



PCI Big Beam Report 2020 - 2021

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Table of Contents

Section 1 Executive Summary	3
Section 2 Certification Form	5
Section 3 Test Summary Form	6
Section 4 Structural Drawings.....	7
Section 4 Concrete Mix Design	8
Section 5 Structural Design	10
Section 6 Beam Fabrication	11
Section 7 Beam Test and Results	13
Section 8 Personal Statements	16
Section 9 References.....	17
Section 10 Permanent Address.....	17
Section 11 Appendices.....	18

Section 1 Executive Summary

PCI Big Beam Competition is a yearly competition that challenges students to apply their skills and abilities to design a precast/prestressed concrete beam. This year, UMKC has entered two teams into the competition under the mentorship of Dr. Ganesh Thiagarajan, who graciously guided the team through the entire process of design and fabrication. We would like to thank him, as well as Terry Fleck (with Coreslab Structures - our PCI producer sponsor) for his involvement and advisement and the many other employees who assisted in the success of our constructed beam. Our thanks to Dr. Matt O'Reilly, professor at University of Kansas for allowing us to use the school's lab to perform the required test and certifying our predictions. Also, thanks to Kent Dye the lab technician in charge of our beam test.

The 2020-2021 PCI competition specified the design criteria that our beam needed to fulfill. These are summarized as follows:

- Simple supported beam, 20 ft clear span (c-c of bearings).
- Two applied loads with a specified midspan offset (see figure 1).
- The beam should not crack under a total applied load of $2P = 20$ kips.
- The ultimate load should be between $32 < 2P < 40$ kips.

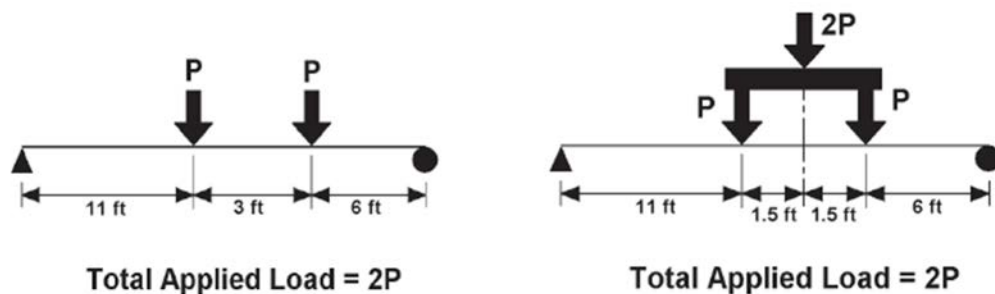


Figure 1. Permitted Loading Configuration

With these conditions and other fabrication limitations in mind, our team designed an I-shaped beam with three prestressing strands in the bottom section and three steel reinforcing bars in the top section. The design was a trial/error approach. Our target ultimate load was the middle value of the allowable range. After several iterations, we achieved the loading criteria specified by PCI.

The beam was fabricated at Coreslab Structure, Kansas City, KS, and then shipped to the lab at University of Kansas, Lawrence, KS, to be tested. The loading configuration was achieved using a jack and spreader beam. The cracking load and peak load is shown in figure 2 and 3. Table 1 summarizes the results:

Table 1. Predictions & Test Outcome Summary

Design Criteria	Prediction	Test Outcome
Cracking Load (kips)	22.9	24.875
Maximum Load (kips)	36.06	36.998
Deflection at Midspan (in)	3.60	3.032

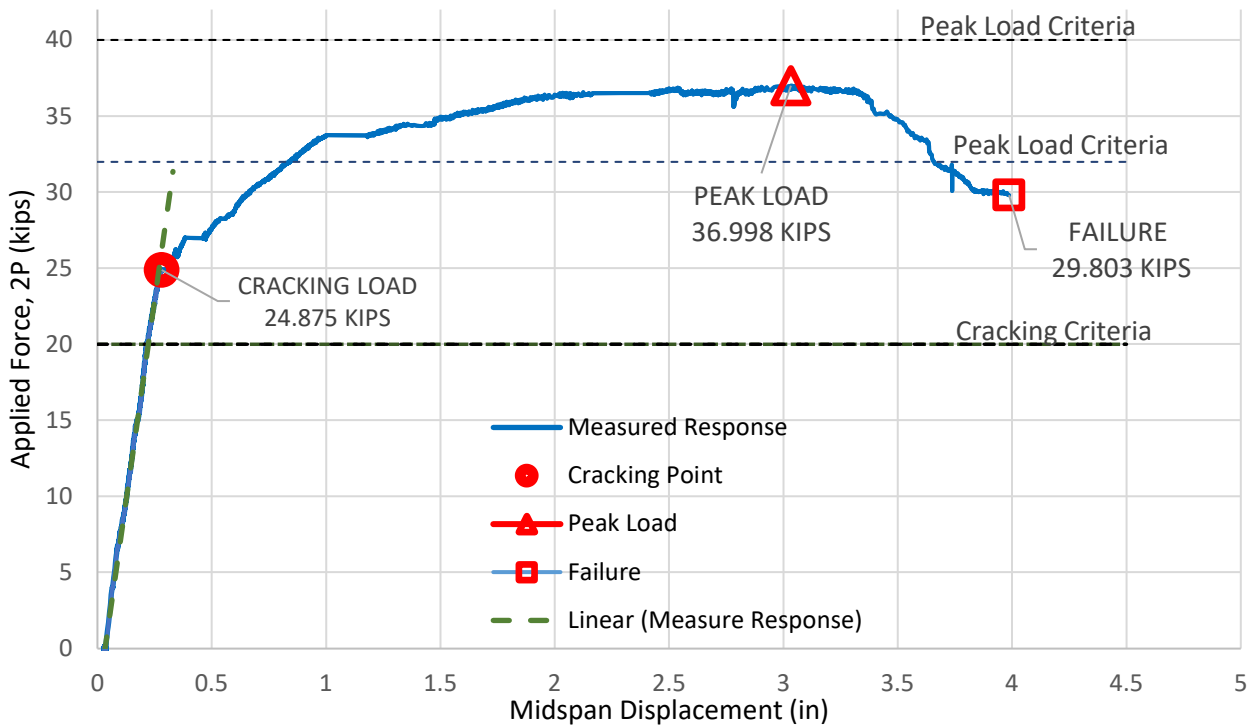


Figure 2. Load vs Midspan Displacement

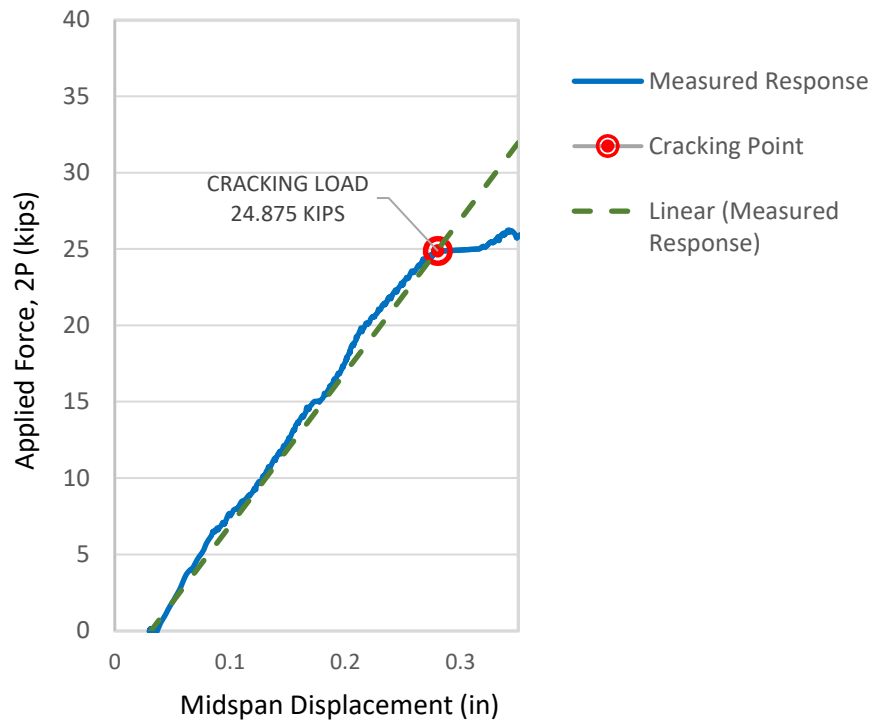


Figure 3. Cracking Load

PCI BIG BEAM COMPETITION 2020-2021

CERTIFICATION

Coreslab Structures Inc. (PCI Producer) / University of Kansas (Lab Test)

As a representative of (name of PCI Producer Member or sponsoring organization)

University of Missouri - Kansas City / Team II

Sponsoring (name of school and team number)

I certify that:

- The beam submitted by this team was fabricated and tested within the contest period.
- The calculations of predicted cracking load, maximum load, and deflection were done prior to testing of the beam.
- The students were chiefly responsible for the design.
- The students participated in the fabrication to the extent that was prudent and safe.
- The submitted test results are, to the best of my knowledge, correct, and the video submitted is of the actual test.

Certified by:

Mark O'Reilly

Signature

Mark O'Reilly

Name (please print)

06/28/2021

Date

36.06 Kips (Total)

Predicted Maximum Load

22.9 Kips (Total)

Predicted Cracking Load

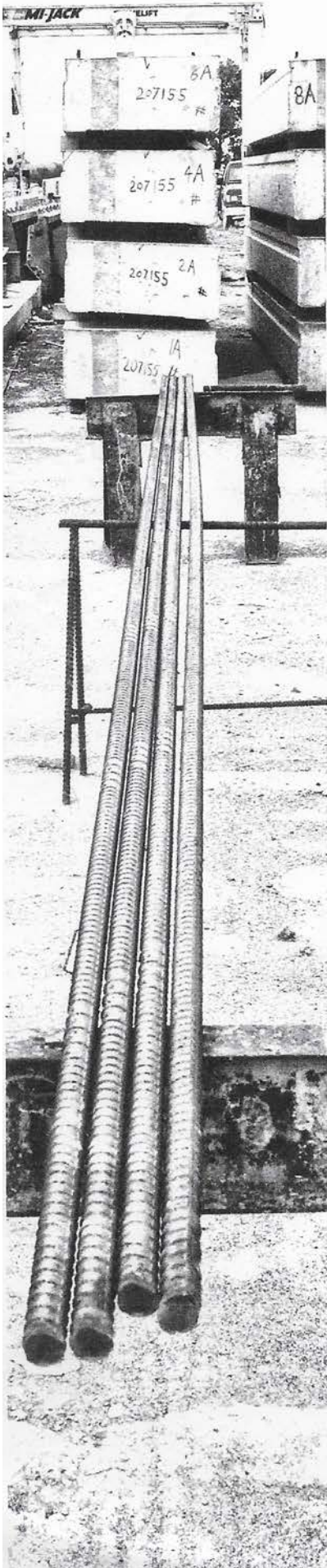
3.60 in (AT PEAK LOAD)

Predicted Deflection at Midspan

THIS CERTIFICATION MUST BE PART OF THE FINAL REPORT

Sponsored by:

ASPIRE®



PCI BIG BEAM COMPETITION 2020-2021

06/28/2021

Date

University of Missouri - Kansas City

Student Team (school name)

Team II

Team Number

05/08/2021

Date of Casting

Basic Information

1. Age of beam at testing (days) 51

2. Compressive cylinder tests*

Number tested 3

Size of cylinders 4"x8"

Average (psi) 9482.2 psi at 48 days

3. Concrete properties

Unit weight of concrete (lb/ft³) 144

Slump (in.) 9

Air content (%) 6.2

Tensile strength (psi) 835

Circle one: Split cylinder MOR beam

4. Pretest calculations

a. Applied load (total) to cause cracking (kip) 22.9

b. Maximum applied point load at midspan (kip) 36.06

c. Maximum anticipated deflection due to applied load only (in.) 3.60

Pretest calculations MUST be completed before testing.

* International entries may substitute the appropriate compressive strength test for their country.

Judging Criteria

Teams MUST fill in these values.

1. Actual maximum applied load (kip) 36.998

2. Measured cracking load (kip)[‡] 24.785

3. Cost (dollars) \$213.28

4. Weight (lb) 1748

5. Largest measured deflection (in.) 3.032

6. Most accurate calculations

a. Absolute value of (maximum applied load – calculated applied load)/calculated applied load 2.60 %

b. Absolute value of (maximum measured deflection – calculated deflection)/calculated deflection 15.78%

c. Absolute value of (measured cracking load – calculated cracking load)/calculated cracking load 8.62 %

Total of three absolute values (a + b + c) = 27.00%

[‡] Measured cracking load is found from the "bend-over" point in the load/deflection curve. Provide load/deflection curve in report.

Test summary forms must be included with the final report, due July 15, 2021.

Sponsored by:



Quantity = 27
fy = 60 ksi

[illegible]
$$3/8'' = 1'-0''$$

Section 4 Concrete Mix Design

Our team defined preliminary concrete mix requirements based on ACI 318-19, Chapter 19. A discussion was held with our PCI producer sponsor regarding possible mix design options. Coreslab Structures (Kansas) provided tentative mix designs that could be produced at their facility.

We chose a concrete mix design that was approved by the Missouri Department of Transportation – Construction and Materials Division based on the criteria stated below.

Type III cement was chosen in order to obtain high early strength (typical in precast plant operations) which is necessary to reduce removal time of the beam from the casting bed. The material cost table provided for this competition outlines the cost for standard concrete and Ultra-High-Performance Concrete. Our team selected the 10ksi mix design from the provided options.

This mix consisted of materials commonly sourced in the Midwest area. It also contained an air-entraining agent and a water-reducing agent, which greatly improved the concrete's workability, and allowed the concrete to flow through the narrow spacing between the reinforcement and form work. It also reduced segregation of the materials and lowered the possible excess amount of bleeding.

The concrete mix ticket that was provided to us by Coreslab Structures lists the individual materials and additives present in the mix. Field and laboratory tests were done on the concrete. This was done to ensure we received the requested mix design and to base our predictions on the resulting test values.

Table 2. Mix Proportion

Material Name		Quantity (lb)
Fine Agg.	Natural Sand for Concrete Class A	1116
Coarse Agg.	Masonry Grade E LS/DO	1752
Cement	Portland Cement Type III	800

Table 3. Admixture Proportion

Admixture Rate Range		oz/cwt
Admixture 1	Air Entraining Agent for Concrete	5-7
Admixture 2	Type A Water Reducing Agent for Concrete	5-10

Testing on the day of the pour indicated that the slump was 9" and air was 6.2%; Both are in the acceptable range and supplied us with a unit weight of 144 lb/ft³. A compressive strength report of the concrete can be found in the appendix. From this report, we found the average strength of compressive and tensile strength at 2 days, 28 days, and 48 days. The last round of concrete compressive and tensile specimens were tested 2 days prior to the big beam test in order to have enough time for the pre-test calculations. The results are shown in tables 3 and 4.

Table 3: ASTM C39 Compressive Concrete Cylinder Results

Specimen Number	Test Age Days	Test Date	Field / Lab Cure Days	Average Cylinder Diameter (in)	Cylinder Area (in ²)	Max Load (lbs)	Strength (psi)
1-1	2	5/10/2021	2	4	12.57	110,680	8,808
1-2	2	5/10/2021	2	4.1	12.63	106,450	8,471
1-3	2	5/10/2021	2	4	12.57	112,370	8,942
2-day average							8,740
1-4	28	6/5/2021	28	4	12.57	126,840	10,094
1-5	28	6/5/2021	28	4	12.57	121,630	9,679
1-6	28	6/5/2021	28	4	12.57	119,020	9,471
28-day average							9,748
1-7	48	6/25/2021	48	4	12.57	121,770	9,690
1-8	48	6/25/2021	48	4	12.57	111,450	8,869
1-9	48	6/25/2021	48	4	12.57	124,250	9,888
48-day average							9,482

ASTM C78 Flexural strength tests were also performed on our MOR concrete beams and produced the following strengths:

Table 4: ASTM C78 Flexural Strength Beam Results

Beam Sample	Test Age Days	Dimensions (in x in)	Moment of Inertia (in ⁴)	Breaking Force (kips)	Midspan Moment (kip-in)	f _r (psi)
A	2	6.03 x 6.10	114.1	7.85	23.55	630
B	28	5.94 x 6.20	118.0	9.38	28.15	740
C	48	6.05 x 6.08	113.0	10.36	31.07	835

From the structural standpoint, the mechanical properties provided by this concrete mix worked fine. We refined our predictions based on the results above. From the constructive perspective, the mix design was good, but could be improved in terms of flowability. After the removal of the form work, we observed some voids in the concrete. We could avoid this by using self-consolidated concrete.

Section 5 Structural Design

The structural design was performed based on the PCI competition design criteria, ACI 318-19, PCI design handbook 8th ed, and some other fabrication constraints specified by our PCI producer. The construction limitations were the following:

- The two UMKC teams will use the same casting bed, the same type and number of strands and the same concrete mix.
- The spacing between strands was limited to 2" center-to-center.
- Straight tendon profile.

With this information, we began to go through the trial-and-error process of dimensioning our beam. We added 6 in at both ends of the beam to allow the installation of the supports, that gave us a total beam length of 21 ft. Initially, we tried T, inverted T, and I-shape sections. Their dimensions were reduced as much as possible based on the loading criteria and minimum concrete cover and rebar spacing requirements of ACI 318-19. We chose I-shaped cross-section because it allowed us to reduced concrete area more than the other 2 sections without having lateral-torsional buckling issues.

