LETTER OF TRANSMITTAL

Date: October 17, 2014

To: Dr. Thomas Boothby
TEBARC@engr.psu.edu

From: Nick Dastalfo
njd5133@psu.edu

Dear Dr. Boothby,

The enclosed documents include my Structural Technical Report 3 for AE481W – Senior Thesis. Technical Report 3 includes a structural analysis and proposed design alternates for the primary structural gravity system of 8621 Georgia Avenue in Silver Springs, Maryland.

This report includes a building abstract and site plans in addition to all necessary calculations for the roof, floor, and wall loads for the building. There will be a detailed analysis of the existing post-tensioned slab as well as 3 new design alternates for the gravity system.

Thank you for taking the time to read and review my report. I am eagerly looking forward to discussing the project with you in the future.

Sincerely,

Nick Dastalfo

Technical Report 3

8621 Georgia Avenue
Silver Spring, Maryland

Nick Dastalfo | Structural

Advisor: Dr. Thomas Boothby

October 17, 2014

Executive Summary

The building at 8621 Georgia Avenue is proposed to be built on an existing 0.69 acre parking lot located in the downtown business district of Silver Spring, Maryland. The 17 story, 347,000 ft^2 project will create more downtown multi-family housing and parking for the booming region. The project has recently finished the permit phase of development and is nearly at the start of construction in early 2015. The building is surrounded by adjacent pre-existing buildings on 3 sides and will also include a side entrance for parking garage egress.

The building will be the tallest of the surrounding buildings and will be clearly visible along specific urban view corridors and pedestrian heavy areas. Therefore, a great attention to detail was put on the architectural impact of the form of the glass curtain wall clad building in these locations. Being the tallest building in the area came along with the challenges of remaining under the zoning height restriction of the area. Efforts were made to decrease the floor to floor height by using post tensioning in order to squeeze the most amount of floors into the building.

The first four stories, used for parking, retail, and café, have flat plate concrete slab floors with minimal use of concrete drop panels and beams when necessary. The 5th through 17th floor utilize post-tensioned concrete flat plates with spans varying from 15’-10” to 24’-0” throughout these 12 floors of apartments. The variation in column locations and the use of transfer girders were eliminated due to strategic placing of columns in a regular grid that was appropriate for both the parking garage and the apartments.

The building was designed considering live loads, gravity loads, snow loads, wind loads, seismic loads, and lateral loads. The lateral force resisting system in the building is primarily made up of shear walls around the two stair towers of the structure.

The design for this building was governed by the International Building Code 2012 as well as the ‘Minimum Design Loads for Buildings and Other Structures” (ASCE 7-10). These codes reference other standards that were integral in the design process and include ACI318-11 and parts 1-5 of the ACI Manual of Standard Practice, PTI’s “Post Tensioning Manual, 6th Edition, the “Manual of Standard Practice” from CRSI, and AISC’s Steel Construction Manual, 14th Edition.

This report will cover all of these features and many more, in greater detail.

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8621 Georgia Avenue
Silver Spring, Maryland

General Building Data:

Building Height: 161 feet
Number of Stories: 17 floors
Size: 347,009 ft^2
Cost: $51 million
Occupancy: Mixed Use
-Residential, Parking Garage, Retail
Construction: Beginning in 2015

Architecture:

The façade of the building brings a refreshing modern addition to the skyline of the developing city of Silver Spring. The position of the building takes advantage of two major view corridors in the urban fabric and has an inviting present on the busy Georgia Avenue.

Structural Systems:

This concrete building utilizes mild reinforced cast-in-place two way flat slabs with full drop panels for the parking garage on floors 1-4 and a post-tensioned cast-in-place two way flat slab for the remainder of the apartment level floors. The lateral system is comprised of 14 concrete shear walls located around stair and elevator cores. The column grid is relatively square vary from 16-24' in length.

Construction:

Construction is scheduled to be 24-28 months and will begin in early 2015. Important factors will be coordinating work with the surrounding existing buildings on all sides and impact of the high water table on the foundation construction.

MEP:

Floors 1-4 (parking garage) will be open and designed as an open structure. Each apartment will be conditioned by a conventional split system heat pump with back-up electric heat. Outdoor air is provided by an exterior louver.

Lighting / Electrical:

The building will have 277/480V as the primary power with 480-120/208V transformers. Branch lighting/power panels will be placed in the cellar and every 4^{th} apartment level. These panels serve the local receptacles, lighting, and HVAC units.

Project Sponsor: Holbert Apple Associates

Nick Dastalfo I Structural
Advisor: Dr. Thomas Boothby

For more information, see my CPEP site:
http://www.engr.edu/ae/thesis/portfolios/2015/njd5133/index.html

Site and Location Plan

Documents Referenced for Report

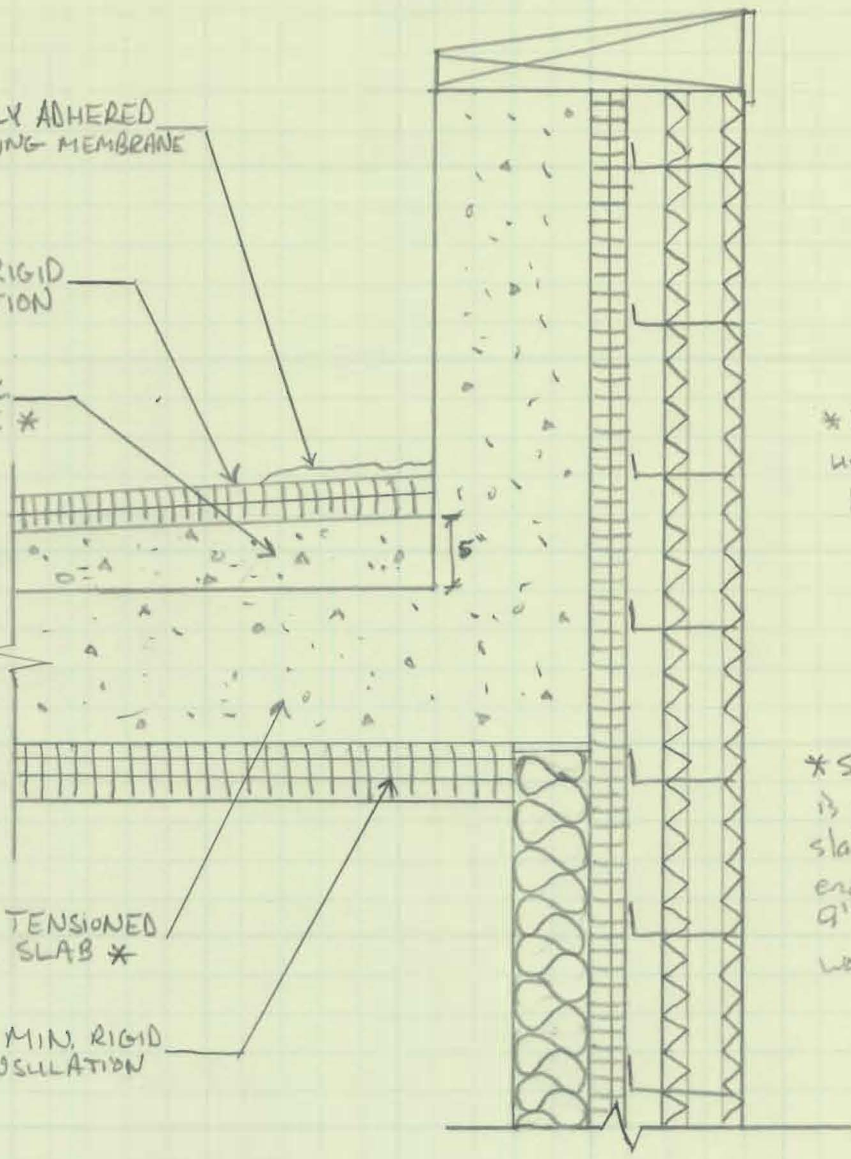
Shown below is a list of the design codes, standards or other references that were used in the structural analysis of 8621 Georgia Avenue for Technical Report 3.

- American Society of Civil Engineers
  - ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
- Montgomery County Building Codes and Standards
- ADAPT Technical Note #292
- American Concrete Institute
  - ACI 318-08:Building Code Requirements for Structural Concrete
- International Building Code 2012
- 8621 Georgia Avenue Silver Spring, MD
  - Construction Drawings
  - Specifications
  - Correspondence with Project Engineers
- American Institute of Steel Construction
  - AISC Manual of Load and Resistance Factor Design, 3rd ed.

Gravity Loads
From Tech Report #2

Typical Roof Dead Load on 17th Floor

Detail Cross-Section at Parapet



* Lightweight Concrete used for roof sloping LWC = 120 PCF

* Slab on south wing is 9", whereas the slab on the north end is 7½". Use 9" to determine worst case loading

Uniformly Distributed Dead Loads

- Rigid Insulation (2×4 = 8")	=	12 psf
- 5" LW Concrete	=	50 psf
- 9" NW Concrete	=	112.5 psf
- Roofing Membrane	=	2 psf
- Collateral	=	3.5 psf

Total = 180 psf

Typical Roof Live Loads

ASCE 07-10, Table 4-1: Minimum Uniformly Distributed Live Loads

Ordinary Flat Roof .... 20 psf

* See Snow loads, for controlling roof live load, where applicable.

* The Engineer also added an additional 30 psf superimposed dead load. This design decision may have been made for a number of foreseeable factors such as: snow accumulation, ponding, roof maintenance, etc.

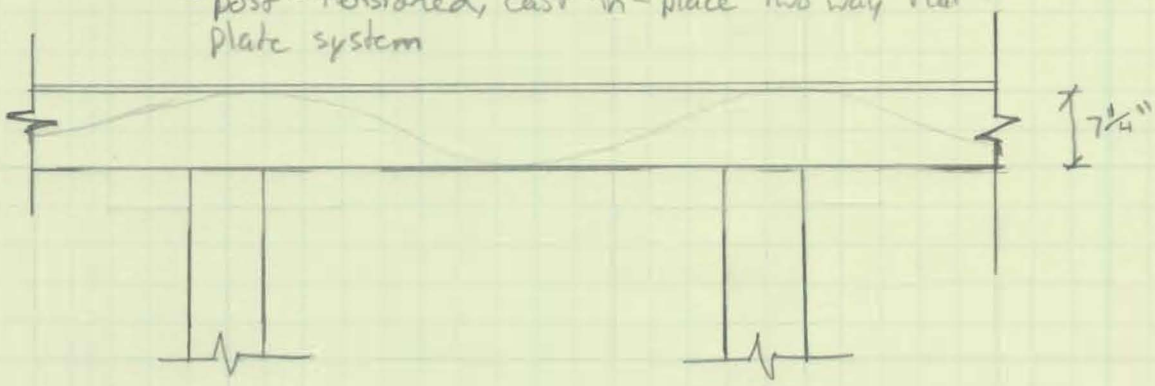
* The Engineer also chose to increase the minimum live load, provided in ASCE 07-10, to 30 psf.

Floor Loads

Floor Dead Loads

Apartment Levels

- post-tensioned, cast-in-place two way flat plate system



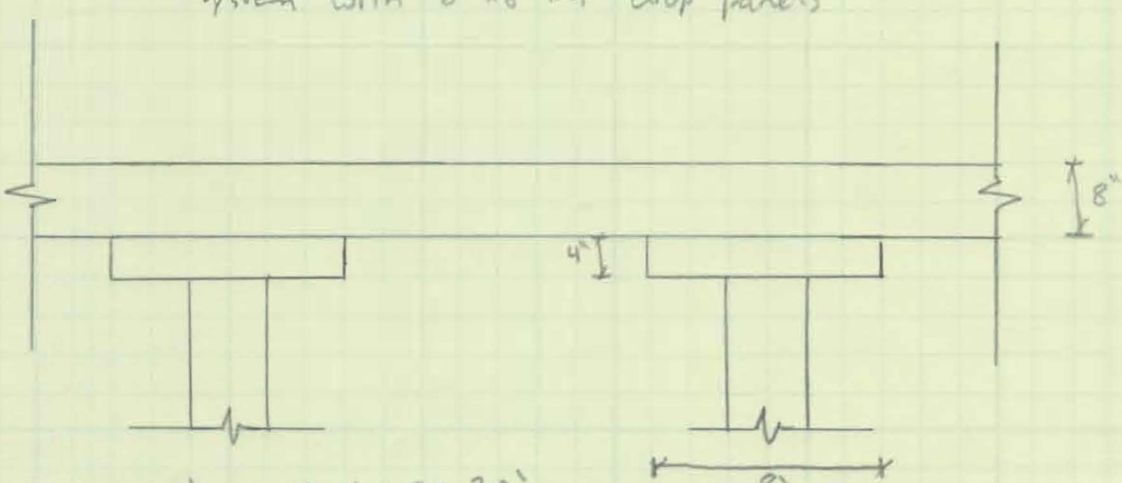
Uniformly Distributed Dead Loads

- 7¼ Concrete = 90.6 psf
- Floor Finish = 2 psf
- Collateral = 5 psf

Total = 97.6 psf

Parking Garage

- 8" mild-reinforced cast-in-place two way flat slab system with 8'×8'×4" drop panels



- average column spacing ≈ 20'

$$\frac{8'}{20'}(12'') + \frac{12'}{20'}(8'') = 9.6'' \rightarrow 10'' \text{ avg. thickness}$$

Uniformly Distributed Loads

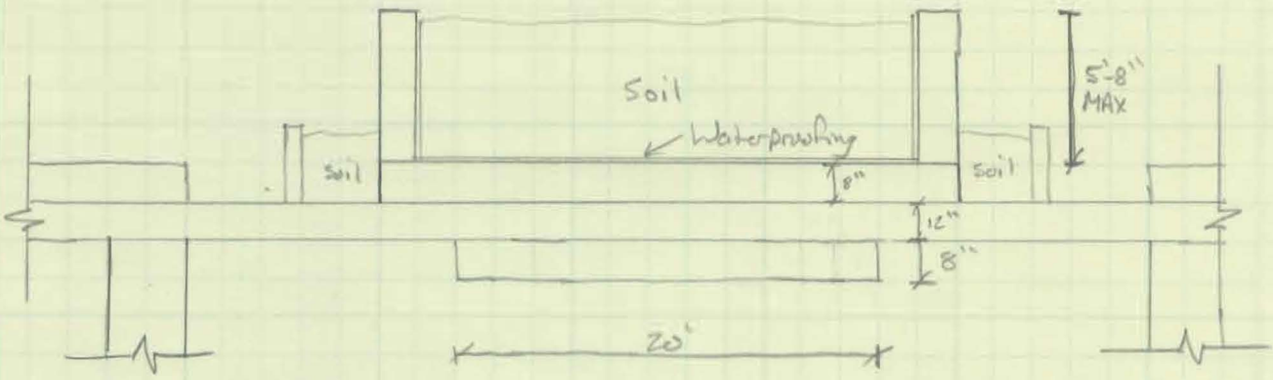
- 10" Concrete = 125 psf
- Collateral = 5 psf

Total = 130 psf

Floor Dead Loads Continued...

Under Bioretension Area:

- The bio-retension area is located on the 5th floor set back and is supported by mild reinforced concrete with a continuous drop panel.



- For worst case, assume planter is saturated $\gamma = 62.4$ pcf

Uniformly Distributed Load

- Avg 20" Concrete = 250 psf
- 5'-8" Water = 353.8 psf
- Water-proofing = 2 psf
- Collateral = 5 psf

Total = 610.8 psf

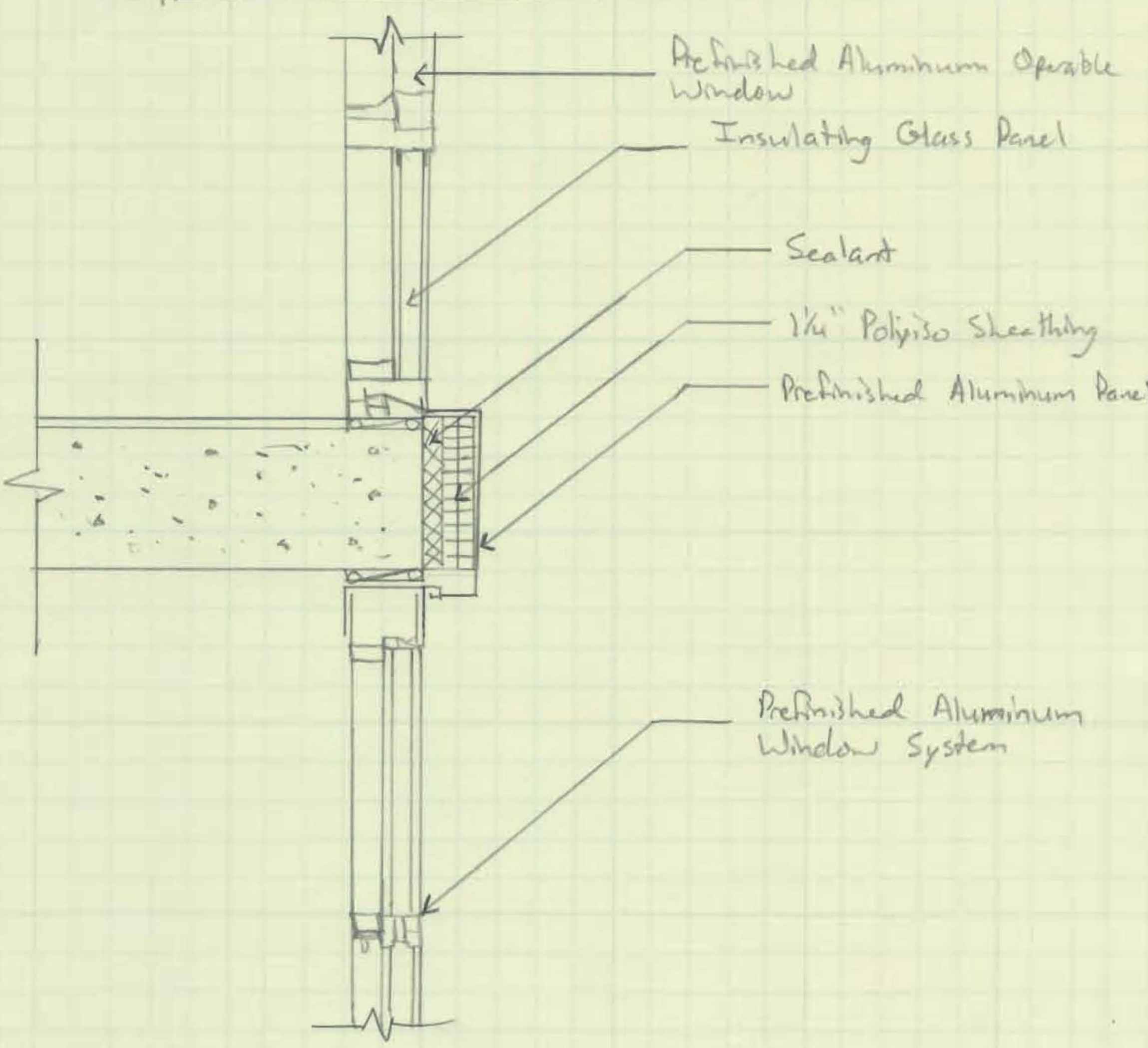
Floor Live Loads

Occupancy	Design Value	ASCE 7-10 Code Minimum
Residential	40 + 10 (partition)	40psf + 10 (partitions)
Parking Garage	50 psf	40 psf
First Floor Retail	100 psf	100psf
Public Space	100 psf	100 psf
Fitness Gym	100 psf	100psf

Exterior Wall Loads

Wall Loads

Typ. Curtain Wall Slab - Operable Window



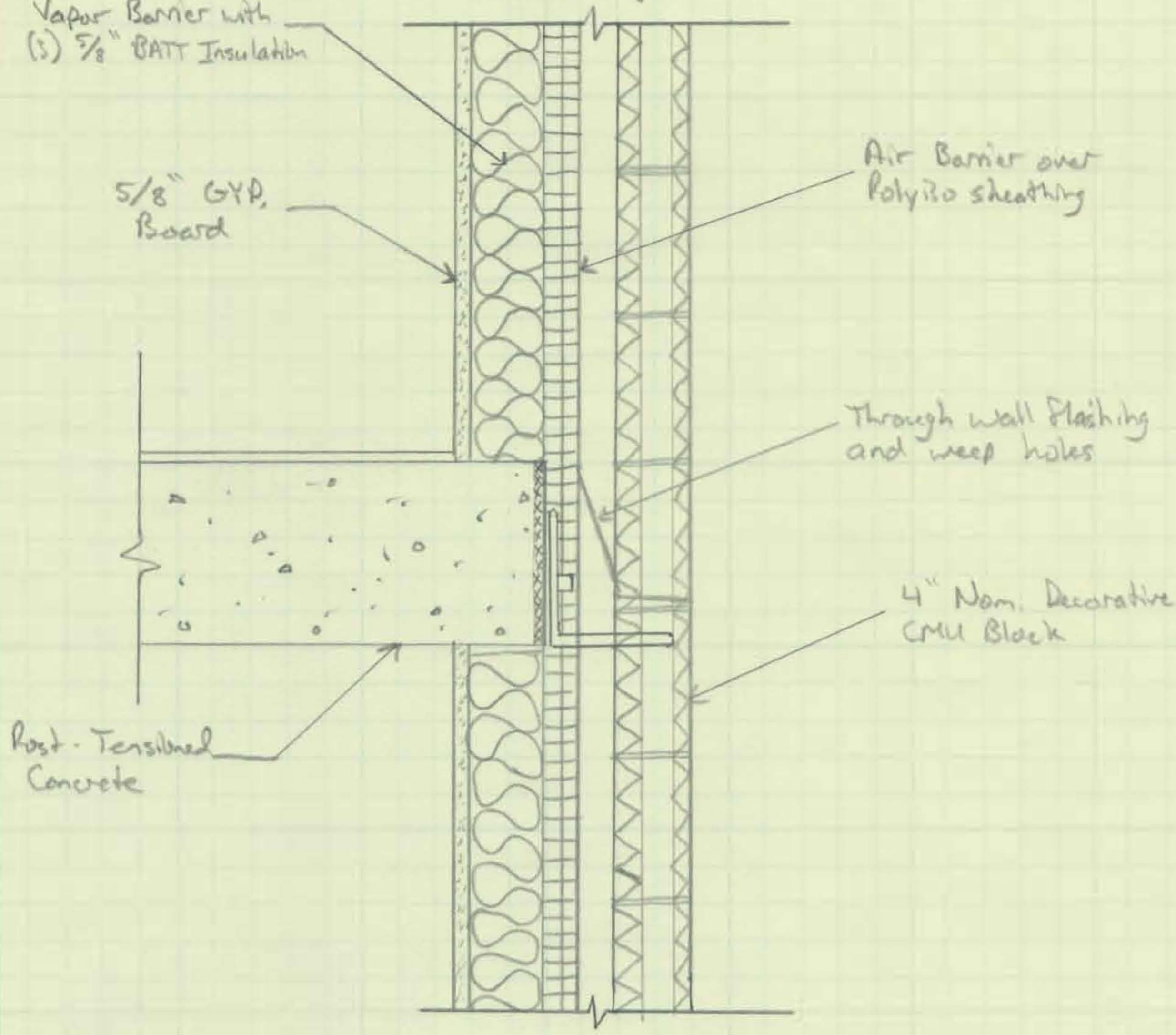
Window System ...... 8 psf × 10' = 80 plf
↳ AISC Manual

Fasteners .... 5 plf

Total = 85 plf

Typical Exterior Masonry Wall

- The facade is covered in windows but the greatest wall load will result from a fully masonry section.
- assume a 10' story height



Gypsum Board - $\frac{5}{8}$" × 4 $\frac{psf}{in}$ × 10' = 25 plf

Batt Insulation - $3(\frac{5}{8}) = 1\frac{7}{8}$" × 1 $\frac{psf}{in}$ × 10' = 18.75 plf

Poly Iso - 1 pcf × $\frac{1}{12}$' × 10' ≈ 1 plf

4" Nom. Concrete. 30 psf × 10' = 300 plf

Total = 345 plf

Existing System Analysis
Post-Tensioned 2-Way Slab

Typical Bay for Analysis

- The typical bay is bound by column lines 6, 7, C, and D.
- Dimensions: 24'-0" x 20'-0"
- Location: Northeast (Plan) of building

