



2025 PCI Big Beam Competition

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Precast/Prestressed
PCI Concrete Institute

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Executive Summary

As part of the 2025 PCI Big Beam Competition, the “Prestressed Wolfpack” team from North Carolina State University (NCSU) designed and tested a Big Beam that met all judging criteria. The NCSU project emphasized innovation by integrating architectural features into a prestressed concrete beam—an approach that is uncommon in traditional construction practice. This concept challenged the team to develop unique and comprehensive design calculations to ensure accurate strength predictions.

Constructability was a key focus, aligning with the practical requirements of precast concrete elements and on-site handling. The beam featured a rectangular cross-section with multiple staggered openings of varying sizes along its span, as detailed in Figure 1. A white architectural concrete mix was used for casting, providing an aesthetically appealing finish that eliminates the need for additional surface treatments like paint.

The final beam weighed 1,820 lbs and the total formwork cost was \$64.02. Due to its novel geometry, beam capacity was predicted using a combination of ACI 318, the PCI Design Handbook, AASHTO LRFD, and relevant literature. Flexural capacity was evaluated at both the midspan and near the openings, and shear checks were performed around all openings to mitigate the risk of shear failure.

The beam was tested under monotonic loading to failure, with two load cycles conducted to measure cracking load, deflection at 32 kips, and ultimate load. The predicted failure mode—concrete crushing—posed an added challenge to the prediction effort. Table 1 summarizes the predictions and test results, while Figure 2 shows the load-midspan deflection. Predictions were based on a combination of finite element modeling (FEM) and hand calculations in accordance with design codes.

Remarkably, the team achieved 0% error in predicting both the cracking and ultimate loads. Deflection prediction proved more challenging with a higher percentage error, although the midspan deflection at 32 kips was under 1 inch—small enough that measurement tolerances could significantly affect accuracy.

The completed test summary form and certification are provided below, followed by engineering drawings in Figure 3 and costing details in Table 2.

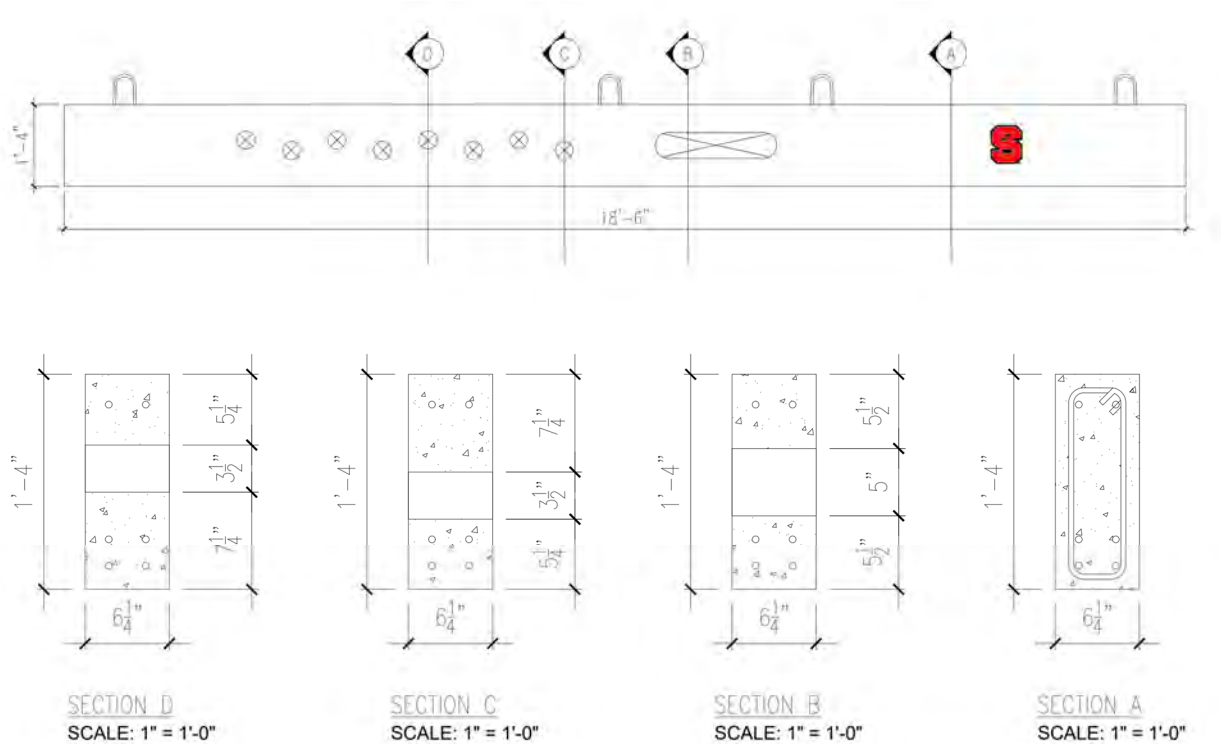
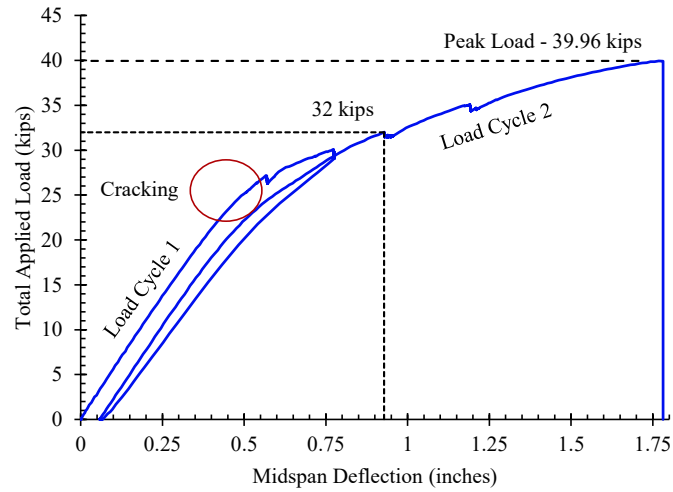