Our first design considered two 1/2" strands, but this was changed to three 1/2" strands to allow the fabrication of the two beams in the same line. Calculations were updated to find an initial pre-stress (or jacking force) that worked for both teams.

For the section properties, we took into account the contribution of the reinforcing steel using the modular ratio at initial stage and final stage. The depth of the PS strands was determined by the beam height minus 2" (specified strand pattern), the top flange dimensions was then determined based on the required concrete compression area to balance the strands' tension on the bottom flange.

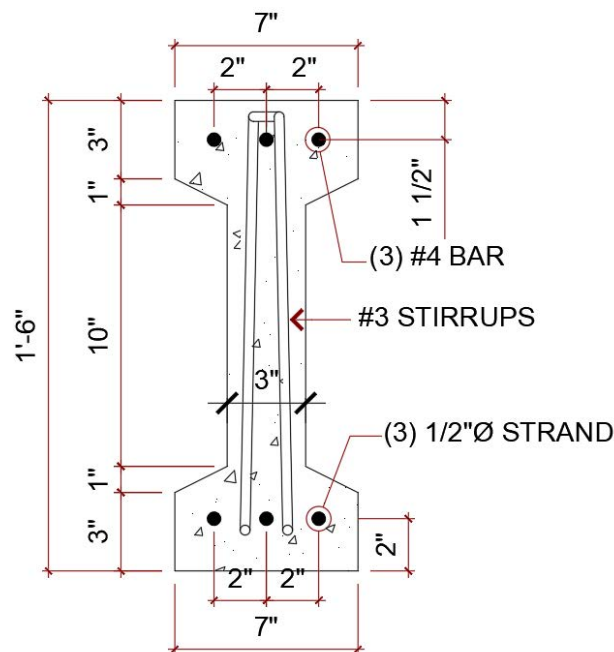


Figure 4. Beam Section

The stresses were calculated based on three different stages:

- At transfer (wire cut), while the beam was still in the casting bed.
- When the beam was transported to the testing facility, we accounted for the effects of the dead load and stresses in the top section of the beam.
- At final-loading configuration.

The stresses check at transfer showed that the stress on top of the beam would exceed the estimated concrete tensile capacity of $7.5\sqrt{f'_c}$, therefore we had to provide mild reinforcement close to the top face to avoid cracking at wire cut and transportation. The final cross-section is shown in figure 4.

The cracking load was back calculated based on cracking moment and load configuration. The cracking moment was initially estimated using ACI 318-19 provisions, and then the predictions were refined based on the concrete test results.

Our next step was calculating ultimate load capacity. We completed this task using the strain compatibility method. We found the moment nominal capacity of the section, then back calculated the ultimate load based on the load configuration. The calculations were refined with the concrete test results prior the beam test. The details are provided in the appendix.

To predict deflection, our team opted to use the Moment-Curvature Method because it associates the strains on the top and bottom fiber prior to cracking, at cracking load stage, and ultimate stage (failure). We defined the concrete strain at failure to be 0.003 in/in based on ACI 318-19. Once the curvatures along the beam were calculated, we used the Unitary Load Method to estimate the deflection at midspan. We implemented a numerical technique, rather than the integration method. The details of this calculation are shown in the appendix.

Section 6 Beam Fabrication

On May 8, 2021, at Coreslab Structures (Kansas) facility, our team arrived to do the final inspection of the form and to assist with cylinder and MOR beam casting. As a team, we inspected the rebar placement for appropriate spacings and dimension deviations. As per our design, we checked that three #4 grade 60 rebar were running through the top with appropriate concrete cover, that the number and spacing of the #3 stirrups were correct, and the size and spacing of the strands were accurate. Cleanliness of the rebar and mold were also checked. Figure 5 (a) shows the form work and rebar after our inspection.

The way the concrete was poured into the mold was using a pouring bucket lifted above the mold. After some amount of concrete was poured into the mold, a wheelbarrow was filled with concrete from the bucket. As both of our team members are ACI certified concrete field-testing grade I technicians, our team performed the required concrete testing, see figure 5 (b). Ten cylinders and three MOR beams were fabricated as shown in figure 5 (c). After the beam was poured, the casting bed was covered to allow steam curing. The concrete specimens were left alongside our beam. This barrier ensured the specimens and beams would be under optimal curing conditions. The strands were cut after 2 days of curing, when the concrete reached the specified initial strength. The beam was shipped to the University of Kansas in Lawrence on 05/21/2021 to await testing day, see picture 5 (d).

A certification and tensioning report was prepared by Coreslab Structures and given to the team to update our predictions based on measured properties. Both can be found in the appendix.



(a)



(b)



(c)

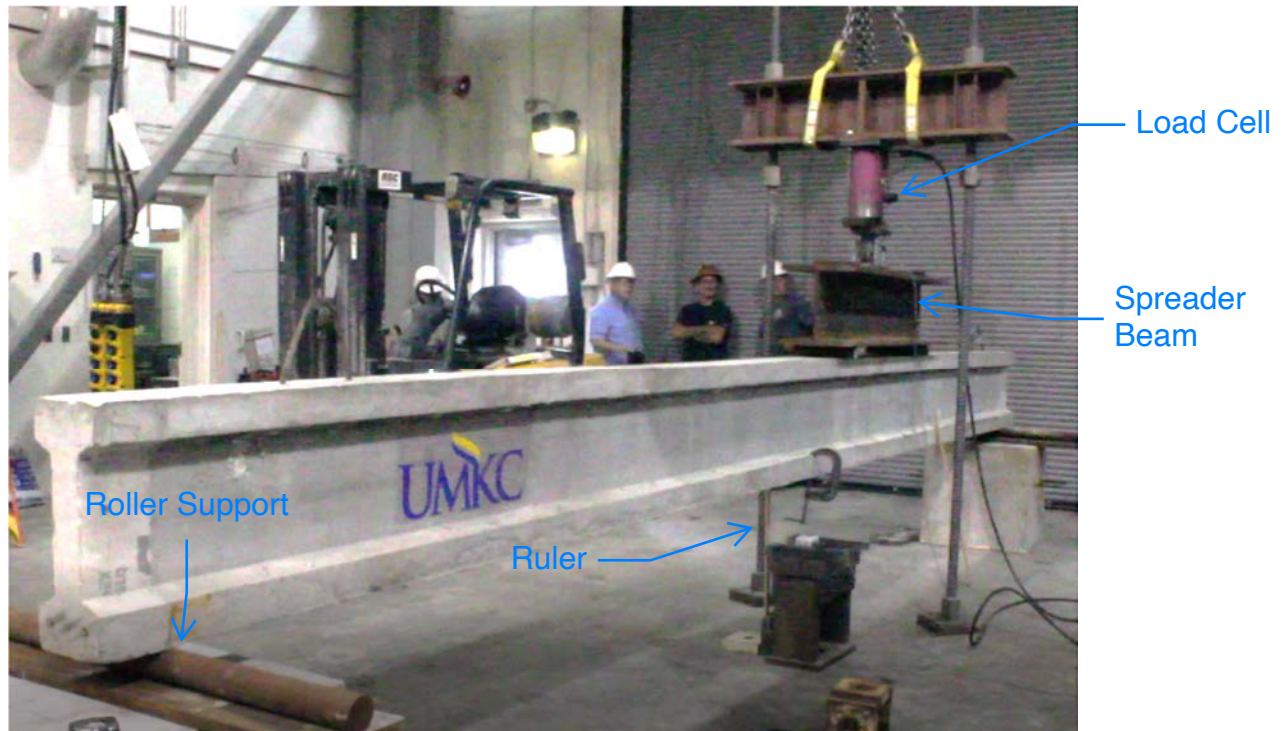


(d)

Figure 5. (a) Form work after inspection. (b) (c) Fresh concrete tests. (d) Shipping to Lab.

Section 7 Beam Test and Results

The beam was tested at the University of Kansas Laboratory on 6/28/2021 (51 days after casting). Once the team arrived at the lab, the team marked the beam to achieve the required loading configuration. The center to center distance between supports was 20 ft. The beam was intentionally made 1 foot longer to allow 6" overhang on each side, see figure 6 (c). A spreader beam was installed at 11 ft from the left support. The beam setup is shown in figure 6 (a). According to the PCI competition rules, a ruler was installed at midspan and the test was video recorded for verification purposes, see figure 6 (b).



(a)



(b)



(c)

Figure 6. (a) Beam Setup. (b) Ruler at midspan. (c) Roller support.

The beam was loaded based on the specified load configuration and the deflection at midspan measured. The load deflection plot is shown in figure 7. This plot was corrected to account for the weight of the testing equipment to represent an accurate force applied to the beam.

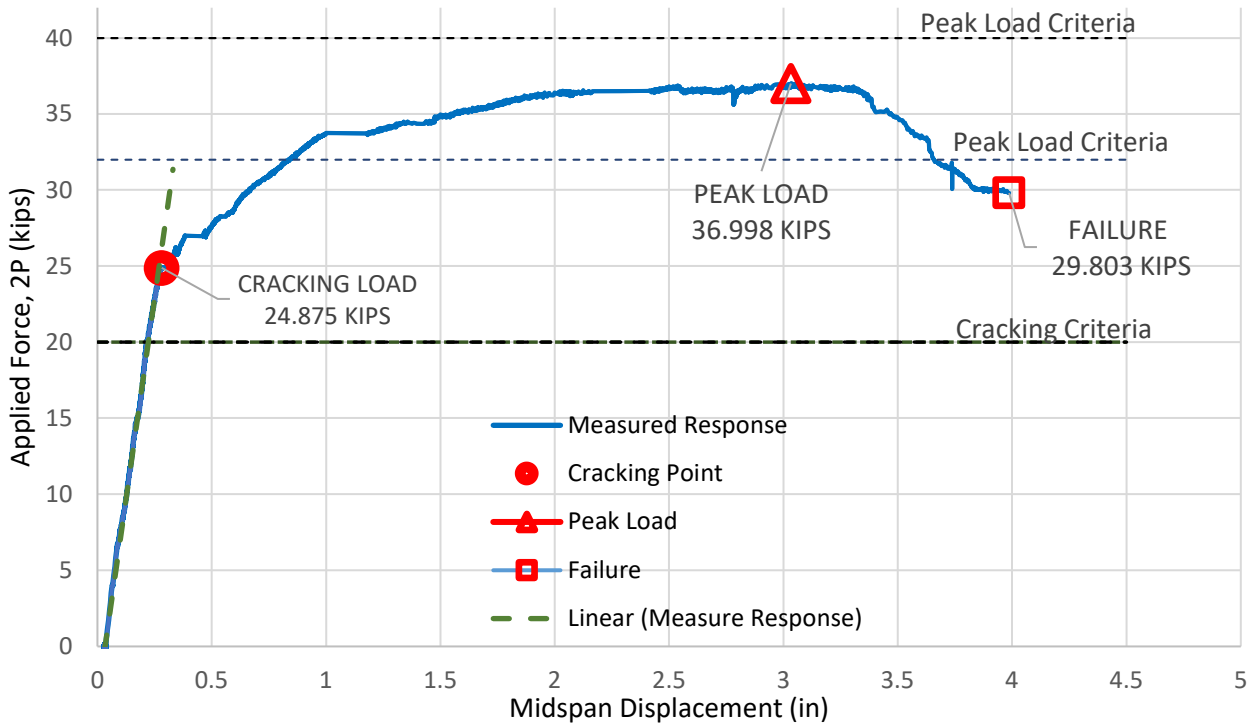


Figure 7. Load vs Midspan Displacement

The first crack, as seen in Figure 8, was due to flexure and was located close to the bottom face of the point load closest to the midspan. Our design goal was to achieve the largest deflection possible. In that sense, flexural cracks were expected.

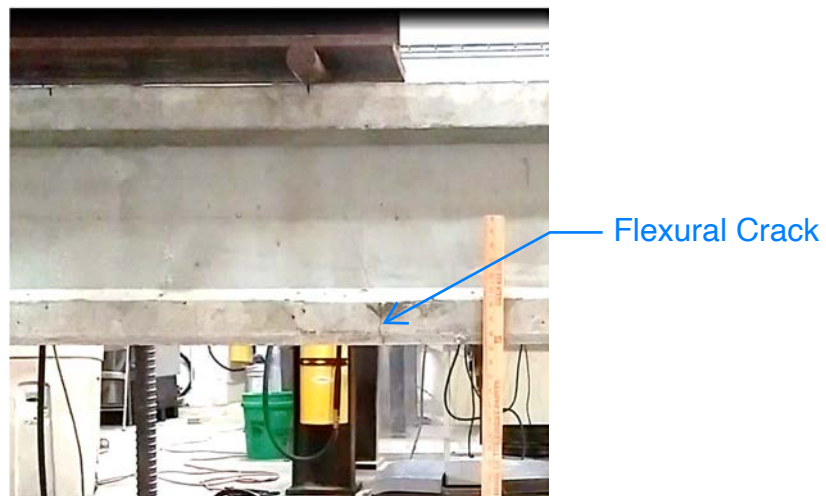


Figure 8. First Crack (flexure)

The “bend-over” point was found from the intersection of the elastic linear response and the load-deflection graph. Figure 9 shows an enhanced version of the load-deflection graph highlighting the cracking point. The beam was able to hold a steady load (between 35 and 36 kips) for about 5 minutes. A larger deflection and wider cracks were observed during this time. Lastly, our beam experienced a flexural failure by crushing of the concrete at the compression side of the beam as shown in figure 10.

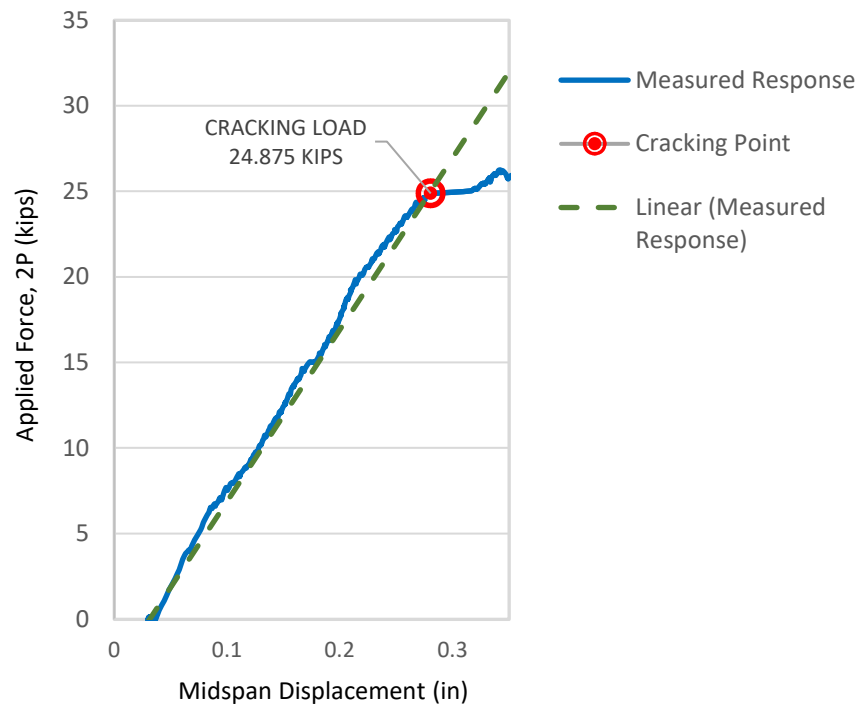


Figure 9. Cracking Load



Figure 10. Beam at failure (flexure).

The beam was designed to have a tension-controlled failure. The formation of flexural cracks and large displacement was anticipated. The beam performed close to our predictions. The difference is summarized in table 5.

Table 5: Comparison of Predicted and test Outcome

Design Criteria	Prediction	Test Outcome	Percent Difference (%)
Cracking Load (kips)	22.9	24.875	8.62%
Maximum Load (kips)	36.06	36.998	2.60%
Deflection at Midspan (in)	3.60	3.032	15.78%
Total			27.00%

Note: % Difference = (Actual - Predicted) / Predicted * 100%

Section 8 Personal Statements

Juan Plasencia – Our design changed quite a bit because of the fabrication constraints and other parameters out of our control. So, the lesson I learned from all of this would be the fabrication and the design should work together with constant communication. I believe that, prior to starting the design, it is very important for the designer to sit down with the manufacturer and discuss all the fabrication constraints to improve the efficiency of the design.

I have also learned about the importance of proper confinement of main reinforcement subjected to flexure-compression. During the testing, we had pretty good deflections until the top section failed. We saw how the mild steel buckled. I believe that with proper confinement, we could achieve more deflection. But this was another construction constraint, we didn't have enough room to provide enclosed stirrups.

Jose Ramirez - Being a part of the PCI Big Beam Competition provided me the opportunity to learn valuable knowledge on designing and utilizing precast prestressed concrete. It gave me the opportunity to dive deeper into subject matters like deflection and flexural strength and allowed me to achieve a better understanding of those topics. I learned just how much one aspect of a design can dramatically affect another. This constant struggle to find a working combination of values and dimensions highlighted just how much effort goes into achieving a workable design. Being a part of this team and competition enriched my knowledge on prestressed concrete and it further demonstrated to me how important communication, and being a proactive member of a team, is for the success of a project such as this one. I will most definitely be using what I have acquired here in my career as an engineer.

Section 9 References

- American Concrete Institute. (2019). *Aci 318-19: Building code requirements for structural concrete: ccommentary on building code requirements for structural concrete (Aci 318R-19)*.
- Precast/Prestressed Concrete Institute. (2017). *PCI design handbook: precast and prestressed concrete*. 8th Edition.
- Nawy, E. G., Nawy, E. G., & Nawy, E. G. (2010). *Prestressed concrete: a fundamental approach*. Prentice Hall.