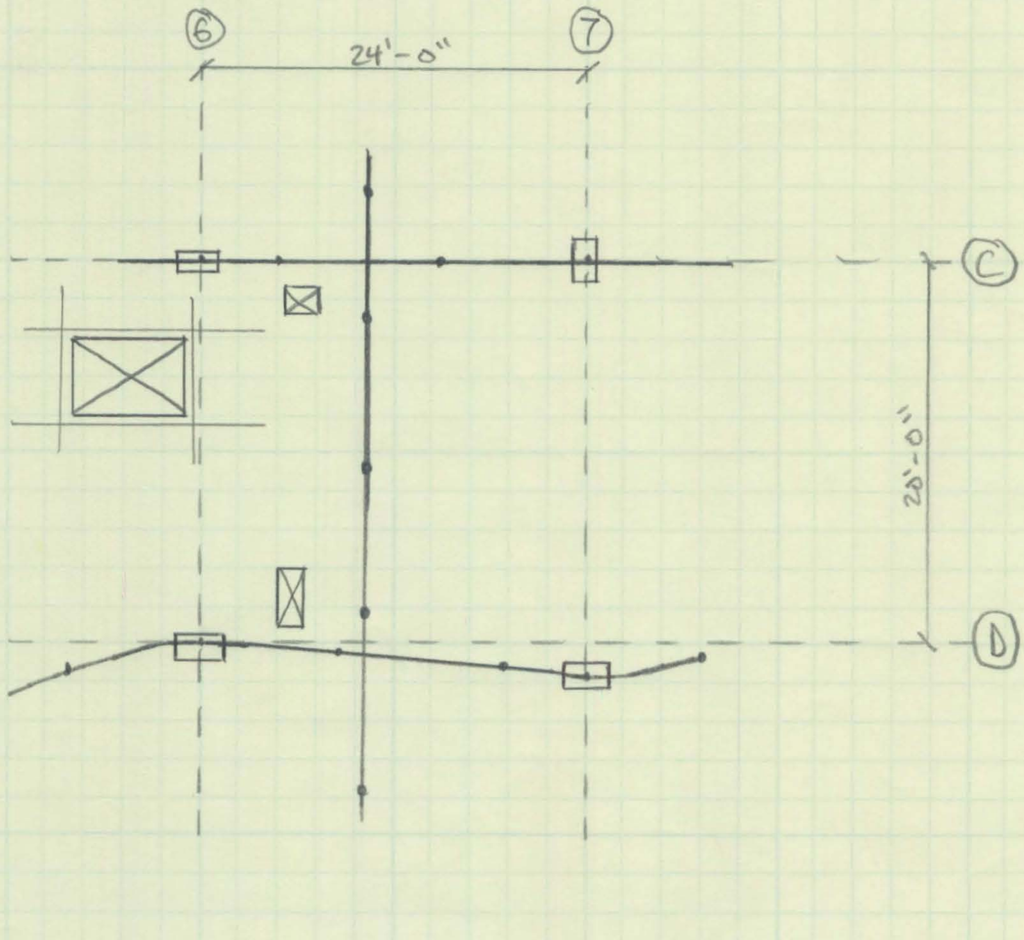
Assumptions:

- Assume that column 7-D is located on the column grid.
- Assume the presence of openings in the slab is neglible

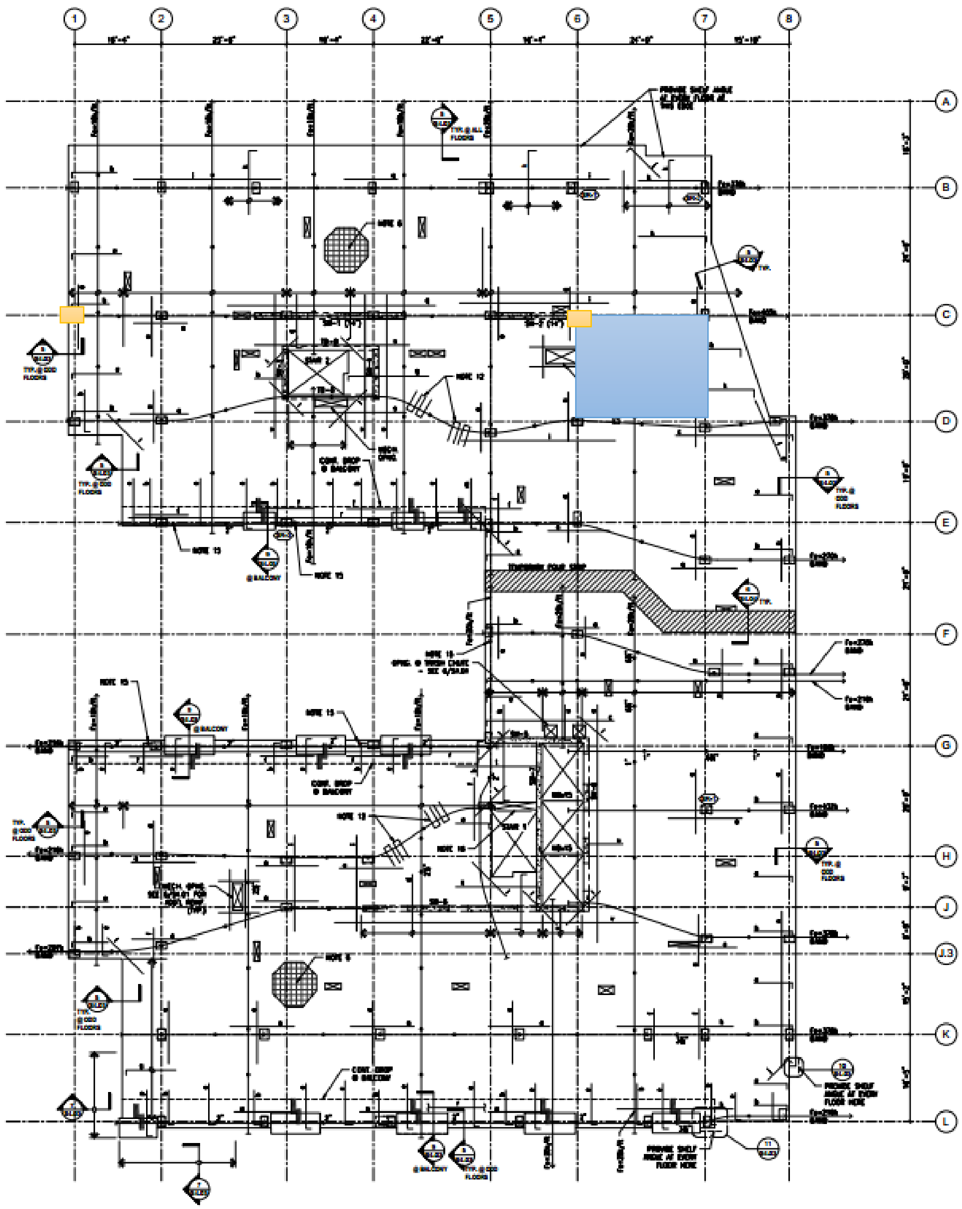
Notes:

- The typical bay will be analyzed for the typical floor; which occurs on floors 5-16 and is a post-tensioned two way flat slab construction and facilitates 25 apartment units.

Typical Bay Detail

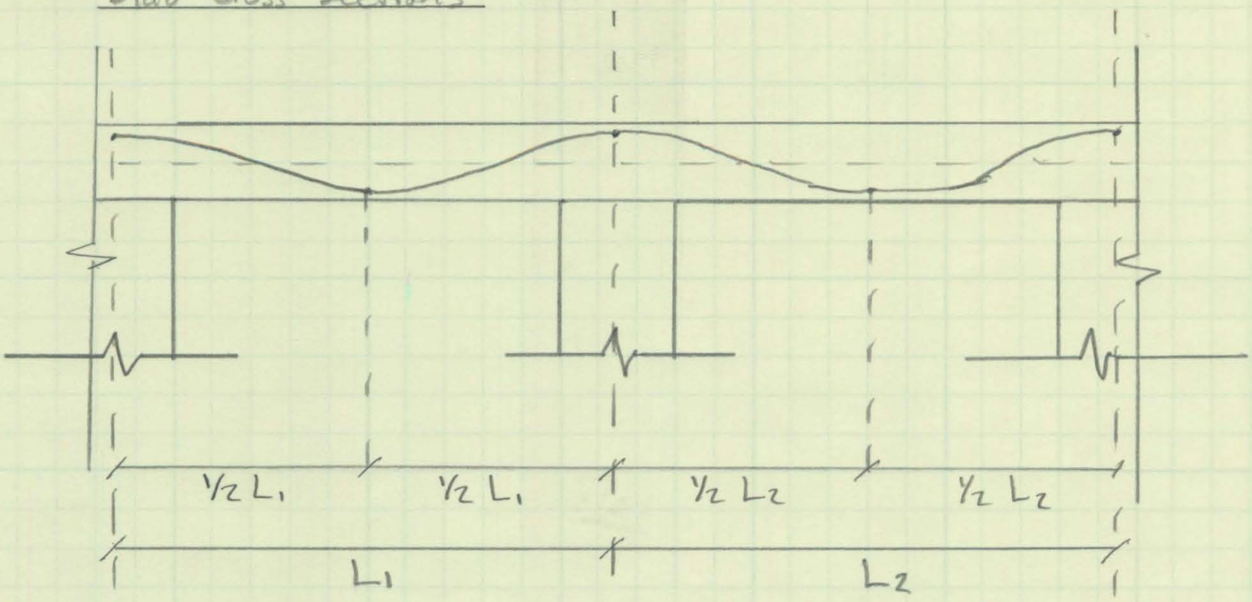


Typical Bay and Columns Analyzed:



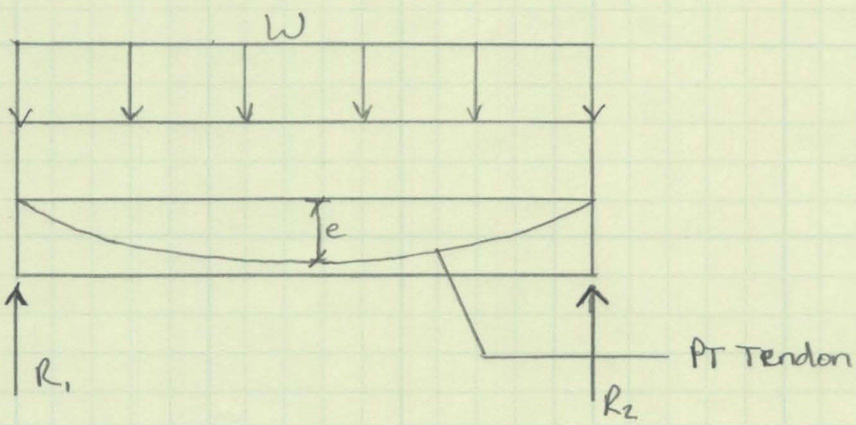
= Analyzed Bay
= Analyzed Columns

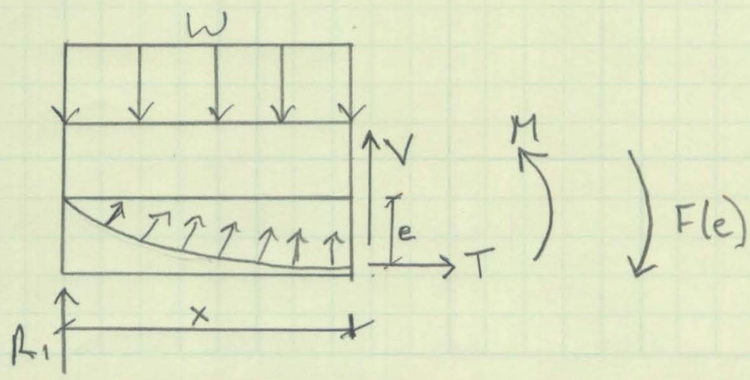
Slab Cross Sections



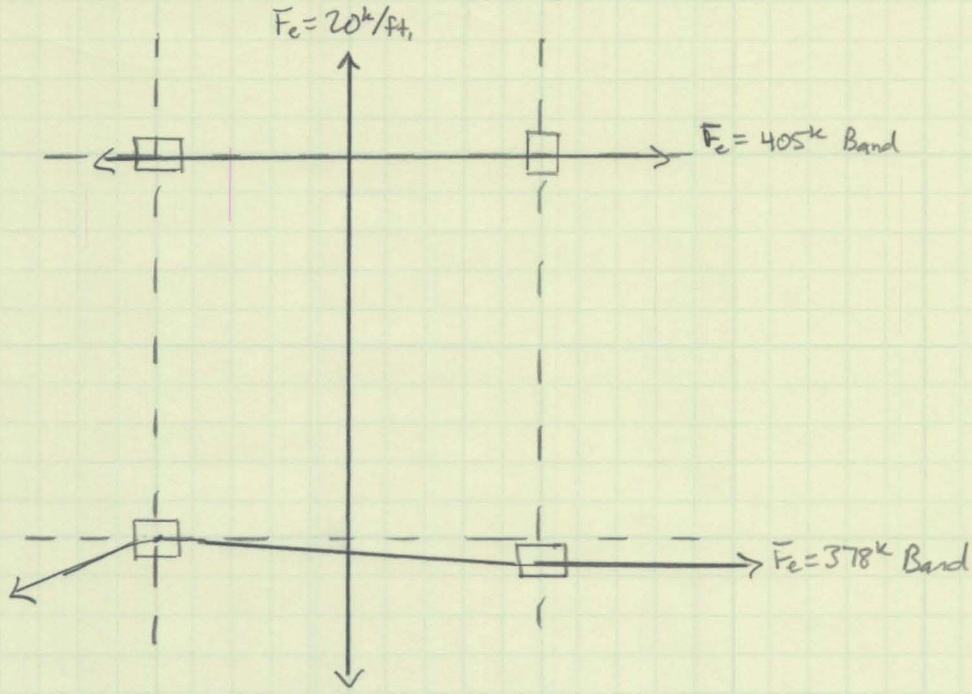
Post Tensioning Theory

- The eccentricity of the draped tendons causes counter-active forces in the tendons which balance out a portion of the dead load.

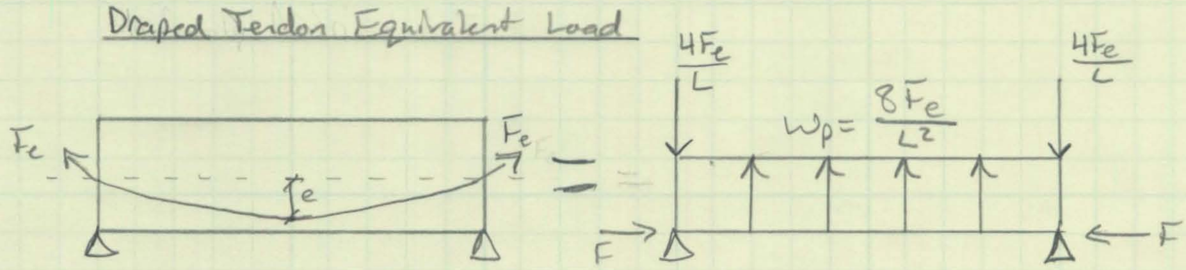




Post Tensioned Balance Load



Draped Tendon Equivalent Load



Banded Tendons

$$e = \frac{7.25}{2} - 1'' = 2.625''$$

↳ Drawings call for tendons set to 1" cover from edge of slab at maximum drape.

$$W_p = \frac{8(378^k)(2.625'')\left(\frac{1'}{12''}\right)}{(24')^2} = 1.148\ ^k/ft.$$

$$\frac{1.148\ k/ft}{24'} = 47.8\ psf$$

Uniformly Distributed Tendons

$e = 2.625''$ $\qquad \frac{F}{L} = 20^{k}/ft.$

$$w_p = \frac{8\,(20^{k}/ft)(2.625'')(\frac{1'}{12''})}{20} = 1.75^{k}/ft.$$

$$1.75^{k}/ft \times \frac{1}{20} = 87.5\ psf$$

Total Upward Force from PT:

$1.148 + 1.75 = 2.898\ ^{k}/ft. \rightarrow 2.9\ ^{k}/ft \uparrow up$

or $47.83 + 87.5 = 135.33\ psf \rightarrow 135\ psf \uparrow up$

-Now subtract this balancing force from the Dead Load and proceed as if it were a 2-way slab analysis...

Load Determination

$LL = 50\ psf$

$DL_{sw} = 7.25/12\,(150) + 5\ psf = 95\ psf$

$Misc.\ DL = 15\ psf$

Load case: $1.2DL + 1.6LL$

$1.2(95+15) + 1.6(50) = 212\ psf.$

- greatest line load would be along the long direction (24 ft).

$212 \frac{lbs}{ft^2} \times 24' = 5.09\ klf$

$$\begin{array}{r l} 5.09\ klf & \leftarrow \text{floor loading} \\ -2.90\ klf & \leftarrow \text{PT balance loading} \\ \hline 2.19\ klf & \end{array}$$

$$\begin{array}{r l} 212\ psf & \leftarrow \text{floor loading} \\ -135\ psf & \leftarrow \text{PT balance loading} \\ \hline 77\ psf & \end{array}$$

- Continue with 2-way slab analysis as if loading was 77psf

Deflection Check

- As published in the ADAPT Technical Note #292, two way post-tensioned floor slabs do not crack under service loads and that any cracks that form are negligible.

- Therefore, it is ok to use gross cross-sectional geometry

- Because gross cross-sectional properties are included I will calculate deflections using the "Closed Form" method given by ADAPT TN292. This reference will be included in appendix A.

- Span to Depth

40-45 is recommend by PTI

$$\frac{24' \times 12}{7.25''} = 39.72 \checkmark$$

- Closed Form

Aspect Ratio: $\gamma = \frac{24'}{20'} = 1.2$

- assume poisson's ratio = 0.25

$q_u = 212$ psf

@ mid-panel deflection:

$$\Delta = k\left(a^4 q / E \cdot h^3\right)$$

- use case 3 from table 5 for K value for "central panel surround by similar panels looking at deflection at the center.

$K = 0.0481$, tabulated value

$a = 24'$, long span

$E_c = 57{,}000\sqrt{5000} = 4030.5 \times 10^3$ psi

$h = 7.25''$, slab thickness

$$\Delta = 0.0481\left(\frac{(24\times12)^4\,(212)\left(\frac{1\ ft^2}{144\ in^2}\right)}{4030.5\times10^3\,(7.25)^3}\right)$$

$$\Delta = 0.0481\,(6.59)$$

$$\Delta = 0.317''$$

$$\frac{L}{360} = \frac{24'\times12''}{360} = 0.8''$$ ← Deflection Limit listed in Table 9.5(b) in ACI-318

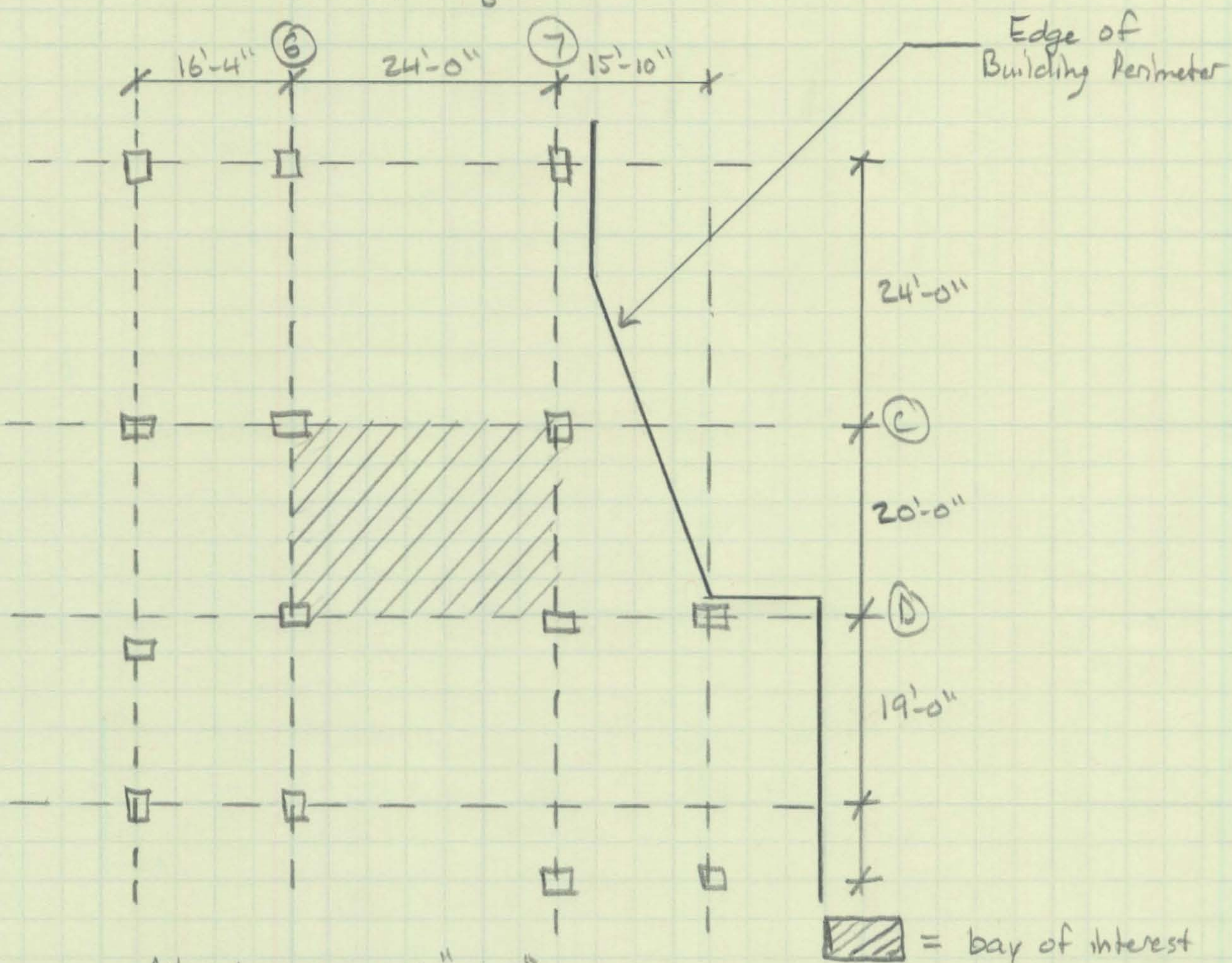
$$0.8'' > 0.317'' \checkmark$$

∴ Slab passes deflection requirements

From post-tensioning load balancing,

$$q_u = 77 \text{ psf}$$

$$M_o = \frac{q_u \ell_2 \ell_n^2}{8}$$



- All columns are 16"x24"

Long Direction:

Along C: $M_o = \frac{(77)(10+12)(24-2)^2}{8\times1000} = 102.5'^k$

Along D: $M_o = \frac{(77)(10+9.5)(24-2)^2}{8\times1000} = 90.8'^k$

Short Direction:

Along 6: $M_o = \frac{(77)(8.16+12)(20-1.25)^2}{8\times1000} = 68.2'^k$

Along 7: $M_o = \frac{(77)(12+12)(20-1.25)^2}{8\times1000} = 81.2'^k$

Coefficients for Factored Moments:

Interior Spans:

- at Midspan ... $0.35\ M_o$
- at Columns ... $0.65\ M_o$

Exterior Edge:

- at Midspan ... $0.33\ M_o$
- at Columns ... $0.67\ M_o$

C6-C7

M-	M+
(0.65)(102.5) = 66.63'k	(0.35)(102.5) = 35.88'k

D6-D7

M-	M+
(0.65)(90.8) = 59.0'k	(0.35)(90.8) = 31.78'k

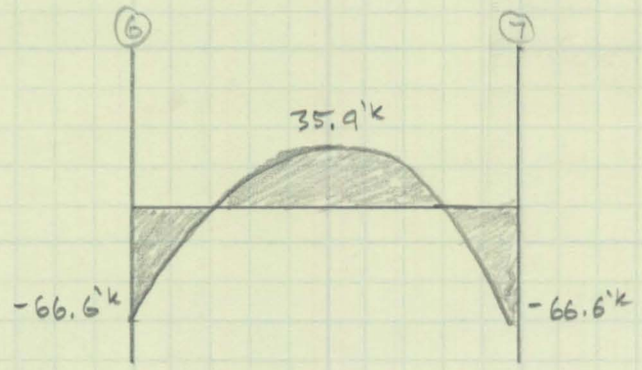
C6-D6

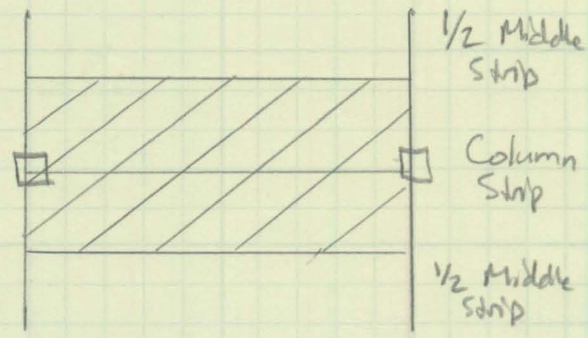
M-	M+
(0.67)(68.2) = 45.7k	(0.33)(68.2) = 22.5'k

C7-D7

M-	M+
(0.67)(81.2) = 54.4'k	(0.33)(81.2) = 26.8'k

For Column line C:





$$\frac{l_2}{l_1} = \frac{20}{24} = 0.83 \qquad \left.\begin{array}{l}\alpha = 0\\ \beta = 0\end{array}\right\} \text{No beams}$$

% of Negative Moment : Section 13.6.4.1 ACI-318

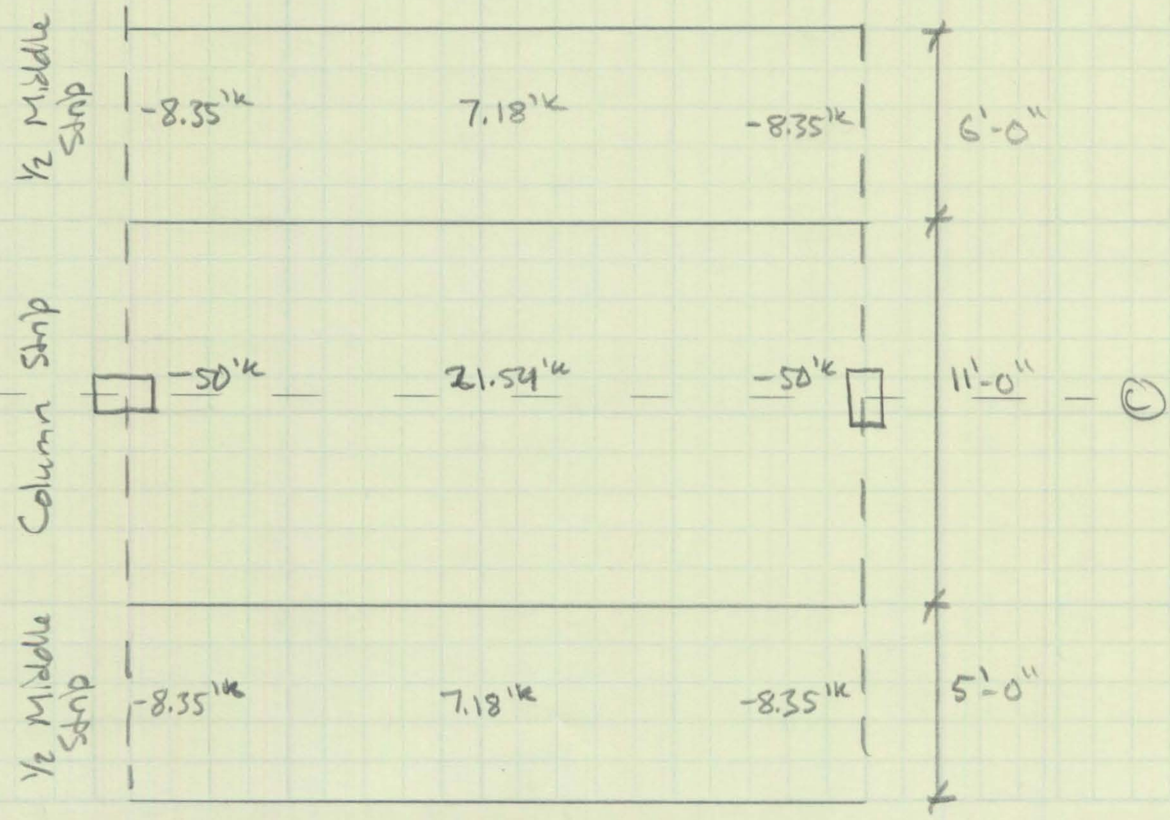
$$\alpha_f \frac{l_2}{l_1} = 0 \rightarrow 75\%$$

$0.75(^-66.6) = {}^-50'^k$ Column
$0.25(^-66.6) = {}^-16.7'^k$ Middle

% of Positive Moment : Section 13.6.4.1 ACI-318

$$\alpha_f \frac{l_2}{l_1} = 0 \rightarrow 60\%$$

$0.6(35.9) = 21.54'^k$ Column
$0.4(35.9) = 14.36'^k$ Middle



Check Reinforcement for Bending Capacity:

Column Strip: $M^+ = 21.54'^k$

$A_s = (6)\#5 @ 12'' \longrightarrow (6)\#5$
$\#4 @ 30'' \longrightarrow (4)\#4$

$A_s = 6(.31) + 4(.2) = 2.66 \text{ in}^2$

$d = 7.25 - \underset{\text{Clear Cover}}{.75} - \underset{\#6}{.75} - \underset{\#5}{\frac{.625}{2}}$

$d = 5.4375''$

$$a = \frac{(2.66 \text{ in}^2)(60{,}000 \text{ psi})}{0.85(5000 \text{ psi})(24' \times 12''/\text{ft})} = 0.13$$

$$\phi M_n = 0.9(2.66)\left(5.438 - \frac{0.13}{2}\right)(60{,}000)\left(\frac{1^k}{1000 \text{ lb}}\right)\left(\frac{1'}{12''}\right)$$

$\phi M_n = 64.3'^k$

$\phi M_n = 64.3'^k \geq M^+ = 21.5'^k$

$\therefore$ Slab is adequate at Column strip

Middle Strip: $M^+ = 14.36'^k$

$A_s = 1.86 \text{ in}^2 \rightarrow$ same reinforcing as above.

$$a = \frac{(2.66 \text{ in}^2)(60{,}000 \text{ psi})}{0.85(5000)(24 \times 12)} = 0.13$$

$$\phi M_n = 0.9(2.66)\left(5.438 - \frac{.13}{2}\right)(60000)\left(\frac{1'^k}{12000''\#}\right)$$

$\phi M_n = 64.3'^k$

$\phi M_n = 64.3'^k \geq M^+ = 14.36'^k$

CS + MS at Column: $M^- = -50^k$

- A_s will remain the same, thus $\phi M_n =$ $'^k$

$\phi M_n = -64.3'^k > M^- = -50'^k$

Moment Capacity:

- All spans, column strips, and middle strips in the bay being analyzed have the same reinforcing pattern and A_s.

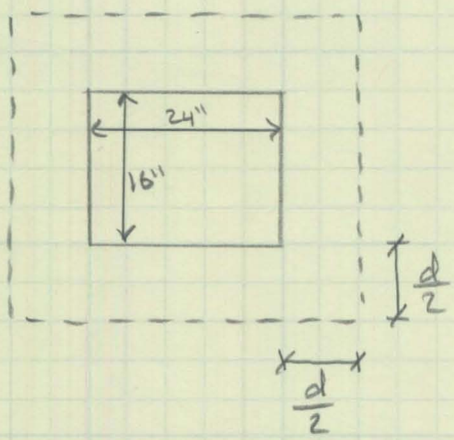
- Column line C had the largest applied moments across the furthest spans. Therefore, by inspection, because the slab has adequate moment capacity along C, it is safe to assume the other spans and strips are of adequate capacity.

Check Shear:

Two Way shear:

@ Column C6

$q_u = 77$ psf



$$\frac{d}{2} = \frac{5.4375}{2} = 2.71'' \rightarrow 2.75''$$

$$\alpha_s = 40$$

$$b_o = 2(29.5) + 2(21.5) = 102 \text{ in.}$$

$$\beta = \frac{24''}{16''} = 1.5$$

$$V_c = \begin{cases} 2 + \frac{4}{\beta} = 2 + \frac{4}{1.5} = 4.67 \\ \frac{\alpha d}{b_o} \times 2 = \frac{40(5.48)}{102} \times 2 = 4.29 \\ \text{min} \quad 4 \longrightarrow \text{Controls} \end{cases}$$

Shear Continued...

$$V_c = 4\sqrt{5000}\,(102)(5.4375'')\left(\frac{1}{1000}\right)$$

$$V_c = 156.87^k$$

$$\phi V_c = 0.75(156.87) = 117.65^k$$

$$V_u = (0.077)\left[(10+12)\times(12+8.16) - \left(\frac{29.5\times21.5}{144}\right)\right]$$

$$V_u = 33.8^k < 117.65^k \ \checkmark$$

- Near columns, the drape of the post-tensioning isn't helping reduce floor loads as much as at the mid-span.

- Consider loading of just floor load without post tensioning

$$q_u = 212\text{ psf}$$

$$V_u = 0.212\left[(22)(20.16) - 4.4\right]$$

$$V_u = 93.1^k$$

$$\phi V_c = 117.65^k > V_u = 93.1^k$$

- Check One-Way Shear with loading strictly due to the floor loading. This is a conservative approach.

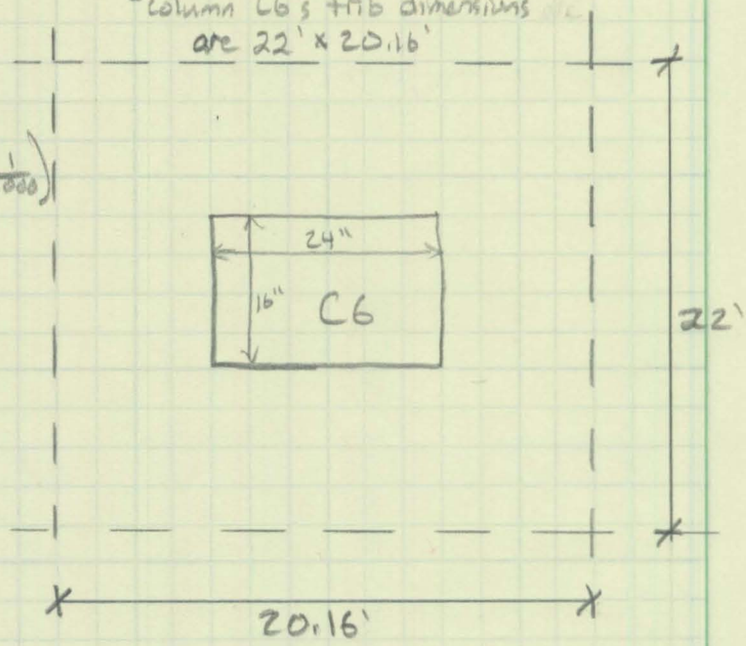
<u>One Way Shear:</u>

@ Column C6

$V_u = (0.212\,ksf)(22' - \frac{16}{12})(20.16 - \frac{24}{12})$

$V_u = 79.57^k$

- span of bay is 24' x 20'
- column C6's trib dimensions are 22' x 20.16'



$V_c = 2\lambda\sqrt{f'_c}\, b_w d$

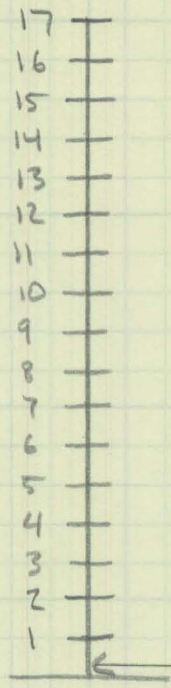
$= 2\sqrt{5000}\,(24 \times 12)(5.4375)(\frac{1}{1000})$

$V_c = 221.47^k$

$\phi V_c = 0.75 = 166^k$

$\phi V_c = 166^k > V_u = 79.6^k$

Column Gravity Checks



- Check Interior Column: C6
- Check Exterior Column: C1

Concrete Strength, $f'c$	
Above 8^{th}	5000 psi
4^{th} – 8^{th}	6000 psi
Below 4^{th}	7000 psi

Size:	C6	C1
Above 4^{th}	16" x 24"	16" x 24"
4^{th} + Below	18" x 24	18" x 24"

Reinforcing:	C6	C1
Above 8^{th}	8 #8 #4 @ 12"	8 #8 #4 @ 12"
5, 6, 7	8 #10 #4 @ 12"	8 #10 #4 @ 12"
4 + Below	8 #11 #4 @ 12"	8 #11 #4 @ 12"

Interior Column – Column C6

$$\text{Trib Area @ each level} = (22')(20.16') = 443.5\ ft^2$$

17th floor

$$\text{Dead} = 180\ psf\ (443.5\ ft^2) = 79.83^k$$
$$\text{Roof} = 20\ psf\ (443.5\ ft^2) = 8.87^k$$

Levels 16 to 5

$$\text{Dead} = 98\ psf\ (443.5\ ft^2) = 43.46^k$$
$$\text{Live} = 50\ psf\ (443.5\ ft^2) = 22.18^k$$

Levels 4 to 1

$$\text{Dead} = 130\ psf\ (443.5\ ft^2) = 57.66^k$$
$$\text{Live} = 50\ psf\ (443.5\ ft^2) = 22.18^k$$

$$DL = 79.83^k + 12(43.46^k) + 4(57.66^k) + \frac{150\ pcf}{1000}\left[\left(\underbrace{\frac{16}{12}\times\frac{24}{12}\times 109}_{290.67}\right) + \left(\underbrace{\frac{18}{12}\times\frac{24}{12}\times 39.83}_{119.5}\right)\right]$$

$$DL = 893.52^k$$

$$LL = 16(22.18) = 354.88^k$$

$$P_u = 1.2DL + 1.6LL + 0.5L_r$$

$$P_u = 1.2(893.52) + 1.6(354.88) + 0.5(79.83)$$

$$\boxed{P_u = 1680^k}$$

Column Strength - Section 10.3.6.2
↳ Non-prestressed members w/ tie reinforcement

$$\phi P_n = 0.8\,\phi\left[0.85 f'_c (A_g - A_s) + f_y A_s\right]$$

$\phi = 0.65$

$A_g = 18'' \times 24'' = 432\ in^2$

$A_s = 8(1.56\ in^2) = 12.48\ in^2$

$f'_c = 7{,}000\ psi$

$$\phi P_{n,max} = 0.8(0.65)\left[0.85(7000)(432 - 12.48) + 60{,}000(12.48)\right]$$

$$\phi P_{n,max} = 1687.4^k$$

$$P_u = 1680^k \le \phi P_n = 1687.4^k$$

- Interior Column C6 passes for compression gravity loads

Exterior Column - Column C1

Trib Area @ each level $= (22')(8.16'+1')$

$= 201.5\ ft^2$

17th Floor — Typical Masonry Wall = 345 plf

Dead $= 180\ psf\,(201.5\ ft^2) + 345(22') = 43.86^k$
Roof $= 20\ psf\,(201.5\ ft^2) = 4.03^k$

Levels 16 to 5

Dead $= 98\ psf\,(201.5\ ft^2) + 345(22') = 27.34^k$
Live $= 50\ psf\,(201.5\ ft^2) = 10.08^k$

Levels 4 to 1

Dead $= 130\ psf\,(201.5\ ft^2) + 345(22') = 33.79^k$
Live $= 50\ psf\,(201.5\ ft^2) = 10.08^k$

$$DL = 43.86^k + 12(27.34^k) + 4(33.79^k) + \frac{150\ pcf}{1000}\left[\left(\frac{16}{12}\times\frac{24}{12}\times 109'\right)+\left(\frac{18}{12}\times\frac{24}{12}\times 39.83'\right)\right]$$

$$DL = 568.6^k$$

$$LL = 16(10.08) = 161.28^k$$

$$P_u = 1.2DL + 1.6LL + 0.5L_r$$ ← controls over 1.4DL by inspection

$$P_u = 1.2(538.6) + 1.6(161.28) + 0.5(4.03)$$

$$P_u = 942.38^k$$

$$\phi P_{n,max} = 0.8(0.65)\left[0.85(7000)(432-12.48) + 60{,}000(12.48)\right]$$
$$= 1687.4^k$$

$$P_u = 942.4^k \le \phi P_n = 1687.4^k \checkmark$$

- Exterior Column passes for compression gravity loads

Proposed Design Alternates

① 2 Way Mild Reinforced Flat Slab

I will redesign the typical bay as a 2 way concrete slab without the use of post tensioning, I will then draw comparisons between the two systems and produce advantages & disadvantages of using one over the other.

② Composite Steel Beam

I will redesign the typical bay as a composite steel deck and beam system. Beam sizes will be selected based on economy of the design

③ Non-Composite Steel Beam

I will redesign the typical bay as a non-composite steel system, I will investigate the economy of orienting the beams in different directions,

I will conclude with comparing the results of the composite and non-composite steel beam design and provide advantages and disadvantages for each.

Design Alternate #1
Mild Reinforced 2 Way Slab

Design Alternate #1: Two way mild reinforced flat slab

- Keep same bay size as existing

1) Determine Slab Thickness: Section 9.5.3 ACI-318

Table 9.5(c)
- $f_y = 60{,}000$ psi
- no drop panels
- interior panel

$\left.\right\} t_{min} = \frac{\ell_n}{33}$

ℓ_n = clear span (face to face) distance in long direction

$$t_{min} = \frac{23'}{33'} = 0.7' \rightarrow 8.4'' \rightarrow \text{Try } 9'' \text{ slab}$$

2). Use Direct Design Method to find slab moments: Section 13.6 ACI-318

- limitations:
  - minimum of three continuous spans $\longrightarrow \therefore$ Ok
  - span lengths should differ by less than 1/3 ℓ_n

    $\frac{1}{3}(24') = 8'$, Successive spans must be greater than 16'.

    $\therefore$ Ok, meets criteria
  - Columns offset from the grid cannot exceed 10% of the span.

    Offset at column 7-D ≈ 18"

    $(0.10)(20') = 24''$ $\quad 18'' < 24''$ $\quad \therefore$ Ok
  - Unfactored Live Load must not exceed twice the unfactored Dead Load

    $$\frac{L}{D} = \frac{50 \text{ psf}}{115 \text{ psf}} \leq 2.0 \quad \therefore \text{Ok}$$

    → Assumes 9" slab and allowance for finishes

  - This bay meets all criteria/limitations to apply the Direct Design Method.

Load Determination

$LL = 50$ psf

$DL_{sw} = 9/12(150) + 2\frac{1}{2} = 115$ psf
↳ allowance for finishes

Misc. DL = 15 psf

Load case : 1.2 DL + 1.6 LL

$1.2(115+15) + 1.6(50) = 236$ psf

$q_u = 236$ psf

- Because the initially sized slab is greater than the values in Table 9.5(c) of ACI-318, deflection control can be neglected.

Proceed with flexural and shear design.

Check Shear Capacity

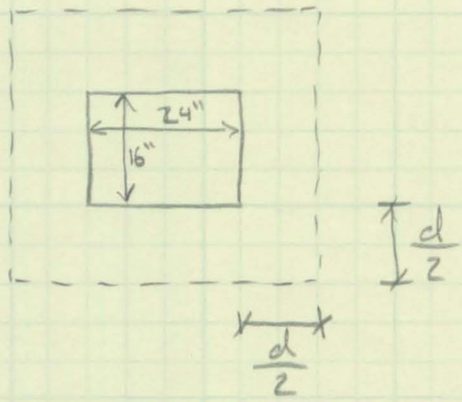
Two-Way Shear - Column C6

$q_u = 236\ \text{psf}$

-assume 3/4" Clear Cover

$d = 9 - \frac{3}{4}" - \frac{3(.625)}{2}$
$= 7.32$

$\frac{d}{2} = 3.66"$



$b_o = 2(24 + 7.32) + 2(16 + 7.32)$
$b_o = 109.28$

$\beta = \frac{24"}{16"} = 1.5 \qquad \alpha_s = 40$

$$V_c = \min \left| \begin{array}{l} 2 + \frac{4}{1.5} = 4.67 \\ \frac{40(7.32)}{109.3} \times 2 = 5.36 \\ \underline{4} \longleftarrow \text{Controls} \end{array} \right.$$

$V_c = 4\sqrt{5000}\,(109.3)(7.32)\left(\frac{1}{1000}\right)$

$V_c = 226.3^k \longrightarrow \phi V_c = 0.75(226.3)$

$\boxed{\phi V_c = 169.7^k}$

$V_u = (0.236)\left[(22)(20.16) - \left(\frac{31.32 \times 23.32}{144}\right)\right]$

$V_u = 103.5^k$

$\phi V_c = 169.7^k > V_u = 103.5^k$ ✓

- Slab passes for punching shear, check one-way shear

<u>One Way Shear</u>

$$V_u = (0.236\,\text{ksf})\left(22 - \tfrac{16}{12}\right)\left(20.16 - \tfrac{24}{12}\right)$$

$$V_u = 88.6^k$$

$$V_c = 2\,(1.0)\sqrt{5000}\,(24 \times 12)(7.32)\left(\tfrac{1}{1000}\right)$$

$$V_c = 298.14^k \rightarrow \phi V_c = 0.75(298.14) = 223.6^k$$

$$\phi V_c = 223.6^k > V_u = 88.6^k$$

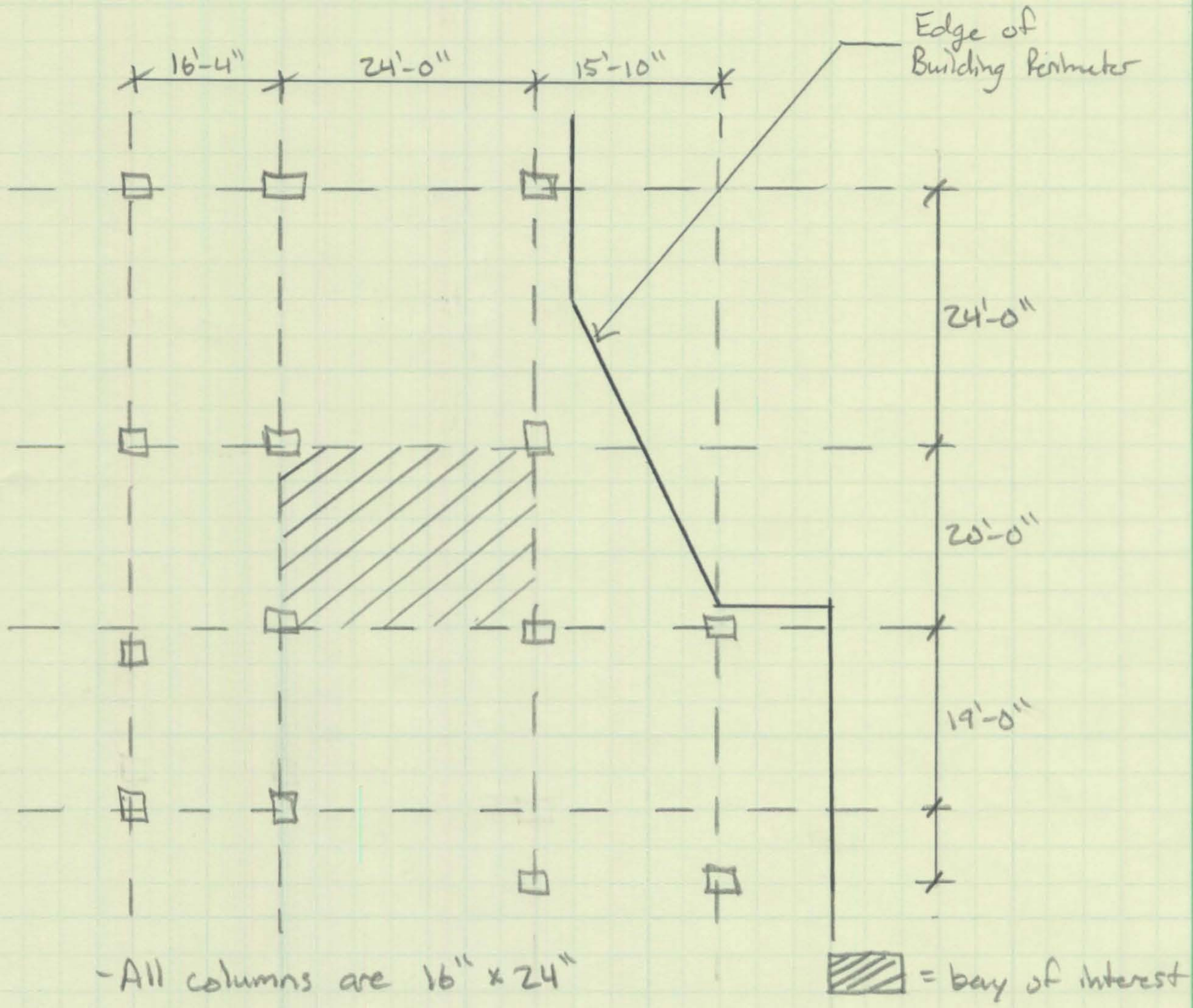
- Slab passes one way shear check

∴ Proceed with flexural design using DDM

Direct Design Method

$q_u = 236$ psf $\qquad M_o = \dfrac{q_u \, l_2 \, l_n^2}{8}$

Plan showing analyzed bay:
- Typ. Apartment floor is shown



- All columns are 16" x 24"