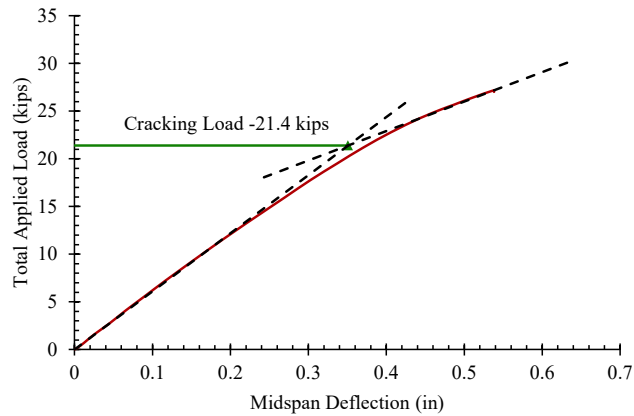
Figure 1: Cross-section and elevation details

Table 1: Summary of predictions and experimental results

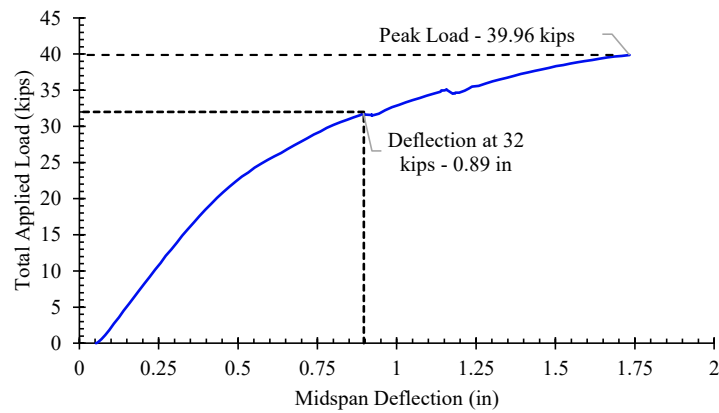
Criteria	Predictions	Experimental Results	Percentage Difference (%)
Cracking Load (kips)	21.50	21.49	0%
Maximum Applied Load (kips)	39.90	39.96	0%
Midspan Deflection at 32 kips (in)	0.50	0.89	78%
Midspan Deflection at 32 kips considering Camber (in) (updated)	0.66	0.89	35%



(a)



(b)



(c)

Figure 2: Load-Deflection at Midspan a) Entire loading protocol b) Cracking load obtained from DIC from first load cycle c) Deflection at 32 kips and peak load obtained from DIC from second load cycle

Summary/Judging Form

PCI BIG BEAM COMPETITION 2024-2025

06/13/2025

Date

Prestressed Wolfpack (North Carolina State University) 1

05/16/2025

Student Team (school name)

Team Number

Date of Casting

Basic Information

1. Age of beam at testing (days) 25
2. Compressive cylinder tests*
 - Number tested 2
 - Size of cylinders 3 in. x 6 in.
 - Average (psi) 8170
3. Concrete properties
 - Unit weight of concrete (lb/ft³) 143.95
 - Slump or spread (in.) 22
 - Air content (%) 5.6
 - Tensile strength (psi) 550
 - Circle one: Split cylinder MOR beam
4. Pretest calculations
 - a. Applied load (total) to cause cracking (kip) 21.5
 - b. Maximum applied point load at midspan (kip) 39.9
 - c. Anticipated deflection due to total live load application of 32 kips 0.5 in.

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. Center to center span (ft) 18
 2. Actual maximum applied load (kip) 39.96
 3. Measured cracking load (kip)¹ 21.49
 4. Cost (dollars) 178.78
 5. Weight (lb) 1819.6
 6. Largest measured deflection (in.) 1.74
 - a. Measured deflection at applied load of 32 kip, 0.89
 7. Most accurate calculations:
 - a. Absolute value of (maximum applied load – calculated applied load)/calculated applied load 0.0015
 - b. Absolute value of (Measured deflection at 32 kips – calculated deflection) / (calculated deflection) 0.78
 - c. Absolute value of (measured cracking load – calculated cracking load)/calculated cracking load 0.0005
- Total of three absolute values (a + b + c) = 0.782**

¹ 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 digital report, due June 13, 2025.

Sponsored by:



Certification



PCI BIG BEAM COMPETITION 2024-2025

CERTIFICATION

Tindall Corporation

As a representative of (name of PCI producer member or sponsoring organization)
North Carolina State University

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:

Digitally signed by Ali
Shokrgozar
DN: cn=Ali
Shokrgozar, email=Ali.Shokrgozar@ncsu.edu,
c=US, ou=North Carolina State University
Date: 2025.06.10 11:55:59 -0400

Signature

Ali (Farshad) Shokrgozar

Name (please print)

06/10/2025

Date

39.9 kips

Predicted maximum load

21.5 kips

Predicted cracking load

0.5 inches

Predicted deflection load at 32 kip

THIS CERTIFICATION MUST BE PART OF THE FINAL REPORT.

