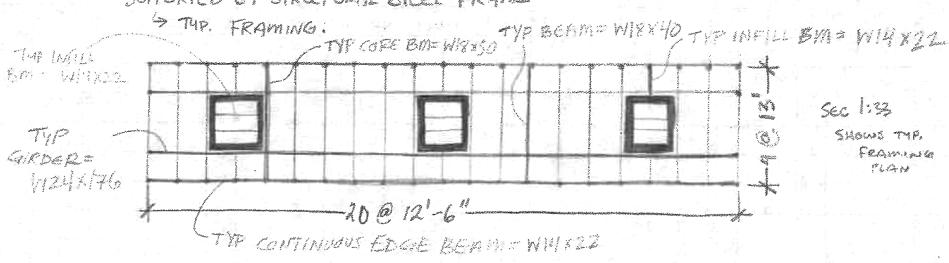
MIN CLEAR SPACING = 3 × BIGGEST ADJACENT PIER ϕ

PREDICTION: FOR ISOLATED PIER, $\phi < 60"$, SETTLEMENT $\leq 1"$

CHARLIE'S NOTES: BLDG DOESN'T HAVE LOCAL? SETTLING B/C SPANS ARE LONGER → NO DIFFERENTIAL SETTLING

- FLOOR SYSTEM - TYP. FOR ALL FLOORS - 3.25" LIGHTWEIGHT CONCRETE ON 3" DECK
  FOR HAND CALCS, CHARLIE ASSUMED 4.5" LIGHTWEIGHT CONCRETE
  ~~WHEELING CORRUGATED 3.0 SB LW.C.~~
  (P 14 IN PDF) 20 GAGE
  GO W/ VULCRAFT

SUPPORTED BY STRUCTURAL STEEL FRAME
→ TYP. FRAMING:



SEC 1:33 SHOWS TYP. FRAMING PLAN

- FRAMING SYSTEM - SEE ABOVE FOR FLOOR FRAMING SYSTEM
  SEE BELOW FOR GRAVITY FRAMING SYSTEM
- LATERAL SYSTEM - (3) CONCRETE CORES: 8" THICK, 25' × 25' ON CENTER (KIND OF)
  SPACED 62'-6" APART (STARTING AT CENTER)

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AE Consultant: Dr. Andres Lepage

Appendix B – Gravity Load Calculations

CALCULATED LOADS

★GRAVITY

– CONSTRUCTION DEAD LOAD

- DECK – 3.0 VLI (VULCRAFT COMPOSITE DECK) 20 GAGE
  3.25" LIGHTWEIGHT CONCRETE
  DECK WEIGHT = 2.14 PSF
  CONCRETE WEIGHT = 46 PSF
  TOTAL = 48.14 PSF

ADDITIONAL NOTES FOR LATER
MAX UNSHORED SPAN
1 SPAN = 10'-6"
2 SPAN = 12'-10"
3 SPAN = 13'-3"
MAX SPAN ON PLAN? 12'-6"
(ASSUME BELOW DECK ORIENTATION)
MAX SUPERIMPOSED LIVE LOAD ON MAX SPAN? 73 PSF
← VERIFY THAT THIS IS LL ONLY
(P55 OF NUCOR DECK MAN.)

- STRUCTURAL STEEL – ASSUME ≈ ~~8 PSF~~
  CHECK CURRENT SIZES:

SIZE	LENGTH	#	WT (K)
W18X50	52	6	15.6 K
W18X40	52	12	25 K
W14X22	13	6	1.7 K
W24X176	262.5	2	92.4 K
W14X22	262.5	2	11.6 K
			146.3 K

TYP FLOOR DIMENSIONS:
250' × 52' = 13 000 SF

APPROX. WEIGHT OF STRUCTURAL STL:

$$\frac{146.3 \times 10^3}{13000} = 11.25 \text{ PSF}$$

STRUCTURAL STEEL – ASSUME $\boxed{11 \text{ PSF}}$ ✓

→ TOTAL CONSTRUCTION DEAD LOAD = 48.14 + 11 = 59.14 → **59 PSF**

– SUPERIMPOSED DEAD LOAD

→ ASSUME SDL OF **15 PSF** ← ASSUMPTION BY ENGINEERS. SHOULD HAVE CONFIRMED – PARTITIONS NOT INCLUDED.

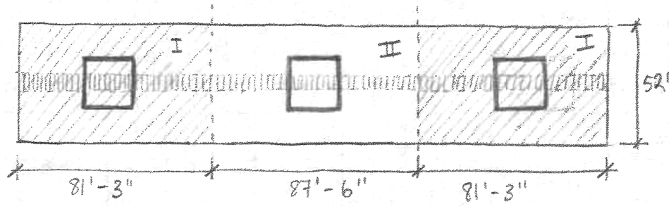
– LIVE LOAD

- RESIDENTIAL = 40 PSF
- PARKING = 40 PSF
- CORRIDORS = $\boxed{80}$ ON FLOORS ABOVE GROUND (PSF)
  = 100 ON GROUND FLOOR (PSF)

(THERE IS A 6' WIDE CORRIDOR RUNNING THRU THE CENTER OF THE BUILDING IN THE LONG DIRECTION)

★ENGINEERS JUST TOOK LL TO BE 40 PSF

LIVE LOAD REDUCTION:



3

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AE Consultant: Dr. Andres Lepage

CALCULATED LOADS (CONTINUED)

REDUCTION FACTORS:

SECTION	AREA/FLOOR	# FLOORS	FACTOR
I	~4390 SF	1	0.5
		>1	@2, 0.41 @>2, 0.4
II	4725 SF	1	0.5
		>1	0.4

} WILL JUST ROUND TO 0.4

$L = L_0\underbrace{\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)}$ $K_{LL} = 1$

≥ 0.5 (1 FLR)
≥ 0.4 (>1 FLR)

SAMPLE CALCULATIONS

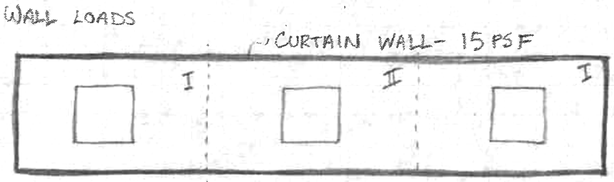
SECTION I, 1 FLOOR:
$0.25 + \frac{15}{\sqrt{1 \times 4390}} = 0.476 \rightarrow 0.5$

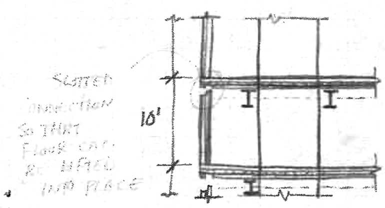
>1 FLOOR:
2? $0.25 + \frac{15}{\sqrt{1 \times 4390 \times 2}} = 0.41$
3? $0.25 + \frac{15}{\sqrt{1 + 4390 \times 3}} = 0.37 \rightarrow 0.4$

$\boxed{80\text{ PSF}} \times 0.5 = \underline{40\text{ PSF}}$ (FOR CORE CALCULATIONS)

— MECHANICAL EQUIPMENT
ON GROUND FLOOR, W/ 2-PIPE SYSTEM FOR HEATING/COOLING ROOMS
CHILLER, PUMPS, ETC. ON GROUND FLOOR

— WALL LOADS





EACH FLOOR SUPPORTS ITS RESPECTIVE "CURTAIN WALL"
∴ FLOORS 2-19 HAVE 10' PERIMETER CURTAIN WALLS @ 15 PSF

— SNOW LOADS
$P_S = 0.7\ C_e C_t I\ P_g \geq 0$
(FLAT ROOF)

$P_S = 0$

FIGURE 7-1 (ASCE 7-05)

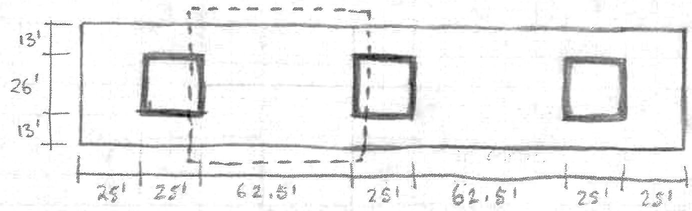
$P_g =$ (5000) 10, (4600) 5, (3500) ZERO

TEMPE, AZ IS @ AN ELEVATION OF... 1140-1495 FEET

$1495 < 3500 \rightarrow P_g = 0$

TECH II

FLOOR SYSTEM CALCULATIONS



DESIGNING FOR THE FLOOR SPAN ENCLOSED IN THE DOTTED LINE.

- ASSUMPTIONS: THE SUPPORTS PROVIDED BY EACH CORE ARE PARTIALLY FIXED → (USE $M = \frac{wl^2}{11}$) ← AS PER RECOMMENDATN OF DR. LEPAGE
- LOADS: CONSTRUCTION LOADS NEED TO BE RECALCULATED
  FIRE RATING FOR FLOOR – 2 HOURS, MIN

  SUPERIMPOSED DEAD LOAD = 15 PSF
  LIVE LOAD (NOT REDUCED) = 80 PSF ← SMALL POTENTIAL FOR REDUCTION ON CERTAIN BEAMS; IT IS SIMPLER TO USE 80 PSF FOR EVERYTHING.

  LATERAL LOADS – WIND WOULD BE UNAFFECTED BY DIFFERENT FLOOR SYSTEMS UNLESS THE FLOOR-TO-FLOOR HEIGHT CHANGED. SEISMIC LOADS WOULD NEED TO BE RECALCULATED TO DEAL WITH DIFFERENT FLOOR ASSEMBLY WEIGHT.