Slab Moment Determination

Long Direction:

Along C: $M_o = \frac{(236)(22)(24-\frac{24}{12})^2}{8000} = 314.1'^k$

Along D: $M_o = \frac{(236)(19.5)(24-2)^2}{8000} = 278.4'^k$

Short Direction:

Along 6: $M_o = \frac{(236)(20.16)(20-1.33)^2}{8000} = 207.3'^k$

Along 7: $M_o = \frac{(236)(24)(18.67)^2}{8000} = 246.8'^k$

Coefficients for Factored Moments:

Interior Spans:

- at Midspan.... $0.35 M_o$
- at Columns.... $0.65 M_o$

Exterior Edge

- at Midspan..... $0.33 M_o$
- at Columns..... $0.67 M_o$

C6-C7

M-	M+
$(0.65)(314.1) = 204.2$	$(0.35)(314.1) = 109.9'^k$

D6-D7

M-	M+
$(0.65)(278.4) = 181.0'^k$	$(0.35)(278.4) = 97.4'^k$

C6-D6

M-	M+
$(0.67)(207.3) = 138.9'^k$	$(0.33)(207.3) = 68.4'^k$

C7-D7

M-	M+
$(0.67)(246.8) = 165.4'^k$	$(0.33)(246.8) = 81.4'^k$

Design Reinforcing for Bending Capacity:

$$A_{s,required} = \frac{M_u}{\phi f_y jd}$$

Worst Case Moment experienced by the slab for M^- in CS

$$= \frac{-153.2'^k \times 12''}{(0.9)(60{,}000\,psi)(0.95)(7.32'')(\frac{1}{1000})}$$

$$A_{s,req} = 4.896\,in^2$$

Spacing: Section 13.3.2 ACI-318

$$S_{max} \leq 2h = 2(9'')$$

$$S_{max} \leq 18''$$

Minimum Reinforcing: Section 13.3.1 ACI-318

$$A_{s,min} \geq 0.0018\,bh$$

b = width of CS = 11'
h = slab thickness = 9"

$$A_{s,min} \geq 0.0018(9)(11)(12)$$

$$A_{s,min} \geq 2.14\,in^2$$

$$A_{s,req} > A_{s,min} \quad \therefore \text{ ok}$$

$A_{s,req} = 4.896\,in^2 \longrightarrow$ Try (16) #5's @ 12"

$$16(0.31) = 4.96\,in^2 > 4.896\,in^2$$

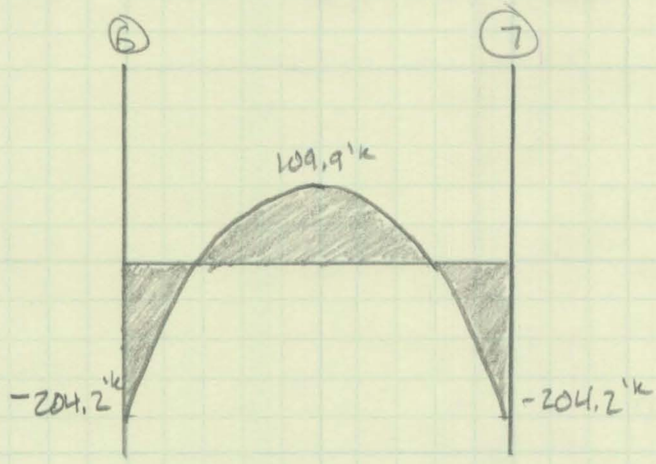
$$a = \frac{(4.96\,in^2)(60{,}000)}{0.85(5000)(11 \times 12)} = 0.53$$

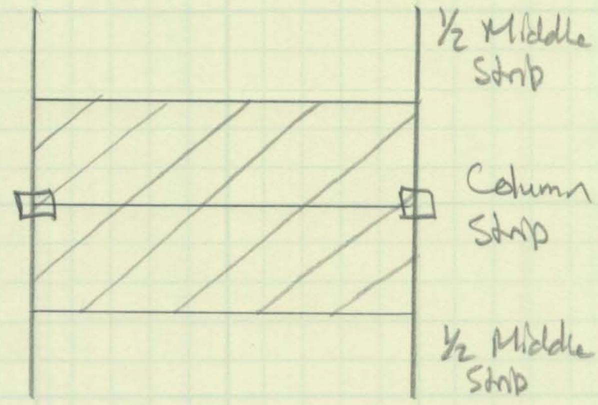
$$\phi M_n = 0.9(4.96)(7.32 - \frac{.53}{2})(60{,}000)(\frac{1}{1000})(\frac{1}{12})$$

$$\phi M_n = 157.5'^k$$

$$\phi M_n = 157.5 \geq M^- = 153.2'^k$$

For Column line C:





$$\frac{\ell_2}{\ell_1} = \frac{20}{24} = 0.83$$

$\left.\begin{array}{l}\alpha = 0 \\ \beta = 0\end{array}\right\}$ No beams

% Negative Moment: Section 13.6.4.1 ACI-318

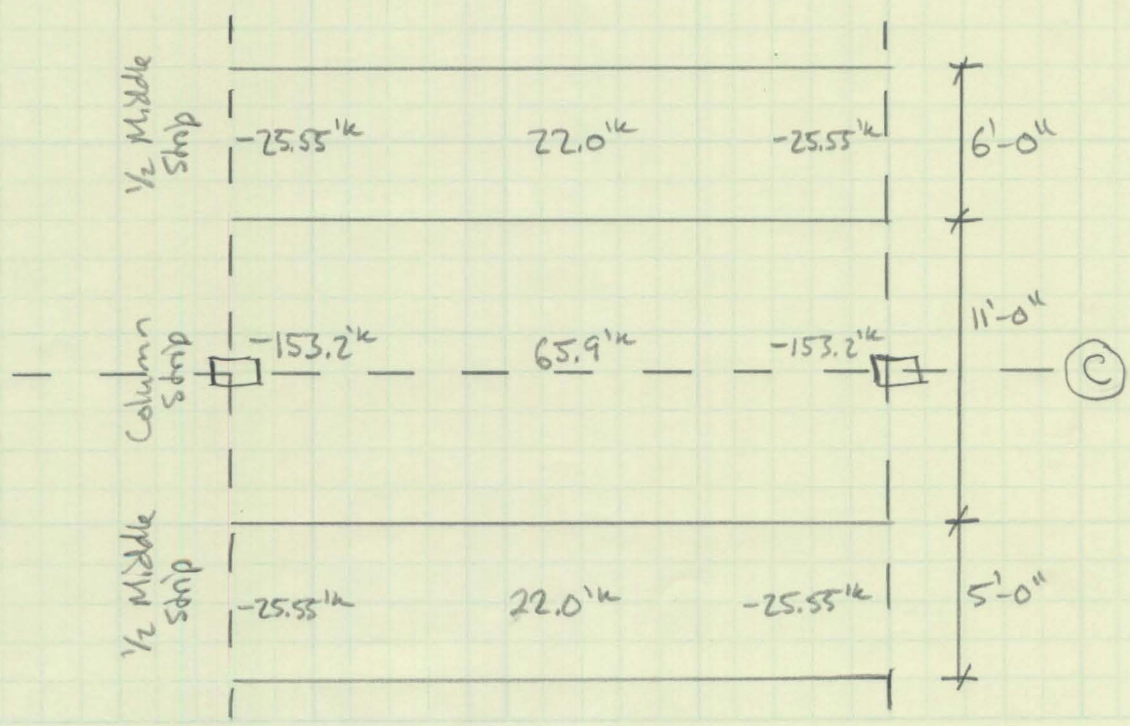
$\alpha_f \frac{\ell_2}{\ell_1} = 0 \rightarrow$ 75% to CS $\quad 0.75(-204.2) = -153.2'^k$

25% to MS $\quad 0.25(-204.2) = -51.1'^k$

% of Positive Moment: Section 13.6.4.1 ACI-318

$\alpha_f \frac{\ell_2}{\ell_1} = 0 \rightarrow$ 60% to CS $\quad 0.6(109.9) = 65.9'^k$

40% to MS $\quad 0.4(109.9) = 44.0'^k$



Middle Strip, M^-

$$A_{s,req'd} = \frac{-51.1 \times 12}{0.9(60,000)(.95)(7.32)(\frac{1}{1000})}$$

$A_{s,req'd} = 1.63\ in^2$ ⟵ $A_{s,req} < A_{s_{min}}$, $A_{s_{min}} = 2.14\ in^2$ and was calculated on the last page. Therefore, use $2.14\ in^2$

$A_{s,req} = 2.14\ in^2 \rightarrow$ Try (7) #5's @ 12"

$(7)(.31) = 2.17\ in^2$

$$a = \frac{(2.17)(60000)}{0.85(5000)(11 \times 12)} = 0.232$$

$$\phi M_n = 0.9(2.17)\left(7.32 - \frac{.232}{2}\right)(60,000)\left(\frac{1}{1000}\right)\left(\frac{1}{12}\right)$$

$\phi M_n = 70.35'^k$

$\phi M_n = 70.35'^k \geq M^- = 51.1'^k$

Column Strip, M^+

$$A_{s,req'd} = \frac{65.9 \times 12}{(0.9)(60,000)(.95)(7.32)(\frac{1}{1000})}$$

$A_{s,req'd} = 2.11\ in^2 \rightarrow$ same as above, $A_{s,req} < A_{s_{min}}$

$\therefore$ Use $A_{s_{req}} = 2.14\ in^2$

- Use same configuration as above...

(7) #5's @ 12"....

$\phi M_n = 70.35'^k \geq M^+ = 65.9'^k$

Middle Strip, M^+

$$A_{s,req'd} = \frac{22 \times 12}{0.9(60{,}000)(.95)(7.32)(\frac{1}{1000})} = 0.7\ in^2$$

$A_{s,req'd} = 0.7\ in^2 \leftarrow$ same as last page, $A_{s,req} < A_{s,min}$

$\therefore$ Use $A_{s,req} = 2.14\ in^2$

- Use same configuration....

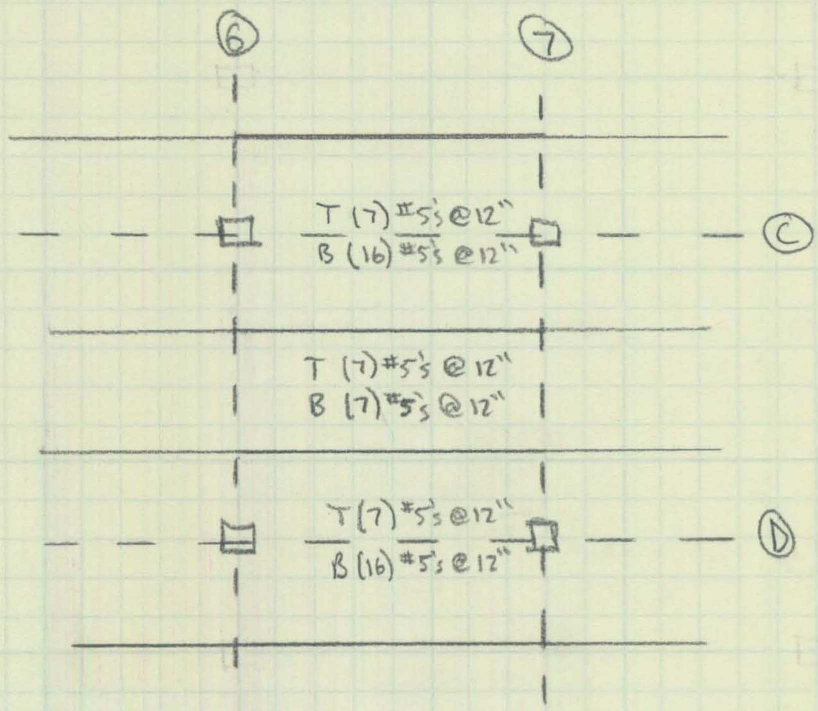
(7) #5's @ 12"

$\phi M_n = 70.35'^k > M^+ = 22.0'^k$

Slab Reinforcing Summary:

Column Strip: Top = (7) #5's @ 12"
Bottom = (16) #5's @ 12"

Middle Strip: Top = (7) #5's @ 12"
Bottom = (7) #5's @ 12"



PT vs. Mild Reinforced 2-Way Slab Comparison

Post-Tensioned Slab	Mild-Reinforced
- 7.25" thickness	9" thickness
- no drop panels	- no drop panels
- Band + Distributed bands + Mild Reinforced bars	- Mild reinforced bars
- $A_s = 2.66 in^2$	- $A_s = 4.96 in^2$

Conclusions:

- The greatest challenge and concern with this system comparison is the thickness of the slab. This is a concern because the building is already at the maximum allowable height. With added thickness to the structure, all of the floors might not be able to fit it.

  Although both systems were able to go without drop panels while maintaining a reasonable depth, the mild reinforced slab is 1.75" deeper. This will add a total of 2'-0" of additional height to the building. Or by maintaining the same height, the typical apartment floor to ceiling height will drop to 7'-7"

  The IBC code minimum for floor to ceiling height is 7'-6". The mild reinforced slab system still meets code but might comprimise tenant satisfaction.

- The mild reinforced slab requires 86% more steel. So this will induce a greater cost of materials and labor.

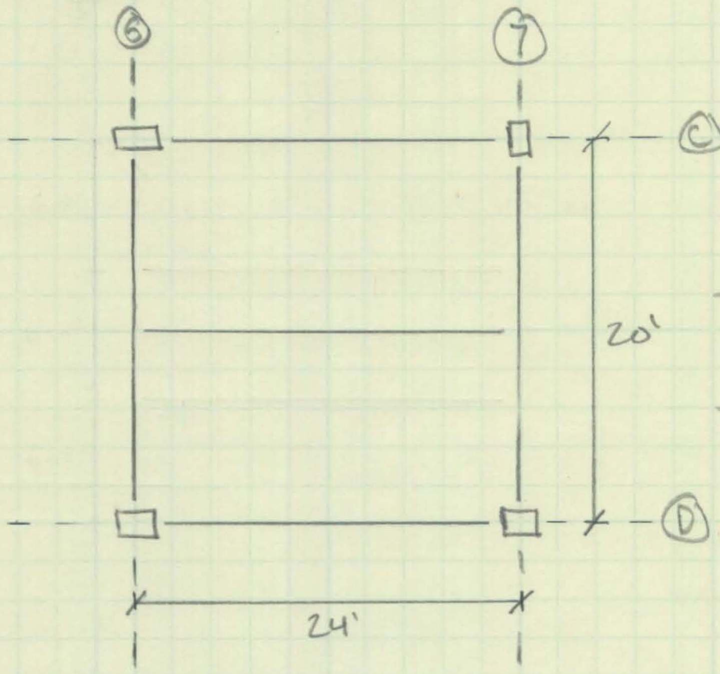
- The mild reinforced slab does not have PT tendons though, which saves steel material and labor there.

- In conclusion, I would recommend the post tensioned slab over the mild reinforced slab.

Design Alternate #2
Composite Steel Beam

Design Alternate #2: Composite Beam + Deck

Layout:



Beam: span = 24'
spacing = 10'

Girder: span = 20'
spacing = 24'

Deck:

2VLI18, 3" NW Concrete (5" total thickness) – 51 psf
– 3 span Clear span = 11'-11"

- From Vulcraft Deck Catalog

Loads:

DL = 57 + 15 + 5 + 4 = 75 psf → 15 psf
5 psf beam allowance
4 psf for finishes

LL = 50 psf

Design Loads:

load case = 1.2DL + 1.6LL

$W_u = 1.2(75) + 1.6(50) = 170$ psf

170 psf (10) = 1.7 klf

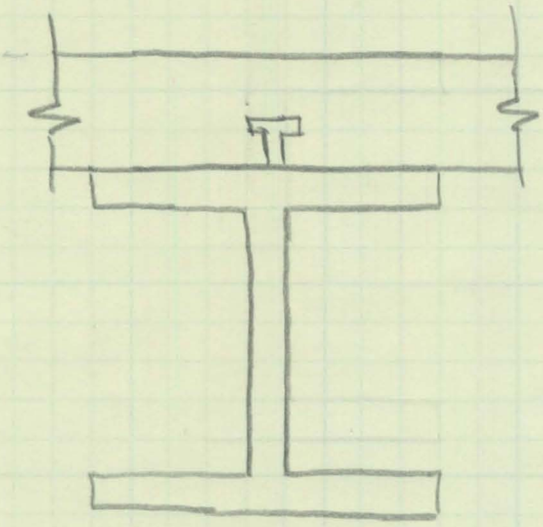
Composite Beam Design

Design Moment:

$$M_u = \frac{w l^2}{8} = \frac{3.4(24)^2}{8}$$

$M_u = 122.4'^k < 128'^k$ ← Max Moment from Vulcraft

How much of the concrete is engaged?



24' beam, max 1 stud/ft,
1 stud/rib = 17.2k/stud
2 stud/rib = 14.6k/stud

- Assume a = 1"

$\longrightarrow Y2 = 5" - \frac{1}{2} = 4.5"$

- Possible Beam sizes (Table 3-19)

W10x19, $\Sigma Q_n = 70.3^k \rightarrow \frac{70.3}{17.2} = 4.09 \rightarrow 5 \times 2 = 10$ studs per beam

W10x17, $\Sigma Q_n = 117^k \longrightarrow \frac{117}{17.2} = 6.8 \rightarrow 7 \times 2 = 14$ studs per beam

W10x15, $\Sigma Q_n = 140^k \longrightarrow \frac{140}{17.2} = 8.14 \rightarrow 9 \times 2 = 18$ studs per beam

W10x12, $\Sigma Q_n = 177^k \longrightarrow \frac{177}{17.2} = 10.3 \rightarrow 11 \times 2 = 22$ studs per beam

Check 'a' assumption:

$$b_{eff} = \min \begin{vmatrix} \frac{span}{8} \\ \frac{spacing}{2} \end{vmatrix} \times 2$$

$$b_{eff} = \min \begin{vmatrix} \frac{24(12)}{8} \times 2 = 72'' \\ \frac{10(12)}{2} \times 2 = 120'' \end{vmatrix}$$

$$a = \frac{177^k}{0.85(4)(72)} = 0.723'' < 1''$$

$\therefore Y2 = 4.5''$ is a conservative value

- Use worst case W_{sw} to check unshored length

Check Unshored Length

Consider 2 Loads Cases:

- $1.4DL \longrightarrow 1.4(75)(10) + 1.4(19) = \underline{1076.6 \text{ plf}}$
- $1.2DL + 1.6LL \rightarrow 1.2((750+19) + 1.6(50) = 1002.8 \text{ plf}$

$\therefore$ 1.4 DL load case controls

$$M_u = \frac{1.077(24)^2}{8} = 77.544'^k$$

<u>W10x19</u>	$\phi M_p = 81.0'^k > 77.54'^k$
W10x17	$\phi M_p = 70.1'^k$ ✗

$\therefore$ Proceed with W10x19, $I_x = 96.3 \text{ in.}^4$

Check Wet Concrete Deflection

$W_{wc} = 75(10) + 19 = 769 \text{ plf} \rightarrow 0.769 \text{ klf}$

$$\Delta_{wc} = \frac{5(0.769)(24)^4(1728)}{(384)(29{,}000)(96.3)} = 2.06''$$

$$-\ \Delta_{wc\,max} = \frac{L}{240} = \frac{24(12)}{240} = 1.2''$$

$\Delta_{wc} = 2.06'' > \Delta_{wc\,max}$ X Beam fails under Δ_{wc}

$\therefore$ Increase beam size to achieve higher I_x

Try W10x26,

$\Sigma Q_n = 95.1^k \rightarrow \frac{95.1^k}{17.2^k} = 5.53 \rightarrow 6 \times 2 = 12$ studs per beam

$a = \frac{95}{0.85(4)(72)} = 0.388'' < 1''$ assumed ✓

Unshored Length:

$1.4DL \rightarrow 1.4(75)(10) + 1.4(26) = \underline{1.086 \text{ klf}}$ → controls

$1.2DL + 1.6LL \rightarrow 1.2(750+26) + 1.6(50) = 1.011 \text{ klf}$

$$M_u = \frac{1.086(24)^2}{8} = 78.2'^k$$

for W10x26 .... $\phi M_p = 117'^k > 78.2'^k$ ✓
$I_x = 144 \text{ in}^4$

Wet Concrete Deflection:

$W_{wc} = 75(10) + 26 = 776 \text{ plf} \rightarrow 0.776 \text{ klf}$

$$\Delta_{wc} = \frac{5(0.776)(24)^4(1728)}{(384)(29000)(144)} = 1.39'' \quad X$$

- W10×26 section fails by deflection limits under wet concrete loading. Possible solutions are to increase the beam size, camber the beam, or shore the beams.

The depth of the structure is an important factor for this project in meeting the height restrictions for the site. Therefore, I will rule out the possibility of increasing the beam size so that I can maintain a W10 shape.

Camber:

W10×26,

$$\Delta_{wc} = 1.38''$$

$$\text{Camber} = 0.8(1.39) = 1.112 \rightarrow 1''$$

- Beams should have a 1" camber

Conclusion:

- Although the cambering solution would work, I recommend shoring the beams. Due to the variability of the bay and span lengths, this camber would be too much for some conditions. To eliminate added fabrication complexity and potential erection errors in the field, I would recommend shoring the beams at mid-span.

Check Live Load Deflection

W10×26 .... $I_{LB} = 286\ in^4$

$$W_{LL} = (50\ psf)(10\ ft)/1000 = 0.5\ klf$$

$$\Delta_{LL} = \frac{5(0.5)(24)^4 1728}{384(29000)(286)} = 0.45''$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{(24)(12)}{360} = 0.8'' > 0.45''$$

$$\Delta_{LL} < \Delta_{max}$$

- Since the beams will be shored anyway, is there any economy in using the originally investigated beam size.