Sponsored by:

ASPIRE

ALP
SUPPLY

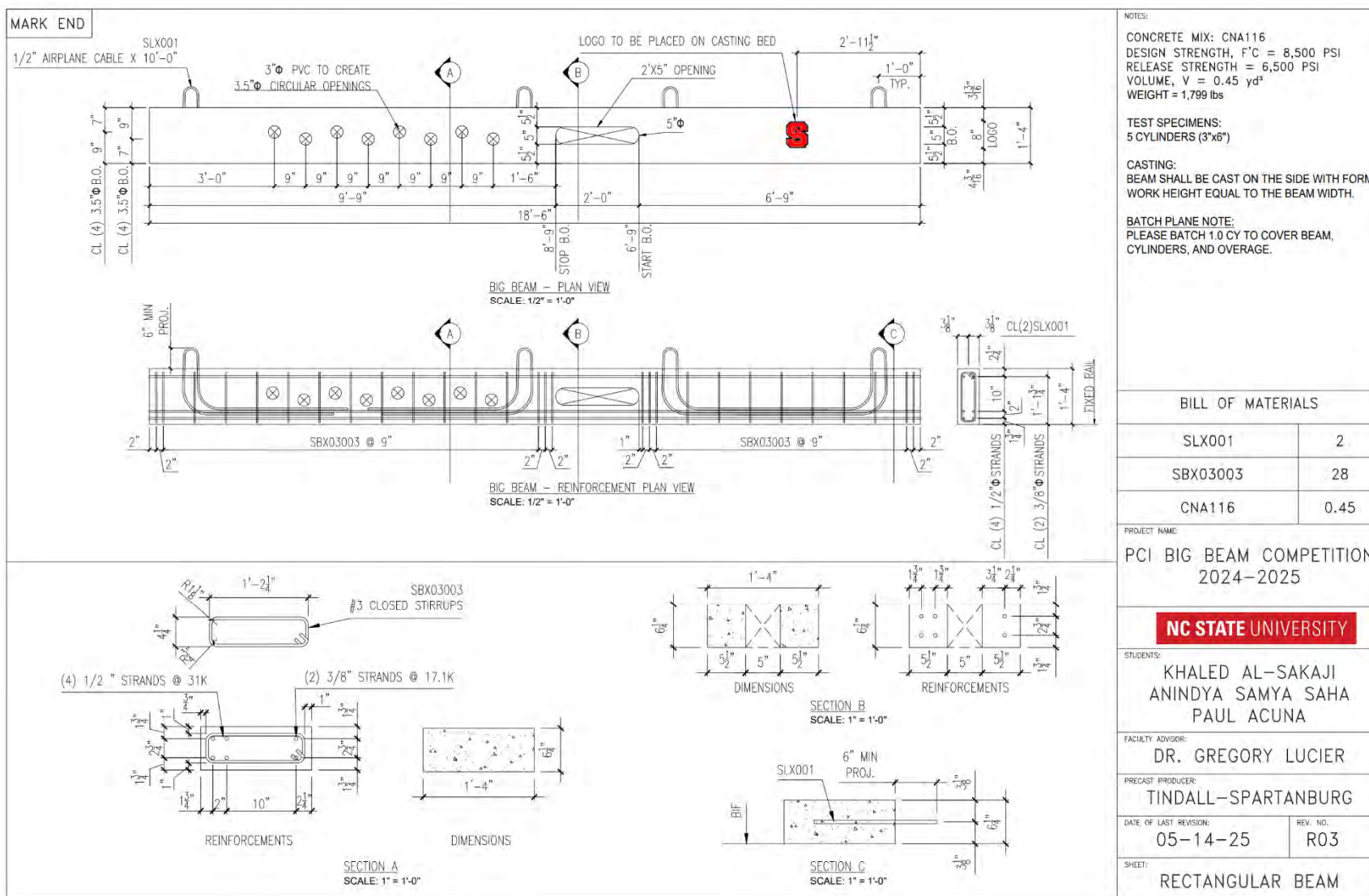


Table 2: Cost calculation

Material	Unit Cost	Amount	Cost
Concrete (8 ksi)	\$145 /yd ³	0.47 yd ³	\$67.88
3/8 in. Diameter Prestressing Strand	\$0.27 /ft	37.00 ft	\$9.99
1/2 in. Diameter Prestressing Strand	\$0.30 /ft	74.00 ft	\$22.20
Steel – A615/A706	\$0.45 /lb	32.64 lb	\$14.69
*Formwork (Cast Flatwise)	\$1.25 /ft ²	51.22 ft ²	\$64.02
Total Cost			\$178.78

*Formwork includes contact surfaces on the sides, bed, and the PVC pipes and foam at openings.

1 Introduction

Prestressed concrete beams came into common existence in the early 1950s. One factor that contributes to the continued popularity of prestressed concrete for long-span flexural members is that these members are often capable of remaining uncracked throughout their service lives. Prestressed concrete resists flexural cracking because the prestressing steel, often referred to as strands or tendons, is tensioned to produce compressive stresses in the concrete. By adding precompression, an applied moment must first relieve the compressive stress in the girder concrete resulting from prestressing before a tensile stress can be produced. Therefore, the moments required to crack a prestressed concrete member are typically significantly higher than those required to crack a similar reinforced concrete member.