FLOOR SYSTEMS TO TEST:
- NONCOMPOSITE (SAME BEAM/GIRDER LAYOUT)
- LONG SPAN DECK (REMOVE ONE OF THE BEAMS FRAMING INTO THE GIRDER)
- HOLLOW CORE (SAME BEAM/GIRDER LAYOUT)
- POST-TENSIONED SLAB (WIDE GIRDERS + BEAMS, MOST LIKELY SAME LAYOUT)

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Appendix C – Existing Floor System: Composite Deck

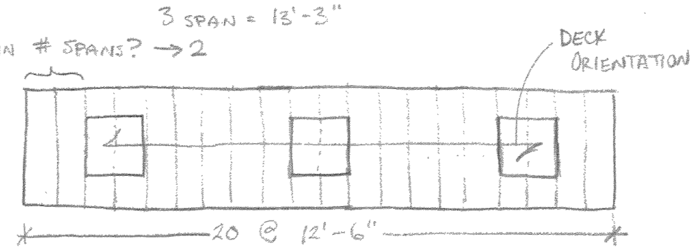
SPOT CHECKS

* METAL DECKING (SEE P55 OF VULCRAFT STEEL DECK MANUAL)

$3\frac{1}{4}$" LIGHTWEIGHT CONCRETE (110 PCF)
3", 20 GAGE DECK

MAX UNSHORED SPAN: 1 SPAN = 10'-6"
2 SPAN = 12'-10"
3 SPAN = 13'-3"

MIN # SPANS? → 2



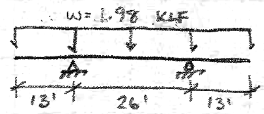
12'-10" ≤ MAX UNSHORED SPAN ≤ 13'-3"
TYP. SPAN = 12'-6" < 12'-10" OK

MAX SUPERIMPOSED LIVE LOAD: MAX SPAN = 12'-6"
MAX LIVE LOAD = 73 PSF
NONREDUCIBLE? → 80 PSF (6' WIDE CORRIDOR)
+ 15 PSF SUPERIMPOSED DEAD LOAD
95 PSF > 73 PSF NG

SUPERSIZE IT!
MAX 18 GAGE (3 VLI 18) SLL = 169 PSF OK (3 VLI 19 = 84 PSF) NG

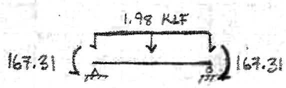
USE 3" DECK (18 GAGE, MINIMUM) w/ $3\frac{1}{4}$" LIGHTWEIGHT CONCRETE, 2 SPANS MINIMUM, & 12'-6" / SPAN

SPOT CHECKS (CONTINUED)
*TYPICAL BEAM



w = FACTORED LOAD = $1.2D + 1.6L$
TRIBUTARY WIDTH = 12.5' (AREA | BM = 650 SF)
→ REDUCIBLE LIVE LOAD

$$L = (80\text{ PSF})\left(0.25 + \frac{15}{\sqrt{4 \times 650}}\right) = 43.5\text{ PSF}$$

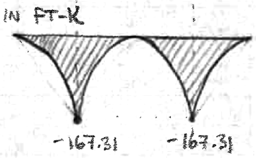


$$w = 1.2(59+15) + 1.6(43.5) = 158.5\text{ PSF}$$
$$w = w \times 12.5 = 1.98\text{ KLF}$$

$$M = \frac{wL^2}{2} = \frac{(1.98)(13^2)}{2} = 167.31\text{ FT-K}$$

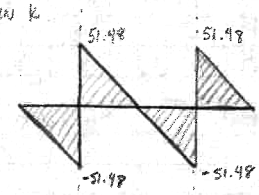
$$M \text{ @ MIDSPAN} = \frac{wL^2}{8} - 167.31 = \frac{1.98\,(26^2)}{8} - 167.31 = 0$$

M IN FT-K



$\underline{\underline{M_u = 167.31\text{ FT-K}}}$

V IN K



$\underline{\underline{V_u = 51.48\text{ K}}}$

$$b_{eff} = \begin{cases} \frac{1}{8}\,(\text{SPAN}) & = \frac{1}{8}(26') = 3.25' \leftarrow \\ \frac{1}{2}\,(\text{SPACING}) & = \frac{1}{2}\,(12.5') = 6.25' \\ \text{EDGE DISTANCE} & = 13' \end{cases}$$

ASSUME $a = 1''$ → $Y_2 = 6.25'' - \frac{1}{2}'' = 5.75''$

TRY W10 X15

$$\phi M_n = 174 + \frac{182-174}{2} = 178\text{ FT-K @ TFL}$$
(INTERPOLATION)

$$a = \frac{221}{0.85\,(4)(3.25 \times 12)} = 1.67 > 1'' \quad \underline{NG}$$

TRY W10 X22

$$\phi M_n = 169 + \frac{174-169}{2} = 171.5\text{ FT-K @ BFL}$$

$$a = \frac{117}{0.85\,(4)(3.25 \times 12)} = 0.882'' < 1'' \quad \underline{OK}$$

$$\frac{117}{17.2} = 7\text{ SHEAR STUDS} \times 2 = 14\text{ STUDS}$$

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SPOT CHECKS (CONTINUED)

DEFLECTIONS? $\frac{L}{360}$ (LIVE) $\frac{L}{240}$ (TOTAL)

$$\Delta_{\text{SIMPLY SUPPORTED W/ END MOMENTS}} = \frac{w(L/2)}{24EI}\left[\left(\frac{L}{2}\right)^3 - (2L)\left(\frac{L}{2}\right)^2 + \left(\frac{12M}{w}\,\frac{L}{2}\right) + L^3 - \frac{4ML}{w}\right]$$

156^3 $\quad \frac{312^3}{2}$ $\quad 1972\frac{M}{w}$ $\quad 312^3$ $\quad 1248\frac{M}{w}$

$$I = 311 + \frac{329-311}{2} = 320 \text{ in}^4$$

$$\Delta_{\text{CANTILEVERED}} = \frac{wL^4}{8EI}$$

LIVE: 43.5 PSF → $w = 0.544$ KLF

TOTAL: 43.5 + 59 + 15 PSF → $w = 1.469$ KLF

END SPANS:

$$\frac{L}{360} = \frac{13 \times 12}{360} = 0.43''$$

$$\Delta_{LIVE} = \frac{(0.544/12)\,13^4}{8(29000)\,320} \times 1728 = 0.03'' \quad \text{OK}$$

$$\frac{L}{240} = \frac{13 \times 12}{240} = 0.65''$$

$$\Delta_{TOTAL} = \frac{(1.469/12)\,13^4}{8(29000)\,320} \times 1728 = 0.081'' \quad \text{OK}$$

MIDDLE SPAN:

$$\frac{L}{360} = 0.86''$$

$$M_{LIVE} = 45.968 \text{ FT-K} = 551.616 \text{ IN-K}$$

$$\Delta_{LIVE} = \frac{(0.544/12)(156'')}{24(29000)\,320}\left[156^3 - \frac{312^3}{2} + 1972\left(\frac{551.616}{0.544/12}\right) + 312^3 - 1248\left(\frac{551.616}{0.544/12}\right)\right] = 0.84'' \quad \text{OK}$$

$$\frac{L}{240} = 1.3''$$

$$M_{TOTAL} = 496.522 \text{ FT-K} = 5958.3 \text{ IN-K}$$

$$\Delta_{TOTAL} = 4.23'' \quad \text{NG}$$

NOTE – THESE DEFLECTIONS DO NOT TAKE INTO ACCOUNT GIRDER DEFLECTIONS

QUICK DEFLECTION CALCS FOR SPECIFIED BM SIZE ON TYP. FRAMING PLAN:

W18×40 ASSUME $a = 2''$ → $Y_2 = 5.25''$ PNA #6 $\phi M_n = 465$ FT-K OK

$a = 1.584'' < 2''$ OK $\quad I = 1190 \text{ in}^4$

END SPANS:

$\Delta_{LIVE} = 0.008''$ OK

$\Delta_{TOTAL} = 0.022''$ OK

MIDDLE SPAN:

$\Delta_{LIVE} = 0.226''$ OK

$\Delta_{TOTAL} = 1.137''$ OK

(WILL CHECK OVERALL DEFLECTION INCLUDING GIRDER IN NEXT SPOT CHECK)

AMPAD

7

Typical Girder Calculations

MOMENT DISTRIBUTION- FACTORED LOADS (k-ft)										
Joint	A	B		C		D		E		F
Distribution Factor	0.500	0.714	0.286	0.286	0.714	0.714	0.286	0.286	0.714	0.500
Fixed End Moment	1575.00	-1575.00	1406.25	-1406.25	225.00	-225.00	1406.00	-1406.00	1575.00	-1575.00
Balance	-787.500	120.488	48.263	337.838	843.413	-843.234	-337.766	-48.334	-120.666	787.500
Carry Over	60.244	-393.750	168.919	24.131	-421.617	421.706	-24.167	-168.883	393.750	-60.333
Balance	-30.122	160.530	64.302	113.681	283.805	-283.843	-113.696	-64.312	-160.555	30.167
Carry Over	80.265	-15.061	56.840	32.151	-141.922	141.902	-32.156	-56.848	15.083	-80.278
Balance	-40.132	-29.831	-11.949	31.394	78.376	-78.359	-31.387	11.945	29.820	40.139
Carry Over	-14.915	-20.066	15.697	-5.974	-39.179	39.188	5.972	-15.694	20.069	14.910
Balance	7.458	3.119	1.250	12.914	32.240	-32.245	-12.916	-1.251	-3.124	-7.455
Carry Over	1.560	3.729	6.457	0.625	-16.122	16.120	-0.626	-6.458	-3.728	-1.562
Balance	-0.780	-7.273	-2.913	4.432	11.065	-11.063	-4.431	2.913	7.272	0.781
Carry Over	-3.636	-0.390	2.216	-1.457	-5.531	5.533	1.457	-2.216	0.391	3.636
Balance	1.818	-1.304	-0.522	1.999	4.989	-4.990	-1.999	0.522	1.303	-1.818
Carry Over	-0.652	0.909	0.999	-0.261	-2.495	2.495	0.261	-0.999	-0.909	0.652
Balance	0.326	-1.363	-0.546	0.788	1.968	-1.968	-0.788	0.546	1.363	-0.326
Carry Over	-0.681	0.163	0.394	-0.273	-0.984	0.984	0.273	-0.394	-0.163	0.681
Balance	0.341	-0.398	-0.159	0.359	0.897	-0.897	-0.359	0.159	0.398	-0.341
Carry Over	-0.199	0.170	0.180	-0.080	-0.449	0.449	0.080	-0.180	-0.170	0.199
Balance	0.099	-0.250	-0.100	0.151	0.377	-0.377	-0.151	0.100	0.250	-0.099
Carry Over	-0.125	0.050	0.076	-0.050	-0.189	0.189	0.050	-0.076	-0.050	0.125
Balance	0.062	-0.089	-0.036	0.068	0.170	-0.170	-0.068	0.036	0.089	-0.062
Carry Over	-0.045	0.031	0.034	-0.018	-0.085	0.085	0.018	-0.034	-0.031	0.045
Balance	0.022	-0.047	-0.019	0.029	0.074	-0.074	-0.029	0.019	0.047	-0.022
Carry Over	-0.023	0.011	0.015	-0.009	-0.037	0.037	0.009	-0.015	-0.011	0.023
Balance	0.012	-0.019	-0.007	0.013	0.033	-0.033	-0.013	0.007	0.019	-0.012
Carry Over	-0.009	0.006	0.007	-0.004	-0.016	0.016	0.004	-0.007	-0.006	0.009
Total	**848**	**-1756**	**1756**	**-854**	**854**	**-854**	**854**	**-1755**	**1755**	**-848**