Composite Beam Options:

W10x26 w/ 12 studs : $\phi M_p = 117'^k$

$26\ ^{lbs}/_{ft} \times 24' + 12 \text{ studs} \left(10^{\#}/\text{stud}\right) = 744$ lbs. of steel

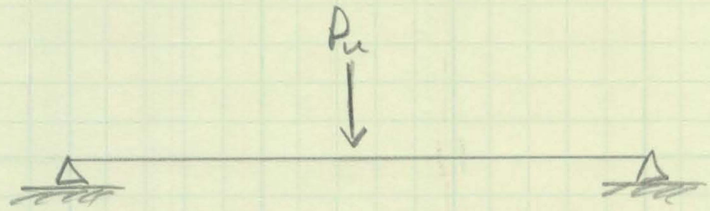
W10x19 w/ 10 studs : $\phi M_p = 81.0'^k$

$19\ ^{lbs}/_{ft} \times 24 + 10 \text{ studs} \left(10^{\#}/\text{stud}\right) = 556$ lbs. of steel

- Choose the 2nd option of W10x19 w/ 10 studs

Girder Design

- span = 20'
- spacing = 24'



$W_u = 170$ psf

$$P_u = (170\text{ psf})(2 \times 12')(20') = 81.6^k$$

Design Moment:

$$M_u = \frac{1}{4}(81.6^k)(20') = 408'^k$$

If $a = 1$, $y_2 = 4.5''$

Possible Girder Sizes:

$W16\times45,\ \Sigma Q_n = 166^k \rightarrow \frac{166}{17.2} = 9.65 \rightarrow 10\times2 = 20$ studs / beam

$W16\times40,\ \Sigma Q_n = 192 \rightarrow \frac{192}{17.2} = 11.16 \rightarrow 12\times2 = 24$ studs / beam

$W16\times36,\ \Sigma Q_n = 305 \rightarrow 2(17.2) + 10(2)(14.6) = 326.4 > 305$ ✓

$(2+20)(2) = 44$ studs / beam

Check a assumption:

$$b_{eff} = \min \begin{cases} \frac{20(12)}{8}\times2 = \underline{60"} \leftarrow \text{Controls} \\ \frac{24(12)}{2}\times2 = 288" \end{cases}$$

$$a = \frac{305}{0.85(4)(60)} = 1.495" > 1" \ \times$$

– By inspection, the W16x36 doesn't seem the most economical. Therefore, by ruling out that option, 192^k becomes our worst case ΣQ_n.

$$a = \frac{192}{0.85(4)(60)} = 0.94 < 1" \ \checkmark$$

Check Unshored Length

Consider 2 Load Cases:

– $1.4DL \rightarrow 1.4(75)(24) + 1.4(45) = \underline{2583 \text{ plf}}$

– $1.2DL + 1.6LL \rightarrow 1.2[(75)(24) + 45] + 1.6(50) = 2294$ plf

$\therefore$ 1.4 DL controls

$$M_u = \frac{2.583(20)^2}{8} = 129.2'^k$$

$M_u = 129.2'^k$

W16×45 . . . $\phi M_p = 205'^k > 129.2'^k$ ✓

$I_x = 586\ in^4$

W16×40 . . . $\phi M_p = 182'^k > 129.2'^k$ ✓

↳ economy shape

$I_x = 518\ in^4$

Check Wet Concrete Deflection

$$W_{wc,beams} = 75(20) + 45 = 1.545\ klf$$

$$\Delta_{wc} = \frac{5(1.545)(20)^4(1728)}{(384)(29000)(518)} = 0.37''$$

$$\Delta_{wc,max} = \frac{L}{240} = \frac{20(12)}{240} = 1'' > 0.37'' ✓$$

$$\Delta_{wc} < \Delta_{max}$$

Check Live Load Deflection

W16×45 . . . $I_{LB} = 1000\ in^4$

W16×40 . . . $I_{LB} = 971\ in^4$

$$W_{LL,beam} = 50\ psf(20') = 1\ klf$$

$$P_{LL} = 1\ klf(20') = 20^k$$

$$\Delta_{LL} = \frac{0.05(20)(20)^3(1728)}{(29000)(971)} = 0.49''$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{20(12)}{360} = 0.67'' > 0.49 ✓$$

$$\Delta_{LL} < \Delta_{max} ✓$$

Check Girder Economy

W16x45 w/ 20 studs

$$45(20) + 20(10) = 1100 \text{ lbs.}$$

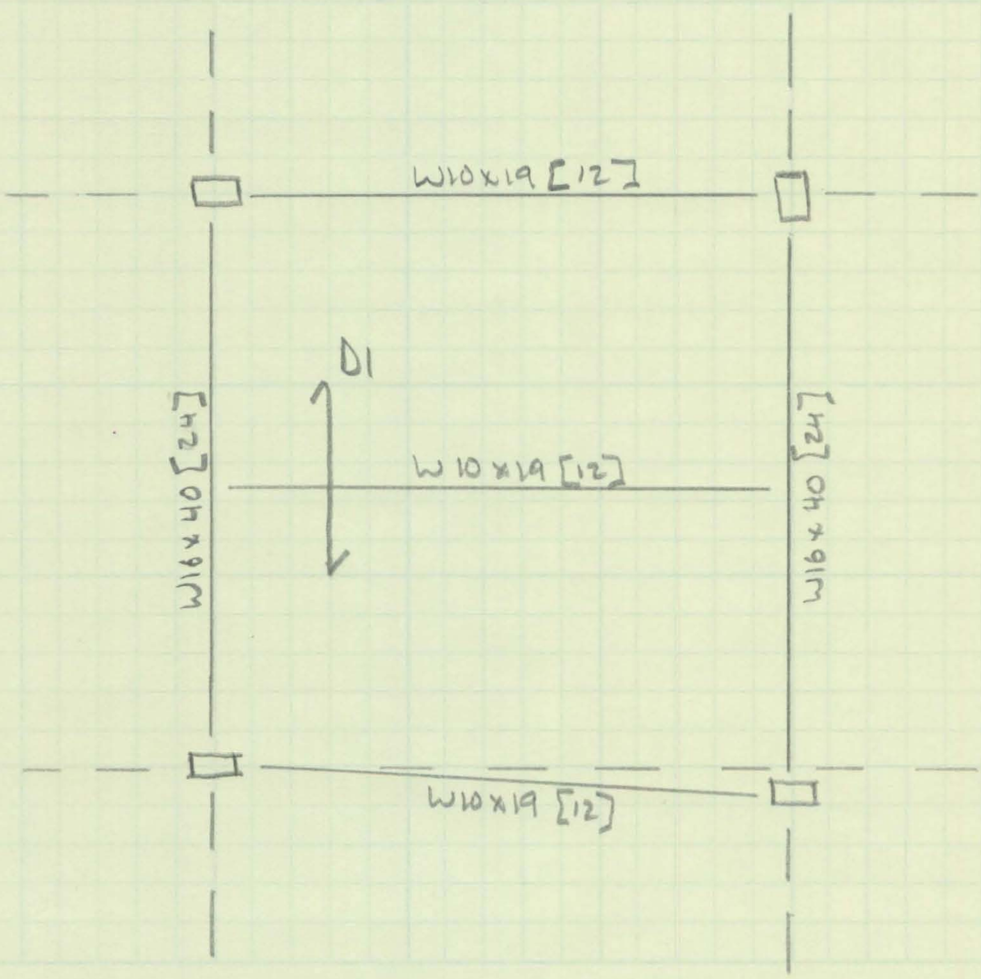
W16x40 w/ 24 studs

$$40(20) + 24(10) = \underline{1040 \text{ lbs.}}$$

∴ Select W16x40 w/ 24 studs spaced evenly across the girder

Design Summary:

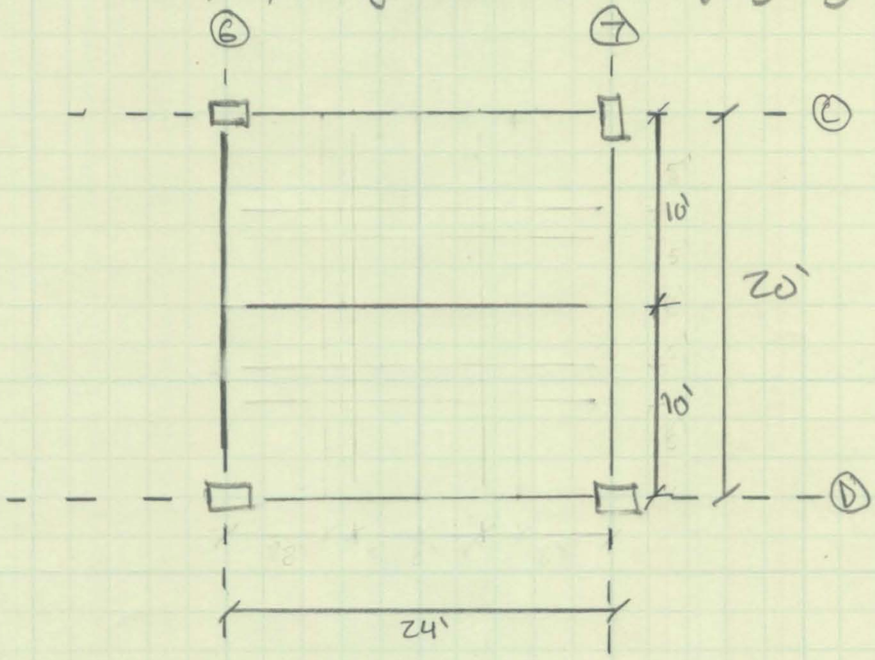
D1 = 2VLI18, 5 in NW Concrete



Design Alternate #3
Non-Composite Steel Beam

Structural Redesign #3 - Non-Composite Steel

- first, try configuration with beams going longway



-Try (1) beam spaced at 10 ft. o.c.

Span = 24'-0"
spacing = 10'-0"

LL = 50 psf

Decking:

Try 1.5 C18, 3.5" NW Concrete Deck
- 38 psf
- max span = 10'-8 for 3 spans

$$DL = \underset{\text{Deck}}{38} + \underset{\text{Superimposed}}{15} + \underset{\text{SW}}{5} + \underset{\text{Collateral}}{2} = 60\,psf$$

$$w_u = [1.2(60) + 1.6(50)] \times 10' = 1.52\ klf$$

$w_u = 1.52$ klf

$$M_u = \frac{(1.52)(24)^2}{8} = 109.4'^k$$

$M_u = 109.4'^k$

Strength

→ Try W12×26 $\phi M_n = 140'^k > 109.4'^k$

Check Deflection

- limit LL deflection to L/360

$$\Delta_{LL} = \frac{5(w_{LL})(L)^4(1728)}{384(29{,}000)(I_x)}$$

$$= \frac{5\left(\frac{50 \times 10}{1000}\right)(24)^4(1728)}{384(29{,}000)(204)} = 0.63''$$

$$\frac{L}{360} = \frac{24(12)}{360} = 0.8''$$

$0.8'' > 0.63''$ ∴ Ok for Deflection Control

Allowance

$$\frac{26\text{ plf}}{10\text{ ft.}} = 2.6 < 5\text{ psf allowance} \quad \therefore \text{Ok}$$

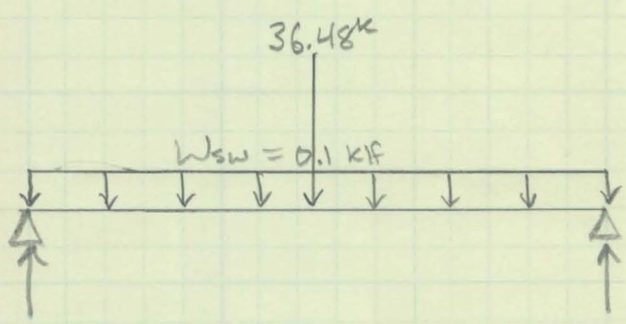
∴ Use W12×26 beams @ 10' o.c.

Girders:

span = 20'
spacing = 24'

$P = \frac{1}{2}(1.52\text{ klf})(24) = 18.24^k \times 2 = 36.48^k$

↑ 2 point loads, one from adjacent bay



$W_{sw} = 5\text{psf}(20) = 100\text{ plf}$
$W_{sw} = 0.1\text{ klf}$

Girder Design Continued..

$$M_u = \frac{1}{4}(36.48^k)(20') + \frac{0.1(20)^2}{8}$$

$$M_u = 187.4'^k$$

Strength:

Try W18x40 $\phi M_n = 196'^k > 187.4'^k$

$I = 612 \text{ in}^4$

Check Deflection

$$\Delta_{LL} = \frac{P_{LL} L^3}{28EI} = \frac{(50)(10)(24)\left(\frac{1}{1000}\right)(20)^3(1728)}{28(29{,}000)(612)} = 0.334''$$

$$\frac{L}{360} = \frac{20(12)}{360} = 0.67'' > 0.334'' \quad \therefore \text{ ok}$$

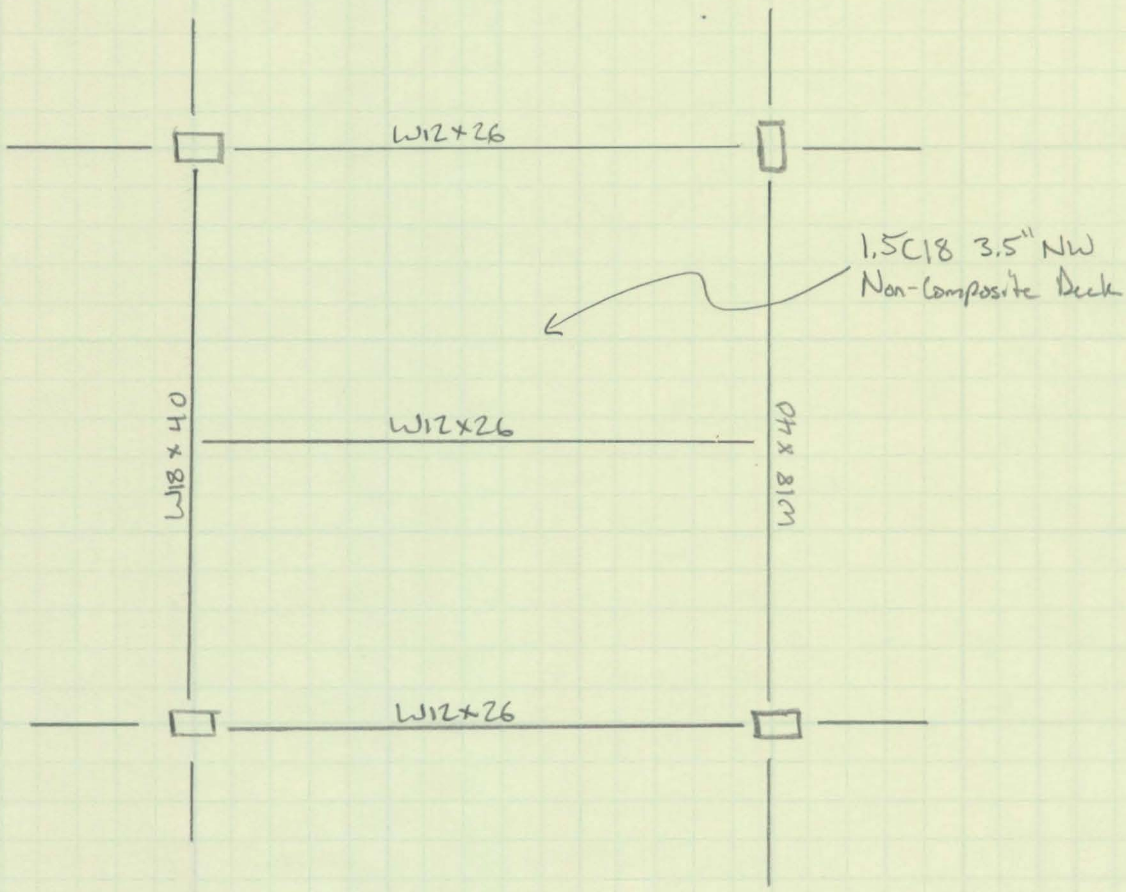
Check Allowance

$$\frac{40}{20} = 2.0 < 5 \quad \therefore \text{Ok, Girder passes}$$

Use W18x40 Girder

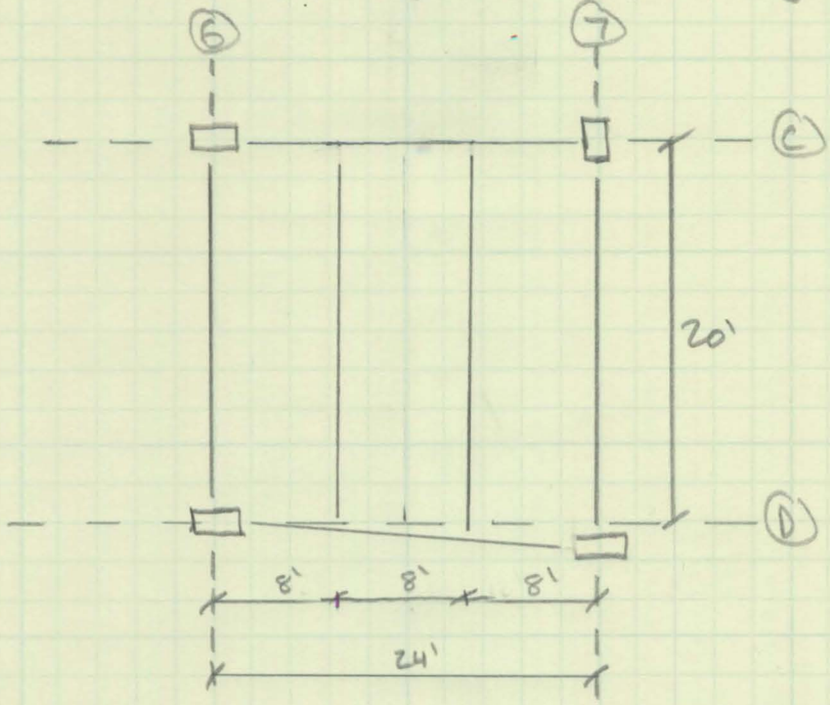
Non-Composite Steel Summary

Typical bay - Option 1



Now, investigate the configuration if the direction of the beams and girders were switched.

- Second Option, try configuration with girders going longway



Beam:

span = 20'
spacing = 8'

- Beam configuration needs two beams because otherwise the deck wouldn't be able to span 12'.

Use same decking....

1.5 C18, 3.5" NW Concrete Deck

Design Loads + Moments

$W_u = 1.52$ klf

$$M_u = \frac{1.52(20)^2}{8} = 76'^k$$

$$M_u = 76'^k$$

Strength:

Try W10×22 .... $\phi M_n = 97.5'^k > 76'^k$

Check Deflection

$$\Delta_{LL} = \frac{5\left(\frac{50 \times 8}{1000}\right)(20)^4(1728)}{384\,(29{,}000)\,(118)} = 0.42''$$

$$\Delta_{LL,max} = \frac{L}{360} = \frac{20 \times 12}{360} = 0.67'' > 0.42''$$

$\Delta_{LL} < \Delta_{max}$ ∴ Ok for Deflection Control

Allowance

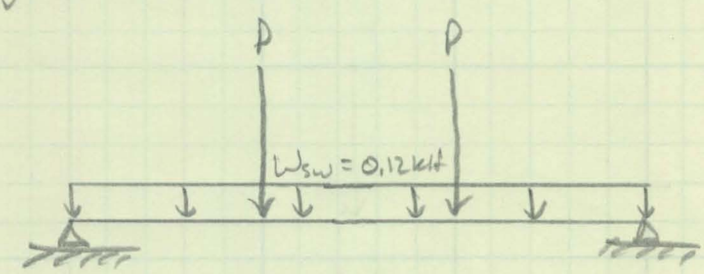
$$\frac{22\text{ plf}}{8} = 2.75 < 5\text{ psf allowance}$$

∴ Use W10×22 beams at 8'-0" o.c.

Girders:

span = 24'
spacing = 20'

$$P = \frac{1}{2}(1.52\text{ klf})(20) = 15.2^k \times 2 = 30.4^k$$



$$W_{sw} = 5\text{ psf}\,(24) = 120\text{ plf} = .12\text{ klf}$$

$$M_u = (8')(30.4^k) + \frac{0.12\,(24)^2}{8} = 251.8'^k$$

$M_u = 251.8'^k$

Strength

Try W16x40 ... $\phi M_n = 274'^k > 251'^k$
$I = 578 in^4$

$$\Delta_{LL} = \frac{50(8)(20)(\frac{1}{1000})(24)^3(1728)}{28(29,000)(578)} = 0.407''$$

$$\Delta_{LL,max} = \frac{24(12)}{360} = 0.8'' > 0.407''$$

$$\Delta_{LL} < \Delta_{max} \checkmark$$

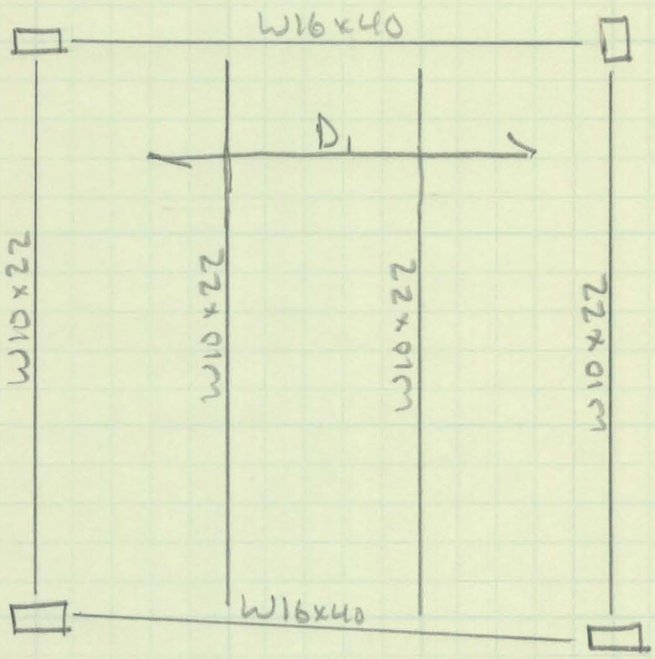
Check Allowance

$$\frac{40}{24} = 1.67 < 5 psf \quad \therefore Ok$$

$\therefore$ Use 16x40 Girder

Non-Composite Steel Summary

Typical Bay – Option 2



D1 = 1.5C18 3.5" NW Concrete Deck

Non-Composite Beam Configuration Selection

Compare Steel Weights

Beams Long Way

$(3)(26)(24') + (2)(40)(20)' = 3472^{\#}$

Girders Long Way

$4(22)(20') + 2(40)(24) = 3680^{\#}$

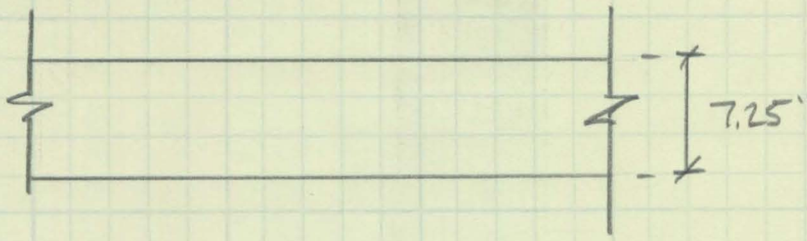
- Orienting the beams along the long direction is more economical.

Composite vs. Non-Composite Comparison

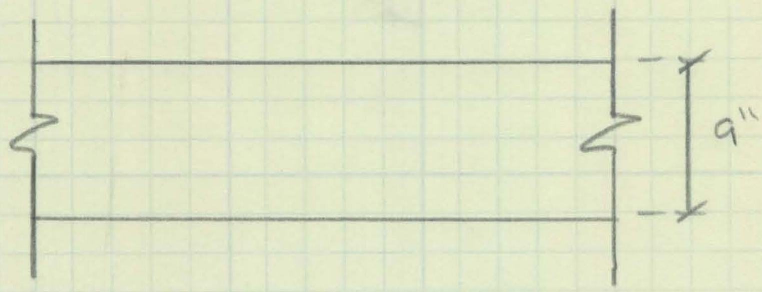
Composite Design	Non-Composite Design
Members: (2) W16×40 [24] (3) W10×19 [12]	Members: (2) W18×40 (3) W12×26
Weight: 3808 #	Weight: 3472 #
Deck: 2VLI18, 5" NW	Deck: 1.5C18 3.5" NW
- needs shoring	- no shoring
Total Depth: 21"	Total Depth: 21.5"

- Both systems result in similarly sized members. The depth of the two systems is nearly the same but because the composite system requires shoring and more steel, I would recommend non-composite beams.