The Precast/Prestressed Concrete Institute (PCI) organizes the Big Beam Competition annually which offers students a unique opportunity to gain hands-on experience designing, constructing, and testing a prestressed concrete beam that is innovative, structurally efficient, and cost-effective—reflecting real-world engineering challenges. The “Prestressed Wolf Pack” team from North Carolina State University is excited to participate in the 2024-2025 edition of this competition and presents this comprehensive report in this regard. The team members are Khaled Al Sakaji, Paul Acuna, and Anindya Samya Saha, advised by Dr. Gregory Lucier from NC State University.

With the generous support and guidance of Tindall Corporation in Spartanburg, South Carolina, our team designed an 18.5 ft-long beam (18 ft tested span), having a rectangular cross-section measuring 6.25 in by 16 in (W×H), however, due to the tolerance limits in the construction the beam width was between 6.25 in & 6.5 in, and the design was refined following to the actual dimensions. Several full-width openings are staggered along the span. The beam consists of four 0.5 in. diameter prestressing strands near the bottom in layers of two, and two 3/8 in. diameter prestressing strands at the top. The beam is lightly reinforced for shear by #3 two legged stirrups. The beam was tested in four-point bending to evaluate its performance under service and ultimate load conditions.

This report outlines our journey—from material selection and structural design to fabrication, testing, and evaluation. It includes detailed calculations for flexural strength, shear capacity, deflection, weight, and cost. Reflections by team members are also shared, highlighting the knowledge and skills gained through this invaluable learning experience.

2 Experimental Program

2.1 Mix Design and Properties

The team opted for a concrete mix that is consistent in strength within the 8000-10000 psi range. Since it was planned to cast the Big Beam on May 16th (less than 1 month before the contest deadline) the team required a mixture that gained the required strength within 21 days. After a discussion with our PCI producer sponsor, Tindall Corporation, Spartanburg Division, and looking at potential mix designs that are regularly produced at their facility, a very unique “Architectural Mix” was selected for the Big Beam. The concrete mix is reliable with a 28-day specified compressive strength around 6 ksi but would readily gain strengths up to 8500 psi, satisfying the structural design calculations. Other reasons for choosing this concrete mix included the white color and smooth texture, giving a surface that is architecturally appealing. Due to the small scale of the Big Beam (compared with typical beams in the industry), it was important that the mix was self-consolidating and had high workability and plasticity allowing it to easily flow into the formwork even though the water-cement ratio was very low (0.39). In order to ensure this, the mix was designed with the following constituents conforming to the specifications of ASTM C260 and ASTM C494: viscosity modifier (VMA-758), water repellants (Waterpel-S), water reducer (Extendflo X90) and air entraining admixture (RAE-260). The combination of admixtures reduced segregation and honeycombing to provide a smooth and even finished surface. The mix uses a white Type I/II cement (ASTM C150) from Florida, #7 limestone aggregates, a ½ in nominal diameter clean stone from Texas (ASTM C33), and a white washed sand which gives this concrete mix a unique white texture. Table 3 summarizes the mix design proportions and complete documentation from Tindall Corporation is included in Appendix A.

Table 3: Concrete mix design proportions

<i>Material</i>	<i>SSD Weight lb/yd³</i>	<i>Volume yd³</i>
Type I/II Cement (Lehigh Alborg White Cement)	800	4.07
Coarse Aggregate (Texas #7 Limestone)	1390	7.86
Coarse Aggregate (Arrowood ½ in. Cleaned)	208	1.14
Fine Aggregate (Sand White Wash)	1217	7.48
Air Entraining Agent (RAE-260)	0.21	0
High-Range Water Reducer/ Plasticizer (Extendflo X90)	5.86	0.09
Water Repellant (Waterpel-S)	1.32	0.02
Viscosity modifier (VMA-758)	2.23	0.03
Water	310	4.97
Air	N/A	1.35
Total	3935	27.02

Figure 5 shows batching at the plant and the fresh concrete undergoing quality control tests to measure unit weight, air content, and a spread. The spread was within the desirable range of 20-26 in. The plant also cast 12 additional 3 in × 6 in cylinders which were used to monitor strength gain with age so that testing of the beam could be timed accurately when it reached the design concrete compressive strength. Table 4 shows the measured properties of the fresh mix.



Figure 4: a) Batching concrete b) Taking a sample for quality control c) Air content meter and unit weight measurement d) Spread test e) Casting 3 in $\times$ 6 in cylinders

Table 4: Properties of the architectural concrete

<i>Property</i>	<i>Value</i>
Spread (in)	22
Air Content (%)	5.6
Unit Weight (lb/ft ³)	143.95
w/c ratio	0.39

2.2 Reinforcement

The beam was designed using a combination of ½ in. and 3/8 in. diameter 7-wire low-relaxation prestressing strands with a specified ultimate strength of 270 ksi. The strands were stressed to 75% of their ultimate strength at jacking. Four ½ in. diameter strands were placed in two layers near the bottom of the beam, while two 3/8 in. strands were positioned at the top, as shown in Figure 3. For shear reinforcement, Grade 60 #3 closed stirrups were used. Reinforcement details are also provided in Figure 3. The material properties of the prestressing strands (per ASTM A416) and rebar are listed in Table 5, with mill certificates included in Appendix A, as provided by Tindall Corporation, our PCI-certified producer.