MOMENT DISTRIBUTION-LIVE LOADS (k-ft)										
Joint	A	B		C		D		E		F
Distribution Factor	0.500	0.714	0.286	0.286	0.714	0.714	0.286	0.286	0.714	0.500
Fixed End Moment	285.00	-285.00	254.00	-254.00	41.00	-41.00	254.00	-254.00	285.00	-285.00
Balance	-142.500	22.134	8.866	60.918	152.082	-152.082	-60.918	-8.866	-22.134	142.500
Carry Over	11.067	-71.250	30.459	4.433	-76.041	76.041	-4.433	-30.459	71.250	-11.067
Balance	-5.534	29.125	11.666	20.480	51.128	-51.128	-20.480	-11.666	-29.125	5.534
Carry Over	14.562	-2.767	10.240	5.833	-25.564	25.564	-5.833	-10.240	2.767	-14.562
Balance	-7.281	-5.336	-2.137	5.643	14.088	-14.088	-5.643	2.137	5.336	7.281
Carry Over	-2.668	-3.641	2.822	-1.069	-7.044	7.044	1.069	-2.822	3.641	2.668
Balance	1.334	0.585	0.234	2.320	5.792	-5.792	-2.320	-0.234	-0.585	-1.334
Carry Over	0.292	0.667	1.160	0.117	-2.896	2.896	-0.117	-1.160	-0.667	-0.292
Balance	-0.146	-1.305	-0.523	0.795	1.984	-1.984	-0.795	0.523	1.305	0.146
Carry Over	-0.652	-0.073	0.397	-0.261	-0.992	0.992	0.261	-0.397	0.073	0.652
Balance	0.326	-0.232	-0.093	0.358	0.895	-0.895	-0.358	0.093	0.232	-0.326
Carry Over	-0.116	0.163	0.179	-0.046	-0.447	0.447	0.046	-0.179	-0.163	0.116
Balance	0.058	-0.244	-0.098	0.141	0.353	-0.353	-0.141	0.098	0.244	-0.058
Carry Over	-0.122	0.029	0.071	-0.049	-0.176	0.176	0.049	-0.071	-0.029	0.122
Balance	0.061	-0.071	-0.028	0.064	0.161	-0.161	-0.064	0.028	0.071	-0.061
Carry Over	-0.036	0.031	0.032	-0.014	-0.080	0.080	0.014	-0.032	-0.031	0.036
Balance	0.018	-0.045	-0.018	0.027	0.068	-0.068	-0.027	0.018	0.045	-0.018
Carry Over	-0.022	0.009	0.014	-0.009	-0.034	0.034	0.009	-0.014	-0.009	0.022
Balance	0.011	-0.016	-0.006	0.012	0.031	-0.031	-0.012	0.006	0.016	-0.011
Carry Over	-0.008	0.006	0.006	-0.003	-0.015	0.015	0.003	-0.006	-0.006	0.008
Balance	0.004	-0.008	-0.003	0.005	0.013	-0.013	-0.005	0.003	0.008	-0.004
Carry Over	-0.004	0.002	0.003	-0.002	-0.007	0.007	0.002	-0.003	-0.002	0.004
Balance	0.002	-0.003	-0.001	0.002	0.006	-0.006	-0.002	0.001	0.003	-0.002
Carry Over	-0.002	0.001	0.001	-0.001	-0.003	0.003	0.001	-0.001	-0.001	0.002
Total	**154**	**-317**	**317**	**-154**	**154**	**-154**	**154**	**-317**	**317**	**-154**

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AE Consultant: Dr. Andres Lepage

Southwest Student Housing

Tempe, Arizona

Technical Assignment #1

MOMENT DISTRIBUTION- TOTAL LOADS										
Joint	A	B		C		D		E		F
Distribution Factor	0.500	0.714	0.286	0.286	0.714	0.714	0.286	0.286	0.714	0.500
Fixed End Moment	810.00	-810.00	723.00	-723.00	116.00	-116.00	723.00	-723.00	810.00	-810.00
Balance	-405.000	62.118	24.882	173.602	433.398	-433.398	-173.602	-24.882	-62.118	405.000
Carry Over	31.059	-202.500	86.801	12.441	-216.699	216.699	-12.441	-86.801	202.500	-31.059
Balance	-15.530	82.609	33.090	58.418	145.840	-145.840	-58.418	-33.090	-82.609	15.530
Carry Over	41.305	-7.765	29.209	16.545	-72.920	72.920	-16.545	-29.209	7.765	-41.305
Balance	-20.652	-15.311	-6.133	16.123	40.252	-40.252	-16.123	6.133	15.311	20.652
Carry Over	-7.656	-10.326	8.062	-3.067	-20.126	20.126	3.067	-8.062	10.326	7.656
Balance	3.828	1.617	0.648	6.633	16.559	-16.559	-6.633	-0.648	-1.617	-3.828
Carry Over	0.808	1.914	3.317	0.324	-8.280	8.280	-0.324	-3.317	-1.914	-0.808
Balance	-0.404	-3.735	-1.496	2.275	5.680	-5.680	-2.275	1.496	3.735	0.404
Carry Over	-1.867	-0.202	1.138	-0.748	-2.840	2.840	0.748	-1.138	0.202	1.867
Balance	0.934	-0.668	-0.268	1.026	2.562	-2.562	-1.026	0.268	0.668	-0.934
Carry Over	-0.334	0.467	0.513	-0.134	-1.281	1.281	0.134	-0.513	-0.467	0.334
Balance	0.167	-0.700	-0.280	0.405	1.010	-1.010	-0.405	0.280	0.700	-0.167
Carry Over	-0.350	0.084	0.202	-0.140	-0.505	0.505	0.140	-0.202	-0.084	0.350
Balance	0.175	-0.204	-0.082	0.185	0.461	-0.461	-0.185	0.082	0.204	-0.175
Carry Over	-0.102	0.087	0.092	-0.041	-0.230	0.230	0.041	-0.092	-0.087	0.102
Balance	0.051	-0.128	-0.051	0.078	0.194	-0.194	-0.078	0.051	0.128	-0.051
Carry Over	-0.064	0.026	0.039	-0.026	-0.097	0.097	0.026	-0.039	-0.026	0.064
Balance	0.032	-0.046	-0.018	0.035	0.087	-0.087	-0.035	0.018	0.046	-0.032
Carry Over	-0.023	0.016	0.018	-0.009	-0.044	0.044	0.009	-0.018	-0.016	0.023
Balance	0.011	-0.024	-0.010	0.015	0.038	-0.038	-0.015	0.010	0.024	-0.011
Carry Over	-0.012	0.006	0.008	-0.005	-0.019	0.019	0.005	-0.008	-0.006	0.012
Balance	0.006	-0.010	-0.004	0.007	0.017	-0.017	-0.007	0.004	0.010	-0.006
Carry Over	-0.005	0.003	0.003	-0.002	-0.008	0.008	0.002	-0.003	-0.003	0.005
Total	**436**	**-903**	**903**	**-439**	**439**	**-439**	**439**	**-903**	**903**	**-436**

Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

SPOT CHECKS (CONTINUED)

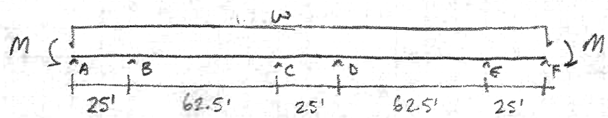
*TYPICAL GIRDER (INTERIOR)

TRIB. WIDTH = 19.5'

$W = \frac{51.48 \times 21}{250'} = 4.32$ KLF (FACTORED)

CONSERVATIVE ESTIMATE: 51.48 KLF LOAD/POINT (21 BMS FRAME INTO EACH GIRDER, 12'-6" SPACING → UNIF LOAD?)

MOMENT DISTRIBUTION (SEE ATTACHED EXCEL SHEET)



$W_{FACTORED} = 4.32$ KLF
$W_{LIVE} = 0.78$ KLF
$W_{TOTAL} = 2.22$ KLF

$M = \frac{WL^2}{2} = \frac{4.32(25')^2}{2} = 1350$ K-FT ← FACTORED

$M_{LIVE} = 243.75$ K-FT

$M_{TOTAL} = 693.75$ K-FT

$$DF = \frac{1/L_{\text{DIRECTION OF CHOICE}}}{1/L_{LEFT} + 1/L_{RIGHT}}$$

(EI IS CONSTANT THRU-OUT)

$$FEM = \frac{WL^2}{12}$$

DISTRIBUTION FACTORS:
AB = 0.5
BA = 0.714
BC = 0.286
CB = 0.286
CD = 0.714
DC = 0.714 (SYMMETRIC)
DE = 0.286
ED = 0.286
EF = 0.714
FE = 0.5

FIXED END MOMENTS: (K-FT)

JOINT	FACTORED	LIVE	TOTAL
AB	225+1350	41+244	116+694
BC	1406.25	254	723
CD	225	41	116
DE	1406.25	254	723
EF	225+1350	41+244	116+694

3 EXCEL SHEETS INCLUDED REGARDING MOMENT DISTRIBUTION:
- FACTORED LOADS
- LIVE LOAD ONLY } (UNFACTORED)
- TOTAL LOAD }

*SEE TYP. BM. SPOT CHECK FOR LOADS USED IN TYP. GIRDER SPOT CHECK.