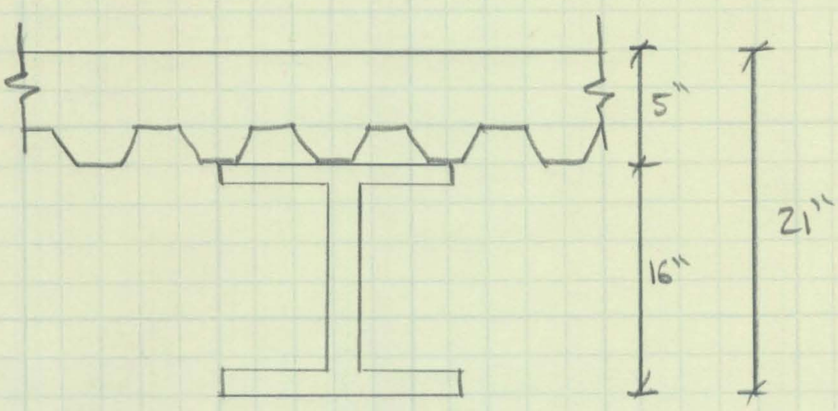
Existing Post-Tension Slab



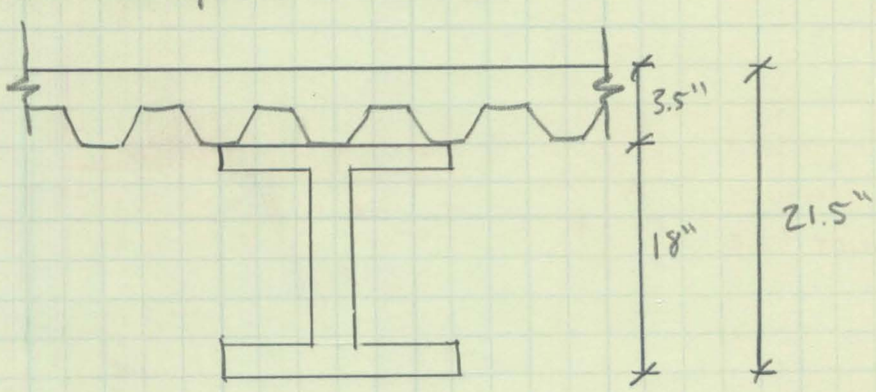
Mild Reinforced 2-Way Slab



Composite Steel Beams



Non Composite Steel Beams



System Comparison:

Floor System Designs				
Criteria	**Post Tensioned Two Way Slab**	**Mild Reinforced Two Way Slab**	**Composite Steel**	**Non-Composite Steel**
System Info				
# of steel	N/A	N/A	3808 lbs.	3472 lbs.
Weight	110 psf	115 psf	75 psf	60 psf
Architectural				
Maximum Depth	7.25"	9"	21"	21.5"
Additional Fire Proofing Required	NO	NO	YES	YES
Fire Rating	2 Hr.	2 Hr.	2 Hr.	2 Hr.
Shoring	NO	NO	YES	NO
Servicability				
Vibrations	Minimal	Minimal	Likely	Likely
Lateral System	Concrete Shear Walls, Concrete Moment Frame	Concrete Shear Walls, Concrete Moment Frame	Concrete Shear Walls	Concrete Shear Walls
Considerations				
Advantages	Smallest depth, no additional fire proofing required, limits vibrations	Limit depth, no additional fire proofing required, limits vibrations	Lightweight, minimal formwork required	Lightest system, limit vibrations
Disadvantages	Higher weight, formwork needed	Higher weight, formwork needed, deeper than existing	Larger depth, needs shoring, needs fireproofing, chance of vibrations	Larger depth, needs fireproofing, chance of vibrations
Feasability	N/A	YES	NO	YES

Appendix A

TN292_Floor_deflection_032109

DEFLECTION OF CONCRETE FLOOR SYSTEMS FOR SERVICEABILITY[1]

Bijan O Aalami[2]

Deflection control is a central considerations in serviceability of floor systems. This Technical Note reviews the levels of acceptable deflections and the currently available methods for their estimate.

OVERVIEW

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There are several reasons to control deflection.

- A concrete floor should have adequate stiffness to prevent changes in deflection that would damage attached partitions or other construction elements likely to be damaged by large deflections.
- The deflection of a floor should not be noticeable by occupants such as to convey a sense of inadequacy or safety concerns.
- Since, in some instances, deflection is used as a measure leading of undesirable vibration in a floor, its value must be controlled.

LIMITS FOR ACCEPTABLE DEFLECTION

Aesthetics and Sense of Comfort

In considering aesthetics and sense of comfort for occupants, the most important criterion is the out-of-level condition of a floor, as opposed to its stiffness. Sensitive individuals, when walking over or viewing a floor in elevation, are claimed to perceive a floor's sag when the vertical out-of-level to span ratio is in excess of 1/250, and for cantilevers in excess of 1/125. The out-of-level condition of a floor system can be controlled through camber at the time of construction, upon estimating the long-term deflection.

Deflection Limits to Mitigate Damage to Non-structural Construction

It is important to note that ACI [ACI 318, 2008] does not impose a limit to deflection under selfweight. ACI's recommendations address the amount of deflection subsequent to the installation of non-structural elements likely to be damaged.

The following table lists the ACI's stipulation on deflections (TABLE 9.5(b)). In the application of ACI's recommended deflection limits, it is important to recognize that the given values are to be compared with "computed" values, not measured out-of-plane amounts.

TABLE 1 MAXIMUM PERMISSIBLE COMPUTED DEFLECTIONS

[1] Copyright ADAPT Corporation, 2008
[2] Professor Emeritus, San Francisco State University; Principal, ADAPT Corporation

Type of member	Deflection to be considered	Deflection limitation
Flat roofs not supporting or attached to nonstructural elements likely to be damaged by large deflection	Immediate deflection due to live load	L/180 *
Floors not supporting or attached to nonstructural elements likely to be damaged by large deflection	Immediate deflection due to live load	L/360
Roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflection	That part of the total deflection occurring after attachment of nonstructural elements(sum of the long-time deflection due to all sustained loads and the immediate deflection due to any additional live load)****	L/480 **
Roof or floor construction supporting or attached to nonstructural elements not likely to be damaged by large deflection		L/240 ***

Notes:

* Limit not intended to safeguard against ponding. Ponding should be checked by suitable calculations of deflection, including added deflections due to ponding of water, and considering long-term effects of all sustained loads, camber, construction tolerances, and reliability of provisions for drainage.

** Limit may be exceeded if adequate measures are taken to prevent damage to supported or attached elements.

*** But not greater than tolerance provided for nonstructural elements. Limit may be exceeded if camber is provided so that total deflection minus camber does not exceed limit.

**** Long-time deflection shall be determined using established procedures, but may be reduced by amount of deflection calculated to occur before attachment of nonstructural elements. This amount shall be determined on basis of accepted engineering data relating to time-deflection characteristics of members similar to those being considered.

DEFLECTION CONTROL THROUGH LIMITATIONS ON SPAN TO DEPTH RATIOS

For common residential and commercial buildings, designers can forego deflection calculation, if the stiffness of the member selected is large enough. Deflection calculation requirements are governed through recommended span-to-depth ratios for different types of floor members. ACI 318 has the recommendations given in Tables 2 and 3.

One-Way Conventionally Reinforced Slabs and Beams

TABLE 2 MINIMUM THICKNESS OF CONVENTIONALLY REINFORCED BEAMS OR ONE-WAY SLABS

Member	Simply supported	One end continuous	Both ends continuous	Cantilever
Solid one-way slabs	L/20	L/24	L/28	L/10
Beams or ribbed one-way slabs	L/16	L/18.5	L/21	L/8

Notes:
L =span length
Values given shall be used directly for members with normal weight concrete and Grade 60 ksi (400 MPa) reinforcement. For other conditions, the values shall be modified as follows:

a) For lightweight concrete having equilibrium density, wc, in the range of 90 to 115 lb/ft^3 (1440-1840 kg/m^3), the values shall be multiplied by (1.65-0.005 wc) but not less than 1.09 [in SI units, (1.65-0.003γ_c) but not less than 1, where γ_c is the density in kg/m^3].
b) For fy other than 60,000 psi(400 MPa), the values shall be multiplied by (0.4+fy/100,000) [in SI units (0.4+fy/670)].

Two-Way Conventionally Reinforced Slabs and Beams

TABLE 3 MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

fy psi**	Without drop panels***			With drop panels***		
	Exterior panels		Interior panels	Exterior panels		Interior panels
	Without edge beams	With edge beams****		Without edge beams	With edge beams****	
40,000	Ln/33	Ln/36	Ln/36	Ln/36	Ln/40	Ln/40
60,000	Ln/30	Ln/33	Ln/33	Ln/33	Ln/36	Ln/36
75,000	Ln/28	Ln/31	Ln/31	Ln/31	Ln/34	Ln/34

Notes:
* For two-way construction, Ln is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
** For fy between the values given in the table, minimum thickness shall be determined by linear interpolation.
*** Drop panels are defined as extension of slab thickening into span not less than span/6, and extension
of thickening below slab not less than slab thickness/4.

**** Slabs with beams between columns along exterior edges. The ratio of edge beam stiffness to the stiffness of the edge beam's design strip shall not be less than 0.44

Post-Tensioned Members

For post-tensioned beams and slabs, the recommended values by the Post-Tensioning Institute [PTI, 1990 are as follows:

TABLE 4 RECOMMENED SPAN TO DEPTH RATIOS FOR POST-TENSIONED MEMBERS

	Continuous Spans		Simple Spans	
	Roof	**Floor**	**Roof**	**Floor**
One-way solid slabs	50	45	45	40
Two-way solid slabs (supported on columns only)	45-48	40-45		
Two-way waffle slabs (1m pans)	40	35	35	30
Beams	35	30	30	26
One-way joists	42	38	38	35

Note: The above ratios may be increased if calculations verify that deflection, camber, and vibrations are not objectionable.

DEFLECTION CALCULATIONS

Under otherwise unchanged conditions, the deformation of an exposed and loaded concrete member continues to increase. The increase is due to creep under applied load and shrinkage from loss of moisture. The engineering approach to estimating of deflection is to determine the instantaneous response of a structure under an applied load, and magnify the instantaneous displacement due to the time-dependent factors of creep and shrinkage. With time, the rate of change in displacement reduces. For building structure it is assumed that five years is sufficient time for the deflections to have reached their final values. While it is practical to calculate the time-dependent deflection for any time interval, the common practice is to estimate the total value at five years and use this value in the design.

Instantaneous Deflection

Instantaneous deflection is generally calculated using concrete's modulus of elasticity at 28 days, gross-cross sectional area and linear elastic theory. The calculated deflection may require adjustment, if the member is likely to crack, when subjected to the design load. Cracking reduces the stiffness of a member and results in increased deflection. The options for calculating instantaneous deflection with due allowance to cracking are:

- Closed form formulas or tables, available primarily for uncracked sections;
- Use of equivalent moment of inertia (Ie) and simplified averaging (ACI-318's simplified procedure);
- Use of equivalent moment of inertia (Ie) combined with numerical integration; and
- Use of Finite Element floor programs that allow for cracking.

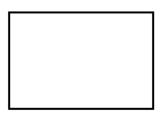
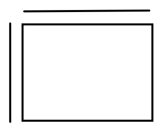
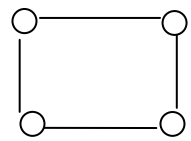
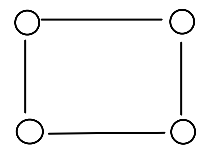
Each of the above procedures is briefly discussed in the following section.

Closed Form Formulas

Closed form formulas are readily available for beams and one-way slabs. The variables that describe the geometry of a two-way panel within a floor system, however, are so extensive that it becomes impractical to compile a meaningful set of tables or relationships without extensive approximation. For non-cracked sections, compilations such as the one listed in Table 5 are readily available in the literature [Bares, 1971].

In the application of data, such as those given in Table 5, the design engineer must use judgment regarding the degree of fixity of the support.

TABLE 5 DEFLECTION COEFFICIENTS k

γ	a  b 1	 2	 3	 4
1	0.0457	0.0143	0.0653	0.0491
1.1	0.0373	0.0116	0.0548	0.0446
1.2	0.0306	0.0094	0.0481	0.0422
1.3	0.0251	0.0075	0.0436	0.0403
1.4	0.0206	0.0061	0.0403	0.0387
1.5	0.0171	0.0049	0.0379	0.0369
2.0	0.0071	0.0018	0.0328	0.0326

Notes:
Poisson's ratio conservatively assumed 0.25
γ = a/b (aspect ratio)
Boundary conditions
1 = rigid supports; rotationally free;
2 = rigid supports; rotationally fixed;
3 = central panel from an array of identical panels supported on columns; deflection at center; and
4 = similar to case 3, but deflection at center of long span at support line

$w = k\,(a^4 * q / E * h^3)$

Where,
w = deflection normal to slab;
a = span along X-direction;
E = Modulus of elasticity; and
h = slab thickness.

EXAMPLE 1

Consider the floor system shown in Fig. EX-1. Estimate the deflection of the slab panel identified in part b of the figure under the follwoing conditions. Other particulars of the floor system are noted in Appendix A.

Given:

Span length along X-X direction	= 30' (9.14 m)
Span length along Y-Y direction	= 26.25' (8.0 m)
Slab thickness	= 8 in (203 mm)
Ec (modulus of elasticity)	= $4.287 * 10^6$ psi (29,558 MPa)
Superimposed dead load	= 25 psf (1.2 kN/m^2)
Live load	= 40 psf (1.9 kN/m^2)

Required

Deflection of the panel at midspan for the following load combination

1*DL + 1*LL

Aspect ratio γ = 30/26.25 = 1.14

Total service load q = [(20+5+40) + 150*8/12]/144 = 1.146 lb/in^2 ($7.9*10^{-3}$ N/mm^2)

Using closed form formulas (Table 5)

$(a^4*q / E*h^3) = [(30*12)^4 * 1.146 / (4.287*10^6 * 8^3) = 8.77$ in (222.76 mm)

For mid-panel deflection, consider case 3 from Table 5

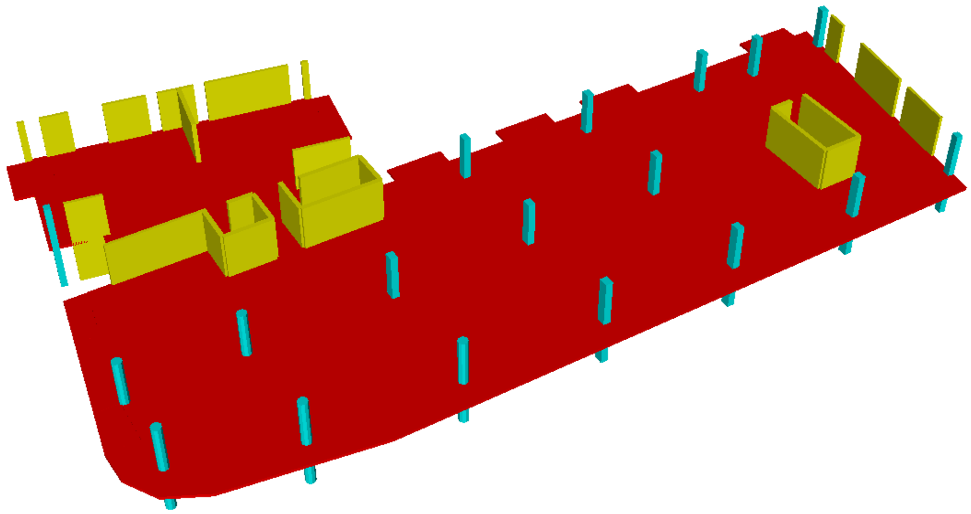
k = 0.0548

Deflection, $\Delta = k\ (a^4*q / E*h^3) = 0.0548*8.77 = 0.48$ in (12.20 mm)

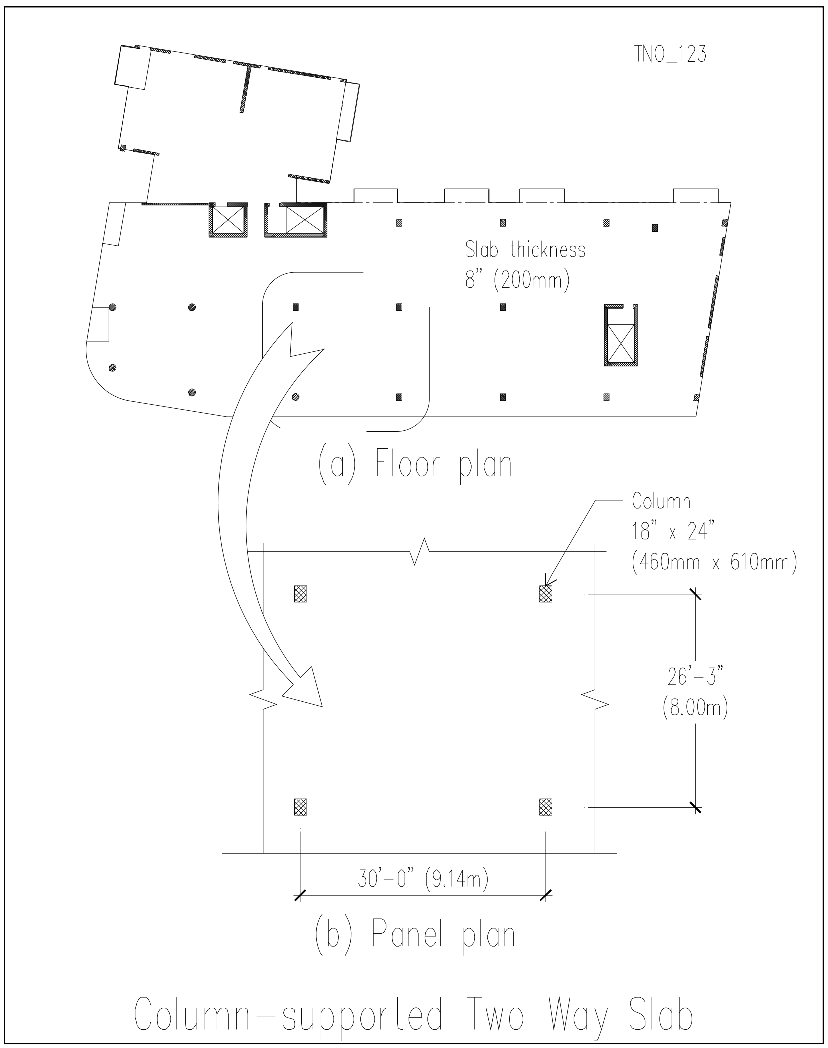
For deflection at midpoint of column lines in X-direction, from Table 5

k = 0.0446

Deflection, $\Delta = k\ (a^4*q / E*h^3) = 0.0446*8.77 = 0.39$ in (9.91 mm)



(a) 3D View of typical floor system



(b) Illustration of design panel

FIGURE EX-1 TYPICAL FLOOR HIGHLIGHTING THE SPAN UNDER CONSIDERATION

Using equivalent moment of inertia (Ie) and (ACI-318's simplified procedure)

In this method allowance is made for crack formation in the slab. The reduction in flexural stiffness due to cracking is accounted for by substituting the otherwise gross moment of inertia "Ig" used in the calculation with a reduced effective moment of inertia "Ie." The equivalent moment of inertia Ie can be applied to the entire span through the "simplified procedure of ACI-318), or applied locally along the entire length of a member for a detailed procedure.

The calculation of the effective moment of inertia Ie will be described next.

Using ACI-318

$$I_e = (M_{cr} / M_a)^3 * I_g + [1-(M_{cr} / M_a)^3] * I_{cr} \leq I_g \tag{1}$$

Where,

Ig	=	Gross moment of inertia;
Icr	=	Moment of inertia of cracked section;
Ie	=	Effective moment of inertia;
Ma	=	Maximum moment in member at stage deflection is computed; and,
Mcr	=	Cracking moment.

The applied moment, M_a, is calculated using elastic theory and the gross moment of inertia (I_g) for the uncracked section. The change in distribution of moment in indeterminate structures resulting from cracking in concrete is generally small, and is already accounted for in the empirical formula (1) for equivalent moment of inertia. The cracking moment is given by:

$$M_{cr} = f_r I_g / y_t \tag{2}$$

Where,

f_r = Modulus of rupture, flexural stress causing cracking. It is given by:

$$f_r = 7.5\, f'_c{}^{1/2} \tag{3}$$

y_t = distance of section centroid to farthest tension fiber

For all-lightweight concrete, f_r is modified as follows:

$$f_r = 0.75 * 7.5\, f'_c{}^{1/2} \tag{4}$$

Figure 2 illustrates the equivalent moment of inertia Ie for a simply supported concrete slab that is partially cracked.

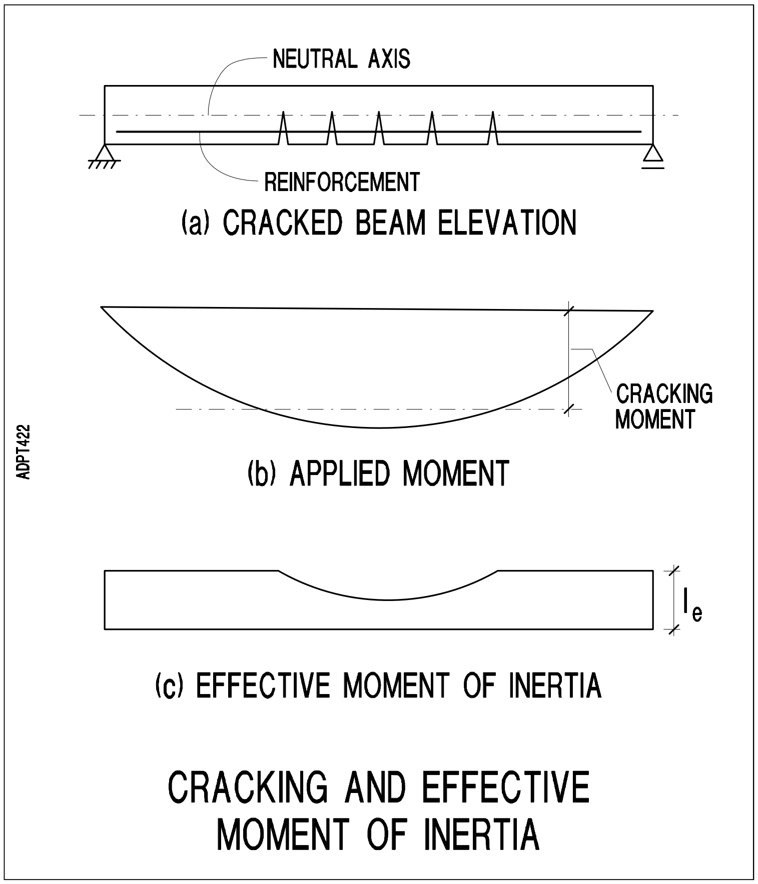


FIGURE 2 ILLUSTRATION OF EFFECTIVE MOMENT OF INTERTIAL IN A PARTIALLY CRACKED SLAB

The value of cracking moment of inertial Icr and the geometry of the section depends on the location and amount of reinforcement. For rectangular sections with single reinforcement (Fig. 2) the value is given by:

$$I_{cr} = (bk^3d^3)/3 + nA_s(d-kd)^2 \qquad (5)$$

Where,

$$kd = [(2dB+1)^{1/2} - 1]/B \qquad (6)$$

d = distance from compression fiber to center of tension reinforcement

$B = b/(nA_s)$

n = Es/Ec

Es = modulus of elasticity of steel

Ec = modulus of elasticity of concrete

For more details and treatment of other cross-sections refer to ADAPT Technical Note TN293.

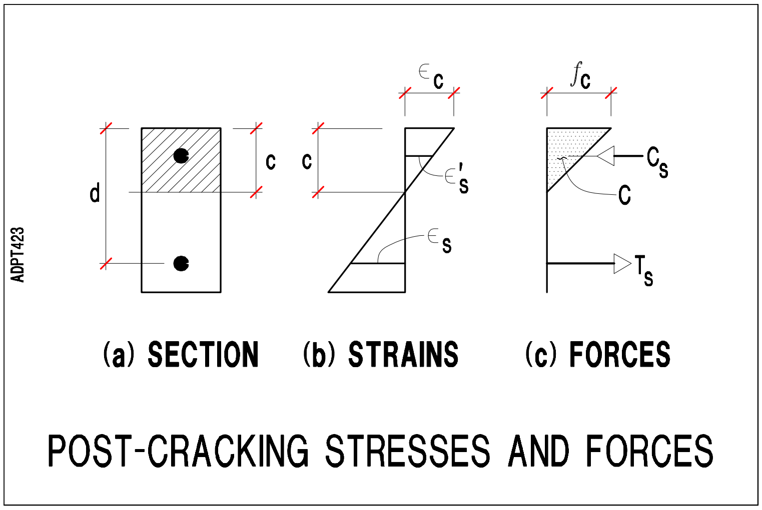


FIGURE 3

In the simplified method an average value of Ie is used for the entire span. For spans, the average value is calculated

$$I_{e,\,av} = 0.5\ [\ (I_{e,\text{left support}} + I_{e,\text{right support}}\ )/2\ +\ I_{e,\ \text{midspan}}\ ] \qquad (7)$$

EXAMPLE 2

Consider the floor system shown in Fig. EX-1. Estimate the deflection of the slab panel identified under the same loading and conditions expressed in Example 1, using the simplified option of ACI-318 for equivalent moment of inertia Ie

Given:
Span length along X-X direction = 30' (9.14 m)
Span length along Y-Y direction = 26.25' (8.0 m)
Slab thickness = 8 in (203 mm)
Ec (modulus of elasticity) = 4.287 *10^6 psi (29558 MPa)
Other details of the slab are given in Example 1 and the Appendix A

Required
Determine the deflection at the center of the panel identified in Example 1 due to the sum of dead and live loads.