Table 5: Reinforcement properties

<i>Reinforcement</i>	<i>Yield Strength f_{py} (MPa)</i>	<i>Ultimate Strength, f_{pu} (MPa)</i>	<i>Modulus of Elasticity, E_p (GPa)</i>	<i>Ultimate Elongation (%)</i>
½ in. Diameter Prestressing Strand	@1% Extension - 1860	1980	198.6	4.7
3/8 in Diameter Prestressing Strand	@1% Extension - 1790	1932	198.6	6.3
#3 Rebar	YS 0.2% – 369	515	184.5	24.2

2.3 Material Testing

The cylinders were tested for compressive strength and splitting tensile strength according to the specifications of ASTM C39 and ASTM C496. For the compressive strength tests, the cylinder ends were first ground to ensure a smooth surface (Figure 5) before testing in a compression machine. Because concrete compressive strength is critical to satisfy stress limits at release and also allows accurately predicting cracking and ultimate beam capacity, the cylinders were closely monitored for compressive strength gain by testing at regular intervals (see Figure 6). A rapid increase in compressive strength was observed within the first 3 days after casting and satisfied the stress limits at release (cast on Friday, released on a Monday). The early strength-time behavior was plotted so the team could predict the concrete age at which the design strength would be achieved.



Figure 5: Grinding cylinders for compression testing

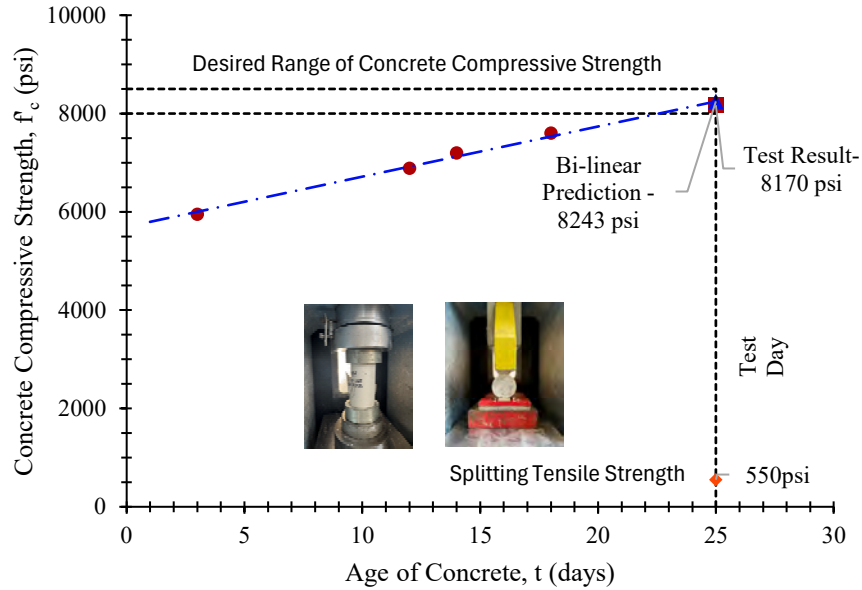


Figure 6: Concrete compressive strength over time

On the day of the Big Beam test (25 days after casting), concrete cylinders were tested for compressive strength and splitting tensile strength and the results were directly used for the final prediction of the beam behavior. The results are listed in Table 6. The expected 28 days compressive strength according to Figure 6 is around 8,250 psi, while the expected tensile strength is around 600psi.

Table 6: Compressive strength and splitting tensile strength on test day (concrete age is 25 days)

Cylinder	Compressive Strength (psi)
1	8080
2	8256
Average	8170
Splitting Tensile Strength (psi)	
1	550

3 Structural Design

3.1 Preliminary Analysis and Summary

This section of the report outlines the Big Beam design approach that the Prestressed Wolfpack team followed to meet the structural performance goals while optimizing cost, weight, strength, and deflection under maximum applied load. The process was iterative and variables such as geometry, materials, and concrete strength were initially discussed with an initial focus on lowering construction cost. The next step involved evaluating the flexural capacity of several cross-sections with comparable dimensions, including I-beams, T-beams, rectangular beams, double-tees, and rectangular sections with openings, a few of which are shown in Figure 7.