DESIGN LOADS:
$M_u = 1756$ K-FT

LIVE LOAD REDUCTIONS: $A = 4875$ SF

$(80\ PSF)\left(0.25 + \frac{15}{\sqrt{4 \times 4875}}\right) = 0.36\,(80\ PSF) \rightarrow L = 40$ PSF

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SPOT CHECKS (CONTINUED)

ASSUME $a = 3.25'' \rightarrow a/2 = 4.625''$

$b_{eff} = \frac{1}{8}(\text{SPAN}) = \frac{1}{8}(62.5) = 7.813' = 93.75''$ ←

$\frac{1}{2}(\text{SPACING}) = \frac{1}{2}(25) = 12.5'$

EDGE DISTANCE = 25'

TRY W27×102

$$\phi M_n = 1810 + (1840-1810)\left(\frac{4.625-4.5}{5-4.5}\right) = 1817.5 \text{ ftk} @ \text{PNA} \#4$$

$$a = \frac{878}{0.85(4)(93.75)} = 2.75 < 3.25'' \text{ OK}$$

$$\frac{878}{17.2} = 51 \text{ SHEAR STUDS} \times 2 = 102 \text{ STUDS}$$

ENGINEER'S PRESCRIBED BEAM IS NOT IN COMPOSITE BM TABLE IN AISC '10 STEEL CONSTRUCTION MANUAL (TABLE 3-19) BUT IT'S A HEFTY W24×176, WHICH IS PROBABLY AS BEEFY OR MORE BEEFY THAN MY W27×105

DEFLECTIONS? $I = 7230 + 0.25(7430-7230) = 7280 \text{ in}^4$

(SYMMETRIC)

1 2 3 4 / 3 2 1

SPAN 1:

$\frac{L}{360} = 0.833$ $\Delta_{LIVE} = \frac{(0.78/12)25^4}{8(29000)7280} \times 1728 = 0.026''$ OK

$\frac{L}{240} = 1.25''$ $\Delta_{TOTAL} = \frac{(2.22/12)25^4}{8(29000)7280} \times 1728 = 0.074''$ OK

$$\Delta_x = \frac{wx}{24EI}\left[x^3 - \left(2L + \frac{4M_1}{wL} - \frac{4M_2}{wL}\right)x^2 + \frac{12M_1}{w}x + L^3 - \frac{8M_1L}{w} - \frac{4M_2L}{w}\right]$$

(SINCE END MOMENTS ARE UNEQUAL... TAKEN FROM CASE 32, AISC STEEL CONSTRUCTION MANUAL, 2010, 14TH ED)

$$\Delta_{MAX} @ x = \frac{L}{2} + \frac{M_1 - M_2}{wL}$$

SPAN 2:
$x = 250.3''$ $\Delta_{LIVE} = 0.06''$ $\Delta_{TOTAL} = 0.17''$

SPAN 3:
$x = 415.1''$ $\Delta_{LIVE} = 0.33''$ $\Delta_{TOTAL} = 0.94''$

SPAN 4:
$x = 150''$ $\Delta_{LIVE} = 0.07''$ $\Delta_{TOTAL} = 0.19''$

OK

SPOT CHECKS (CONTINUED)

NEW TYPICAL BEAM DEFLECTIONS: (W18×40)

END SPANS:

$\Delta_{LIVE} = 0.008 + 0.33 = 0.338"$ OK

$\Delta_{TOTAL} = 0.022 + 0.94 = 0.962"$ NG (CAMBERED)

MIDDLE SPAN:

$\Delta_{LIVE} = 0.226 + 0.33 = 0.556"$ OK

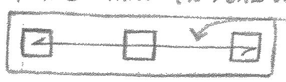
$\Delta_{TOTAL} = 1.137 + 0.94 = 2.077"$ NG (CAMBERED)

Appendix D – Alternative Floor System: Non-Composite Deck

NONCOMPOSITE FLOOR SYSTEM

FIRE REQUIREMENTS: 2HR, EXPOSED GRID ⇒ 2-½" NW w/ 0.6C, 1.0C, 1.3C, 1.5C
3" NW w/ 0.6C, 1.0C, 1.3C, 1.5C

MIN 2 SPANS (TO DEAL W/ CANTILEVER)



DECK ORIENTATION
TYP. SPAN = 12.5'

WILL GO W/ 2-½" NW → INITIAL SIZING: (BASED ON MAX UNSHORED CONDITION)

0.6C22 – 4' 7" (2 & 3 SPAN)
1.0C20 – 7' 3" (2 & 3 SPAN)
1.3C20 – 8' 4" (2 & 3 SPAN)
1.5C18 – 9' 9" (2 SPAN) 10' 1" (3 SPAN) } NG
$\boxed{2C16}$ – 12' 6" (2 SPAN) 12' 11" (3 SPAN) OK

ASSUME FLOOR WILL BE COATED W/ A FINISH (AFFECTS ALLOWABLE SLAB LOAD)

*INITIAL SIZING FROM VULCRAFT STEEL DECK CATALOG

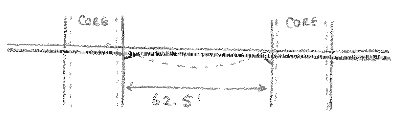
2C16 CANNOT HOLD LOADS ON SUCH WIDE SPANS
↓
3C16 W/ 3" NW?
MAX ALLOWABLE UNIFORM LOAD FOR $\delta \le \frac{\ell}{240}$ = 100 PSF

LOADS:
- CONSTRUCTION:
  DECK WEIGHT = 58 PSF
  ASSUMED STRUCTURAL STL = 15 PSF
  TOTAL = 58 + 15 = 73 PSF
- SUPERIMPOSED: 15 PSF

TOTAL DEAD LOAD = 73 + 15 = 88 PSF
TOTAL LIVE LOAD = 80 PSF (COULD POTENTIALLY BE REDUCED BY ~7 PSF)
TOTAL SUPERIMPOSED UNIFORM LOAD = 15 PSF + 80 PSF = 95 PSF < 100 PSF OK

↳ USE 3" CONFORM DECK, 16 GAGE, WITH 3" N.W. CONCRETE
2 SPANS, MIN. TYP SPAN = 12'-6"

GIRDER DESIGN:



REDUCTIONS:
$$(80\text{ psf})\left(0.25 + \frac{15}{\sqrt{4 \times 1625}}\right) = 0.44\,(80)$$
↳ LIVE LOAD = 0.5(80) = 40 PSF

TRIBUTARY WIDTH = 26' ⟶ A = 26(62.5) = 1625 SF
↳ ASSUMING THE EDGE GIRDER PROVIDES NO SUPPORT, THUS NO REDUCTION IN TRIB.

$$w_{FACTORED} = \left[1.2(88) + 1.6(40)\right] \times 26' \cong 4.41\text{ KLF}$$
$$w_{LIVE} = (40\text{ PSF}) \times 26' = 1.04\text{ KLF}$$
$$w_{TOTAL} = \left[88 + 40\right] \times 26' = 3.33\text{ KLF}$$

Southwest Student Housing

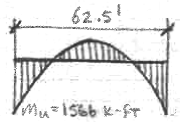
Tempe, Arizona

Technical Assignment #1

NONCOMPOSITE FLOOR SYSTEM (CONT'D)

GIRDER DESIGN (CONT'D)

MAX MOMENT $\approx \frac{w\ell^2}{11} \rightarrow M_u = \frac{(4.41)(62.5)^2}{11} = 1566$ K-FT



→ MAINTAIN SAME FLOOR HEIGHTS ∴ W24 X ___

FROM TABLE 3-2 OF AISC STEEL CONSTRUCTION MANUAL, 13TH ED.:

$\phi_b M_{px} = 1570$ K-FT FOR W24×146 $> M_u = 1566$ K-FT

OK

DEFLECTIONS

$I = 4580 \text{ in}^4$ $\quad \delta_{max} = \frac{w\ell^4}{384EI}\left(\frac{12}{11}\right) = \frac{w\ell^4}{352EI}$

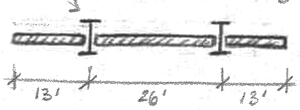
$\delta_{LIVE} = \frac{(1.04/12)(62.5\times12)^4}{352(29000)(4580)} = 0.587"$ $\quad \frac{\ell}{360} = 2.08"$ OK

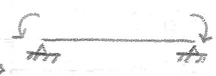
$\delta_{TOTAL} = \frac{(3.33/12)(62.5\times12)^4}{352(29000)(4580)} = 1.878"$ $\quad \frac{\ell}{240} = 3.125"$ OK

→ USE W24×146 FOR 62'-6" SPAN BETWEEN CORES (AKA GIRDER)

BEAM DESIGN:

CONTINUOUS OVER BOTH SUPPORTS





⇒ END MOMENT: $\frac{w\ell^2}{2}$ (END LOADED CANTILEVER)

FOR UNIFORM LOAD.

TO INCLUDE CURTAIN WALL:

10' × 12.5' × 15 PSF = 1.875 K END LOAD

+ $P\ell$

TRIB. WIDTH = 12.5'

→ 12.5 × 52 = 650 SF

$(80)\left(0.25 + \frac{15}{\sqrt{4\times650}}\right) = 0.544(80) = 43.5$ PSF

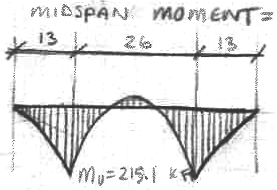
$w_{FACTORED} = [1.2(88) + 1.6(43.5)] \times 12.5' = 2.2$ KLF

$w_{LIVE} = 43.5 \times 12.5' = 0.544$ KLF

$w_{TOTAL} = (88+43.5)\times12.5' = 1.644$ KLF

END MOMENTS $= \frac{(2.2)(13')^2}{2} + 1.875(13') = 215.1$ K-FT

MIDSPAN MOMENT $= \frac{w\ell^2}{8} - \frac{2(M_{END})}{2} = \frac{(2.2)(26)^2}{8} - 215.1 = -29.8$ K-FT



$M_u = 210.3$ K-FT

FOR EASE OF APPLYING DECK.