Calculate Cracking Moment Mcr

Ig = 15,360 in^4 (6.40e+10 mm^4)

yt = 4 " (101.60 mm)

fr = $7.5\sqrt{f'c} = 7.5*\sqrt{5000}$ = 530.33 psi (3.66 MPa)

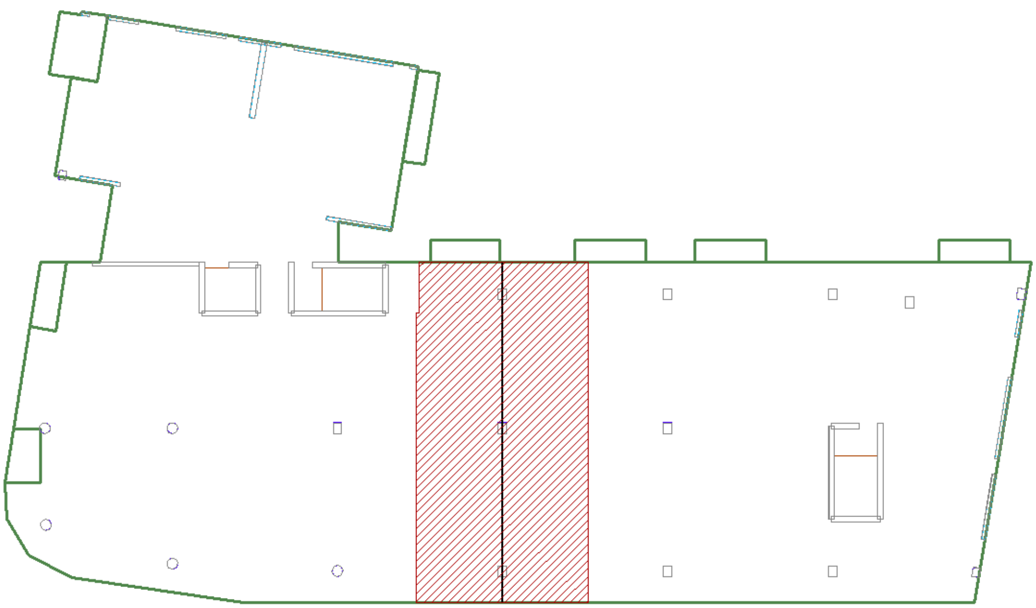
$$M_{cr} = f_r I_g / y_t = 530.33*15,360/(4*12000) = 169.70 \text{ k-ft } (230 \text{ kNm})$$

To determine the deflection at center, the applied moment (Ma) for the "design strip" associated with the panel in question must be determined. Refer to Fig. EX2 – 1a

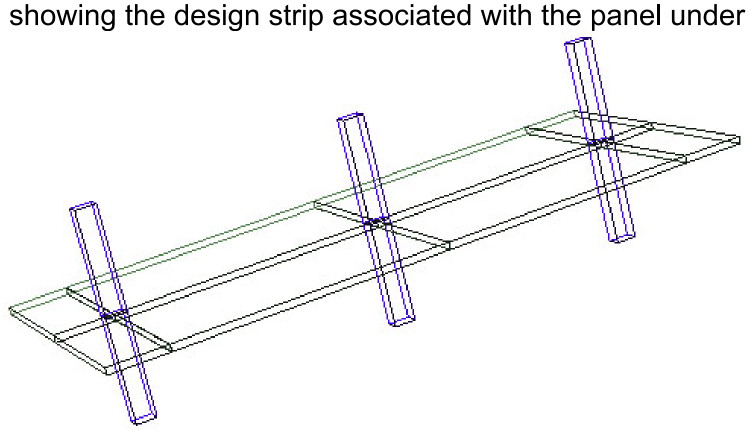
$$I_e = (M_{cr} / M_a)^3 * I_g + [1-(M_{cr} / M_a)^3] * I_{cr} \le I_g \qquad (1)$$

The design strip associated with the panel under consideration is shown in Fig. EX-2a. It connects the line of columns and extends on each side to the midspan line of the adjacent panels.

The design strip extracted from the floor system is shown in its idealized form in Fig. EX-2b. Using a computer program, the applied moment Ma in the idealized design strip is calculated.



(a) Plan of slab showing the design strip associated with the panel under consideration



(b) View of the design strip extracted from the floor system

FIGURE EX2 -1 PLAN OF TYPICAL FLOOR HIGHLIGHTING THE DESIGN STRIP OF THE SPAN UNDER CONSIDERATION

A solution obtained from the computer program ADAPT-RC[3] [ADAPT RC, 2008] gives the following values:

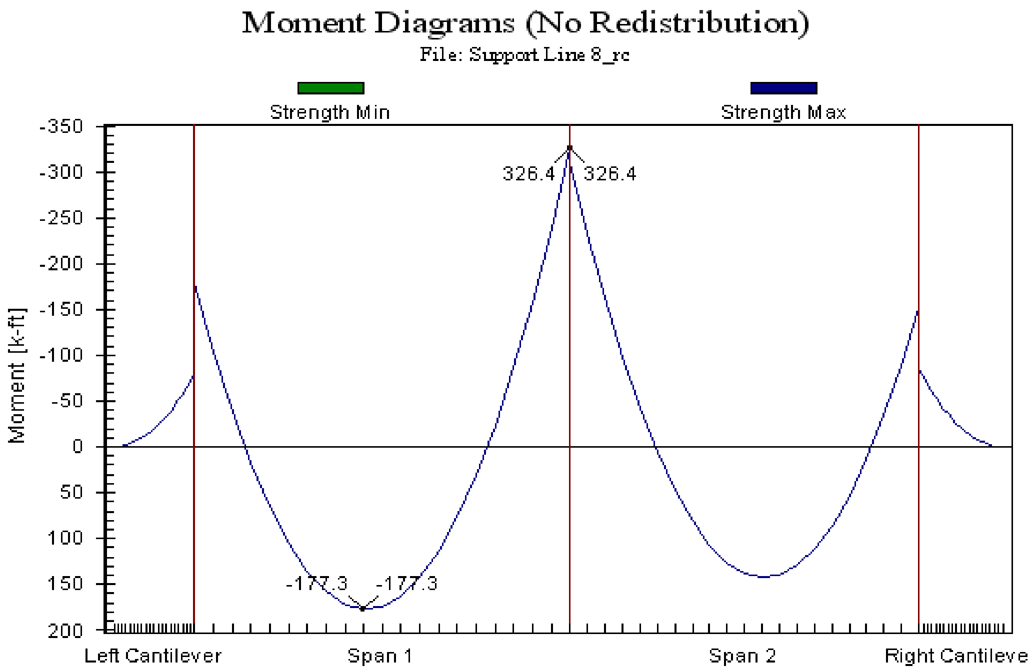


FIGURE EX2-2 DISTRIBUTION OF MOMENTS DUE TO DEAD PLUS LIVE LOAD

The computed deflection for the first span without accounting for the crack formation associated with moments of Fig. EX2-2 is 0.231 in. (5.9 mm).

It is noteworthy that the strip method, as outlined herein, provides the deflection value in the direction of analysis, not accounting for the deflection in the transverse direction. For a complete analysis of mid-panel displacement, the deflection in the transferse direction must also be calculated and added to the deflection calculated for this direction (Fig. EX2-3). For panels that are fairly square, it is acceptable to multiply the deflection calculated for one direction by a factor of 2. For this example, the total deflection is estimated as:

Total deflection = 2 * 0.231 = 0.462 in. (11.7 mm)

ADD FIGURE ???

FIGURE EX2-3 COMBINATION OF DEFLECTIONS FROM ORTHOGONAL DIRECTIONS

on the assumption that there is no transv that is representative of both the midpoint of the panel and midpoint of the line of support.

[3] ADAPT-RC is a computer program for design and analysis of conventionally reinforced beam frames and slabs. It is based on Equivalent Frame Method (www.adaptsoft.com)..

It is noteworthy that the strip method, as outlined herein, provides ??

Using the moment values given in Fig. EX2-2 for the right support of span 1 the Ie is given by:

M_a = 326.4 k-ft (442.53 kNm)
I_g = 17,019 in^4 (6.40e+10 mm^4)

M_{cr} = $f_r I_g / y_t$ = 530.33*17,019/(4*12000) = 188.04 k-ft (254.94 kNm)

$I_e = (M_{cr} / M_a)^3 * Ig + [1-(M_{cr} / M_a)^3] * I_{cr} \leq I_g$

Where,

$I_{cr} = (bk^3d^3)/3 + nAs\ (d-kd)^2$

kd = $[(2dB+1)^{1/2} - 1]/B$

d = 6.81 in (173 mm)

B = $b/(nA_S)$

n = Es/Ec = 30000/4287 = 7.0

As = 10.12 in^2 (6529 mm^2)

B = 360/(7.0* 10.12) = 5.08 /in (0.2/mm)
Kd = $[(2*6.81*5.08+1)^{1/2} - 1]/5.08$
= 1.45 in (36.83 mm)

I_{cr} = $(360*1.45^3)/3 + 7.0*10.12* (6.81-1.45)^2$
= 2401 in^4 (9.99e+8)

I_e = $(188.04 / 326.4)^3 * 17019 + [1-(188.04 / 326.4)^3] *2401$
= 5196 in^4 (2.16e+9)= 0.31 Ig

Using the same procedure, the value of Ie at other locations required by the code formula are calculated and listed below:

Left cantilever:
I_e at face of support = I_g = 1.536e+4 in^4 (6.39e+9 mm^4)

First Span:
I_e at left support centerline = 1.70e+4 in^4 (7.08e+9 mm^4)
I_e at midspan = 1.44e+4 in^4 (5.99e+9 mm^4)
I_e at right support centerline = 5.20e+3 in^4 (2.16e+9 mm^4)

Second Span:
I_e at left support centerline = 5.63e+3 in^4 (2.34e+9 mm^4)
I_e at midspan = 1.536e+4 in^4 (6.39e+9 mm^4)
I_e at right support centerline = 1.702e+4 in^4 (7.08e+9 mm^4)

Right cantilever:

I_e at face of support = I_g = 1.536e+4 in^4 (6.39e+9 mm^4)

Using the averaging procedure suggested by ACI-318, the Ie values to be used for deflection calculation are:

Left and right cantilevers $I_e = I_g$

First span
Average I_e = $[(1.70*10^4+5.20*10^3)/2 +1.44*10^4]/2$
= 12.755e+3 in^4 (5.31e+9 mm^4)

Second span
Average I_e = $[(5.63*10^3+1.702*10^4)/2 +1.536*10^4]/2$
= 13.343e+3 in^4 (5.55e+9 mm^4)

In order to use the same frame program for the calculation of deflected shape, the calculated equivalent moments of inertia are used to determine an equivalent thickness (h_e) for each of the spans. The equivalent thickness is given by

$$I_e = b*h_e^3 /12$$

Where, b is the width of the tributary of the design strip.

Left cantilever:	h_e = 8 in (203 mm)
First span	h_e = 7.52 in (191 mm)
Second span	h_e = 7.63 in (193.8 mm)
Right cantilever	h_e = 8 in (203 mm)

Using the same computer program, material values, boundary conditions and loads, but with the reduced slab thickness of modified moment of inertia a new solution is obtained.

The maximum value of deflection for span 1 is 0.264 in (6.71 mm), compared to 0.231 in (5.87 mm), without allowing for reduction of stiffness due to cracking. Note that the above deflections do not account for the flexure of the slab in the orthogonal direction, as indicated in Example 2. For engineering design, where panels are fairly square, the calculated values are commonly multiplied by 2 to represent the deflection at the middle of panel. Hence, mid-panel deflections would be 0.528 in. (13.42 mm) and 0.462 in. (11.74 mm).

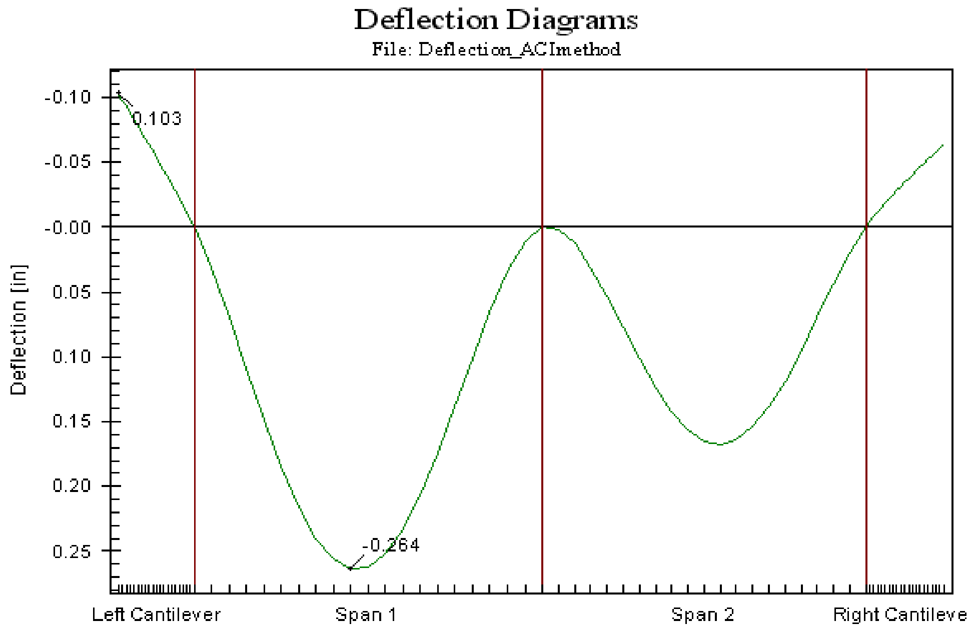


FIGURE EX2-3 DEFLECTED SHAPE WITH ALLOWANCE FOR CRACKING, USING SIMPLIFIED METHOD.

Using Equivalent Moment of Inertia (Ie) Combined with Numerical Integration

The next step in increased accuracy of deflection calculation is (i) the use of equivalent moment of inertia Ie, (ii) the strip method as outlined in the preceding example, and (iii) numerical integration. In this scheme each span will be subdivided in a number of segments, typically 10 to 20 divisions. The equivalent moment of inertia for each division will be calculated separately, and a solution obtained with recognition of a variable moment of inertia along the length of each span. This procedure along with a detailed numerical example is described in ADAPT TN294

Figure 3 is an example showing the variation of moment along the first span of a two-span member, subdivision of the span into smaller segments, and the equivalent moment of inertia for each segment due to cracking.

Using the computer program ADAPT-RC, the above procedure is employed to determine the deflection of the design strip shown in Fig, 4, with due consideration for cracking. The calculated deflection by the program is 0.235 in (5.97 mm), compared to 0.264 in (6.71 mm) where the simplified averaging of effective moment of inertia was used in calculation of cracked deflection.

Using this method, the total deflection is estimated as:

Total deflection = 2 * 0.235 = 0.470 in. (11.94 mm)

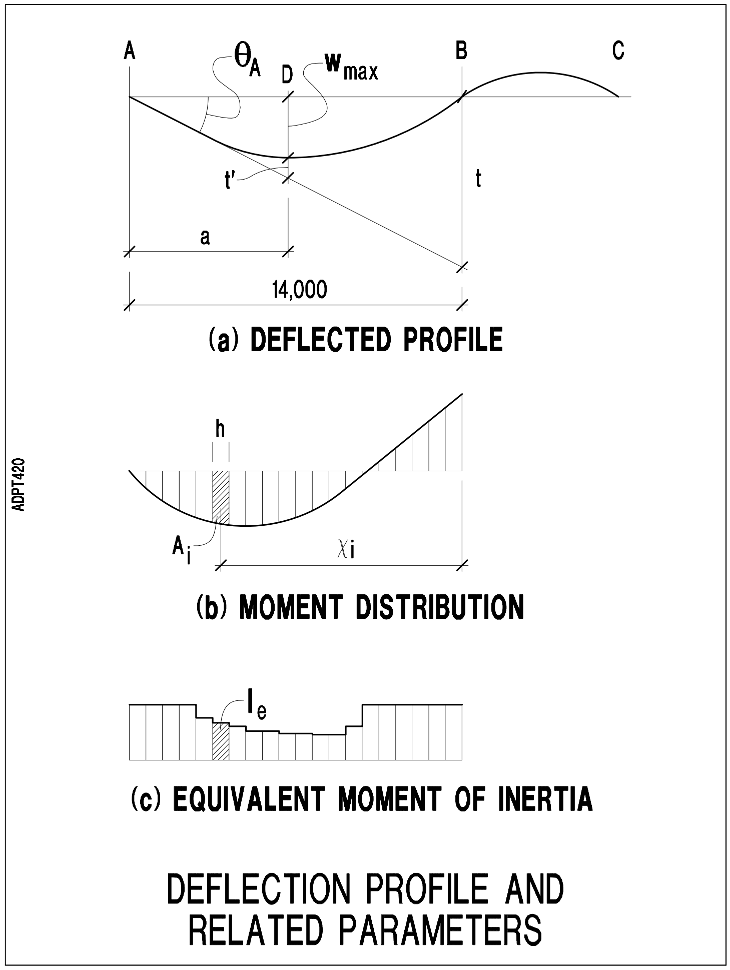
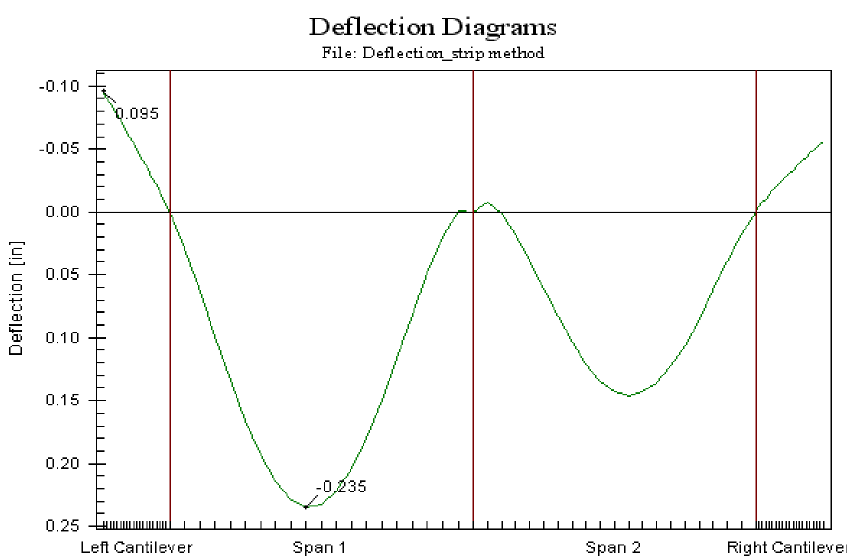


FIGURE 3 VARIABLE MOMENT INERTIA ALONG A MEMBER DUE TO CRACKING



FIUGRE 4 DEFLECTED PROFILE OF THE DESIGN STRIP WITH ALLOWANCE FOR CRACKING USING NUMERICAL INTEGRATION

Using Finite Element Method With No Allowance for Cracking

Using finite element method (FEM), the salient features of the geometry and loading that are idealized in other previously explained options can be faithfully modeled. This leads to a more valid estimate of slab deflection. Figure 5 shows the Discretization of the floor system used in the previous examples into finite element cells.

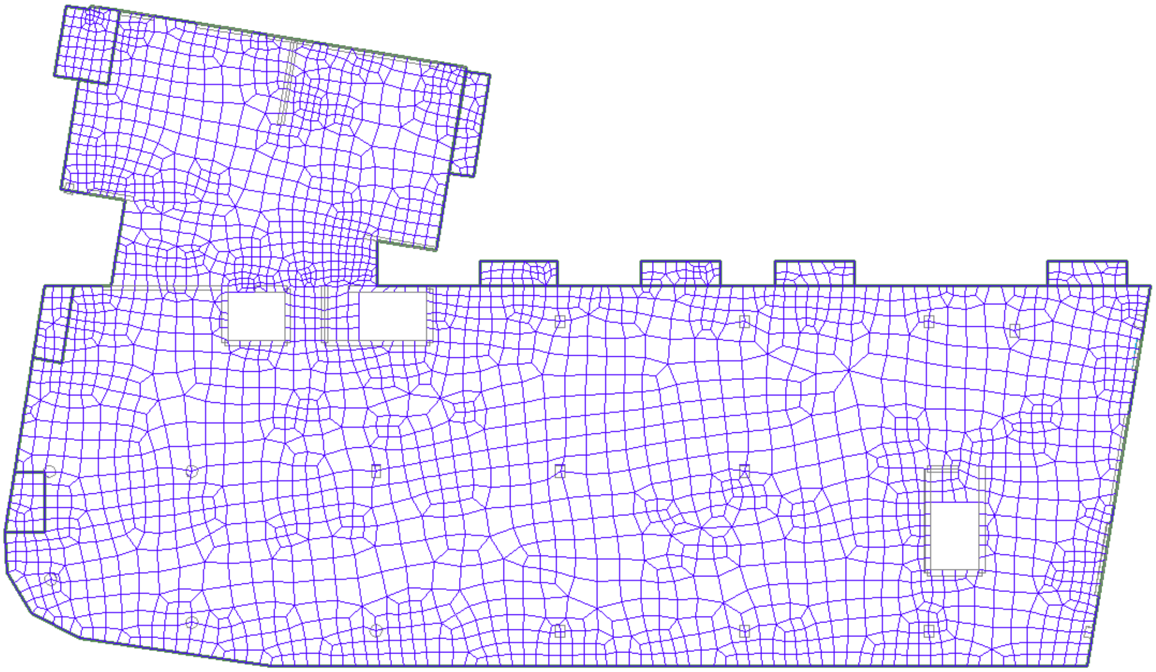


FIGURE 5 DISCRETIZATION OF THE TYPICAL FLOOR SLAB FOR FINITE ELEMENT ANALYSIS (FLOOR-PRO)

EXAMPLE
For the same geometry and parameters of examples 1 and 2, using a finite element program determine the deflection at center of the panel identified in Fig. EX1-1.

ADAPT-FLOOR Pro[4] program was used to model the slab and obtain a solution. The distribution of deflection for the given load is shown in Fig. EX3-1. The maximum deflection at the center of the panel under consideration is reported as 0.54 in.