Section 10 Permanent Address

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APPENDIX A

Flexural Design

PETITION
DESIGN

Table with 3 columns: Property, Value, Unit. Rows include: s, 0.50, ft; 20.0, ft; l, 21.00, ft.

Table with 3 columns: Property, Value, Unit. Rows include: h, 18.1, in; h flange beam avg, 3.5, in; b Bottom Flange, 7.06, in; b Top Flange, 7.06, in; b_w, 3.0, in.

Table with 3 columns: Property, Value, Unit. Rows include: A_c, 83.32, in^2; I_c, 3,049.79, in^4; r^2 = I_c/A_c, 36.60, in^2; c', 9.10, in; c_b, 9.03, in; S', 335.29, in^3; S_b, 337.78, in^3.

Table with 3 columns: Property, Value, Unit. Rows include: f'_c, 9,482.2, psi; f'_{ci}, 8,740.0, psi; E_{ci}, 4,690,574, psi; E_c, 4,844,512, psi.

Table with 3 columns: Property, Value, Unit. Rows include: f_y, 60,000.0, psi; E_s, 29,000.0, ksi; n_i = E_s/E_{ci}, 6.2; n_f = E_s/E_c, 6.0.

Table with 3 columns: Property, Value, Unit. Rows include: f_{pu}, 270,000.0, psi; P_e/P_l = gamma, 0.85; E_{ps}, 28,800.0, ksi; f_{pe}, 172,519, psi; n_{ip} = E_{ps}/E_{ci}, 6.1; n_{ip} = E_{ps}/E_c, 5.9.

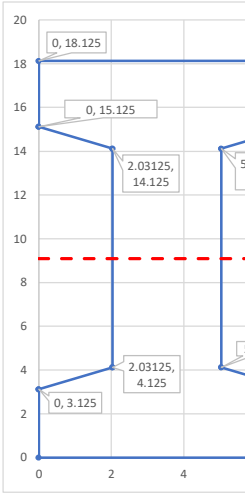
Table with 3 columns: Property, Value, Unit. Rows include: f_{pi}, 203,052, psi; f_{ti} = 6*sqrt(f'_{ci}), 630, psi; f_{ti} = 3*sqrt(f'_{ci}), 630, psi; f_{ci} = 0.70*f'_{ci}, -6,118, psi; f_{ci} = 0.60*f'_{ci}, -5,244, psi; f_t = 7.5*sqrt(f'_c), 835, psi; f_c = 0.45*f'_c, -4,267, psi.

Table with 2 columns: Property, Value. Rows include: Top Flange Width, 7.06, in; Top Flange Thickness, 3.00, in; Top Transition Height, 1.00, in; Web Thickness, 3.03, in; Web Height, 10.00, in; Bottom Flange Width, 7.06, in; Bottom Flange Thickness, 3.125, in; Bottom Transition Height, 1.00, in.

Table with 1 column: Formula. Rows include: ACI 363R, for f'c > 6ksi; E_c (psi) = [40,000*sqrt(f'_c) + 10^6]*(w_c/145)^1.5; f'_c = psi and w_c = lb/ft^3.

Table with 3 columns: Stage, Location, Maximum tensile stress. Rows include: During stressing, At jacking end, Least of: 0.94*f_{py}, 0.80*f_{py}, Maximum jacking force recommended by the supplier of anchorage device; Immediately after force transfer, At post-tensioning anchorage devices and couplers, 0.70*f_{py}.

Table with 2 columns: Location, Concrete compressive stress limits. Rows include: End of simply-supported members, 0.70*f'_{ci}; All other locations, 0.60*f'_{ci}.



ACI 318 – 19
24.5.4 Permissible concrete compressive stress limits at service loads

24.5.4.1 For Class U and T members, extreme concrete fiber stress in compression after allowance for all prestress losses shall not exceed the limits in Table 24.5.4.1.

Table with 2 columns: Load condition, Concrete stress. Rows include: Prestress plus sustained load, Concrete stress; Prestress plus total load, Concrete stress.

Table 24.5.3.2—Concrete tensile stress limits immediately after transfer of prestress and additional bonded reinforcement in

Table with 2 columns: Location, Concrete stress. Rows include: Ends of simply-supported members, Concrete stress; All other locations, Concrete stress.

Table 24.5.2.1—Classification of prestress flexural members based on f_t

Table with 2 columns: Assumed behavior, Class. Rows include: Uncracked, Class I; Transition between uncracked and cracked, Class T; Cracked, Class C.

(1) Prestressed two-way slabs shall be designed as Class

IMENTS

ght	wc	=	144.00	lb/ft ³	determined with actual test at beam pour
elf weight dead load (Untopped)	w _D	=	83.32	lb/ft	
d (sustained)	P _L	=	10,000	lb	

NT (POINT LOAD 1)	Distance from end, X _{LOAD 1}	=	11.5	ft
oment (Beam)	M _D	=	4,114	lb-ft
ment	M _L	=	82,500	lb-ft

POINT LOAD 2)	Distance from end, X _{LOAD 2}	=	14.5	ft
oment (Beam)	M _D	=	3,489	lb-ft
ment	M _L	=	75,000	lb-ft

T MID SPAN	Distance from end, X _{MIDSPAN}	=	10.5	ft
oment (Beam)	M _D	=	4,156	lb-ft
ment	M _L	=	75,000	lb-ft

T X ₁	Distance from end, X ₁	=	6.0	ft
oment (Beam)	M _D	=	3,312	lb-ft
ment	M _L	=	41,250	lb-ft

T X ₂	Distance from end, X ₂	=	18.0	ft
oment (Beam)	M _D	=	1,812	lb-ft
ment	M _L	=	31,250	lb-ft

OMENT L=11.5 FT

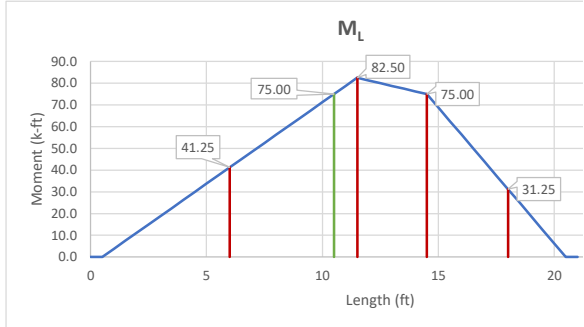
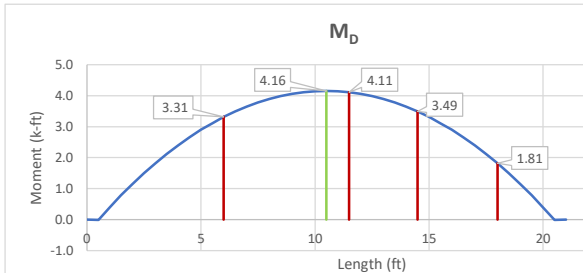
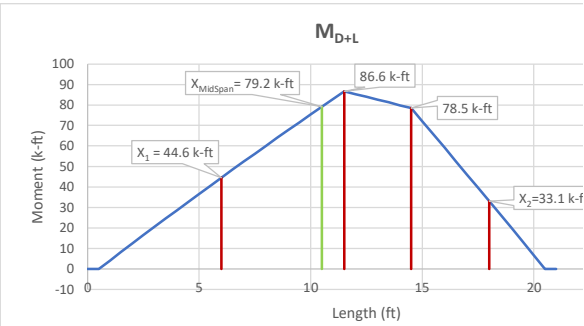
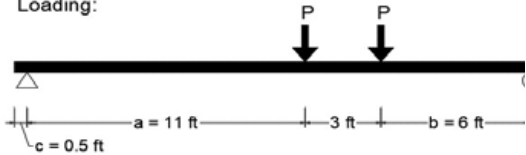
- M _U = 1.0 D + 1.6 L	M _U min	=	136,114	lb-ft	32,000 lb
- M _U = 1.0 D + 2.0 L	M _U max	=	169,114	lb-ft	40,000 lb

ECTION MODULUS

Eccentricity	=	Constant
This check will be done at	Max. Moment Location, L= 11.5ft	

Section Modulus - Top	S ^t	=	216	in ³	<	335 in ³	OK
Section Modulus - Bottom	S _b	=	196	in ³	<	338 in ³	OK

Loading:



Constant Eccentricity:

$$S^t \geq \frac{M_D + M_{SD} + M_L}{\gamma f_a - f_c}$$

$$S_b \geq \frac{M_D + M_{SD} + M_L}{f_t - \gamma f_d}$$

Y & NUMBER OF STRANDS

Stress at Transfer @ centroid level	$\bar{f}_{ci}$ =	-2,756.5	psi	Ends Control
Initial Tension	Pi =	229,671	lb	
Eccentricity	e =	4.9	in	Ends Control
Area of PS steel	A_{ps} =	1.13	in ²	
1 strand		0.1507	in ²	1/2" (7) wire strand Grade 270
Number of strands	N =	8		

$P_i = A_{ps} f_{pi} \rightarrow$

Constant Eccentricity:

$P_i = \bar{f}_{ci} A_c$

$e_e = (f_u - \bar{f}_{ci}) \frac{S'}{P_i}$

$\bar{f}_{ci} = f_u - \frac{e_i}{h} (f_u - f_{ci})$

where tensile stress is $\oplus$ and compressive stress is $\ominus$

TENDONS (TRIAL/ERROR)

Say Use	N =	3	
Area of PS steel	A_{ps} =	0.45	in ²
Distance from bottom to PS strands	y_{ps} =	2.0	in

Assumed based on Table ACI 318-19, Table 20.5.1.3.3

PRE STRESS REBAR (TRIAL/ERROR)

REBAR - TENSION

Say use #	4	bars
Yield strength	f_y =	60,000 psi
Bar area	A_{bar} =	0.20 in ²

Use	N =	0.0
Area of steel	A_s =	0.00 in ²

Effective depth	h =	18.1 in
Clearance to reinforcement	cc =	1 in
Clearance due to construction	=	0.00 in
Clearance to reinforcement	=	3/4 in
Bar diameter	bar #	3
Clearance for non-prestressed rebar	d =	15.75 in

REBAR - COMPRESSION

Say use #	4	bars
Yield strength	f_y =	60,000 psi
Bar area	A_{bar} =	0.20 in ²

Use	N =	3.0
Area of steel	A'_s =	0.60 in ²

Effective depth due to construction issues	cc =	0.87 in
Clearance to reinforcement	=	0.00 in
Bar diameter	bar #	3

Clearance for non-prestressed rebar	d' =	1.50 in
Effective depth $d' < 0.15 * d_p$	d' max =	2.42 in

OK

CRITERIA USED TO DETERMINE THE PS PATTERN AT MIDDLE AND ENDS:

CODE

Table 20.5.1.3.3—Specified concrete precast-nonprestressed or prestressed members manufactured under plan

Concrete exposure	Member	Reinforcement
Exposed to weather or in contact with ground	Walls	No. 14 and No. 18 bars, tendons larger than 1-1/2 in. diameter
		No. 11 bars and small W31 and D31 wire at smaller; tendons and strands 1-1/2 in. diameter and smaller
	All other	No. 14 and No. 18 bars, tendons larger than 1-1/2 in. diameter
Not exposed to weather or in contact with ground	Slabs, joists, and walls	No. 6 through No. 11 bars, tendons and strands larger than 5/8 in. diameter through 1-1/2 in. diameter
		No. 5 bar, W31 or D31 and smaller; tendons and strands 5/8 in. diameter and smaller
	Beams, columns, pedestals, and tension ties	Primary reinforcement: No. 14 and No. 18 bars, tendons larger than 1-1/2 in. diameter and smaller; Tendons and strands 1-1/2 in. diameter and smaller; No. 11 bar, W31 or D31 wire, and smaller
Not exposed to weather or in contact with ground	Stirrups, ties, spirals, and hoops	Primary reinforcement
		Stirrups, ties, spirals, and hoops

25.2.4 For pretensioned strands at ends of a member, minimum center-to-center spacing s shall be the greater of the value in Table 25.2.4, and $[(4/3)d_{agg} + d_b]$.

Table 25.2.4—Minimum center-to-center spacing of pretensioned strands at ends of members

f'_c , psi	Nominal strand diameter, in.	Minimum s
< 4000	All	$4d_b$
	< 0.5 in.	$4d_b$
≥ 4000	0.5 in.	1-3/4 in.
	0.6 in.	2 in.

25.2.5 For pretensioned wire at ends of a member, minimum center-to-center spacing, s , shall be the greater of $5d_b$ and $[(4/3)d_{agg} + d_b]$.

25.2.6 Reduced vertical spacing including bundling of prestressed reinforcement shall be permitted in the middle portion of a span.

25.2—Minimum spacing of reinforcement

25.2.1 For parallel nonprestressed reinforcement in a horizontal layer, clear spacing shall be at least the greatest of 1 in., d_b , and $(4/3)d_{agg}$.

25.2.2 For parallel nonprestressed reinforcement placed in two or more horizontal layers, reinforcement in the upper layers shall be placed directly above reinforcement in the bottom layer with a clear spacing between layers of at least 1 in.

IECK STRESSES AT INITIAL STAGE

SFORMED SECTION - INITIAL STAGE (See section Properties Calculation)

ea	A	=	88.8	in ²
uared radius of gyration	r ² _i	=	37.7	in ²
stance from CG to top fiber	C ^t _i	=	9.0	in
stance from CG to btm fiber	C _{bi}	=	9.1	in
p fiber section modulus	S ^t _i	=	371.0	in3
m fiber section modulus	S _{bi}	=	367.0	in3

nstant Eccentricity e_i = C_{bi} - Y_{ps} = 7.11 in

sting bed

Initial Prestress Load	M _D	=	0.0	lb-ft
Initial PS force	P _i	=	91,800	lb
Top Stress at Initial Load	f ^t	=	726	psi
Bottom Stress at Initial Load	f _b	=	-2,813	psi

Mid Span

Stress at Transfer Load				
Dead load Moment	M _D	=	4,156	lb-ft
Top Stress at Initial Load	f ^t	=	591	psi
Bottom Stress at Initial Load	f _b	=	-2,677	psi

K NON-PS STEEL ON TOP

Top Stress at Initial Load	f ^{top}	=	726	psi	Stresses at Casting Bed Constrols
Bottom Stress at Initial Load	f _{bottom}	=	-2,813	psi	Stresses at Casting Bed Constrols
Distance from top to NA at initial Prestress Load	NA	=	3.7	in	
Allowable Yield Stress of Compression Steel	f _s = 0.5*f _y	=	30,000.0	psi	
Top Flange Width	b	=	7.1	in	(conservative)
Tensile Force to be Supported by Mild Steel	T _{st}	=	9,520	lb	
Area of Top Steel Required	A _{s req}	=	0.32	in ²	
Area of Top Steel Provided	A _s	=	0.60	in ²	OK

NA = $\frac{h \times f^{top}}{|f^{top}| + |f_{bottom}|}$

T_{st} = $\frac{1}{2} f_s \times NA \times b$

IECK STRESSES AT SERVICE STAGE

SFORMED SECTION - SERVICE STAGE (See section Properties Calculation)

ea	A	=	88.5	in ²
uared radius of gyration	r ² _f	=	37.6	in ²
stance from CG to top fiber	C ^t _f	=	9.0	in
stance from CG to btm fiber	C _{bf}	=	9.1	in
p fiber section modulus	S ^t _f	=	369.6	in3
m fiber section modulus	S _{bf}	=	365.9	in3

nstant Eccentricity e_f = C_{bf} - Y_{ps} = 7.11 in

Point Load 1 (Max Moment)

Stress at service Load				
Dead load Moment	M _D	=	4,113.9	lb-ft
Live Load Moment	M _L	=	82,500.0	lb-ft
Effective PS force	P _e	=	77,996	lb
Stress at top of beam	f ^t	=	-2,193	psi
Stress at Bottom of beam	f _b	=	445	psi

Point Load 2

Stress at service Load				
Dead load Moment	M _D	=	3,489.0	lb-ft
Live Load Moment	M _L	=	75,000.0	lb-ft
Effective PS force	P _e	=	77,996	lb
Stress at top of beam	f ^t	=	-1,929	psi
Stress at Bottom of beam	f _b	=	178	psi

ENDS

Stress at service Load				
Dead load Moment	M _D	=	0.0	lb-ft
Live Load Moment	M _L	=	0.0	lb-ft
Effective PS force	P _e	=	77,996	lb
Stress at top of beam	f ^t	=	619	psi
Stress at Bottom of beam	f _b	=	-2,396	psi

Stress at Transfer

f' = $-\frac{P_i}{A_c} \left(1 - \frac{ec_i}{r^2} \right) - \frac{M_D}{S'} \leq f_{ti}$

f_b = $-\frac{P_i}{A_c} \left(1 + \frac{ec_b}{r^2} \right) + \frac{M_D}{S_b} \leq f_{ci}$

P_i = A_{ps}f_{pi}

Unshored Slab Case. Before the top slab is situ cast:

f' = $-\frac{P_i}{A_c} \left(1 - \frac{ec_i}{r^2} \right) - \frac{M_D + M_{SD}}{S'}$
f_b = $-\frac{P_i}{A_c} \left(1 + \frac{ec_b}{r^2} \right) + \frac{M_D + M_{SD}}{S_b}$

f' = $-\frac{P_i}{A_c} \left(1 - \frac{ec_i}{r^2} \right) - \frac{M_D + M_{SD}}{S'}$
f_b = $-\frac{P_i}{A_c} \left(1 + \frac{ec_b}{r^2} \right) + \frac{M_D + M_{SD}}{S_b}$

f' = $-\frac{M_{CSD} + M_L}{S'_{cs}}$ ← at Top of Slab
f_{bs} = $-\frac{M_{CSD} + M_L}{S'_{bs}}$ ← at Bottom of Slab