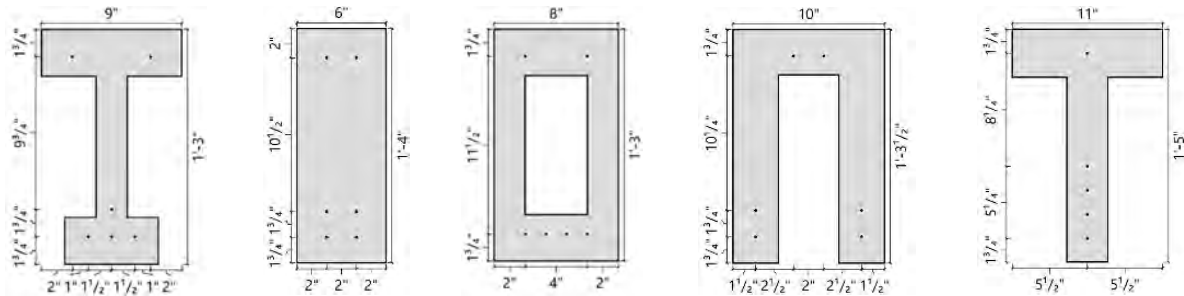


Figure 7: Some of the sections considered in preliminary design

Concrete cover and reinforcement spacing were conservatively determined following the guidelines of ACI-318, and AASHTO LRFD Tables 20.5.1.3.3 and 25.2.4, which helped to determine the initial strand profiles. The section was designed for a total applied load of 38 kips, below the 40 kip maximum, to account for some variability in concrete strength. The strands were low-relaxation strands with assumed jacking stress of 75% of f_{pu} (where $f_{pu} = 270$ ksi). The ultimate flexural capacity at the midspan section was then evaluated based on the selected reinforcement. Cracking moments were calculated using both ACI 318 and AASHTO LRFD limits— $7.5\sqrt{f'_c}$ (psi) and $0.37\sqrt{f'_c}$ (ksi), respectively. All sections cracked above 20 kips under both standards, satisfying design requirements (details in the Appendix). The design assumed flexural failure at the midspan where the flexural stress is maximum, with adequate shear capacity which will be verified later. For the initial design, only bottom strands were used, resulting in high tensile stress at the top concrete fiber at release, potentially causing early cracking. To prevent this, two top strands were added and each cross-section was re-analyzed using Eriksson software to predict cracking and ultimate loads at midspan. The full member response was also developed using Eriksson and stress limits were checked. Performance of each type of section was assessed based on competition judging criteria—design accuracy, cost, weight, and deflection. Results showed that rectangular, I-beam and T-beam sections performed best when deflection was excluded, while the I-beam was more efficient when deflection was considered (see Table 7 **Error! Reference source not found.** and Table 8). Rectangular and I-beam sections were then selected for further optimization.

Table 7: Section optimization according to judging criteria using Eriksson Software

Options	Service Load (kips)	Ultimate Load (kips)	Cost (\$)	Weight (lb)	Deflection (in)	Max. Points
I-Beam (1)	20.6	38.1	180	1305	0.695	44
I-Beam (2)	20.5	37.9	178	1142	0.685	45
Rect (1)	20.2	38.4	197	2175	0.900	42
Rect (2)	20.5	35.9	155	1740	0.452	39
Rect Hollow (1)	20.6	39.3	231	2175	0.798	38
DT-Beam (1)	20.2	36.4	263	3045	0.643	28
T-Beam (1)	20.2	38.4	188	1432	0.417	37

Table 8: Section optimization according to judging criteria (without deflection)

Options	Service Load (kips)	Ultimate Load (kips)	Cost (\$)	Weight (lb)	Max. Points
I-Beam (1)	20.6	38.1	180	1305	36
I-Beam (2)	20.5	37.9	178	1142	37
Rect (1)	20.2	38.4	197	2175	30
Rect (2)	20.5	35.9	155	1740	36
Rect Hollow (1)	20.6	39.3	231	2175	28
DT-Beam (1)	20.2	36.4	263	3045	21
T-Beam (1)	20.2	38.4	188	1432	35

The goal was to innovate within strength and serviceability limits while maintaining confidence in predictions of cracking moment and ultimate flexural capacity. As part of it, an opening was created between the loading points where there exists a relatively constant moment region with significantly lower applied shear because of the asymmetrical loading configuration. Therefore, the beam was still expected to fail in flexure, assuming the small shear force at the opening could be carried by concrete. This move towards a large opening between the loads helped reduce both weight and the amount of concrete used, but did present the risk of increased deflection. In addition, a series of smaller openings along the longer shear span were created to also minimize the weight while introducing an architectural aspect. No openings were created in the shorter span which higher shear to avoid any potential localized shear failures. The number of small openings involved optimizing the trade-off between reduced self-weight and increased formwork cost as detailed in Appendix E. The location of the openings was also carefully selected since the M/V ratio changes along the shear span and openings closer to the support pose a risk of shear failure. Analysis in Eriksson software showed that both the rectangular and I-beam met strength and serviceability criteria, with the I-beam offered slightly lower production cost. However, due to the novelty of incorporating openings in a prestressed concrete beam, possible localized high shear stress concentration in the web of an I-beam, and the short time available for fabrication and testing, the team proceeded with the rectangular section. Small openings in beam webs are common to allow ducts for utilities, but the larger opening between the load points makes the NCSU Big Beam incredibly unique. The design was further validated through hand calculations, layered sectional analysis, and finite element modeling, as detailed in the following sections.

The finalized beam cross-section is shown in Figure 3. The beam measures 16 in. high, 6.5 in. wide and 18.5 ft long. It has an opening with rounded edges located at 9 ft 9 in from the end of longer shear span that measures 2 ft long by 5 in. deep. All openings are full thickness of the web. The longer shear span has a series of 8 circular openings measuring 3.5 inches in diameter each. The first opening is located at 3 ft from with a center-to-center spacing between the opening of 9 in. The openings have a staggered layout in terms of their location along the depth, alternating between a distance of 7 in. and 9 in. measured to the center from the top surface. The beam has two 3/8 in. prestressing strands at the top and four 1/2 in. prestressing strand layout near the bottom and no strands are harped. All strands have a jacking stress of 75% of f_{pu} . The beam was reinforced with #3 two legged stirrups at 9 in. spacing throughout the span of the beam except for the ends and near the opening where the beam was designed for bursting reinforcement due to prestressing and included stirrups at 2 in. c/c spacing as shown in Figure 3, bursting reinforcement calculation can be found in Appendix B. The beam had a specified design compressive strength of 8500 psi and had reached a strength of 8170 psi on the test day.