→ MAINTAIN SAME FLOOR HEIGHTS ∴ W18 X ___

$\phi_b M_{px} = 249$ K-FT FOR W18 × 35 $> M_u = 210.3$ K-FT

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NONCOMPOSITE FLOOR SYSTEM (CONT'D)

BEAM DESIGN (CONT'D)

DEFLECTIONS

$I = 510 \text{ in}^4$

$$\delta_x = \frac{wx}{24EI}\left[x^3 - \left(2l + \frac{4M_1}{wl} - \frac{4M_2}{wl}\right)x^2 + \frac{12M_1}{w}x + l^3 - \frac{8M_1 l}{w} - \frac{4M_2 l}{w}\right]$$

$M_1 = M_2$, $x = \frac{l}{2}$

$$M_{LIVE} = \frac{(0.544)(13)^2}{2} + \cancel{1.875(13)} = 46 \text{ k-ft}$$

$$\delta_x = \frac{wl}{48EI}\left[\frac{l^3}{8} - \frac{l^3}{2} + \frac{16Ml}{w} + l^3 - \frac{12Ml}{w}\right]$$

$M_{TOTAL} = 163.3$ k-ft

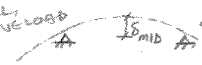
$$\delta_{LIVE} = \frac{(0.544/12)(26\times12)}{48(29000)(510)}\left[(26\times12)^3 - \frac{3(26\times12)^3}{8} - \frac{6(46\times12)}{(0.544/12)}(26\times12)\right] = -0.076''$$

$$\delta_{TOTAL} = \frac{(1.644/12)(26\times12)}{48(29000)(510)}\left[(26\times12)^3 - \frac{3(26\times12)^3}{8} - \frac{6(163.3\times12)}{(1.644/12)}(26\times12)\right] = -0.469''$$

$\frac{l}{360} = 0.867'' > |-0.076''|$ OK

$\frac{l}{240} = 1.3'' > |-0.469''|$ OK

(DEFLECTIONS ARE NEGATIVE BECAUSE THE END MOMENTS ARE LARGE ENOUGH TO CAUSE THE BEAM TO CURVE UP @ MIDSPAN:



CURTAIN WALL, NOT LIVE LOAD

← EXAGGERATED. MIGHT NEED TO CHECK CANTILEVER DEFLECTIONS)

$$\delta_{CANTILEVER} = \frac{wl^4}{8EI} + \frac{Pl^3}{3EI}$$

$$\delta_{LIVE,END} = \frac{(0.544/12)(13\times12)^4}{8(29000)(510)} + \cancel{\frac{1.875(13\times12)^3}{3(29000)(510)}} = 0.227'' < \frac{13\times12}{360} = 0.433''\ \text{OK}$$

$$\delta_{TOTAL,END} = \frac{(1.644/12)(13\times12)^4}{8(29000)(510)} + \frac{1.875(13\times12)^3}{3(29000)510} = 0.846'' > \frac{13\times12}{240} = 0.65\ \text{NG}$$

W18x46? $I = 712$

- MIDSPAN –

$\delta_{LIVE} = (-0.076)\left(\frac{510}{712}\right) = |-0.054''| < 0.867''$ OK

$\delta_{TOTAL} = (-0.469)\left(\frac{510}{712}\right) = |-0.336''| < 1.3''$ OK

- END SPAN –

$\delta_{LIVE} = (0.227)\left(\frac{510}{712}\right) = 0.163'' < 0.433''$ OK

$\delta_{TOTAL} = (0.846)\left(\frac{510}{712}\right) = 0.606'' < 0.65''$ OK

→ USE W18x46 FOR BEAMS @ 12'-6" SPACING.

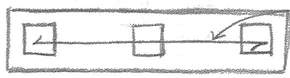
Appendix E – Alternative Floor System: Long-Span Deck

1/3

LONG SPAN DECK

FIRE REQUIREMENTS: 2HR, EXPOSED GRID → $3\frac{1}{4}$" L.W. CONCRETE

MIN 2 SPANS (TO DEAL W/ CANTILEVER)

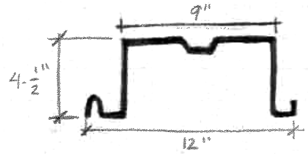


DECK ORIENTATION

GOAL: ELIMINATE A FEW BEAMS

LOOKED AT DIAMEDE ENTERPRISES ROOF & FORM DECK

$4\frac{1}{2}$" DEEP DECK:



MAX SPAN = ?

↳ $S.W.: \left(\underbrace{\frac{3.25" \times 12"}{144} + \frac{3" \times 4.5"}{144}}_{\text{X-SEC. AREA}}\right) / 1\,FT \times 110\,PCF = 40.1\,PSF$ [CONCRETE]

MAX DECK S.W = 5.68 PSF ⇒ 40.1 + 5.68 = 45.78 ≈ 46 PSF

LOADS:

- CONSTRUCTION –
  - DECK WEIGHT = 46 PSF
  - ALLOWANCE FOR STRUCTURAL STL = 15 PSF
- SUPERIMPOSED – 15 PSF
- LIVE – 80 PSF

FOR DECK, TOTAL (DL+LL) UNIFORM LOAD = 46 + 15 + 80 = 141 PSF

↳ NO $4\frac{1}{2}$" DECK GAGE WORK FOR SINGLE SPAN > 12'

∴ 6" DECK?

NEW S.W: $\left(\frac{3.25" \times 12"}{144} + \frac{3" \times 6"}{144}\right) / 1\,FT \times 110\,PCF = 43.5\,PSF$ [CONCRETE]

MAX 6" DECK WEIGHT = 6.61 PSF ⇒ 43.5 + 6.61 = 50.2 PSF

NEW TOTAL (DL + LL) UNIFORM LOAD = 50.2 + 15 + 80 = 145.2 PSF

6" DEEP DECK, 14 GAUGE: FOR STRESS & DEFLECTION $\left(\frac{L}{240}\right)$, LOAD = 161 PSF FOR SINGLE 16' SPAN.

→ USE 6" DEEP DECK, 14 GAUGE, W/ $3\frac{1}{4}$" L.W. CONCRETE.
TYP. SPAN ≤ 16' AS APPROPRIATE FOR BAY.

$\frac{62.5}{16}$ ≈ 4 SPANS (1 LESS BEAM PER 62.5' SECTION BETWEEN CORES)

↑ ABOVE DECK IS NON COMPOSITE (FORM).

WILL NEED TO ACCOMODATE +3" OF DECK IN FLOOR HEIGHT OR BEAM SIZING.

Ksenia Tretiakova, Structural Option
AE Consultant: Dr. Andres Lepage

LONG SPAN DECK (CONT'D)

GIRDER DESIGN:

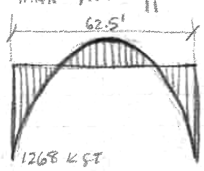
SPAN = 62.5' → LIVE LOAD REDUCTION TO 40 PSF (SEE NONCOMPOSITE CALCS)

$W_{FACTORED} = \left[1.2(46+15+15) + 1.6(40)\right] \times 26' = 3.57$ KLF

$W_{LIVE} = (40) \times 26 = 1.04$ KLF

$W_{TOTAL} = (40+46+15+15) \times 26 = 3.02$ KLF

MAX $M \approx \frac{w\ell^2}{11} \rightarrow M_u = \frac{(3.57)(62.5)^2}{11} = 1268$ K-FT



→ MAINTAIN SAME FLOOR HEIGHTS ∴ W24 × ___

TABLE 3-2 OF AISC STEEL CONSTRUCTION MANUAL, 13TH ED. ∴

$\phi_b M_{px} = 1390$ FT-K FOR W24×131 $> M_u = 1268$ K-FT OK

DEFLECTIONS

$I_x = 4020$ IN^4 $\quad \delta_{max} = \frac{w\ell^4}{352EI}$

$\delta_{LIVE} = \frac{(1.04/12)(62.5\times12)^4}{352(29000)(4020)} = 0.679''$ $\quad \frac{\ell}{360} = 2.08''$ OK

$\delta_{TOTAL} = \frac{(3.02/12)(62.5\times12)^4}{352(29000)(4020)} = 1.94''$ $\quad \frac{\ell}{240} = 3.125''$ OK

BEAM DESIGN:

M (———) $M = \frac{w\ell^2}{2} + P\ell$ $\quad P = 1.875$

TRIB WIDTH ≈ 16' → (16' × 52') = 832 SF → $(80)\left(0.25 + \frac{15}{\sqrt{4\times832}}\right) = 0.51(80) = 40.8$ PSF

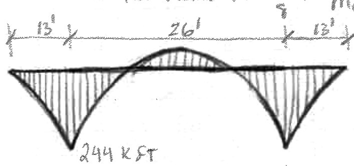
$W_{FACTORED} = \left[1.2(46+15+15) + 1.6(40.8)\right] \times 16' = 2.504$ KLF

$W_{LIVE} = (40.8) \times 16 = 0.653$ KLF

$W_{TOTAL} = (40.8+46+15+15) \times 16 = 1.869$ KLF

END MOMENT $M_u = \frac{(2.504)(13')^2}{2} + (1.875\times1.2)(13') = 244$ KFT

MIDSPAN MOMENT $= \frac{w\ell^2}{8} - M_{END} = \frac{(2.504)(26)^2}{8} - 244 = -32.4$ KFT



6" DECK, NOT 3"