SSUMED ECCENTRICITY TO KERN BOUNDARY

PS force	$P_i = A_s f_{pi}$	P_i	=	91,800	lb
ive PS force (after losses)	$P_e = A_s f_{pe}$	P_e	=	77,996	lb

KERN POINTS - BEAM

it top of beam	k_t	=	4.13	in
it bottom of beam	k_b	=	4.18	in

IND UPPER ENVELOPE - ZERO STRESS AT TOP/BOTTOM OF SECTION

Envelope (initial stage):

mid span	M_D	=	4,155.6	lb-ft
	a_{min}	=	0.54	in
	e_b mid span	=	4.72	in

Point Load 1	M_D	=	4,113.9	lb-ft
	a_{min}	=	0.54	in
	e_b PL1	=	4.72	in

Point Load 2	M_D	=	3,489.0	lb-ft
	a_{min}	=	0.46	in
	e_b PL2	=	4.64	in

X_1	M_D	=	3,312.0	lb-ft
	a_{min}	=	0.43	in
	e_b X1	=	4.61	in

X_2	M_D	=	1,812.2	lb-ft
	a_{min}	=	0.24	in
	e_b X2	=	4.42	in

ends	M_D	=	0.0	lb-ft
	a_{min}	=	0.00	in
	e_b ends	=	4.18	in

Envelope (Service)

mid span	$M_D + M_L$	=	79,155.6	lb-ft
	a_{max}	=	12.18	in
	e_t mid span	=	8.04	in

Point Load 1	$M_D + M_L$	=	86,613.9	lb-ft
	a_{max}	=	13.33	in
	e_t PL1	=	9.19	in

Point Load 2	$M_D + M_L$	=	78,489.0	lb-ft
	a_{max}	=	12.08	in
	e_t PL2	=	7.94	in

X_1	$M_D + M_L$	=	44,562.0	lb-ft
	a_{max}	=	6.86	in
	e_t X1	=	2.72	in

X_2	$M_D + M_L$	=	33,062.2	lb-ft
	a_{max}	=	5.09	in
	e_t X2	=	0.95	in

ends	$M_D + M_L$	=	0.0	lb-ft
	a_{max}	=	0.00	in
	e_t ends	=	-4.13	in

NAL "e" ENVELOPE - FROM ZERO TO MAX TENSILE STRESS AT TOP/BOTTOM OF SECTION

nc. tensile stress limit at Service	f_t	=	835.0	psi
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	k_t	=	4.13	in
ditional eccentricity at bottom @ ends	$e'_{b \text{ ends}}$	=	3.17	in
ditional eccentricity at bottom @ all other locations	$e'_{b \text{ else}}$	=	3.17	in
ditional eccentricity at top - All Locations	e'_t	=	3.7	in

Lower cgs Envelope.

$$e_b = (a_{min} + k_b)$$

$$k_b = \frac{r^2}{c_t} \quad a_{min} = \frac{M_D}{P_i}$$

This bottom eccentricity is calculated based on ZERO tension stress in the top of concrete

Upper cgs Envelope.

$$e_t = (a_{max} - k_t)$$

$$k_t = \frac{r^2}{c_b} \quad a_{max} = \frac{M_T}{P_e}$$

This top eccentricity is calculated based on ZERO tension stress in the bottom of concrete

Table 24.5.3.2—Concrete tensile stress limits immediately after transfer of prestress, without additional bonded reinforcement in tension zone

Location	Concrete tensile stress limits
Ends of simply-supported members	$6\sqrt{f'_c}$
All other locations	$3\sqrt{f'_c}$

Table 24.5.2.1—Classification of prestressed flexural members based on f_t

Assumed behavior	Class	Limit
Uncracked	$I^{(1)}$	$f_t \leq 7$
Transition between uncracked and cracked	T	$7.5\sqrt{f'_c} <$
Cracked	C	$f_t > 1$

⁽¹⁾Prestressed two-way slabs shall be designed as Class U with $f_t \leq 6$

additional eccentricity e'_b, e'_t

$$e'_b = \frac{f_t^{(0)} A_c k_b}{P_i} \leftarrow \text{Bottom ecc } P_i \text{ controls}$$

This bottom envelope eccentricity is calculated assuming allowable tension stress in the top of concrete at initial st (COMPOSITE ACTION)

$$e'_t = \frac{f_{(b)} A_c k_t}{P_e} \leftarrow \text{Top ecc } P_e \text{ controls}$$

Additional top eccentricity is calculated assuming the maxim tension stress in the bottom of concrete at final stage $f_{(b)} = f$ (COMPOSITE ACTION)

SUMMARY

Lower Envelope (Initial stress)

At mid span	$e_{b \text{ mid span}} + e'_{b \text{ else}}$	=	7.9	in	>	7.11	in	OK
AT Point Load 1	$e_{b \text{ PL1}} + e'_{b \text{ else}}$	=	7.9	in	>	7.11	in	OK
AT Point Load 2	$e_{b \text{ PL2}} + e'_{b \text{ else}}$	=	7.8	in	>	7.11	in	OK
AT X ₁	$e_{b \text{ X1}} + e'_{b \text{ else}}$	=	7.8	in	>	7.11	in	OK
AT X ₂	$e_{b \text{ X2}} + e'_{b \text{ else}}$	=	7.6	in	>	7.11	in	
At ends	$e_{b \text{ ends}} + e'_{b \text{ ends}}$	=	7.3	in	>	7.11	in	

Eccentricity Provided

NOTE: (+) Positive eccentricity "e" is downward ↓ & (-) Negative eccentricity "e" is upward ↑

Upper Envelope - Composite (Service)

At mid span	$e_{t \text{ mid span}} + e'_{t \text{ else}}$	=	4.36	in	<	7.11	in	OK
AT Point Load 1	$e_{t \text{ PL1}} + e'_{t \text{ else}}$	=	5.50	in	<	7.11	in	OK
AT Point Load 2	$e_{t \text{ PL2}} + e'_{t \text{ else}}$	=	4.25	in	<	7.11	in	OK
AT X ₁	$e_{t \text{ X1}} + e'_{t \text{ else}}$	=	-0.97	in	<	7.11	in	OK
AT X ₂	$e_{t \text{ X2}} + e'_{t \text{ else}}$	=	-2.74	in	<	7.11	in	OK
At ends	$e_{t \text{ ends}} + e'_{t \text{ ends}}$	=	-7.8	in	<	7.11	in	OK

CAPACITY OF SECTION

[A] CALCULATION OF f_{ps}: STRAIN COMPATIBILITY METHOD

f _{pe}	=	172,518.9	psi
E _{ps}	=	28,800.0	ksi
E _c	=	4,844,512	psi
P _e	=	77,995.8	lb
A _c	=	88.5	in ²
e _{mid span}	=	7.11	in
r ²	=	37.6	in ²

b _{top Flange}	=	7.1	in
h _{beam flange avg}	=	3.5	in
b _w	=	3.0	in
d _p = h - Y _{ps}	=	16.13	in

Effective depth of PS steel

Area of PT strands
Area of Non-PS steel in tension
Area of Non-PS steel in Compression

A _{ps}	=	0.45	in ²
A _s	=	0.00	in ²
A' _s	=	0.60	in ²
f _y	=	60,000.00	psi
f' _c	=	9,482.20	psi
β ₁	=	0.65	
f _{decomp}	=	2,063.4	psi

Assume & Iterate

PS at Nominal Capacity

f_{ps} = 267,714 psi iterate

DETERMINE THE 3 STRAINS

Strain at effective stress of PS steel (after losses)
Strain at concrete due to decompression

ε ₁	=	0.0060	in/in
ε ₂	=	0.0004	in/in

Depth of compressive block - Rectangular Behavior
Depth of compressive block - T-Behavior

a	=	1.5	in
a	=	-1.2	in

NA within top flange, Rectangular Behavior

Use a = 1.5 in

Strain at concrete due to overload stress

ε ₃	=	0.0180	in/in
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Total strain at PS steel

ε ₁ + ε ₂ + ε ₃	=	0.0245	in/in
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f_{ps} = 267,714 psi OK

$$f_{decomp} = \frac{P_e}{A_c} \left(1 + \frac{e^2}{r^2} \right)$$

$$e_1 = \frac{f_{pe}}{E_{ps}}$$

$$e_2 = \frac{f_{decomp}}{E_c}$$

Rectangular Behavior

$$a = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f_y}{0.85 f'_c b}$$

T-behavior

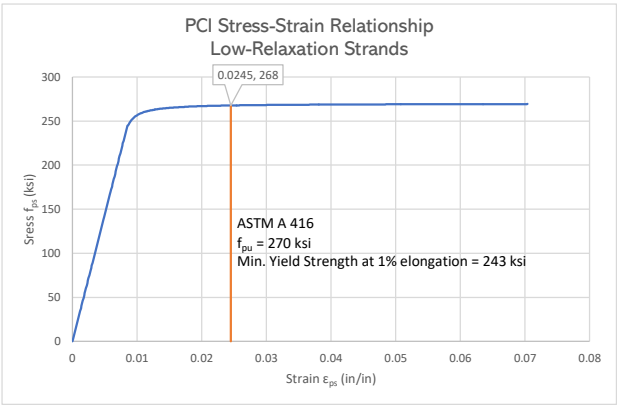
$$a = \frac{A_{ps} f_{ps} + A_s f_y - A'_s f_y - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c b_w}$$

Strain at concrete due to overload stress, ε₃

For All Beams

Net tensile strain: α = β₁ c

$$\epsilon_t = \frac{0.003(d_t - c)}{c} = \frac{0.003(\beta_1 d_t - a)}{a}$$



Note: Approximate strain at rupture is 0.05 to 0.07 in./in. Minimum elongation at the ultimate strength ≈ 3.5%.

These curves can be approximated by the following equations:

270 ksi strand:

$$\epsilon_{ps} \leq 0.0085: f_{ps} = 28,800 \epsilon_{ps} \text{ (ksi)}$$

$$\epsilon_{ps} > 0.0085: f_{ps} = 270 - \frac{0.04}{\epsilon_{ps} - 0.007} \text{ (ksi)}$$

E CALCULATION OF fps: APPROXIMATE METHOD (BONDED PRESTRESSED REINFORCEMENT)

CRITICAL SECTION)

ultimate strength	f_{pu}	=	270,000.0	psi
relaxation (ACI 318-19, Table 20.3.2.3.1)	γ_p	=	0.28	Low Relaxation
compressive strength	f'_c	=	9,482.2	psi
equivalent rectangular block to depth of NA	β_1	=	0.65	ACI 318-19, table 22.2.4.3
depth of non-prestressed rebar in Tension	d	=	15.75	in
depth of PS steel	$d_p = h - \gamma_{ps}$	=	16.13	in
	bd	=	111.23	in ² (Calculated Graphically)
	bd_p	=	113.88	in ² (Calculated Graphically)
flange width	$b_{top\ Flange}$	=	7.1	in
flange strength	f_v	=	60,000	psi
flange reinforcement	A_{ps}	=	0.45	in ²
PS steel in tension	A_s	=	0.00	in ²
PS steel in Compression	A'_s	=	0.60	in ²

cement ratio (As in tension)	ρ	=	0.0000
cement ratio (A's in Compression)	ρ'	=	0.0054
reinforcement ratio (Aps in tension)	ρ_p	=	0.0040
on 10.3.2.3.1 (b) > 0.17 if compression reinforcement is used		=	0.08 Use 0.17

Ultimate Nominal Capacity	f_{ps}	=	250,228	psi
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COMPATIBILITY METHOD	f_{ps}	=	267,714	psi
ITERATIVE METHOD	f_{ps}	=	250,228	psi
Use	f_{ps}	=	267,714	psi

FALLS IN THE TOP FLANGE (RECTANGULAR or "T" BEHAVIOR)

$b_{Top\ flange}$	=	7.06	in
a	=	1.49	in
$h_{beam\ flange\ avg}$	=	3.5	in
NA within top flange, Rectangular Behavior			

$$\frac{+ A_s f_y - A'_s f_y}{0.85 f'_c b}$$

$$\frac{+ A_s f_y - A'_s f_y - 0.85 f'_c (b - b_w) h_f}{0.85 f'_c b_w}$$

Use, a	=	1.5	in
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UNDER-REINFORCED (DUTILE BEHAVIOR) WHEN $\epsilon_{steel} > 0.005$

PS steel strain ϵ_{steel}	=	0.018	Tension Controlled Section
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bMn (AT point load 1)

Nominal Capacity	M_n	=	152,861	lb-ft
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ALTERNATIVE METHOD FOR THE CALCULATION OF "fps" USED TO ESTIMATE SEC

PRE-TENSIONED CASE

20.3.2.3 Stress in bonded prestressed reinforcement at nominal flexural strength, f_{ps}

Pre-Tensioned

20.3.2.3.1 As an alternative to a more accurate calculation of f_{ps} based on strain compatibility, values of f_{ps} calculated in accordance with Eq. (20.3.2.3.1) shall be permitted for members with bonded prestressed reinforcement if all prestressed reinforcement is in the tension zone and $f_{se} \geq 0.5 f_{pu}$.

$$f_{ps} = f_{pu} \left\{ 1 - \frac{\gamma_p}{\beta_1} \left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} \frac{f'_c}{f'_c} (\rho - \rho') \right] \right\} \quad (20.3.2.3.1)$$

where γ_p is in accordance with Table 20.3.2.3.1.

If compression reinforcement is considered for the calculation of f_{ps} by Eq. (20.3.2.3.1), (a) and (b) shall be satisfied.

(a) If d' exceeds $0.15 d_p$, the compression reinforcement shall be neglected in Eq. (20.3.2.3.1).

(b) If compression reinforcement is included in Eq. (20.3.2.3.1), the term

$$\left[\rho_p \frac{f_{pu}}{f'_c} + \frac{d}{d_p} \frac{f'_c}{f'_c} (\rho - \rho') \right]$$

shall not be taken less than 0.17.

Table 20.3.2.3.1—Values of γ_p for use in Eq. (20.3.2.3.1)

f_{pu}/f'_c	γ_p
≥ 0.80	0.55
≥ 0.85	0.40
≥ 0.90	0.28

20.3.2.3.2 For pretensioned strands, the strand design stress at sections of members located within l_d from the free end of strand shall not exceed that calculated in accordance with 25.4.8.3.

Table R20.3.2.3.1—Ratio of f_{ps}/f_{pu} associated with reinforcement type

Prestressing reinforcement type	f_{ps}/f_{pu}
High-strength prestressing bars (Plain)	≥ 0.85
High-strength prestressing bars (Deformed)	≥ 0.80
Stress-relieved strand and wire	≥ 0.85
Low-relaxation strand and wire	≥ 0.90

POST-TENSIONING

20.3.2.4 Stress in unbonded prestressing reinforcement at nominal flexural strength, f_{ps}

20.3.2.4.1 As an alternative to a calculation of f_{ps} , values of f_{ps} calculated in accordance with Eq. (20.3.2.4.1) shall be permitted for members with unbonded tendons if $f_{se} \geq 0.5 f_{pu}$.

Table 20.3.2.4.1—Approximate nominal flexural strength for u

l_d/h	f_{ps}
≤ 35	The least of: $f_{se} + 11$ f_{se} $f_{se} + 11$
> 35	The least of: f_{se} $f_{se} + 11$

f_{ps} = stress in prestressed reinforcement at nominal strength, psi

f_{se} = effective stress in prestressing reinforcement at nominal strength, psi

" f_{se} " ---> ACI 318-19, f_{pe} ---> text

f_{py} = specified yield strength of reinforcement, psi

β_1 = factor relating depth of equivalent rectangular stress block to depth of beam

Table 22.2.2.4.3—Values of rectangular concrete stress

f'_c , psi	
$2500 \leq f'_c < 4000$	
$4000 < f'_c < 8000$	0.85
$f'_c \geq 8000$	

$$\left(\frac{a}{2}\right) + A_s f_y (d - d_p) + 0.85 f'_c (b - b_w) h_f \left(d_p - \frac{h_f}{2}\right)$$

$$\begin{aligned} A_{pw} &= 0.14 \text{ in}^2 \\ M_n &= 226,887.6 \text{ lb-ft} \end{aligned}$$

NA within top flange, Rectangular Behavior			
	M_n	=	152,861 lb-ft
ad Associated			
32 kips	Demand Min	M_U	= 136,113.9 lb-ft
40 kips	Demand Max	M_U	= 169,114 lb-ft

KING MOMENT CAPACITY

ure

$$\overline{f'_c}$$

nt:

$$S_b + P_e \left(e + \frac{r^2}{c_b} \right)$$

not crack at total service load

$$\begin{aligned} \lambda &= 1.0 && \text{Normal Weight Concrete} \\ f_r &= 835 \text{ psi} \\ S_b &= 365.9 \text{ in}^3 \\ e &= 7.11 \text{ in} \\ r^2 &= 37.6 \text{ in}^2 \\ c_b &= 9.1 \text{ in} \\ M_{cr} &= 98,517 \text{ lb-ft} \\ 2 * P_L &= 20 \text{ kips} \end{aligned}$$

NCHORAGE STEEL (Closed Stirrups)

in-PT Steel stress for cracking control
e (no losses)
steel stress (after losses)
s at Nominal Capacity
l
gth

Length (Tansfer length + Flexural Bond)

ps
losed stirrups
irrupps

$$\begin{aligned} \text{Say use \#} &= 3 && \text{bars} \\ A_{bar} &= 0.11 \text{ in}^2 \\ f_s &= 20,000.0 \text{ psi} \\ P_t = f_{pi} * A_{ps} &= 91,800.00 \text{ lb} \\ f_{pe} &= 172,518.9 \text{ psi} \\ f_{ps} &= 250,228 \text{ psi} \\ h &= 18.1 \text{ in} \\ l_t &= 21.56 \text{ in} \\ l_d &= 50.71 \text{ in} \\ l_d &= 4.2 \text{ ft} \\ \text{Use } l_d &= 5.0 \text{ ft} \\ A_t &= 0.08 \text{ in}^2 \\ 2 * A_{bar} &= 0.2 \text{ in}^2 \\ N &= 1.0 && \text{Each end} \end{aligned}$$

To be added to the Shear rebar

ee #3 ties to provide the envelope for all the main longitudinal reinforcement.
he tendons with helical steel wire through the development length, l_n in order
l transfer.