3.2 *Hand Calculations*

3.2.1 *Flexural Capacity*

Since the maximum bending moment occurs at midspan and there is an opening between the two applied loads, the flexural capacity of the critical section near the opening was determined using a Whitney stress block approach by considering the equilibrium of concrete compression and tension in the strands. Due to the highly reinforced section (necessary to control cracking and stress at release within the code and competition limits), flexural failure was expected to be brittle by concrete crushing instead of ductile by yielding of the strand. To sharpen the pencil in the prediction, the concrete cylinders strength (8,170 psi) obtained on the day of testing and was used. The predicted nominal moment capacity was 149 k-ft corresponding to a total applied peak load of 39.5 kips. According to ACI318 & PCI Design Handbook, the predicted cracking moment was 82.12 k-ft and a corresponding total applied service load of 21.32 kips.

The flexural capacity at midspan was also determined according to an alternative approach which assumes a ratio of top strain to peak compressive strain, uses the corresponding stress block factors, and then iterates on the depth of neutral axis to find equilibrium of tension and compression. The predicted nominal moment capacity according to this approach was 153.5 k-ft and the predicted total applied load was 39.8 kips. For the cracking moment, the precompression strain and the decompression force were first determined using the geometry, jacking stress, and approximate prestress losses. Using the cracking strength from the tensile splitting strength test, and therefore the cracking strain, we could correlate with the elastic moment-curvature equation and obtain the cracking moment prediction of 84.3 k-ft, equivalent to a total applied load of 21.4 kips. Since there was also a smaller opening near one load point, the flexural capacity at the middle of this opening was also checked and was found to be higher than the midspan flexural capacity ($2P_s = 25.03$ kips, $2P_u = 40.3$ kips) and therefore didn't govern according to AASHTO LRFD.

Detailed calculations of cracking moment, prestress losses, and ultimate flexural capacity can be found in Appendix B.

3.2.2 *Layered sectional analysis*

The layered sectional analysis (LSA) technique was also used to predict the flexural capacity of the beam. LSA is a powerful method for estimating the strength of concrete members by subdividing the cross-section into thin horizontal layers, each of which can be analyzed individually. This process is analogous to numerical integration, where the finer the layer discretization, the higher the accuracy of the results. Each layer contributes an internal force and, consequently, a moment to the section. The key objective is to iterate on the depth of the neutral axis until an axial force equilibrium is achieved across the section.

One of the main advantages of this method is its ability to incorporate the full stress-strain behavior of each material, rather than relying on simplified assumptions from design codes. This enables more refined predictions that accurately reflect the actual material performance observed during testing. In our case, the analysis was automated using a MATLAB script, allowing the use of hundreds or even thousands of layers. The script also verifies that no material property limits are violated at any point across the section.

For our beam, the failure mode was compression-controlled, with crushing of the top concrete fibers occurring at a strain of 0.003. The script automatically generated the strain profile and the stress distribution along the height of the beam (see Figure 8). An additional benefit of the LSA method is its ability to produce a moment-curvature diagram, which can be used to compute equilibrium conditions for a series of increasing concrete strains. The script used for this project is included in Appendix B.

For the material models, we employed a parabolic stress-strain relationship for concrete and a simplified bilinear model for prestressing steel, as recommended in the PCI Design Handbook. Using these assumptions, the estimated maximum load was found to be 40.8 kips.

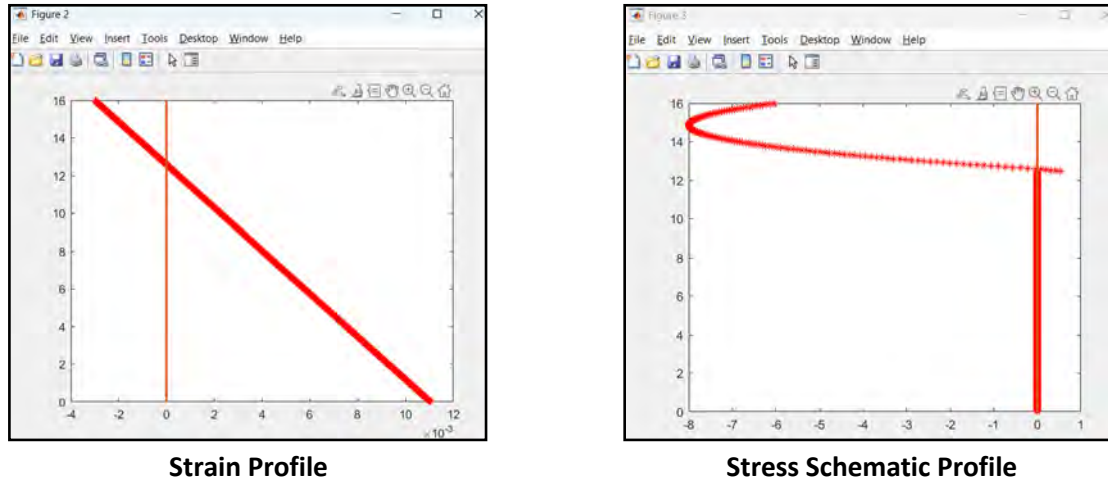


Figure 8. Strain and stress profiles at failure (LSA method)

To validate the MATLAB results, the same section was modeled using Response, a specialized tool for sectional analysis that allows the incorporation of more advanced material models such as compression softening and tension stiffening. While Response predictions were slightly more conservative, they remained in good agreement with the internal LSA results. Figure 9, shows the Response interface and the resulting moment-curvature diagram.