[4] ADAPT-Floor Pro is a finite element program for analysis and design of conventionally reinforced or post-tensioned floor systems. www.adaptsoft.com

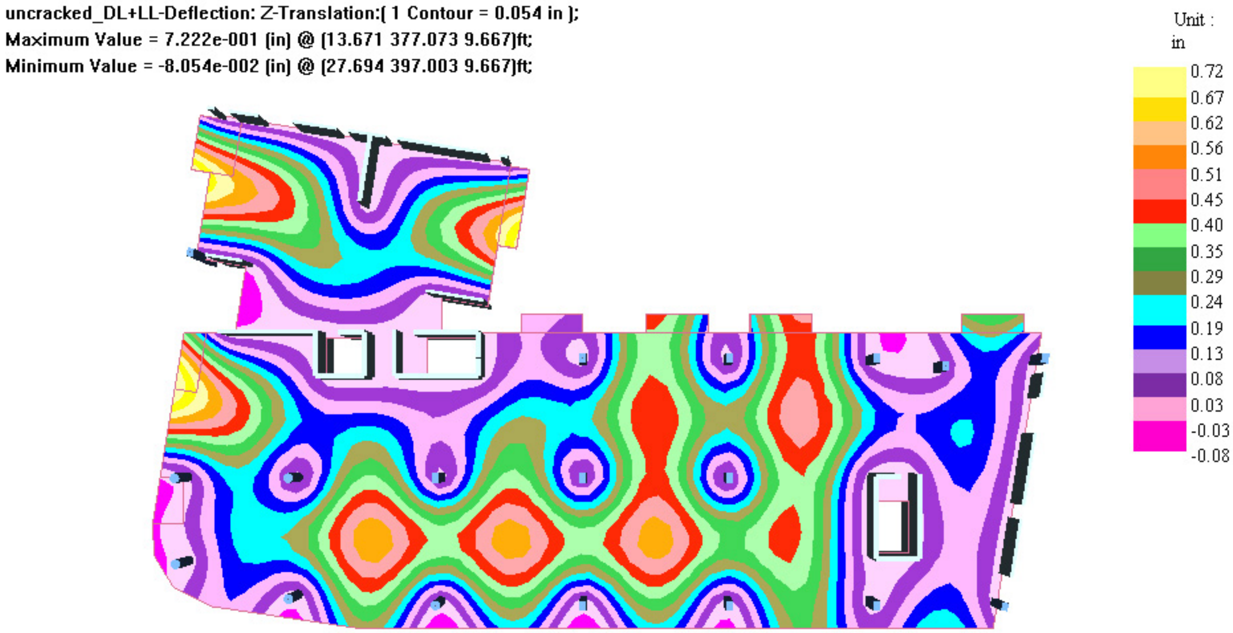


FIGURE EX3-1 DEFLECTION CONTOUR OF THE FLOOR SYSTEM UNDER THE COMBINED ACTION OF DEAD AND LIVE LOADS

Using Finite Element Method With Due Allowance for Cracking

Formulation of finite elements with allowance for cracking is somewhat complex. The complexity arises from the fact that cracking and reduction in stiffness depend on the presence, amount and orientation of reinforcement. Before a solution is obtained, the reinforcement detailing of a floor system must be fully known, since the loss of stiffness in each finite element cell depends on the availability and exact location of the reinforcement in that cell.

The following briefly describes the steps for a finite element deflection calculation, with allowance for cracking.

1. Using the geometry, boundary conditions, material properties, and the load combination for which the deflection is sought, the program discretizes the structure, sets up the system stiffness matrix of the structure based on gross moment of inertia (Ig), and obtains the distribution of moments (Ma) over the entire structure.

2. If required the program performs a design check, using a building code, and adds the required reinforcement to the floor system for the specified loads.

3. The Program scans the entire floor system to detect the reinforcement available in the beams and the slab regions. The available reinforcement is either determined by the Program prior to the initiation of deflection calculation, or is a combination of program calculated and user defined/edited reinforcement.

Once the deflection calculation is initiated, the available reinforcement remains unchanged. The reinforcement can be in one or more of the following forms, with no restriction on the orientation, length, or the position of each reinforcement within the floor system.

a - User defined one or more top and bottom reinforcement mesh;
b -. User defined grouped or distributed reinforcement bars at top and/or bottom of

slab and beams;
c - Reinforcement calculated and reported by the program for minimum requirements of the code, strength check, initial condition, or other code related criteria; and
d - Post-tensioning tendons defined by the user, each with its own location and force.

4. The program matches the calculated moment (Ma) of each finite element cell with the existing reinforcement in that cell. Using the parameters given below, the Program calculates the effective moment of inertia for each cell:

a. Finite element cell thickness (to obtain uncracked second moment of area);
b. Available reinforcement associated with each cell (nonprestressed and prestressed), with recognition of orientation and height of each individual reinforcement;
c. Cracking moment of inertia associated with each cell in each direction (Icr);
d. Cracking moment associated with each cell (Mcr); and
e. Applied moment (Ma calculated in 2).

5. Having determined the effective second moment of area of each finite element cell in each of the principal directions, the Program re-constructs the stiffness matrix of each cell.

6. The Program re-assembles the system stiffness matrix and solves for deflections. At this stage the solution given from the first iteration is a conservative estimate for the floor deflection with cracking. For practical engineering design, it is recommended to stop the computations at this stage.

For a more accurate solution the newly calculated deflection can be compared with that of a previous iteration. If the change in maximum deflection is more than a pre-defined tolerance, the Program will go into another iteration starting from step (2). In this scenario, the iterations are continued, until the solution converges to within the pre-defined tolerance.

The Program accounts for loss of bending stiffness in beams and slab regions and its combinations..

EXAMPLE
Using finite elements determine the deflection of the panel identified in Fig. EX1 for the loads and conditions described in Example 1.

Calculate the deflection for the combination of dead and live loads.
Use ACI318-08 to determine the reinforcement necessary for both the in-service and strength requirements of the code.
Use the calculated minimum reinforcement of the code to determine the cracked deflection. No other reinforcement is added.

Using the above requirements, the cracked deflection of the floor system is calculated and illustrated as a contour image in Fig. EX4-1. The deflection at the center of the panel under consideration is 0.70 in. (17.78 mm) compared to 0.54 in. (13.72 mm) for the uncracked slab. The cracked deflection can be reduced by adding reinforcement at the locations of crack formation in addition to the minimum requirements of the code already included in the analysis.

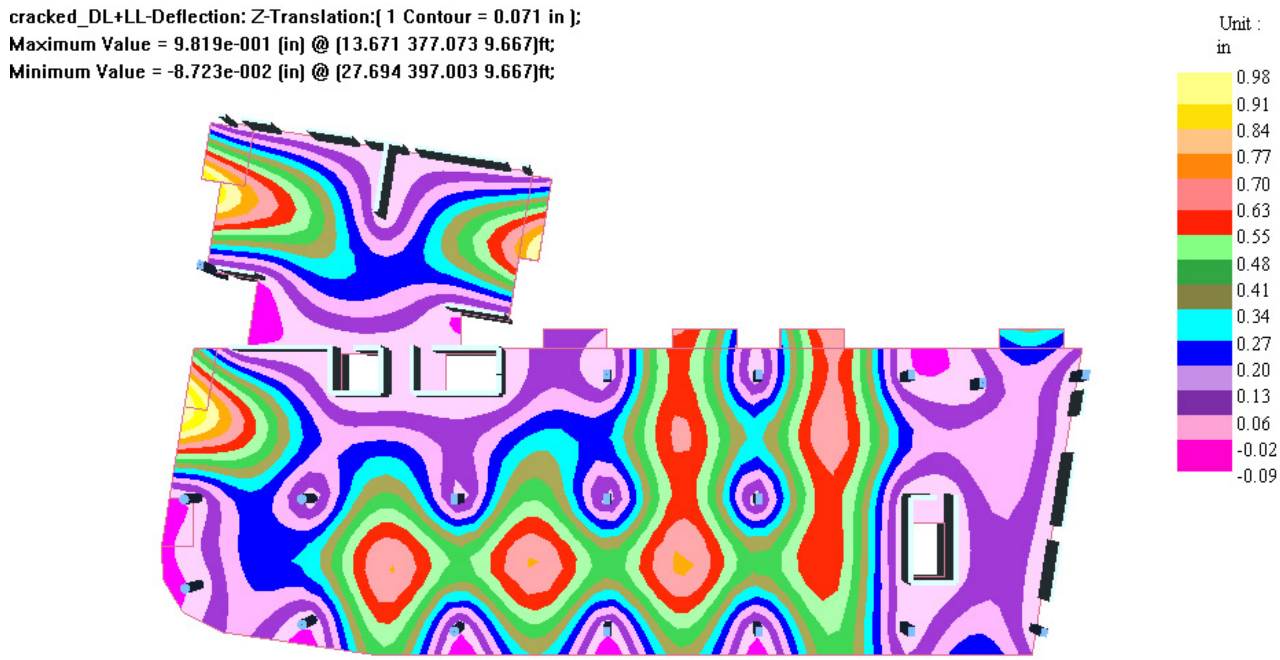


FIGURE EX4-1 DEFLECTION CONTOUR OF SLAB WITH CRACKING

The locations of crack formation and the extent of cracking are illustrated in Figs. EX4-2 and EX4-3. At each location, the reduction in effective moment of inertia is based on the calculated moment at that location and the amount, position and orientation of reinforcement at the same location. The largest loss of stiffness occurs over the columns and the support lines joining the columns. The maximum loss of stiffness is 69% reducing the effective moment of inertia to 31% of its uncracked value.

FIGURE EX4-2 EXTENT OF CRACKING SHOWN THROUGH REDUCTION IN EFFECTIVE MOMENT OF INERTIA Ie ABOUT Y-Y AXIS

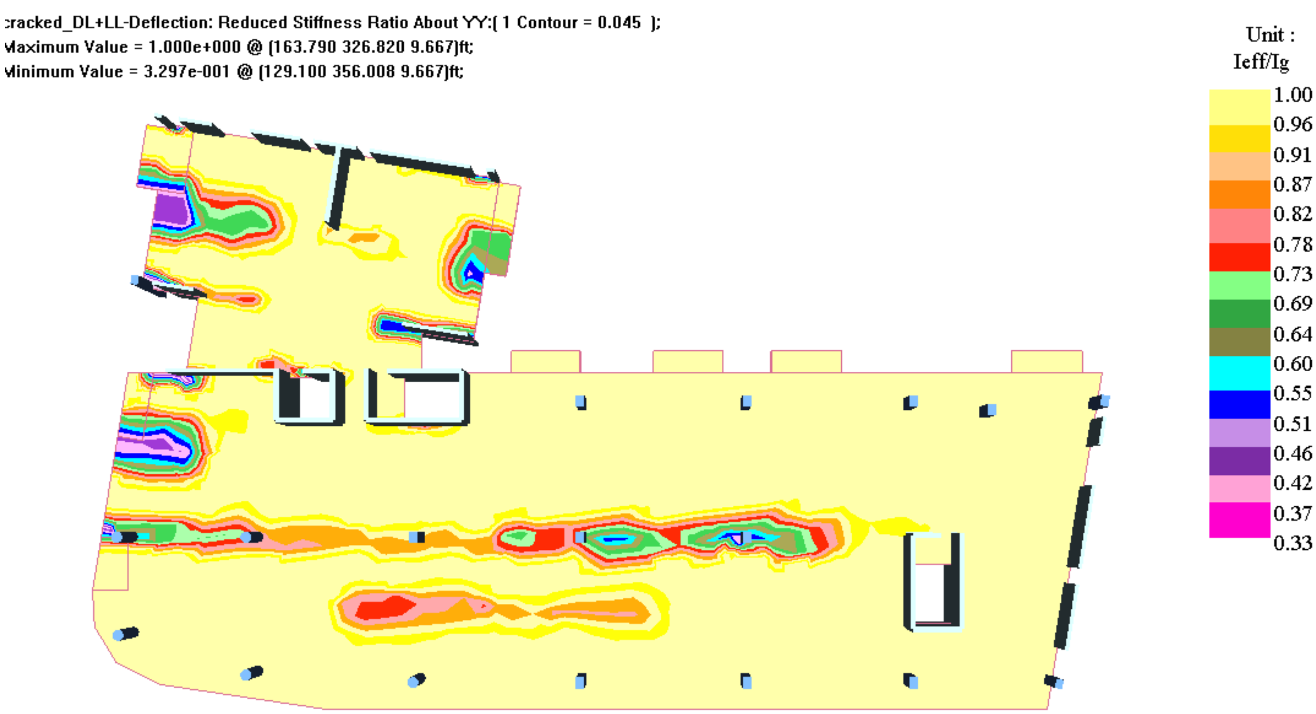


FIGURE EX4-3 EXTENT OF CRACKING SHOWN THROUGH REDUCTION IN EFFECTIVE MOMENT OF INERTIA Ie ABOUT X-X AXIS

Deflection of Post-Tensioned Floor Systems

Two-way post-tensioned floor systems designed to ACI-318 provisions and the PTI-recommended slab-to-depth ratios (Table 4), either do not crack under service condition, or crack to an extent that does not invalidate calculations based on gross cross-sectional geometry and linear elastic theory. This is because, unlike other major non-US codes, the allowable tensile stresses in ACI are relatively low.

The preceding observation does not hold true for post-tensioned one-way slab and beams, where ACI-318 permits designs based on post-cracking regime. For such post-tensioned floor systems, designers must include allowance for cracking in their designs.

COMPARISON OF DEFLECTION CALCULATION METHODS

As illustrated in the design examples, the calculated deflections design engineers use from the ??? currently available procedures for conventionally reinforced concrete vary greatly. Table 6 lists the outcome of the various methods. Note that for the typical floor system selected, the difference between the various methods can be as much as three times. Finite Element Method with due allowance for crack formation gives the largest deflection. The strip method with no allowance for cracking produces the smallest value.

TABLE 6 DEFLECTION VALUES AT CENTER OF PANEL OF THE NUMERICAL

	Calculation Method	Deflection in(mm)	Normalized Deflection
1	Closed form formulas	0.480(12.19)	69 %
2	ACI318 – Simplified method	0.528(13.42)	75 %
3	Strip method (uncracked)	0.462(11.74)	66 %
4	Strip method with cracking and numerical integration	0.470(11.94)	67 %
5	Finite Element Method (FEM) No allowance for cracking	0.540(13.72)	77 %
6	Finite Element Method (FEM) With allowance for cracking	0.700(17.78)	100 %

LONG-TERM DEFLECTIONS

A concrete member's deformation changes with time due to shrinkage and creep. Shrinkage of concrete is due to loss of moisture. Creep is increase in displacement under stress. Under constant loading, such as selfweight, the effect of creep diminishes with time. Likewise, under normal conditions, with loss of moisture, the effect of deformation due to shrinkage diminishes. Restraint of supports to free shortening of a slab due to shrinkage or creep can lead to cracking of slabs and thereby an increase in deflection due to gravity loads.

While it is practical to determine the increase in instantaneous deflection of a floor system due to creep and shrinkage at different time intervals, the common practice for residential and commercial buildings is to estimate the long-term deflection due to ultimate effects of creep and shrinkage.

Shrinkage

It is the long-term shrinkage due to loss of moisture through the entire volume of concrete that impacts a slab's deformation. Plastic shrinkage that takes place within the first few hours of placing of concrete does not play a significant role in slab's deflection and its impact in long-term deflection is not considered.

Long-term shrinkage results in shortening of a member. On its own, long-term shrinkage does not result in vertical displacement of a floor system. It is the presence of non-symmetrical reinforcement within the depth of a slab that curls it (warping) toward the face with less or no reinforcement. The slab curling is affine to its deflection due to selfweight, and hence results in a magnification of slab's natural deflection.

It is important to note that, deflection due to shrinkage alone is independent of the natural deflection of slab. It neither depends on the direction of deflection due to applied loads, nor the magnitude. The shrinkage deflection depends primarily on the amount and position of reinforcement in slab.

A corollary impact of shrinkage is crack formation due to restraint of the supports. This is further discussed in connection with the restraint of supports. It is the crack formation due to shrinkage that increases deflection under gravity loads.

Shrinkage takes place over a time period extending beyond a year. While the amount of shrinkage and its impact on deflection can be calculated at shorter intervals, the common practice is to estimate the long-term deflection due to the ultimate shrinkage value.

Shrinkage values can vary from zero, when concrete is fully immersed in water to 800 micro strain. Typical ultimate shrinkage values are between 400 to 500 micro strain.

Creep

Creep is stress related. It is a continued magnification of the spontaneous displacement of a member with reduced rate of creep with time. Values of creep vary from 1.5 to 4. Typical ultimate creep values for commercial and building structures are between 2 to 3.

Restraint of Supports

Restraint of supports, such as walls and columns to free movement of a slab due to shrinkage can lead to tensile stresses in the slab and early cracking under applied loads. Early cracking will reduce the stiffness of the slab and increase its deflection.

Multiplier Factors for Long-Term Deflections
For design purposes, the long-term deflection of a floor system due to creep and shrinkage can be expressed as a multiplier to its instantaneous deflection.

Long-term deflection due to sustained load:

$$\Delta_l = C * \Delta_i \tag{8}$$

Where

Δ_l = long-term deflection;
Δ_i = instantaneous deflection; and
C = multiplier.

ACI-318 suggests the multiplier factor shown in Fig. 5 to estimate long term deflections due to sustained loads

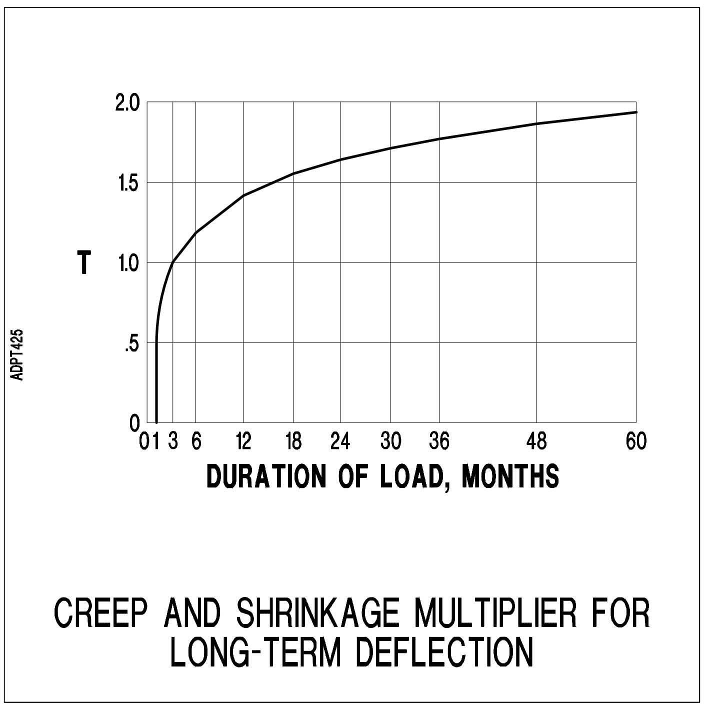


FIGURE 5 MULTIPLIER FOR LONG-TERM DEFLECTION

The multiplier can be reduced, if compression reinforcement is present. The factor (λ) for the reduction of the multiplier is given by:

$$\lambda = C / (1 + 50\rho') \tag{9}$$

Where ρ' is the value of percentage of compression rebar at mid-span for simple and continuous members and at support for cantilevers.

ACI's recommended multipliers account for the cracking of slab. Hence, they are intended to be applied to cracked deflections. Several Investigators recommend long--term multiplier coefficients for deflections based on gross cross-sectional area. These coefficients are higher than the ACI-318 multiplier (Fig. 5). Table 7 lists the recommended values of multipliers for non-prestressed slabs.

$$\Delta_l = (1 + \lambda_c + \lambda_{sh}) * \Delta_i \qquad (10)$$

Where

λ_c = creep multiplier;

λ_{sh} = shrinkage multiplier;

Or, simply $\Delta_l = C * \Delta_l$, where $C = (1 + \lambda_c + \lambda_{sh})$

TABLE 7 MULTIPLIERS FOR LONG-TERM DEFLECTIONS

Source	**Immediate deflection**	**Creep** λ_c	**Shrinkage** λ_{sh}	**Total** C
Sbarounis(1984)	1.0	2.8	1.2	5.0
Branson(1977)	1.0	2.0	1.0	4.0
Graham and Scanlon (1986b)	1.0	2.0	2.0	5.0
ACI-318	1.0	2.0		3.0

Based on the author's observation and experience, it is recommended that structures built in California use the following values:

For conventionally reinforced floor systems C = 4
For post-tensioned floor systems C = 3

LOAD COMBINATIONS

The load combination proposed for evaluating the deflection of a floor system depends on the objective of the floors evaluation. The following describe several common scenarios.

Total Long-Term Displacement From Removal of Forms

(1.0*SW + 1.0*SDL + 1.0*PT + 0.3*LL)* C

Where
SW = selfweight;
SDL = superimposed dead load, (floor cover and partitions);
PT = post-tensioning; and
LL = design live load.

The above load combination is conservative as it assumes the application of superimposed loads as well as the application of sustained live load of the structure to take place at the time of removal of the supports below the cast floort. The factor 0.3 suggested for live load is for “sustained” load combination. The significance of the above load combination is that it provides a measure for the total deflection from the position of the forms at the time of concrete casting. Its magnitude must be evaluated for aesthetics and drainage of surface water, if applicable. It is used for checking the deflection of parking structure decks or roofs, where the floor is placed in service in its as-cast condition.

Load Combination for Code Checks

For the acceptability of a floor deflection in connection with the code specified maximum values listed in Table 1, the following two load combinations apply.

1.0*LL

C1*C*(SW + SDL + PT) + 0.3*C2*C*LL + 0.7*LL

Where C1 is the fraction of long-term deflection coefficient related to the balance of long-term deflection subsequent to construction installation likely to be damaged by deflection of slab. Partitions and other fixtures are generally installed when more than one-half of long-term deflection has taken place. As a result, C1 is generally less than 50% of the long-term deflection multiplier, assuming that the superimposed dead load and partitions are not installed before 40 days from date of casting the floor (Fig. 6). C2 relates to the time, when construction is complete and in-service live load applied. This is generally less than 20% of the long-term multiplier. Figure 6 can be used as a guideline for values of C1 and C2. For example, if the in-service live load of a structure is put in place six months subsequent to casting the floor, the value of C2 will be approximately 0.25 (value associated with 180 days in Fig. 6).

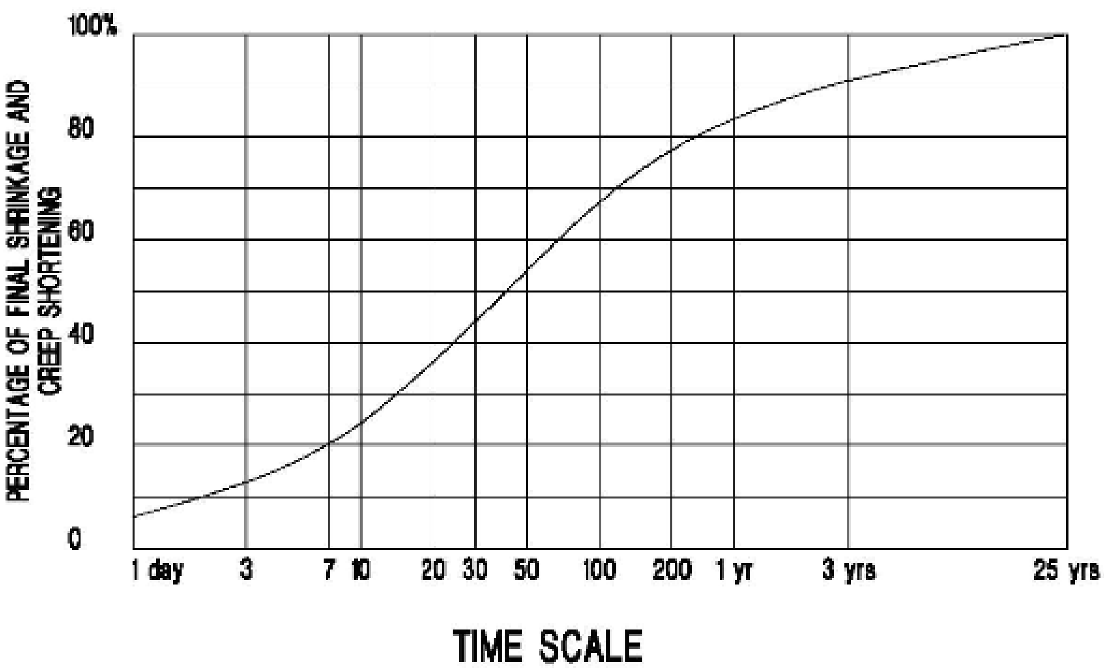


FIGURE 6 LONG-TERM SHORTENING OF CONCRETE MEMBERS DUE TO CREEP AND SHRINKAGE WITH TIME

LIVE LOAD DEFLECTION

Even when using linear elastic theory to calculate a floor system's deflection, cracking will result in a non-linear response. For the same load, the deflection of a slab depends on the extent of cracking prior to the application of the load. Therefore, when calculating the deflection due to the instantaneous application of live load, one must use the following procedure:

Deflection due to LL = (deflection due to DL+LL) – (deflection due to DL)

The above accounts for loss of stiffness due to dead load prior to the application of live load.

APPENDIX A

CHARACTERISTICS OF PARISSA APARTMENTS TYPICAL FLOOR

Geometry
Slab thickness and support dimensions (see plan)

Concrete

f'c (28 day cylinder strength)	= 5000 psi (34.47 MPa)
Wc (unit weight)	= 150 pcf (2403 kg/m^3)
Ec (modulus of elasticity at 28 days)	= 4,287 ksi (29558 MPa)

Non-Prestressed Reinforcement

Yield stress	= 60 ksi (400 MPa)

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