PS rebar should be placed as closest as possible to the tension fiber. That way those bars will "activate" after decompression stage, when there is a significant increase of strain in the tension side.
cover should be controlled during the manufacturing process.
nforcement will be added

Ultimate Load at Failure		
X=	11.5	ft
M_D =	4,114	lb-ft
M_L =	148,747	lb-ft
P_U =	18.03	kips
$2P_U$ =	36.06	kips

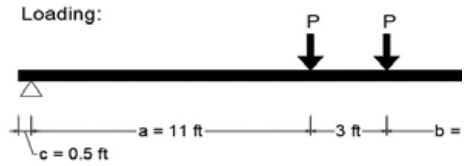
OK

0 iterate to zero

Cracking Load		
X=	11.5	ft
M_D =	4,114	lb-ft
M_L =	94,403	lb-ft
P_{cr} =	11.4	kips
$2P_{cr}$ =	22.9	kips

OK

0 iterate to zero



Development Length:

$$\text{Min } l_d = \frac{1}{1,000} \left(f_{ps} - \frac{2}{3} f_{pe} \right) d_b$$

Area of stirrups

$$A_t = 0.021 \frac{P_i h}{f_s l_t}$$

$$l_t = \frac{f_{pe}}{3000} d_b$$

$f_s = f_y$, but is taken as
 $f_s = 20,000 \text{ psi}$ for cracking control

ES CALCULATIONS

Width	7.0625 in	Bottom Flange	0	3.125
Thickness	3 in		0	0
Flange Height	1 in		7.0625	0
			7.0625	3.125
Flange Width	3 in	Bottom Transition	0	3.125
Flange Height	10 in		2.03125	4.125
			7.0625	3.125
			5.03125	4.125
Web Thickness	7.0625 in	Web	2.03125	4.125
Web Height	3.125 in		2.03125	14.125
			5.03125	4.125
			5.03125	14.125
	18.125 in	Top Transition	2.03125	14.125
			0	15.125
			5.03125	14.125
			7.0625	15.125
Top Flange Ave, Thickness	3.5 in	Top Flange	0	15.125
Bottom Flange Ave, Thickness	3.6 in		0	18.125
			7.0625	18.125
			7.0625	15.125
Volume	1749.727 ft ³			
Area	1342 ft ²			
Perimeter	1.30 ft			

ON

	Area	Dist. from top to centroid	A*d _t	Inertia at centroid	Distance NA to centroid	Inertia at NA
	A (in ²)	d _t (in)	A*d _t (in ³)	I _c (in ³)	d (in)	I _c + A*d ² (in ³)
Flange	21.1875	1.5	31.8	16	7.6	1,238.4
Transition	5.03125	3.4	17.3	0	5.7	161.8
	30	9	270.0	250	0.1	250.3
Transition	5.03125	14.6	73.3	0	5.5	151.0
Flange	22.07031	16.5625	365.5	18	7.5	1,248.3
	83.32031		757.9			3,050

ED SECTION - INITIAL STAGE

Non-PS Modular Ratio at Initial Stage $n_t = E_s/E_{ci} = 6.18$
PS Modular Ratio at Initial Stage $n_{tp} = E_{sp}/E_{ci} = 6.14$

	Steel Reinf. A_{steel} (in ²)	Transfo. A. Steel Ratio (n-1)	Concrete Area A (in ²)	Dist. from top to centroid d _t (in)	A*d _t (in ³)	Inertia at centroid I _c (in ³)	Distance NA to centroid d (in)	Inertia at NA I _c + A*d ² (in ³)
Flange			21.1875	1.5	31.8	16	7.5	1,212.1
Transition			5.03125	3.4	17.3	0	5.6	157.1
			30	9	270.0	250	0.0	250.0
Transition			5.03125	14.6	73.3	0	5.6	155.6
Flange			22.07031	16.5625	365.5	18	7.5	1,275.6
Steel, A _s	0.60	5.18	3.1	1.50	4.6	0	7.5	176
Steel, A _s	0.00	5.18	0.0	15.75	0.0	0	6.7	0
	0.45	5.14	2.3	16.13	37.5	0	7.1	118
			88.8		800.0			3,344

ED SECTION - FINAL STAGE

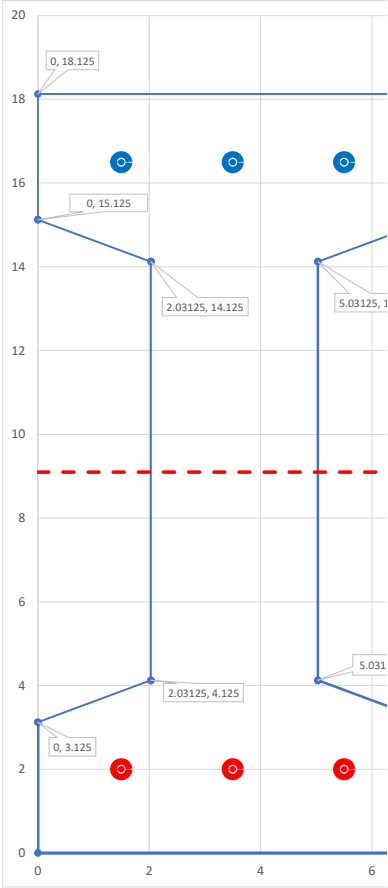
Non-PS Modular Ratio at Final Stage $n_t = E_s/E_{ci} = 5.99$
PS Modular Ratio at Final Stage $n_{tp} = E_{sp}/E_{ci} = 5.94$

	Steel Reinf. A_{steel} (in ²)	Transfo. A. Steel Ratio (n-1)	Concrete Area A (in ²)	Dist. from top to centroid d _t (in)	A*d _t (in ³)	Inertia at centroid I _c (in ³)	Distance NA to centroid d (in)	Inertia at NA I _c + A*d ² (in ³)
Flange			21.1875	1.5	31.8	16	7.5	1,213.0
Transition			5.03125	3.4	17.3	0.4	5.6	157.3
			30	9	270.0	250	0.0	250.0
Transition			5.03125	14.6	73.3	0.4	5.6	155.4
Flange			22.07031	16.5625	365.5	18	7.5	1,274.6
Steel, A _s	0.60	4.99	3.0	1.50	4.5	0	7.5	169
Steel, A _s	0.00	4.99	0.0	15.75	0.0	0	6.7	0
	0.45	4.94	2.2	16.13	36.0	0	7.1	113
			88.5		798.4			3,333

NA axis	
0	9.10
7.0625	9.10
Act =	41.81

$r^2 = 37 \text{ in}^2$
 $C^t = 9.10 \text{ in}$
 $C_b = 9.03 \text{ in}$
 $S^t = 335 \text{ in}^3$
 $S_b = 338 \text{ in}^3$

$r^2 = 38 \text{ in}^2$
 $C^t = 9.01 \text{ in}$
 $C_b = 9.11 \text{ in}$
 $S^t = 371 \text{ in}^3$
 $S_b = 367 \text{ in}^3$



Casting:	5/8/2021	Beam Tested on:	6/28/2021	Age:	51	days
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ENING LOSSES

$f_{pES} = n f_{cs}$	$f_{cs} = -\frac{P_i}{A_c} \left(1 + \frac{e^2}{r^2} \right) + \frac{M_{DE}}{I_c}$
of inertia	$A_c = 88.8 \text{ in}^2$
radius of gyration	$I_g = 3,343.7 \text{ in}^4$
d moment at Midspan	$r^2 = I_c/A_c = 37.7 \text{ in}^2$
axial force	$M_D = 4,156 \text{ lb-ft}$
tendon eccentricity	$P_i = A_s f_{pi} = 91,800.0 \text{ lb}$
crete stress at steel centroid	$e_c = 7.11 \text{ in}$
ular Ratio	$f_{cs} = -2,074.39 \text{ psi}$
ortening PS loss	$n = E_s/E_{ci} = 6.14$
	$\Delta f_{pES} = n f_{cs} = -12,737 \text{ psi}$

From PCI handbook:

$$f_{cir} = K_{cir} \left(\frac{P_i}{A_g} + \frac{P_i e^2}{I_g} \right) - \frac{M_{ge}}{I_g}$$

where:

- $K_{cir} = 0.9$ for pretensioned components
- P_i = initial prestress force (jacking force after anchorage loss), lb
- e = eccentricity of center of gravity of tendons with respect to center of gravity of concrete at the cross section considered, in.
- A_g = area of gross concrete section at the cross section considered, in.²
- I_g = moment of inertia of gross concrete section at the cross section considered, in.⁴
- M_{ge} = bending moment due to dead weight of prestressed component and any other permanent loads in place at time of prestressing, in./lb

$C_u = \frac{\epsilon_{CR}}{\epsilon_{EL}}$	$C_t = \frac{t^{0.60}}{10 + t^{0.60}} C_u$	$\Delta f_{PCR} = C_t \frac{E_{ps}}{E_c} f_{cs}$
creep coefficient for standard conditions	$C_u = 2.00$	
n factor for moist-cured at 7 days or more	$K_a = 0.8$	
humidity	$RH = 75.0 \%$	assumed
n factor for humidity greater than 40%	$K_{ct} = 0.8$	
creep	$C_u = 1.21$	
ween casting and testing	$t_1 = 51.0 \text{ days}$	
efficient	$C_{t1} = 0.62$	
ular Ratio	$n = E_{ps}/E_c = 5.94$	
ep PS loss	$\Delta f_{PCR} = -7,649 \text{ psi}$	

SES

Shrinkage Strain for standard conditions	$\epsilon_{SH,u} = 7.8E-04 \text{ in/in}$	ACI 209R
ween casting and testing	$t_1 = 51.0 \text{ days}$	
e Strain at time t_1 - Standard conditions	$\epsilon_{SH,t} = 4.3E-04 \text{ in/in}$	moist cured assumed
humidity	$RH = 75.0 \%$	assumed
n factor for humidity greater than 40%	$K_{SH} = 0.7$	
e at time t_1	$\epsilon_{SH,t} = 2.8E-04 \text{ in/in}$	
modulus of elasticity	$E_{ps} = 28,800.0 \text{ ksi}$	

rinkage PS loss	$\Delta f_{pSH} = -8,133 \text{ psi}$
-----------------	---------------------------------------

ATION LOSSES (stress relieved stands)

$$\Delta f_{pR} = f'_{pi} \frac{\log t}{10} \left(\frac{f'_{pi}}{f_{py}} - 0.55 \right) \quad f'_{pi} = f_{pi} + \Delta f_{pSH} + \Delta f_{PCR} + \Delta f_{pES}$$

For low relaxation steel, the divider is 45 instead of 10

stress for calculating Relaxation	$f'_{pi} = 174,534 \text{ psi}$	
irs	$t = 1,224 \text{ hrs}$	51.0 days
strength (Based on ACI 318, table R20.3.2.3.1)	$f_{py} = 0.90 * f_{pu} = 243,000 \text{ psi}$	(Low Relaxation)
axation Loss	$\Delta f_{pR} = -2,014.9 \text{ psi}$	

2-BY-STEP METHOD

6. Loss Ratio γ

	51 days		PS transfer tensile strength limit	f_{pi}	=	203,052	psi
ening	-12,737	6%	Total Losses	Total Loss =		-30,534	psi
ion	-2,015	1%	Effective PS tendon (after losses)	$f_{pe} = f_{pi} - \text{Total Loss}$	=	172,519	psi
ep	-7,649	4%		Calculated Loss % =		15.0%	
rinkage	-8,133	4%		$\gamma = f_{pe}/f_{pi}$	=	0.85	
TOTAL	-30,534	15%					

When conditions are not standard, creep correction factors have to be applied to Equations 2.10 or 2.11 as follows:

(a) For moist-cured concrete loaded at an age of 7 days or more,

$$k_a = 1.25t^{-0.118} \quad (2.12)$$

(b) For steam-cured concrete loaded at an age of 1 to 3 days or more,

$$k_a = 1.13t^{-0.095} \quad (2.13)$$

For greater than 40 percent relative humidity, a further multiplier correction factor of

$$k_{ci} = 1.27 - 0.0067H \quad (2.14)$$

Branson (Ref. 2.18) recommends the following relationships for the shrinkage strain as a function of time for standard conditions of humidity ($H \geq 40$ percent):

(a) For moist-cured concrete any time t after 7 days,

$$\epsilon_{SH,t} = \frac{t}{35 + t} (\epsilon_{SH,u}) \quad (2.15)$$

where $\epsilon_{SH,u} = 800 \times 10^{-6} \text{ in./in.}$ if local data are not available.

(b) For steam-cured concrete after the age of 1 to 3 days,

$$\epsilon_{SH,t} = \frac{t}{55 + t} \epsilon_{SH,u} \quad (2.16)$$

For other than standard humidity, a correction factor has to be applied to Equations 2.15 and 2.16 as follows:

(a) For $40 < H \leq 80$ percent,

$$k_{SH} = 1.40 - 0.010H \quad (2.17a)$$

(b) For $80 < H \leq 100$ percent,

$$k_{SH} = 3.00 - 0.030H \quad (2.17b)$$

APPENDIX B

Deflection Calculation

DEFLECTION

Moment-Curvature Method

Step 1: Find curvature due to effective PS tendons only:

TRANSFORMED SECTION - SERVICE STAGE (See section Properties Calculation)

Area	A =	88.5	in²
Squared radius of gyration	r² =	37.6	in²
Distance from CG to top fiber	C _t =	9.0	in
Distance from CG to btm fiber	C _{bf} =	9.1	in
Top fiber section modulus	S _t =	369.6	in³
Btm fiber section modulus	S _{bf} =	365.9	in³
Beam height	h =	18.1	in
Modulus of Elasticity at Testing	E _c =	4,844,512	psi
Constant Eccentricity	e _t = C _{bf} - Y _{ps} =	7.11	in

$$f^t = -\frac{P_i}{A_c}\left(1 - \frac{ec_t}{r^2}\right) - \frac{M_D}{S^t} \leq f_{ti}$$

$$f_b = -\frac{P_i}{A_c}\left(1 + \frac{ec_b}{r^2}\right) + \frac{M_D}{S_b} \leq f_{ci}$$

AT Point Load 1 (Max Moment)

Moment	M =	0.0	lb-ft
Effective PS force	P _e =	77,995.8	lb
Stress at top of beam	f ^t =	619	psi
Stress at Bottom of beam	f _b =	-2,396	psi
Strain at top of beam	ε ^t =	0.0001278	
Strain at bottom of beam	ε _b =	-0.0004946	
Curvature due to PS load	ϕ ₁ =	-0.0000343	1/in

Calculated in flexure design, step 6
Tension
Compression

$$\epsilon^t = \frac{f^t}{E_c}$$
$$\epsilon_b = \frac{f_b}{E_c}$$

Step 2: Find curvature due to effective PS tendons and Cracking Moment:

AT Point Load 1 (Max Moment)

Cracking Moment	M _{cr} =	98,517.1	lb-ft
Effective PS force	P _e =	77,995.8	lb
Stress at top of beam	f ^t =	-2,579	psi
Stress at Bottom of beam	f _b =	835	psi
Strain at top of beam	ε ^t =	-0.0005324	
Strain at bottom of beam	ε _b =	0.0001724	
Curvature at Cracking Moment	ϕ _{cr} =	0.0000389	1/in

Calculated in flexure design, step 13
Calculated in flexure design, step 6
Compression
Tension

Step 3: Find curvature due to effective PS tendons and ultimate load (nominal capacity):

AT Point Load 1 (Max Moment)

Ultimate Moment (nominal moment)	M _n =	152,861.2	lb-ft
Ultimate strain of concrete in compression	ε _{cu} =	0.003	
Depth of compression block	a =	1.5	in
Ratio depth equivalent rectangular block to depth of N	β ₁ =	0.65	
Distance to neutral axis	c =	2.30	in
Curvature at Ultimate Moment	ϕ _u =	0.0013054	1/in