→ MAINTAIN FLOOR HEIGHTS → W14 × ___

$\phi_b M_{px} = 383$ K-FT FOR W14×61 $> M_u = 244$ K-FT

$M_{LIVE} = \frac{(0.653)(13)^2}{2} = 55.2$ K-FT

$M_{TOTAL} = \frac{(1.869)(13^2)}{2} + 1.875(13) = 182.3$ K-FT

$I_x = 640$ IN^4

$\delta_x = \frac{w\ell}{48EI}\left[\frac{5}{8}\ell^3 - \frac{6M\ell}{w}\right]$ @ MID SPAN

SIMPLIFIED B/C END MOMENTS ARE SYMMETRIC & MAX MIDSPAN δ IS IN THE VERY MIDDLE $\left(\frac{\ell}{2}\right)$

3/3

LONG SPAN DECK (CONT'D)

BEAM DESIGN (CONT'D)

DEFLECTIONS

MIDSPAN

$$\delta_{LIVE} = \frac{(0.653/12)(26\times12)}{48(29000)(640)}\left[\frac{5}{8}(26\times12)^3 - \frac{6(55.2\times12)}{0.653/12}(26\times12)\right] = |-0.073''| < 0.867 \text{ OK}$$

$$\delta_{TOTAL} = \frac{(1.869/12)(26\times12)}{48(29000)(640)}\left[\frac{5}{8}(26\times12)^3 - \frac{6(182.3\times12)}{1.869/12}(26\times12)\right] = |-0.399''| < 1.3'' \text{ OK}$$

$$\delta_{END} = \frac{w\ell^4}{8EI} + \frac{P\ell^3}{3EI}$$ (AS APPROPRIATE)

$$\delta_{LIVE} = \frac{(0.653/12)(13\times12)^4}{8(29000)(640)} = 0.217'' < 0.43\bar{3} \text{ OK}$$

$$\delta_{TOTAL} = \frac{(0.869/12)(13\times12)^4}{8(29000)(640)} + \frac{1.875(13\times12)^3}{3(29000)(640)} = 0.749'' > 0.65'' \text{ NG}$$

W14x74? $I = 795 \text{ in}^4$

- MIDSPAN

$$\delta_{LIVE} = (-0.073)\left(\frac{640}{795}\right) = |-0.059''| < 0.867'' \text{ OK}$$

$$\delta_{TOTAL} = (-0.399)\left(\frac{640}{795}\right) = |-0.321''| < 1.3'' \text{ OK}$$

- END SPAN

$$\delta_{LIVE} = (0.217)\left(\frac{640}{795}\right) = 0.175'' < 0.43\bar{3}'' \text{ OK}$$

$$\delta_{TOTAL} = (0.749)\left(\frac{640}{795}\right) = 0.603'' < 0.65'' \text{ OK}$$

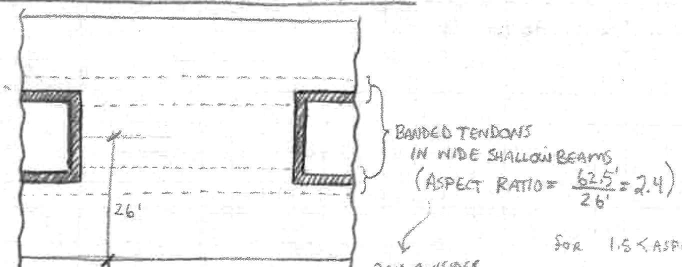
→ USE W14x74 FOR BEAMS @ $\frac{62.5}{4} \cong 15'-6''$ SPACING (4 BAYS / 62.5' SPAN)

AMPAD

Appendix F – Alternative Floor System: Post-Tensioned Concrete

1/6

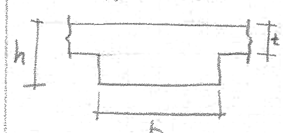
POST-TENSIONED CONCRETE SLAB (2-WAY)



CAN CONSIDER INITIAL SLAB t SIZING FROM SHORTER DIRECTION, CAN USE EQUIVALENT FRAME METHOD FOR ANALYSIS

FOR 1.5 < ASPECT RATIO < 2.5, RECOMMENDATION IS TO USE WIDE SHALLOW BEAMS (PTI TECH NOTE #3)

DIMENSIONAL RESTRICTIONS ON WIDE SHALLOW BMS:



$h \leq 2t \quad h \leq \frac{b}{3}$

(SEE FIGURE 5.1-1 OF TECH NOTE)

MATERIALS:

CONCRETE – NW (150 PCF) $f'_c = 4000$ PSI $\quad f'_{ci} = 3000$ PSI

REBAR – $F_y = 60\,000$ PSI

POST-TENSIONING – UNBONDED

$\frac{1}{2}$" ϕ 7-WIRE $\rightarrow A = 0.153$ IN^2

$f_{pu} = 270\,000$ PSI (ASSUME 30 KSI PRESTRESS LOSS)

$f_{py} = 243\,000$ PSI

$f_{pe} = 0.7\,f_{pu} - \text{LOSSES} = 0.7(270) - 30 = 159\,000$ PSI

$\hookrightarrow P_{eff} = A\,f_{pe} = 24.327$ K / TENDON

INITIAL SIZING:

FOR FULLY CONTINUOUS – $\frac{\ell}{t} \leq 45 \quad t \geq \frac{\ell}{45} \quad t \geq 6.933$ IN $\rightarrow t = 7$ IN

$\therefore h \leq 2t \rightarrow h \leq 2(7) = 14$, SAY $h = 14$ IN

$h \leq \frac{b}{3} \rightarrow 14 \leq \frac{b}{3}$, SAY $b = 42$ IN

LOADS / LOADING:

DEAD – SELF WEIGHT = $\frac{\text{BM SW}}{\text{TRIB}}$ + SLAB SW = $\frac{150 \text{ PCF}}{12 \text{ IN/FT}}\left[\frac{(14-7)(42)}{26 \times 12} + 7"\right] = 99.3$ PSF

SUPERIMPOSED = 15 PSF

LIVE = 80 PSF $\rightarrow$ 26' × 62.5' = 1625 SF $\quad (80)\left(0.25 + \frac{15}{\sqrt{1625 \times 1.0}}\right) = 0.622\,(80) \cong 50$ PSF

$\frac{\text{LIVE}}{\text{DEAD}} = \frac{50}{114.3} = 0.44 < 0.75$ (NO PATTERN LOADING REQUIRED)

ASSUME GROSS SECTION PROPERTIES FOR CLASS U

2/6

POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

DESIGN PARAMETERS:

ALLOWABLE STRESSES – @ JACKING – $0.6 f'_{ci} = 0.6(3000) = 1800$ PSI [COMPRESSION]
$3\sqrt{f'_{ci}} = 164$ PSI [TENSION]

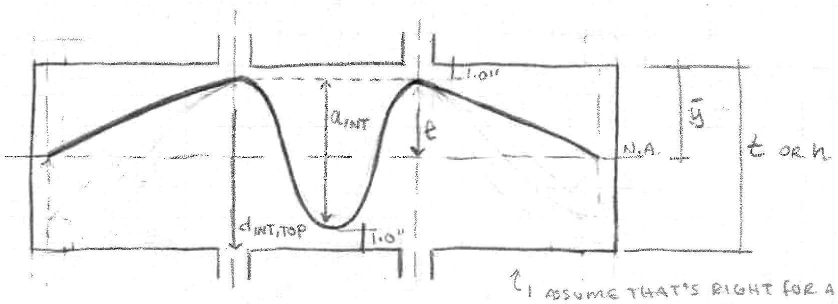
@ SERVICE – $0.45 f'_c = 0.45(4000) = 1800$ PSI [COMPRESSION]
$6\sqrt{f'_c} = 379.5$ PSI [TENSION]

AVERAGE PRECOMPRESSION LIMITS – $\frac{P}{A} = 125$ PSI (MIN)
$= 300$ PSI (MAX. FOR ECONOMICAL REASONS)

TARGET LOAD BALANCES – 0.6–0.8 → 0.8 × (SELF WEIGHT) = 0.8 (99.3) = 79.44 PSF

COVER – TOP = 0.75"
BOTTOM = 0.75" (RESTRAINED)
= 1.5" (UNRESTRAINED)
} ADD $\frac{\phi}{2}$ TO C_c

TENSION PROFILE (HEIGHTS FOR REFERENCE) –



↑ I ASSUME THAT'S RIGHT FOR A CANTILEVER?

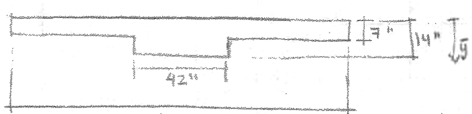
EAST-WEST DIRECTION DESIGN:

PRESTRESS FORCE TO BALANCE – 80% S.W.

$w_{BAL} = (79.44 \text{ PSF}) \times 26' = 2065.44$ PLF

FOR E-W DIRECTION, ONLY LOOKING @ INTERIOR BAY.