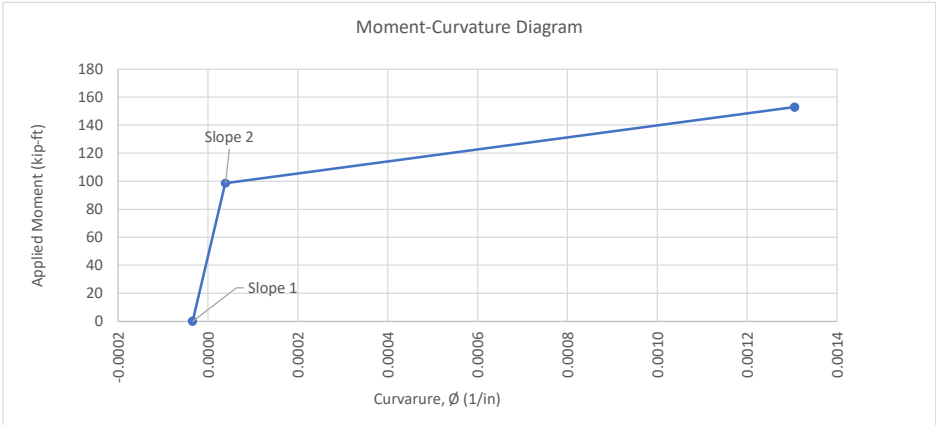
Calculated in flexure design, step 12
Top fiber
Calculated in flexure design, step 10
Calculated in flexure design, step 9

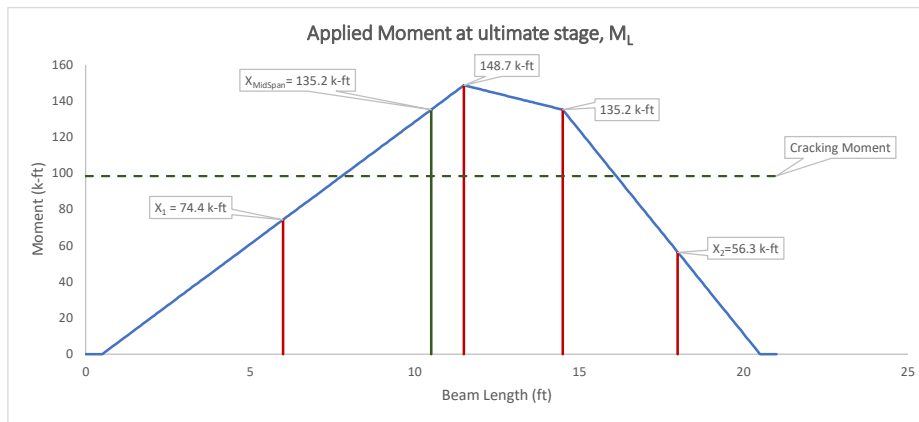
$$c = \frac{a}{\beta_1}$$
$$\phi_u = \frac{\epsilon_{cu}}{c}$$

Step 4: Moment Curvature Summary

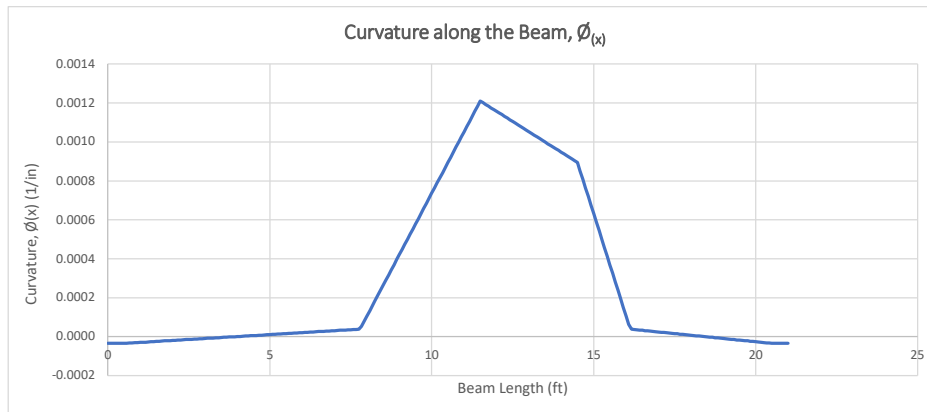
	ϕ (1/in)	Applied Moment (lb-ft)
Due to PS load	-0.000034	0.0
At Cracking Moment	0.000039	98,517.1
At Ultimate Moment	0.001305	152,861.2
Slope 1, before cracking	6.19E-11	1/lb-in²
Slope 2, after cracking	1.94E-09	1/lb-in²

Criteria used to calculate curvature along the span:
If Applied moment it's less than Cracking moment, interpolate with slope 1
If Applied moment it's greater than Cracking moment, interpolate with slope 2





See calculation on next page:



Step 4: Deflection by Unit Load Method (see next page for complete calculation)

The displacement of any point on the beam caused by external loads is found by applying a unit load at that point that corresponds to the desired displacement. In this case, the displacement will be calculated at midspan.

$$\Delta = \int_0^L m(x) \frac{M(x)}{EI} dx \quad \phi(x) = \int_0^L \frac{M(x)}{EI} dx \quad \Delta = \int_0^L m(x) \phi(x) dx$$

Where:

- Δ : displacement at point of application of unit load (+ in direction of unit load)
- m_l moment equation in member caused by the unit load
- M_l moment equation in member caused by loads applied to the beam
- L = length of member
- I = moment of inertia of member
- ϕ_0 Curvature along the beam

if dx is small enough, Integration can be obtained by:

$$\Delta = \sum_x^L (\bar{m}(x) \bar{\phi}(x) dx)$$

where:

$$dx = 1 \text{ in}$$

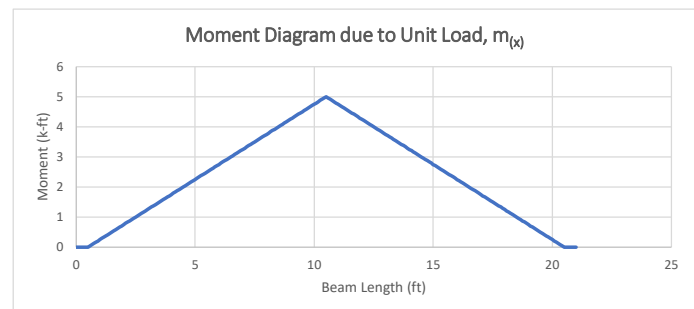
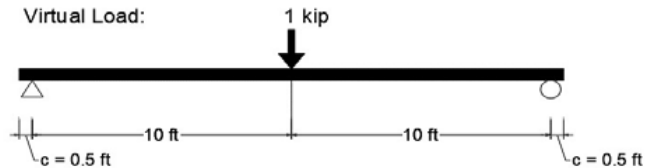
$$\bar{m}(x) = \frac{(m(x) + m(x+dx))}{2}$$

$$\bar{\phi}(x) = \frac{(\phi(x) + \phi(x+dx))}{2}$$

Calculated Deflection due to Live Load:

$\Delta_L = 3.60 \text{ in}$ (see next page)

Virtual Load:



Calculation of Deflection using Moment - Curvature and Unitary Load Method

Dead Load

Overhanging distance =	0.5 ft
Distance between supports =	20 ft
Total Beam length =	21 ft
Ay =	875 lb
By =	875 lb
W ₁	144 lb/ft
W ₀	83 lb/ft

Live Load, P

P ₁ =	18,030 lb
a =	11 ft
b =	6 ft
c ₁ =	0.5 ft
d ₁ =	3 ft
clear span L =	20 ft
Reaction A ₁ =	13,523 lb
Reaction B ₁ =	22,538 lb

Calculated in flexure design, step 12

Formulas:

$$\Delta = \int_0^L m(x) \frac{M(x)}{EI} dx \qquad \phi(x) = \int_0^L \frac{M(x)}{EI} dx \qquad \Delta = \int_0^L m(x) \phi(x) dx$$

$$\Delta = \sum_x^{L_1} (\bar{m}(x) \bar{\phi}(x) dx)$$

$$dx = 1 \text{ in}$$

$$\bar{m}(x) = \frac{(m(x) + m(x+dx))}{2}$$

$$\bar{\phi}(x) = \frac{(\phi(x) + \phi(x+dx))}{2}$$

Δ = displacement at point of application of unit load (+ in direction of unit load)
m_u = moment equation in member caused by the unit load
M_u = moment equation in member caused by loads applied to the beam
L = length of member
I = moment of inertia of member
φ_u = Curvature along the beam

Moment - Curvature Data:

Curvature Due to PS load	φ ₁ =	-0.000034	1/in
At Cracking Moment	φ _c =	0.000039	1/in
Curvature at Ultimate Moment	φ _u =	0.000039	1/in
Applied Moment at PS	M ₁ =	0.00	lb-ft
Cracking Moment	M _c =	98,517.1	lb-ft
Ultimate Moment (nominal moment)	M _u =	152,861.2	lb-ft
Slope 1, before cracking		6.19E-11	1/lb-in ²
Slope 2, after cracking		1.94E-09	1/lb-in ²
Interpolated Slope 1, when applied moment < M ₁		-0.000034	1/in
Interpolated Slope 2, when applied moment < Cracking Moment		-0.002257	1/in
Criteria used to calculate curvature along the span: If Applied moment it's less than Cracking moment, interpolate with slope 1 If Applied moment it's greater than Cracking moment, interpolate with slope 2			

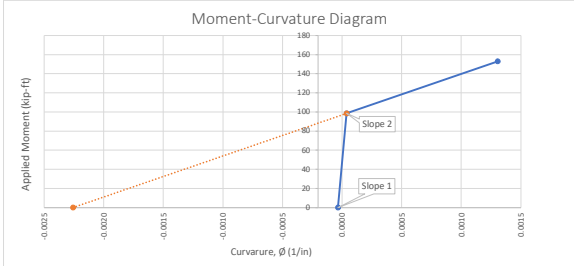
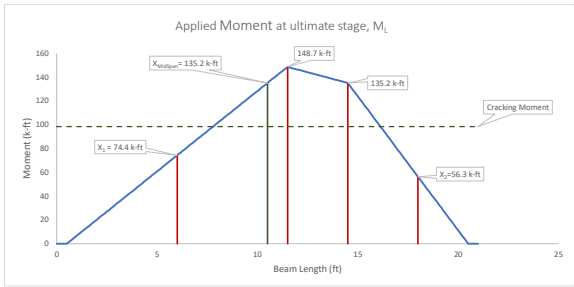
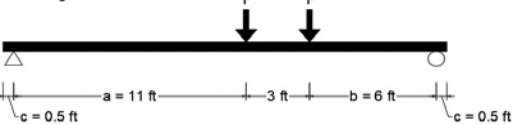
Calculated in flexure design, step 13
Calculated in flexure design, step 12

Virtual Load	1.0
Distance from support to virtual load	10.0 ft
Reaction at supports	0.5

$$\Delta = \sum_x^L (\bar{m}(x) \bar{\phi}(x) dx) = 3.60 \text{ in}$$

Distance from Left End				Applied Moment		Curvature φ (1/in)	m(x) (1-ft)	φ̄(x) + dx (in/in)	m̄(x) (1-in)	(m̄(x) φ̄(x) dx) (in)
X (ft)	M ₀ (lb-ft)	M ₁ (lb-ft)	M ₂ (lb-ft)							
0	0	0	0	-0.000034	0.00	-3.43411E-05	0	0	0.0000	
0.08	0	0	0	-0.000034	0.00	-3.43411E-05	0	0	0.0000	
0.17	-1	0	0	-0.000034	0.00	-3.43411E-05	0	0	0.0000	
0.25	-3	0	0	-0.000034	0.00	-3.43411E-05	0	0	0.0000	
0.33	-5	0	0	-0.000034	0.00	-3.43411E-05	0	0	0.0000	
0.42	-7	0	0	-0.000034	0.00	-3.43411E-05	0	0	0.0000	
0.50	-10	0	0	-0.000034	0.00	-3.39232E-05	0.25	0.0000		
0.58	59	1,127	1	-0.000034	0.04	-3.30847E-05	0.75	0.0000		
0.67	127	2,254	2	-0.000033	0.08	-3.22471E-05	1.25	0.0000		
0.75	195	3,381	3	-0.000032	0.13	-3.14095E-05	1.75	-0.0001		
0.83	263	4,508	5	-0.000031	0.17	-3.05719E-05	2.25	-0.0001		
0.92	330	5,634	6	-0.000030	0.21	-2.97344E-05	2.75	-0.0001		
1.00	396	6,761	7	-0.000029	0.25	-2.88968E-05	3.25	-0.0001		
1.08	461	7,888	8	-0.000028	0.29	-2.80592E-05	3.75	-0.0001		
1.17	527	9,015	9	-0.000028	0.33	-2.72216E-05	4.25	-0.0001		
1.25	591	10,142	10	-0.000027	0.38	-2.6384E-05	4.75	-0.0001		
1.33	655	11,269	11	-0.000026	0.42	-2.55464E-05	5.25	-0.0001		
1.42	718	12,396	12	-0.000025	0.46	-2.47088E-05	5.75	-0.0001		
1.50	781	13,523	14	-0.000024	0.50	-2.38712E-05	6.25	-0.0001		
1.58	843	14,649	15	-0.000023	0.54	-2.30336E-05	6.75	-0.0002		
1.67	905	15,776	16	-0.000023	0.58	-2.2196E-05	7.25	-0.0002		
1.75	966	16,903	17	-0.000022	0.63	-2.13584E-05	7.75	-0.0002		
1.83	1,026	18,030	18	-0.000021	0.67	-2.05208E-05	8.25	-0.0002		
1.92	1,086	19,157	19	-0.000020	0.71	-1.96832E-05	8.75	-0.0002		
2.00	1,146	20,284	20	-0.000019	0.75	-1.88457E-05	9.25	-0.0002		
2.08	1,204	21,411	21	-0.000018	0.79	-1.80081E-05	9.75	-0.0002		
2.17	1,263	22,538	23	-0.000018	0.83	-1.71705E-05	10.25	-0.0002		
2.25	1,320	23,664	24	-0.000017	0.88	-1.63329E-05	10.75	-0.0002		
2.33	1,377	24,791	25	-0.000016	0.92	-1.54953E-05	11.25	-0.0002		
2.42	1,434	25,918	26	-0.000015	0.96	-1.46577E-05	11.75	-0.0002		
2.50	1,489	27,045	27	-0.000014	1.00	-1.38201E-05	12.25	-0.0002		
2.58	1,545	28,172	28	-0.000013	1.04	-1.29825E-05	12.75	-0.0002		
2.67	1,599	29,299	29	-0.000013	1.08	-1.21449E-05	13.25	-0.0002		
2.75	1,653	30,426	30	-0.000012	1.13	-1.13073E-05	13.75	-0.0002		
2.83	1,707	31,553	32	-0.000011	1.17	-1.04697E-05	14.25	-0.0001		
2.92	1,760	32,679	33	-0.000010	1.21	-9.63213E-06	14.75	-0.0001		
3.00	1,812	33,806	34	-0.000009	1.25	-8.79454E-06	15.25	-0.0001		
3.08	1,864	34,933	35	-0.000008	1.29	-7.95695E-06	15.75	-0.0001		
3.17	1,915	36,060	36	-0.000008	1.33	-7.11936E-06	16.25	-0.0001		
3.25	1,966	37,187	37	-0.000007	1.38	-6.28176E-06	16.75	-0.0001		
3.33	2,016	38,314	38	-0.000006	1.42	-5.44417E-06	17.25	-0.0001		
3.42	2,065	39,441	39	-0.000005	1.46	-4.60658E-06	17.75	-0.0001		
3.50	2,114	40,568	41	-0.000004	1.50	-3.76899E-06	18.25	-0.0001		
3.58	2,163	41,694	42	-0.000003	1.54	-2.93139E-06	18.75	-0.0001		
3.67	2,210	42,821	43	-0.000003	1.58	-2.0938E-06	19.25	0.0000		
3.75	2,257	43,948	44	-0.000002	1.63	-1.25621E-06	19.75	0.0000		
3.83	2,304	45,075	45	-0.000001	1.67	-4.18617E-07	20.25	0.0000		
3.92	2,350	46,202	46	0.000000	1.71	4.18976E-07	20.75	0.0000		
4.00	2,395	47,329	47	0.000001	1.75	1.25657E-06	21.25	0.0000		
4.08	2,440	48,456	48	0.000002	1.79	2.09416E-06	21.75	0.0000		
4.17	2,485	49,583	50	0.000003	1.83	2.93175E-06	22.25	0.0001		
4.25	2,528	50,709	51	0.000003	1.88	3.76935E-06	22.75	0.0001		
4.33	2,571	51,836	52	0.000004	1.92	4.60694E-06	23.25	0.0001		
4.42	2,614	52,963	53	0.000005	1.96	5.44453E-06	23.75	0.0001		
4.50	2,656	54,090	54	0.000006	2.00	6.28212E-06	24.25	0.0002		
4.58	2,697	55,217	55	0.000007	2.04	7.11972E-06	24.75	0.0002		
4.67	2,738	56,344	56	0.000008	2.08	7.95731E-06	25.25	0.0002		
4.75	2,778	57,471	57	0.000008	2.13	8.7949E-06	25.75	0.0002		
4.83	2,818	58,598	59	0.000009	2.17	9.63249E-06	26.25	0.0003		
4.92	2,857	59,724	60	0.000010	2.21	1.04701E-05	26.75	0.0003		
5.00	2,895	60,851	61	0.000011	2.25	1.13077E-05	27.25	0.0003		
5.08	2,933	61,978	62	0.000012	2.29	1.21453E-05	27.75	0.0003		
5.17	2,971	63,105	63	0.000013	2.33	1.29829E-05	28.25	0.0004		
5.25	3,007	64,232	64	0.000013	2.38	1.38205E-05	28.75	0.0004		
5.33	3,044	65,359	65	0.000014	2.42	1.4658E-05	29.25	0.0004		
5.42	3,079	66,486	66	0.000015	2.46	1.54956E-05	29.75	0.0005		
5.50	3,114	67,613	68	0.000016	2.50	1.63332E-05	30.25	0.0005		
5.58	3,149	68,739	69	0.000017	2.54	1.71708E-05	30.75	0.0005		
5.67	3,182	69,866	70	0.000018	2.58	1.80084E-05	31.25	0.0006		
5.75	3,216	70,993	71	0.000018	2.63	1.8846E-05	31.75	0.0006		
5.83	3,248	72,120	72	0.000019	2.67	1.96836E-05	32.25	0.0006		
5.92	3,280	73,247	73	0.000020	2.71	2.05212E-05	32.75	0.0007		
6.00	3,312	74,374	74	0.000021	2.75	2.13588E-05	33.25	0.0007		
6.08	3,343	75,501	76	0.000022	2.79	2.21964E-05	33.75	0.0007		
6.17	3,373	76,627	77	0.000023	2.83	2.3034E-05	34.25	0.0008		
6.25	3,403	77,754	78	0.000023	2.88	2.38716E-05	34.75	0.0008		
6.33	3,432	78,881	79	0.000024	2.92	2.47092E-05	35.25	0.0009		
6.42	3,461	80,008	80	0.000025	2.96	2.55468E-05	35.75	0.0009		
6.50	3,489	81,135	81	0.000026	3.00	2.63843E-05	36.25	0.0010		
6.58	3,517	82,262	82	0.000027	3.04	2.72219E-05	36.75	0.0010		
6.67	3,543	83,389	83	0.000028	3.08	2.80595E-05	37.25	0.0010		
6.75	3,570	84,516	85	0.000028	3.13	2.88971E-05	37.75	0.0011		
6.83	3,596	85,643	86	0.000029	3.17	2.97347E-05	38.25	0.0011		
6.92	3,621	86,769	87	0.000030	3.21	3.05723E-05	38.75	0.0012		
7.00	3,645	87,896	88	0.000031	3.25	3.14099E-05	39.25	0.0012		
7.08	3,669	89,023	89	0.000032	3.29	3.22475E-05	39.75	0.0013		
7.17	3,693	90,150	90	0.000033	3.33	3.30851E-05	40.25	0.0013		
7.25	3,716	91,277	91	0.000034	3.38	3.39227E-05	40.75	0.0014		
7.33	3,738	92,404	92	0.000034	3.42	3.47603E-05	41.25	0.0014		
7.42	3,760	93,531	94	0.000035	3.46	3.55979E-05	41.75	0.0015		
7.50	3,781	94,657	95	0.000036	3.50	3.64355E-05	42.25	0.0015		

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7.58	3.801	95.784	96	0.000037	3.54	3.7273E-05	42.75	0.0016
7.67	3.821	96.911	97	0.000038	3.58	3.81106E-05	43.25	0.0016
7.75	3.841	98.038	98	0.000039	3.63	4.62575E-05	43.75	0.0020
7.83	3.859	99.165	99	0.000054	3.67	6.71163E-05	44.25	0.0030
7.92	3.878	100.292	100	0.000080	3.71	9.23779E-05	44.75	0.0042
8.00	3.895	101.419	101	0.000107	3.75	0.000119639	45.25	0.0054
8.08	3.912	102.546	103	0.000133	3.79	0.000145901	45.75	0.0067
8.17	3.929	103.673	104	0.000159	3.83	0.000172162	46.25	0.0080
8.25	3.945	104.799	105	0.000185	3.87	0.000198424	46.75	0.0093
8.33	3.960	105.926	106	0.000212	3.92	0.000224685	47.25	0.0106
8.42	3.975	107.053	107	0.000238	3.96	0.000250947	47.75	0.0120
8.50	3.989	108.180	108	0.000264	4.00	0.000277209	48.25	0.0134
8.58	4.003	109.307	109	0.000290	4.04	0.00030347	48.75	0.0148
8.67	4.016	110.434	110	0.000317	4.08	0.000329732	49.25	0.0162
8.75	4.028	111.561	112	0.000343	4.12	0.000355993	49.75	0.0177
8.83	4.040	112.688	113	0.000369	4.17	0.000382255	50.25	0.0192
8.92	4.051	113.814	114	0.000395	4.21	0.000408516	50.75	0.0207
9.00	4.062	114.941	115	0.000422	4.25	0.000434778	51.25	0.0223
9.08	4.072	116.068	116	0.000448	4.29	0.000461039	51.75	0.0239
9.17	4.082	117.195	117	0.000474	4.33	0.000487301	52.25	0.0255
9.25	4.091	118.322	118	0.000500	4.38	0.000513562	52.75	0.0271
9.33	4.099	119.449	119	0.000527	4.42	0.000539824	53.25	0.0287
9.42	4.107	120.576	121	0.000553	4.46	0.000566085	53.75	0.0304
9.50	4.114	121.703	122	0.000579	4.50	0.000592347	54.25	0.0321
9.58	4.121	122.829	123	0.000605	4.54	0.000618608	54.75	0.0339
9.67	4.127	123.956	124	0.000632	4.58	0.00064487	55.25	0.0356
9.75	4.132	125.083	125	0.000658	4.63	0.000671131	55.75	0.0374
9.83	4.137	126.210	126	0.000684	4.67	0.000697393	56.25	0.0392
9.92	4.141	127.337	127	0.000711	4.71	0.000723654	56.75	0.0411
10.00	4.145	128.464	128	0.000737	4.75	0.000749916	57.25	0.0429
10.08	4.148	129.591	130	0.000763	4.79	0.000776177	57.75	0.0448
10.17	4.151	130.718	131	0.000789	4.83	0.000802439	58.25	0.0467
10.25	4.153	131.844	132	0.000816	4.88	0.0008287	58.75	0.0487
10.33	4.154	132.971	133	0.000842	4.92	0.000854962	59.25	0.0507
10.42	4.155	134.098	134	0.000868	4.96	0.000881224	59.75	0.0527
10.50	4.156	135.225	135	0.000894	5.00	0.000907485	59.75	0.0542
10.58	4.155	136.352	136	0.000921	4.96	0.000933747	59.25	0.0553
10.67	4.154	137.479	137	0.000947	4.92	0.000960008	58.75	0.0564
10.75	4.153	138.606	139	0.000973	4.88	0.00098627	58.25	0.0575
10.83	4.151	139.733	140	0.000999	4.83	0.001012531	57.75	0.0585
10.92	4.148	140.859	141	0.001026	4.79	0.001038793	57.25	0.0595
11.00	4.145	141.986	142	0.001052	4.75	0.001065054	56.75	0.0604
11.08	4.141	143.113	143	0.001078	4.71	0.001091316	56.25	0.0614
11.17	4.137	144.240	144	0.001104	4.67	0.001117577	55.75	0.0623
11.25	4.132	145.367	145	0.001131	4.63	0.001143839	55.25	0.0632
11.33	4.127	146.494	146	0.001157	4.58	0.0011701	54.75	0.0641
11.42	4.121	147.621	148	0.001183	4.54	0.001196362	54.25	0.0649
11.50	4.114	148.748	149	0.001209	4.50	0.001205116	53.75	0.0648
11.58	4.107	148.372	148	0.001201	4.46	0.001196362	53.25	0.0637
11.67	4.099	147.996	148	0.001192	4.42	0.001187608	52.75	0.0626
11.75	4.091	147.621	148	0.001183	4.38	0.001178854	52.25	0.0616
11.83	4.082	147.245	147	0.001174	4.33	0.0011701	51.75	0.0606
11.92	4.072	146.869	147	0.001166	4.29	0.001161346	51.25	0.0595
12.00	4.062	146.494	146	0.001157	4.25	0.001152593	50.75	0.0585
12.08	4.051	146.118	146	0.001148	4.21	0.001143839	50.25	0.0575
12.17	4.040	145.743	146	0.001139	4.17	0.001135085	49.75	0.0565
12.25	4.028	145.367	145	0.001131	4.13	0.001126331	49.25	0.0555
12.33	4.016	144.991	145	0.001122	4.08	0.001117577	48.75	0.0545
12.42	4.003	144.616	145	0.001113	4.04	0.001108823	48.25	0.0535
12.50	3.989	144.240	144	0.001104	4.00	0.00110007	47.75	0.0525
12.58	3.975	143.864	144	0.001096	3.96	0.001091316	47.25	0.0516
12.67	3.960	143.489	143	0.001087	3.92	0.001082562	46.75	0.0506
12.75	3.945	143.113	143	0.001078	3.88	0.001073808	46.25	0.0497
12.83	3.929	142.738	143	0.001069	3.83	0.001065054	45.75	0.0487
12.92	3.912	142.362	142	0.001061	3.79	0.0010563	45.25	0.0478
13.00	3.895	141.986	142	0.001052	3.75	0.001047546	44.75	0.0469
13.08	3.878	141.611	142	0.001043	3.71	0.001038793	44.25	0.0460
13.17	3.859	141.235	141	0.001034	3.67	0.001030039	43.75	0.0451
13.25	3.841	140.859	141	0.001026	3.63	0.001021285	43.25	0.0442
13.33	3.821	140.484	140	0.001017	3.58	0.001012531	42.75	0.0433
13.42	3.801	140.108	140	0.001008	3.54	0.001003777	42.25	0.0424
13.50	3.781	139.733	140	0.000999	3.50	0.000995023	41.75	0.0415
13.58	3.760	139.357	139	0.000991	3.46	0.00098627	41.25	0.0407
13.67	3.738	138.981	139	0.000982	3.42	0.000977516	40.75	0.0398
13.75	3.716	138.606	139	0.000973	3.38	0.000968762	40.25	0.0390
13.83	3.693	138.230	138	0.000964	3.33	0.000960008	39.75	0.0382
13.92	3.669	137.854	138	0.000956	3.29	0.000951254	39.25	0.0373
14.00	3.645	137.479	137	0.000947	3.25	0.0009425	38.75	0.0365
14.08	3.621	137.103	137	0.000938	3.21	0.000933747	38.25	0.0357
14.17	3.596	136.728	137	0.000929	3.17	0.000924993	37.75	0.0349
14.25	3.570	136.352	136	0.000921	3.13	0.000916239	37.25	0.0341
14.33	3.543	135.976	136	0.000912	3.08	0.000907485	36.75	0.0334
14.42	3.517	135.601	136	0.000903	3.04	0.000898731	36.25	0.0326
14.50	3.489	135.225	135	0.000894	3.00	0.000889977	35.75	0.0312
14.58	3.461	133.347	133	0.000851	2.96	0.0008287	35.25	0.0292
14.67	3.432	131.469	131	0.000807	2.92	0.000784931	34.75	0.0273
14.75	3.403	129.591	130	0.000763	2.88	0.000741162	34.25	0.0254
14.83	3.373	127.712	129	0.000719	2.83	0.000697393	33.75	0.0235
14.92	3.343	125.834	126	0.000676	2.79	0.000653624	33.25	0.0217
15.00	3.312	123.956	124	0.000632	2.75	0.000609854	32.75	0.0200
15.08	3.280	122.078	122	0.000588	2.71	0.000566085	32.25	0.0183
15.17	3.248	120.200	120	0.000544	2.67	0.000522316	31.75	0.0166
15.25	3.216	118.322	118	0.000500	2.63	0.000478547	31.25	0.0150
15.33	3.182	116.444	116	0.000457	2.58	0.000434778	30.75	0.0134
15.42	3.149	114.566	115	0.000413	2.54	0.000391008	30.25	0.0118
15.50	3.114	112.688	113	0.000369	2.50	0.000347239	29.75	0.0103
15.58	3.079	110.809	111	0.000325	2.46	0.00030347	29.25	0.0089
15.67	3.044	108.931	109	0.000282	2.42	0.000259701	28.75	0.0075
15.75	3.007	107.053	107	0.000238	2.38	0.000215932	28.25	0.0061
15.83	2.971	105.175	105	0.000194	2.33	0.000172162	27.75	0.0048
15.92	2.933	103.297	103	0.000150	2.29	0.000128393	27.25	0.0035
16.00	2.895	101.419	101	0.000107	2.25	8.4624E-05	26.75	0.0023
16.08	2.857	99.541	100	0.000063	2.21	5.04948E-05	26.25	0.0013
16.17	2.818	97.662	98	0.000038	2.17	3.75522E-05	25.75	0.0010
16.25	2.778	95.784	96	0.000037	2.13	3.61563E-05	25.25	0.0009
16.33	2.738	93.906	94	0.000035	2.08	3.47603E-05	24.75	0.0009
16.42	2.697	92.028	92	0.000034	2.04	3.33643E-05	24.25	0.0008
16.50	2.656	90.150	90	0.000033	2.00	3.19683E-05	23.75	0.0008
16.58	2.614	88.272	88	0.000031	1.96	3.05723E-05	23.25	0.0007
16.67	2.571	86.394	86	0.000030	1.92	2.91763E-05	22.75	0.0007
16.75	2.528	84.516	85	0.000028	1.88	2.77803E-05	22.25	0.0006
16.83	2.485	82.637	83	0.000027	1.83	2.63843E-05	21.75	0.0006
16.92	2.440	80.759	81	0.000026	1.79	2.49884E-05	21.25	0.0005
17.00	2.395	78.881	79	0.000024	1.75	2.35924E-05	20.75	0.0005
17.08	2.350	77.003	77	0.000023	1.71	2.21964E-05	20.25	0.0004
17.17	2.304	75.125	75	0.000021	1.67	2.08004E-05	19.75	0.0004
17.25	2.257	73.247	73	0.000020	1.63	1.94044E-05	19.25	0.0004
17.33	2.210	71.369	71	0.000019	1.58	1.80084E-05	18.75	0.0003
17.42	2.163	69.491	69	0.000017	1.54	1.66124E-05	18.25	0.0003
17.50	2.114	67.613	68	0.000016	1.50	1.52164E-05	17.75	0.0003
17.58								

19.08	1,086	31,928	32	-0.000011	0.71	-1.13073E-05	8.25	-0.0001
19.17	1,026	30,050	30	-0.000012	0.67	-1.27033E-05	7.75	-0.0001
19.25	966	28,172	28	-0.000013	0.63	-1.40993E-05	7.25	-0.0001
19.33	905	26,294	26	-0.000015	0.58	-1.54953E-05	6.75	-0.0001
19.42	843	24,416	24	-0.000016	0.54	-1.68913E-05	6.25	-0.0001
19.50	781	22,538	23	-0.000018	0.50	-1.82873E-05	5.75	-0.0001
19.58	718	20,659	21	-0.000019	0.46	-1.96832E-05	5.25	-0.0001
19.67	655	18,781	19	-0.000020	0.42	-2.10792E-05	4.75	-0.0001
19.75	591	16,903	17	-0.000022	0.38	-2.24752E-05	4.25	-0.0001
19.83	527	15,025	15	-0.000023	0.33	-2.38712E-05	3.75	-0.0001
19.92	461	13,147	13	-0.000025	0.29	-2.52672E-05	3.25	-0.0001
20.00	396	11,269	11	-0.000026	0.25	-2.66632E-05	2.75	-0.0001
20.08	330	9,391	9	-0.000027	0.21	-2.80592E-05	2.25	-0.0001
20.17	263	7,512	8	-0.000029	0.17	-2.94552E-05	1.75	-0.0001
20.25	195	5,634	6	-0.000030	0.13	-3.08511E-05	1.25	0.0000
20.33	127	3,756	4	-0.000032	0.08	-3.22471E-05	0.75	0.0000
20.42	59	1,878	2	-0.000033	0.04	-3.36431E-05	0.25	0.0000
20.50	-10	0	0	-0.000034	0.00	-3.43411E-05	0	0.0000
20.58	-7	0	0	-0.000034	0.00	-3.43411E-05	0	0.0000
20.67	-5	0	0	-0.000034	0.00	-3.43411E-05	0	0.0000
20.75	-3	0	0	-0.000034	0.00	-3.43411E-05	0	0.0000
20.83	-1	0	0	-0.000034	0.00	-3.43411E-05	0	0.0000
20.92	0	0	0	-0.000034	0.00	-3.43411E-05	0	0.0000
21.00	0	0	0	-0.000034	0.00	0.00432698	0	0.0000

APPENDIX C

Shear Design

PCI Big Beam Shear Design

$$\lambda := 1.0$$

$$\phi := 1.0$$

$$f'_c := 9482.2 \text{ psi}$$

$$f_{yt} := 60000 \text{ psi}$$

$$A_{ps} := 0.4521 \text{ in}^2$$

$$d := 18 \text{ in}$$

$$d_p := \frac{16}{12} \text{ ft}$$

$$d' := 1.5 \text{ in}$$

$$b_w := \frac{3}{12} \text{ ft}$$

NWC

Shear Resistance Factor

Concrete Compressive strength

Steel Yield Strength

PS Steel Area

Beam depth

Depth to PS Steel

Depth to compressive steel

Web Width

Design at d/2 from support

$$V_u := 23.26 \text{ kip}$$

Design Shear at d/2 from right side=19.16'
(Calcs on Excel sheet)

$$M_u := 31.076 \text{ kip} \cdot \text{ft}$$

Design moment at 19.16'

$$f_{pu} := 270 \text{ ksi}$$

PS Tendon Tensile Strength

$$f_{pe} := 172519 \text{ psi}$$

PS Tendon Effective Strength

$$f_{pe} > 0.4 f_{pu} = 1 \quad (\text{OK})$$

Alternative Method Check

$$V_u \cdot \frac{d}{M_u} = 1.123$$

$$V_c := \left(0.6 \lambda \cdot \sqrt{f'_c} \cdot \frac{1}{\sqrt{\text{psi}}} + \left(700 \cdot V_u \cdot \frac{d}{M_u} \right) \right) \cdot \frac{b_w \cdot d_p \cdot \text{kip}}{\text{ft}^2} = 281.446 \text{ kip}$$

$$V_{cmin} := \left(0.6 \lambda \cdot \sqrt{f'_c} \cdot \frac{1}{\sqrt{\text{psi}}} + (700) \right) \cdot \frac{b_w \cdot d_p \cdot \text{kip}}{\text{ft}^2} = 252.809 \text{ kip}$$