↳ $P = \frac{w_{BAL}\, \ell^2}{8\, a_{INT}}$



$A = 7" \times 312" + 7" \times 42" = 2478 \text{ IN}^2$

$\bar{y} = \frac{(7\times312)(3.5) + (7\times42)(10.5)}{7\times312 + 7\times42} = 4.33"$ (FROM TOP)

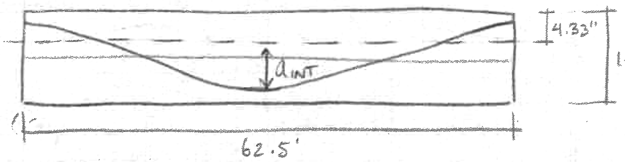
$$I = \left[\frac{312(7)^3}{12} + (312\times7)(4.33-3.5)^2\right] + \left[\frac{42(7)^3}{12} + (42\times7)(10.5-4.33)^2\right] = 22815.3 \text{ IN}^4$$

$S_{TOP} = \frac{I}{y_{TOP}} = \frac{22815.3}{4.33} = 5269 \text{ IN}^3$ $\quad S_{BOTT} = \frac{I}{y_{BOTT}} = \frac{22815.3}{14-4.33} = 2359 \text{ IN}^3$

3/6

POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

E-W TENDON PROFILE IN WIDE SHALLOW BEAM:



SLAB CONTINUES OFF ON BOTH ENDS, ∴ TENDONS DON'T NEED TO RETURN TO THE NEUTRAL AXIS, AS THEY ARE CONTINUOUS

$a_{INT} = 14 - 4.33 - 1.0 = 8.67"$

SO $P = \frac{(2065.44)(62.5)^2}{8(8.67/12)} = 1395.9$ KIPS

$\frac{1395.9}{24.327} = 57.4$ → MAX ACCEPTABLE TENDON AREA: $\frac{P}{A} = 300$ PSI $P = 300\,(A)$

$300(2478) = 743.4$ K

↳ $\frac{743.4}{24.327} = 30$ TENDONS

$P_{ACTUAL} = 30\,(24.327) = 729.8$ K

$W_b = \frac{P_{ACTUAL}\,(8 \times a_{INT})}{l^2} = \frac{729.8\,(8 \times 8.67/12)}{62.5^2} = 1.08$ KLF

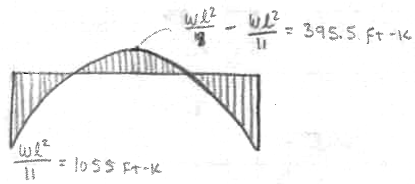
$W_{S.W.} = 2.06$ KLF

$\frac{W_b}{W_{S.W.}} = \frac{1.08}{2.06} = 0.523 < 1.0$ OK

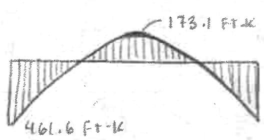
$P_{EFF} = 729.8$ K $\quad \frac{P}{A} = \frac{729.8}{2478} = 0.2945$ KSI

SLAB STRESSES:

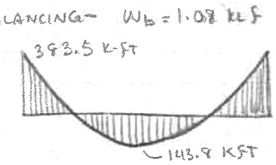
DEAD – $W_D = (99.3 + 15)(26') = 2.97$ KLF



LIVE – $W_L = (50)(26') = 1.3$ KLF



BALANCING – $W_b = 1.08$ KLF



IMMEDIATELY AFTER JACKING:

• MIDSPAN:

$f_{TOP} = \frac{(-395.5 + 143.8)}{5269} - 0.2945 = -342$ PSI

$f_{BOTT} = \frac{(395.5 - 143.8)}{-2359} - 0.2945 = -188$ PSI

• SUPPORTS:

$f_{TOP} = \frac{(1055 - 383.5)}{5269} - 0.2945 = -182$ PSI

$f_{BOTT} = \frac{(-1055 + 383.5)}{2359} - 0.2945 = -579$ PSI

WITHIN CLASS U DESIGN PARAMETERS, OK

4/6

POST - TENSIONED CONCRETE (E-W AY) (CONT'D)

STRESSES AT SERVICE LOAD:

- MIDSPAN:

$$f_{Top} = \frac{(-395.5 - 173.1 + 143.8)}{5269} - 0.2945 = -375 \text{ PSI}$$

$$f_{Bott} = \frac{(395.5 + 173.1 - 143.8)}{2359} - 0.2945 = -114 \text{ PSI}$$

- SUPPORTS:

$$f_{Top} = \frac{(1055 + 461.6 - 383.5)}{5269} - 0.2945 = -79 \text{ PSI}$$

$$f_{Bott} = \frac{(-1055 - 461.6 + 383.5)}{2359} - 0.2945 = -774 \text{ PSI}$$

WITHIN CLASS U DESIGN PARAMETERS, OK

ULTIMATE STRENGTH:

$$M_1 = Pe = (729.8k)(4.33 - 1.0) = 202.5 \text{ k-IN}$$

$$M_{Sec} = M_{BAL} - M_1 = 383.5 - \frac{202.5}{12} = 366.6 \text{ K FT} \leftarrow \text{EQUAL @ BOTH SUPPORTS}$$

MINIMUM BONDED REINFORCEMENT:

OVERALL, NO STRESSES IN TENSION ⇒ TENSILE REINFORCEMENT IS UNNECESSARY

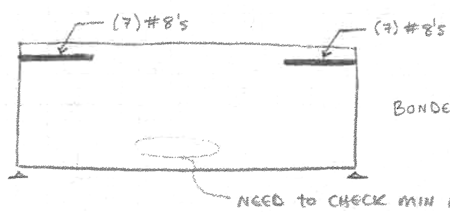
NEGATIVE MOMENT REGIONS:

$$A_{s,min} = 0.00075 A_{cf} \qquad A_{cf} = \left\{ \begin{array}{l} \frac{(25 + 62.5)}{2} = 43.75 \\ 26' \end{array} \right|_{MAX} \times 12"/' \times 14" = 7350 \text{ IN}^2$$

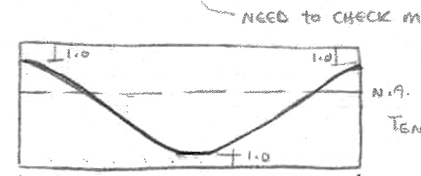
$$A_{s,min} = 0.00075(7350) = 5.5125 \text{ IN}^2$$

(7) #8's TOP (5.53 IN²)

AT BOTH SUPPORTS



BONDED REINFORCING



TENDON PROFILE

5/

POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

$A = 7'' \times (62.5' \times 12) = 5250\ in^2$

$S = \frac{bh^2}{6} = \frac{(750'')(7'')^2}{6} = 6125\ in^3$

NORTH-SOUTH DIRECTION DESIGN:

$\omega_{BAL} = (79.44)(62.5') = 4965\ PLF$

$\hookrightarrow P = \frac{w_{BAL}\, l^2}{8\, a_{INT}} = 3355.4\ K \qquad \frac{3355.4}{24.327} = 137.9$

MAX ACCEPTABLE TENDONS:

$\frac{P}{A} \geq 300\ PSI \qquad 300(5250) = 1575\ K$

$\hookrightarrow \frac{1575}{24.327} = 64$ TENDONS

$P_{ACTUAL} = 64(24.327) = 1556.9\ K$

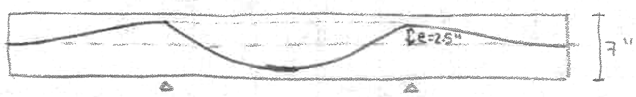
$w_b = \frac{1556.9\,(8 \times 8.67/12)}{(62.5)^2} = 2.3\ KLF$

$w_{S.W.} = 4.965\ KLF$

$\frac{w_b}{w_{SW}} < 1.0$ OK

$\frac{P}{A} = \frac{1556.9}{5250} = 0.2965\ KSI$

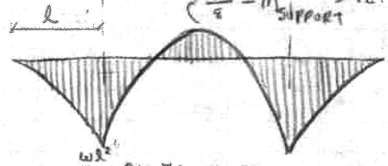
TENDON PROFILE:



SLAB STRESSES:

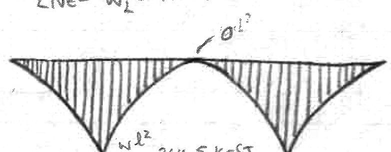
DEAD – $w_D = (99.3 + 15)(62.5') = 7.14\ KLF \qquad P = (10' \times 62.5') \times 15\ psf = 9.375\ K$

$\left(\frac{wl^2}{8} - M_{SUPPORT}\right) = 121.87\ K\text{-}FT$



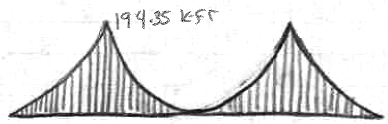
$\frac{wl^2}{2} + Pl = 725.2\ K\text{-}FT$

LIVE – $w_L = (50)(62.5') = 3.13\ KLF$



$\frac{wl^2}{2} = 264.5\ K\text{-}FT$

BALANCING – $\omega_{BAL} = 2.3\ KLF$



IMMEDIATELY AFTER JACKING:

- MIDSPAN

$f_{TOP} = \frac{(-121.87 + 0)}{6125} - 0.2965 = -317\ PSI$

$f_{BOTTOM} = \frac{(121.87 + 0)}{6125} - 0.2965 = -277\ PSI$

- SUPPORTS

$f_{TOP} = \frac{(-725.2 + 194.35)}{6125} - 0.2965 = -383\ PSI$

$f_{BOTT} = \frac{(725.2 + 194.35)}{6125} - 0.2965 = -210\ PSI$

WITHIN CLASS U DESIGN PARAMETERS. OK

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POST-TENSIONED CONCRETE (2-WAY) (CONT'D)

STRESSES AT SERVICE LOAD:

- MIDSPAN

$$f_{TOP} = \frac{(-121.87 - 0 + 0)}{6125} - 0.2965 = -317 \text{ PSI}$$

$$f_{BOTT} = \frac{(121.87 + 0 - 0)}{6125} - 0.2965 = -277 \text{ PSI}$$

- SUPPORTS

$$f_{TOP} = \frac{(-725.2 - 264.5 + 194.35)}{6125} - 0.2965 = -426 \text{ PSI}$$

$$f_{BOTT} = \frac{(725.2 + 264.5 - 194.35)}{6125} - 0.2965 = -167 \text{ PSI}$$

WITHIN CLASS U PARAMETERS OK

ULTIMATE STRENGTH:

$M_1 = Pe = (1556.9)(2.5") = 3892.25$ K-IN

$M_{SEC} = 194.35 - \frac{3892.25}{12} = -130$ K-FT ← EQUAL @ BOTH SUPPORTS

MINIMUM BONDED REINFORCEMENT:

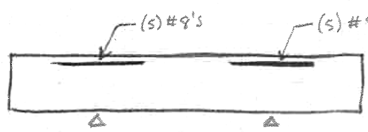
OVERALL, NO STRESSES IN TENSION ⇒ TENSILE REINFORCEMENT IS UNNECESSARY

NEGATIVE MOMENT REGIONS:

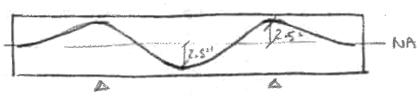
$A_{S,MIN} = 0.00075 A_{CS}$ $A_{CS} = \left\{ \begin{array}{l} \frac{13+26}{2} = 19.5 \\ 62.5' \end{array} \right|_{MAX} \times 12"/' \times 7" = 5250 \text{ IN}^2$