Concrete Shear
Strength

$$V_{cmax} := 5 \lambda \cdot \frac{\sqrt{f'_c}}{\sqrt{\text{psi}}} \cdot b_w \cdot d_p \cdot \frac{\text{kip}}{\text{ft}^2} = 162.294 \text{ kip}$$

$$V_c := V_{cmax} = 162.294 \text{ kip}$$

$$\frac{V_u}{\phi} < 0.5 V_c = 1$$

Check if Stirrups are needed

Stirrup steel is not needed at this section; minimum steel will be provided

$$A_{vmin} := \frac{A_{ps} \cdot f_{pu}}{80 \cdot f_{yt} \cdot dp} \cdot \sqrt{\frac{dp}{bw}} = 0.004 \text{ in} \quad \text{Minimum Steel}$$

$$A_{stirrup} := 2 \cdot 0.11 \text{ in}^2 \quad \text{Area of a \#3 U stirrup}$$

$$s := \frac{A_{stirrup}}{A_{vmin}} = 59.936 \text{ in}$$

Design at 14 ft

$$V_u := 22.829 \text{ kip}$$

$$M_u := 141.12 \text{ kip} \cdot \text{ft}$$

$$V_c := \left(0.6 \lambda \cdot \sqrt{f'_c} \cdot \frac{1}{\sqrt{\text{psi}}} + \left(700 \cdot V_u \cdot \frac{d}{M_u} \right) \right) \cdot \frac{bw \cdot dp \cdot \text{kip}}{\text{ft}^2} = 76.095 \text{ kip}$$

$$V_{cmin} := \left(0.6 \lambda \cdot \sqrt{f'_c} \cdot \frac{1}{\sqrt{\text{psi}}} + (700) \right) \cdot \frac{bw \cdot dp \cdot \text{kip}}{\text{ft}^2} = 252.809 \text{ kip} \quad \text{Concrete Shear Strength}$$

$$V_{cmax} := 5 \lambda \cdot \frac{\sqrt{f'_c}}{\sqrt{\text{psi}}} \cdot bw \cdot dp \cdot \frac{\text{kip}}{\text{ft}^2} = 162.294 \text{ kip}$$

$$V_c = 76.095 \text{ kip}$$

$$\frac{V_u}{\phi} < 0.5 V_c = 1 \quad \text{Check if Stirrups are needed}$$

Stirrup steel is not needed at this section; minimum steel will be provided

$$A_{vmin} := \frac{A_{ps} \cdot f_{pu}}{80 \cdot f_{yt} \cdot dp} \cdot \sqrt{\frac{dp}{bw}} = 0.004 \text{ in} \quad \text{Minimum Steel}$$

$$A_{stirrup} := 2 \cdot 0.11 \text{ in}^2 \quad \text{Area of a \#3 U stirrup}$$

$$s := \frac{A_{stirrup}}{A_{vmin}} = 59.936 \text{ in}$$

Design at 11.5 ft

$$V_u := 13.77 \text{ kip}$$

$$M_u := 152.86 \text{ kip} \cdot \text{ft}$$

$$V_c := \left(0.6 \lambda \cdot \sqrt{f'_c} \cdot \frac{1}{\sqrt{\text{psi}}} + \left(700 \cdot V_u \cdot \frac{d}{M_u} \right) \right) \cdot \frac{b_w \cdot d_p \cdot \text{kip}}{\text{ft}^2} = 51.004 \text{ kip}$$

$$V_{cmin} := 1.7 \lambda \cdot \frac{\sqrt{f'_c}}{\sqrt{\text{psi}}} \cdot b_w \cdot d_p \cdot \frac{\text{kip}}{\text{ft}^2} = 55.18 \text{ kip}$$

$$V_{cmax} := 5 \lambda \cdot \frac{\sqrt{f'_c}}{\sqrt{\text{psi}}} \cdot b_w \cdot d_p \cdot \frac{\text{kip}}{\text{ft}^2} = 162.294 \text{ kip}$$

$$V_c = 51.004 \text{ kip}$$

$$\frac{V_u}{\phi} < 0.5 V_c = 1$$

Check if Stirrups are needed

Stirrup steel is not needed at this section; minimum steel will be provided

$$A_{vmin} := \frac{A_{ps} \cdot f_{pu}}{80 \cdot f_{yt} \cdot d_p} \cdot \sqrt{\frac{d_p}{b_w}} = 0.004 \text{ in} \quad \text{Minimum Steel}$$

$$A_{stirrup} := 2 \cdot 0.11 \text{ in}^2$$

Area of a #3 U stirrup

$$s := \frac{A_{stirrup}}{A_{vmin}} = 59.936 \text{ in}$$

Use 2U#3 (engineering judgement on location)

APPENDIX D

Cost Calculation

BEAM COST

PROJECT : PCI BIG BEAM COMPETITION 2020-2021

Prepared by : Jose Ramirez

: Juan Plasencia

LOCATION : Kansas City, MO

DATE : 7/11/2021

ITEM	DESCRIPTION	Unit	Quantity	Unit Cost	Price (\$)
01	FORMWORK				
01.01	Forming	ft ²	85.62	1.25	107.03
02	CONCRETE				
02.01	f'c = 9,482 PSI Concrete	yd ³	0.45	114.82	51.62
03	REINFORCEMENT				
03.01	Mild Steel A615/A706	lb	79.44	0.45	35.75
03.02	Prestressing Strands	ft	62.94	0.30	18.88
Beam Cost					\$ 213.28

<u>Weight of Beam</u>	Unit Weight (lb/ft ³)	Beam Section (ft ²)	Length (ft)	Total Weight (lb)
Concrete Volume	144	0.58	20.98	1,748

Quantities

PROJECT : PCI BIG BEAM COMPETITION 2020-2021

Prepared by : Jose Ramirez

: Juan Plasencia

LOCATION : Kansas City, MO

DATE : 7/11/2021

01	FORMWORK
-----------	-----------------

01.01	Forming			Unit	ft ²
Drawing	Description	Number	Length (ft)	Perimeter (ft)	Partial (ft ²)
S1	Sides	2	20.98	1.72	72.12
	Bottom	1	20.98	0.59	12.35
	Ends	2	area=	0.58	1.16
				Total	85.62

02	CONCRETE
-----------	-----------------

02.01	f'c = 9,482 PSI Concrete			Unit	yd ³
Drawing	Description	Section (ft ²)	Length (ft)	Volume (ft ³)	Volume (yd ³)
S1	Beam Volume	0.58	20.98	12.14	0.45
					-
				Total	0.45

03	REINFORCEMENT
-----------	----------------------

03.01	Mild Steel A615/A706			Unit	lb
Plan	Description	Number	Length (ft)	wt (lb/ft)	Partial (lb)
Top Bars	#4 Rebars	3	21.61	0.668	43.31
					-
					-
Stirrups	#3 Rebars	27	3.56	0.376	36.13
					-
					-
				Total	79.44

03.02	Prestressing Strands			Unit	ft
Plan	Description	Number	Length (ft)		Partial (ft)
	1/2 in. Diameter Strands	3	20.98		62.94
					-
					-
				Total	62.94

APPENDIX E

Material Properties

6208 Richards Dr.
Shawnee, KS 66216
Phone: 913-495-2360

Client:
University of Missouri-Kansas City
5110 Rockhill Rd, Kansas City, MO 64110

Project:
UMKC Big Beam Competition

Sample Details

Set #:	1	Technician:	Jose Ramirez	Batched:	09:41 CDT
Specimen Size:	4" X 8"	Cast By:	Jose Ramirez	Sampled:	09:46 CDT
Specimens In Set:	10	Date Cast:	05/08/21	Cast:	09:50 CDT
Truck / Ticket #:	00 / 2824	Sampled From:	Direct	Truck Empty:	10:10 CDT
Contractor:	N/A	Placement Method:	Direct	Placement Time:	29 (min)

Location

Placement Location: I-Beam Mold
Location Details: Coreslab Structures Facility
Sample Location / Notes: 10 cylinders made next to form

Batch Log

Supplier: Coreslab Structures Inc
House Supply
Plant: 01
On-Site Admixtures: None

Mix Design: CS-800-L

Specifications

Strength: 10,000 (psi)
Air: 5.0 (%)
Slump: 10 (in)

Field Measurements

Weather: Clear
Air Temperature (F): 59
Slump (in): 9 (ASTM C143)
Concrete Temp (F): 71 (ASTM C1064)
Plastic Unit Weight: 144 (lb/ft³)
Air Content: 6.2 (ASTM C231)
Load Volume: 2 (yd³)

Lab Test Results

Specimen Number	Test Age Days	Test Date	Field / Lab Cure Days	Average Cylinder Diameter (in)	Cylinder Area (in ²)	Max Load (lbs)	Strength (psi)	Fracture Type	Break Remark	Capping Method
1-1	2	05/10/21	2	4.00	12.57	110680	8807.6	3		NP
1-2	2	05/10/21	2	4.01	12.63	106450	8471.0	3		NP
1-3	2	05/10/21	2	4.00	12.57	112370	8942.1	3		NP
1-4	28	06/05/21	28	4.00	12.57	126840	10093.6	3		NP
1-5	28	06/05/21	28	4.00	12.57	121630	9679.0	3		NP
1-6	28	06/05/21	28	4.00	12.57	119020	9471.3	3		NP
1-7	48	06/25/21	48	4.00	12.57	121770	9690.2	3		NP
1-8	48	06/25/21	48	4.00	12.57	111450	8868.9	3		NP
1-9	48	06/25/21	48	4.00	12.57	124250	9887.5	3		NP
1-10	H									



TYPE 1 TYPE 2 TYPE 3 TYPE 4 TYPE 5 TYPE 6

CORESLAB STRUCTURES (KANSAS) INC.

DATE: 5/8/2021

TICKET NO: 2824

Batched Time : 9:41 AM

JOB NO
210 U.M.K.C

JOB DESCRIPTION
Q. C. Miguel Madigal

ORDER NO

PO NUMBER

JOB LOCATION
210 U.M.K.C Test

MIX	PRODUCT DESCRIPTION	QTY
S-800-L	ANYTHING WITH 6500 AND OVER FOR RELEASE	2.00

BATCH SIZE	SHIPPED	LOAD NO	UNIT NO	WC RATIO	SLUMP
2.00	2.00	1	99	0.28	9.50

BATCH DETAILS

MATERIAL	MIX	TARGET	ACTUAL	MOISTURE
WATER	27	50	50	
HOT WATER		0	0	
TYPE 3	800	1600	1591	
LIMESTONE	1752	3495	3565	0.80
SAND	1116	2275	2255	2.23
VR10	113.00	224.00	224	
PS1466	15.00	240.00	238	

TESTED SLUMP: _____ IN

TESTED AIR: _____ %

TEMP: _____ F

Missouri Department of Transportation-Construction and Materials Division

Concrete Mix Design Submittal Form

Date: 4/19/2021

rete Supplier: Coreslab Structures (Kansas)

Plant Location: Kansas City, K

Designer Name: Terry Fleck

Telephone No.: 913-287-5725

This form should be submitted with a cover letter acknowledging which MoDOT contracts a mix is planned to be used on if any are k

nts

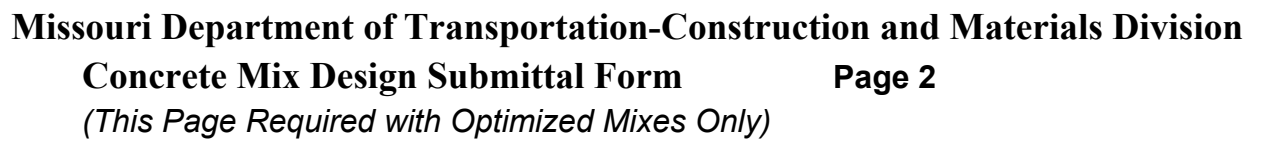
Producer	Type	Unit Wt (lb/cf)	G	Absorptivity
Midwest Concrete Materials	Natural Sand for Concrete Class A	115	2.636	0.2
Capital Materials, Tightwad Quarry	Masonry Grade E LS/DO	99	2.664	0.9
Central Plains Cement, Sugar Creek	Portland Cement Type III		3.15	

intermediate aggregate is essentially a second course aggregate.

Source	Type	Brand Name
BASF Constr. Chemicals, (IL)	Air Entraining Agent for Concrete	VR 10
BASF Constr. Chemicals, (IL)	Type A Water Reducing Agent for Concrete	PS 1466

ns and Properties (for up to five mixes)

[illegible]

[illegible]

Straight Elongation Calculations 210

Job Name UMKC Bed 210 Date May 6, 2021

Job Number 0 Pour No. 1

Strand Size & Type .5 270k lolax (WMC) Reel No. 1355-S2-6

AE= 4340160 Area 0.15070 in²

Bed Length 225.1771 ft MOE 2.88E+01 lb/in²x10⁶

Actual Length 225.18 Number of Strand 3

Total Force in Strand P 30600.00 lbs

Initial Force 3000.00 lbs

Elongation 17.1834 in

Total 17.1834 in

Elongation Corrections:

Dead End Chuck Slippage 0.1875 in

Anchor Movement 0.15625 in

Live End Seating Slippage 0.375 in

Temperature Correction Delta 0 Fahrenheit
(estimated Concrete temp-steel temp)

Thermal Effects 0.000000 in

Splice Chuck 0

Jacking Elongation 17.9021 in

Final Elongation 17.5271 in

Jacking Pressure 31453 lbs

Final Gauge Pressure 31500 lbs

Final Spread 17.5271 in + **18 6/16** in
- **16 10/16** in



18490 MAIN STREET, CONROE, TX 77385
678-633-7091

PRODUCT TEST CERTIFICATION

CUSTOMER: Coreslab Structures

DATE: 01/18/2021

LOCATION: 759 S. 65th Street,
Kansas City, KS 66111
Ph 913-287-5725

CERTIFICATION # 80004336

PURCHASE ORDER # 3085

CERTIFICATION STANDARD: ASTM A416/A416M-12

WMC HEREBY CERTIFIES THAT THE SPECIMENS TAKEN FROM PRODUCTION LOT(S) CONSISTING OF ONE OR MORE OF THE FOLLOWING LOT/SERIAL NUMBERS WERE TESTED IN ACCORDANCE WITH AND MET THE SPECIFICATION REQUIREMENTS OF ASTM A 416-10.

THE ATTACHED TEST REPORT(S) REPRESENTS THE RESULT OF SUCH TEST(S).

SIZE: 0.500 " DIAMETER

GRADE: 270K GRADE LR SEVEN WIRE STRAND ASTM A416:

COIL NUMBERS		HEAT NUMBER	
7700013954	1354-S1-3	Pile 4/4	OT0022772
7700013956	1354-S1-5	x Pile 1-27	OT0022772
7700013957	1354-S1-6	x Pile 3/17	OT0022772
7700013959	1354-S1-8	x Pile 3/4	OT0022772
7700013943	1355-S2-1	135 3/10	OT0022772
7700013945	1355-S2-3	135 3/10	OT0022772
7700013948	1355-S2-6	Pile 4/26	OT0022772

THE PRODUCTS LISTED IN THIS CERTIFICATION WERE MANUFACTURED AND FABRICATED IN THE UNITED STATES OF AMERICA

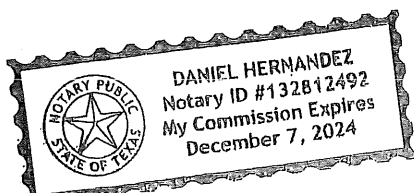
WMC HEREBY CERTIFIES THAT THE PRESTRESSING STRAND DESCRIBED ABOVE MEETS OR EXCEEDS THE MINIMUM BONDING REQUIREMENTS AS CURRENTLY ACCEPTED IN THE NASP (NORTH AMERICAN STRAND PRODUCERS) PULL-OUT TEST AND THE MOUSTAFA BLOCK PULL-OUT TEST.

ALL DOMESTIC PRESTRESSING STRAND WAS MADE FROM STEEL ROD THAT WAS MANUFACTURED AND PROCESSED COMPLETELY IN AMERICA. THE ROD WAS THEN MANUFACTURED INTO PC STRAND IN THE UNITED STATES AT WMC LLC, 18490 MAIN STREET CONROE, TX 77385, WITH AN AVERAGE MODULUS OF 28.8.

THE MATERIAL MEETS THE "BUY AMERICA" REQUIREMENTS OF 23 CFR 635.410.

THE PRODUCTS LISTED IN THIS CERTIFICATION WERE MANUFACTURED IN THE USA FROM WIRE ROD WHICH WAS MANUFACTURED IN THE USA.

CERTIFICATION PREPARED BY:



Richard Gane



WMC

18490 MAIN STREET
CONROE, TX 77385

Customer: Stock
Size: 0.500
Test Start Date and Time: 1/17/2021 12:36 P

Heat Number: OT0022772
Grade: 270K Low Relaxation
Tested by: WG

Coil No: 1355-S2-2
Core Diameter: 0.1711 in
Diameter: 0.1646 in
Area: 0.1507 in²
Ultimate Force: 42900 lbf
EUL Force @ 1%: 39800 lbf
Avg. Modulus Mpsi: 28.8 Mpsi
Total Elongation: 4.83 %

1355-S2-1
1355-S2-3
1355-S2-6