$A_{S,MIN} = 3.94 \text{ IN}^2$

(5) #8's TOP (3.95 IN²)
AT BOTH SUPPORTS



BONDED REINFORCING



TENDON PROFILE

Appendix G – Cost Estimate Documentation

Material		Properties	Overall Cost	Material Cost	Units
Steel Floor Deck					
	Non-Cellular 3" Composite Deck, Galvanized				
		20 Gauge	2.93	1.86	ft^2
		18 Gauge	3.45	2.29	ft^2
	Open Deck, Wide Rib				
		3", 16 Gauge	4.26	3.16	ft^2
		6", 14 Gauge	9.48	7.3	ft^2
Structural Steel Members					
		W14x22	35.45	31.5	ft
		W14x74	106.12	89.5	ft
		W18x40	61.15	48.5	ft
		W18x46	69.15	55.5	ft
		W18x50	75.05	60.5	ft
		W24x131	201.43	177	ft
		W24x146	201.43	177	ft
		W24x176	201.43	177	ft
		W27x102	158.78	138	ft
Prestressing Steel					
	Grouted, 100' Span, 300 kip		3.27	1.79	lb
Reinforcing Steel					
	Typ. In-Place Average, #8		1495	985	ton
Concrete					
	Normal Weight				
		3000 psi	110	100	yd^3
		4000 psi	117	106	yd^3
	Light Weight				
		3000 psi	161	146	yd^3
		4000 psi	163	149	yd^3

Prestressing strands/floor		
Weight (lb/1000 ft)	Estimated feet of tendons*	Weight
775	32484	25175.1 lb
* (#tendons x #bays x length of bay x 1.5 for drape and any additional factors)		

Reinforcing Bars			
Weight (lb/ft)	Estimated feet	Weight (lb)	Weight (tons)
2.67	1860	4966.2	2.4831

Existing - Composite Deck			
Material	**Units/floor**	**Total**	**Base Total**
Design			
3" 20 Gauge Deck	13000.00	38090.00	24180.00
W14x22	722.00	25594.90	22743.00
W18x40	624.00	38157.60	30264.00
W18x50	312.00	23415.60	18876.00
W24x176	500.00	100715.00	88500.00
L.W. 3000 psi, 3.25"	130.40	20994.60	19038.58
	SUM=	246967.70	203601.58
	Cost/ft^2=	**19.00**	**15.66**
Calculated			
3" 18 Gauge	13000.00	44850.00	29770.00
W14x22	722.00	25594.90	22743.00
W18x40	936.00	57236.40	45396.00
W27x102	500.00	79390.00	69000.00
L.W. 3000 psi, 3.25"	130.40	20994.60	19038.58
	SUM=	228065.90	185947.58
	Cost/ft^2=	**17.54**	**14.30**

Alternative - Non-Composite Deck			
Material	**Units/floor**	**Total**	**Base Total**
3" Form, 16 Gauge	13000.00	55380.00	41080.00
W14x22	722.00	25594.90	22743.00
W18x46	936.00	64724.40	51948.00
W24x146	500.00	100715.00	88500.00
N.W. 3000 psi, 3"	120.37	13240.74	12037.04
	SUM=	259655.04	216308.04
	Cost/ft^2=	**19.97**	**16.64**

Alternative - Long Span Deck			
Material	**Units/floor**	**Total**	**Base Total**
6" Form, 14 Gauge	13000.00	123240.00	94900.00
W14x22	722.00	25594.90	22743.00
W14x74	832.00	88291.84	74464.00
W24x131	500.00	100715.00	88500.00
L.W. 3000 psi, 3.25"	130.40	20994.60	19038.58
	SUM=	358836.34	299645.58
	Cost/ft^2=	**27.60**	**23.05**

Alternative - Post-Tensioned Concrete			
Material	**Units/floor**	**Total**	**Base Total**
300 kip prestressing tendons	25175.10	82322.58	45063.43
#8 reinforcing steel	2.48	3712.23	2445.85
N.W. 4000 psi, 7" slab, 14" beams	299.77	35072.92	31775.46
	SUM=	121107.73	79284.75
	Cost/ft^2=	**9.32**	**6.10**

Appendix H – Additional References

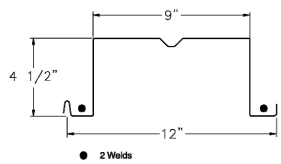
Preformed Metal Roof Deck

MORE >

< RETURN TO DECK INDEX

ASC STEEL DECK

Deep Deck 4 1/2″ and 6″



Deep Deck 4 1/2"

1. The top value reflects the allowable reaction at the panel end supports.
2. The bottom value reflects the allowable reaction at the interior supports.
3. Values are in pounds per linear foot.

Allowable Reactions

Gauge	Bearing Length (in) 3"
20	390
	853
18	815
	1477
16	1362
	2316
14	2282
	3721

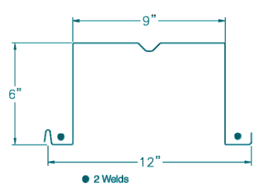
4 1/2″ Deep Deck Section Properties

Gauge	Weight (psf)	I (In4)	S+ (In3)	S– (In3)
20	2.86	2.44	0.924	0.957
18	3.74	3.42	1.266	1.313
16	4.69	4.36	1.608	1.635
14	5.86	5.49	2.056	2.056

1. Section properties are based on minimum 33 ksi steel (Fy).

4 1/2″ Deep Deck Allowable Total (DL + LL) Uniform Load (psf) [1, 2 (footnote page 26)]

Span Condition	Gauge		Span 10'0"	12'0"	14'0"	16'0"	18'0"	20'0"	22'0"	24'0"	26'0"	28'0"	30'0"
SINGLE SPAN	20	Stress	123	86	63	48	38	31	25	21	18	16	14
		Deflection	123	86	58	39	27	20	15	12	9	7	6
	18	Stress	169	117	86	66	52	42	35	29	25	22	19
		Deflection	169	117	82	55	38	28	21	16	13	10	8
	16	Stress	214	149	109	84	66	54	44	37	32	27	24
		Deflection	214	149	104	70	49	36	27	21	16	13	11
	14	Stress	274	190	140	107	85	69	57	48	41	35	30
		Deflection	274	190	131	88	62	45	34	26	20	16	13



Deep Deck 6"

1. The top value reflects the allowable reaction at the panel end supports.
2. The bottom value reflects the allowable reaction at the interior supports.
3. Values are in pounds per linear foot.

Allowable Reactions

Gauge	Bearing Length (in) 3"
20	351
	793
18	757
	1403
16	1289
	2226
14	2188
	3610

6″ Deep Deck Section Properties

Gauge	Weight (psf)	I (In4)	S+ (In3)	S– (In3)
20	3.22	4.79	1.386	1.314
18	4.22	6.68	1.892	1.966
16	5.29	8.56	2.406	2.451
14	6.61	10.78	3.085	3.087

1. Section properties are based on minimum 33 ksi steel (Fy).

6″ Deep Deck Allowable Total (DL + LL) Uniform Load (psf) [1, 2 (footnote page 26)]

Span Condition	Gauge		Span 10'0"	12'0"	14'0"	16'0"	18'0"	20'0"	22'0"	24'0"	26'0"	28'0"	30'0"
SINGLE SPAN	20	Stress	185	128	94	72	57	46	38	32	27	24	21
		Deflection	185	128	94	72	54	39	29	23	18	14	12
	18	Stress	252	175	129	99	78	63	52	44	37	32	28
		Deflection	252	175	129	99	75	55	41	32	25	20	16
	16	Stress	321	223	164	125	99	80	66	56	47	41	36
		Deflection	321	223	164	125	96	70	53	41	32	26	21
	14	Stress	411	286	210	161	127	103	85	71	61	52	46
		Deflection	411	286	210	161	121	88	66	51	40	32	26

SWPC

Specifications for PC Strand

ASTM A416 – SEVEN-WIRE UNCOATED LOW RELAXATION STRAND							
Grade	Nominal Strand Diameter in [mm]	Strand Tolerance in [mm]	Minimum Breaking Strength Lbs [kgs]	Min. Yield Strength at 1% Extension Lbs [kgs]	Minimum Elongation at 24" Gauge	Nominal Area In^2 [mm^2]	Nominal Weight Lbs/1000 ft Kg/1000 m
250K	3/8" [9.5]	0.3910/0.3590 [9.93/9.13]	20,000 [9,072]	18,000 [8,165]	3.5%	0.080 [51.61]	272 [405]
	7/16" [11.1]	0.4535/0.4215 [11.51/10.71]	27,000 [12,247]	24,300 [11,022]		0.108 [69.68]	367 [548]
	1/2" [12.7]	0.5160/0.4840 [13.1/12.3]	36,000 [16,329]	32,400 [14,696]		0.144 [92.9]	490 [730]
270K	3/8" [9.5]	0.4010/0.3690 [10.18/9.38]	23,000 [10,433]	20,700 [9,389]	3.5%	0.085 [55.03]	290 [432]
	7/16" [11.1]	0.4635/0.4315 [11.76/10.96]	31,000 [14,061]	27,900 [12,655]		0.115 [74.19]	390 [582]
	1/2" [12.7]	0.5260/0.4940 [13.35/12.55]	41,300 [18,733]	37,170 [16,860]		0.153 [98.71]	520 [775]
	0.52" (1/2"HBS) [13.2]	0.5460/0.5140 [13.86/13.06]	45,000 [20,412]	40,500 [18,368]		0.165 [106.45]	563 [874]
	9/16" [14.3]	0.5885/0.5565 [14.94/14.14]	51,700 [23,451]	46,530 [21,102]		0.192 [123.87]	650 [967]
	0.6" [15.2]	0.6260/0.5940 [15.89/15.09]	58,600 [26,581]	52,740 [23,922]		0.217 [140.00]	740 [1,102]

RELAXATION PROPERTIES	
Initial Stress	Maximum Relaxation after 1000 Hours
70% G.U.T.S.	2.5%
80% G.U.T.S.	3.5%

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