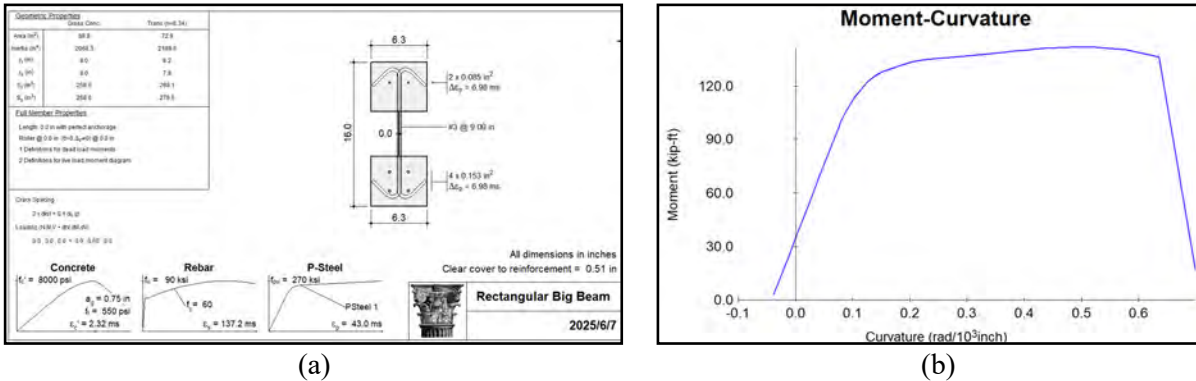


Figure 9: a) Response model interface. b) Moment curvature from response.

3.2.3 Shear Design

The beam was designed to resist shear and ensure flexural failure near midspan in the constant moment region. The evaluated shear capacity at midspan according to ACI 318 and AASHTO 2019 are 27.1 kips and 28.1 kips respectively. To have a sufficient factor of safety against shear failure, the beam was reinforced with #3 rebar at a minimum of 9 in. spacing. Shear capacity was evaluated at critical locations along the beam: at the two loading points, at the center of the large opening, and at the small openings distributed along the span. Since neither ACI 318 nor AASHTO provides direct guidance for calculating shear capacity in sections with web openings, the net concrete area approach was adopted—subtracting the area of each opening from the gross cross-sectional area to estimate the effective shear capacity. For the big opening the expected shear force is less than 4 kips while the calculated concrete shear capacity at that section is around 10.3 kips without considering the effect of the prestressing using ACI 318, therefore no

shear links were provided in the top or bottom part of the big opening. For the opening located closest to the support, the applied shear force was significantly higher than the corresponding moment, and code-based predictions would suggest a potential failure in the absence of stirrups. However, due to the presence of stirrups spaced at 9 inches—effectively reinforcing both sides of the opening—it was conservatively assumed that the shear capacity at this location is comparable to that of a section with uniform 9 in. c/c stirrup reinforcement. The results are tabulated in Table 9 and the locations of sections are shown in Figure 10. Detailed calculations can be found in Appendix C.

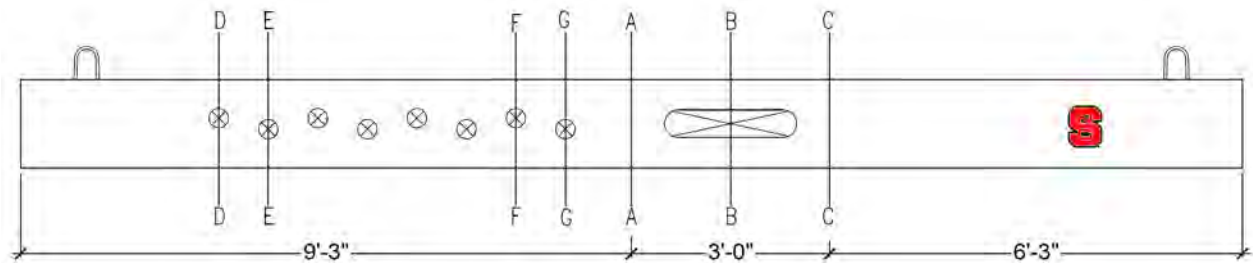


Figure 10: Checking shear at critical locations

Table 9: Shear strength calculations

Section	V	M	M/V	No Shear Reinforcement				Min. Shear Reinforcement			
				ϵ_s	V_u	P_{Shear}	F.S. $\cdot$ Shear	ϵ_s	V_u	P_{Shear}	F.S. $\cdot$ Shear
A-A (*L)	5P/6	7.5P	9.0	0.00106	16.55	19.86	0.90	0.00425	23.44	28.12	1.21
A-A (**R)	P/6	7.5P	45.0	0.00568	5.75	34.50	1.36	0.01211	8.77	52.62	2.05
B-B	P/6	7.25P	43.5	0.00557	5.89	35.36	1.35	0.01166	8.85	53.07	2.00
B-B (**M)	P/6	7.25P	43.5	0.00339	4.84	29.01	1.11	0.01090	8.48	50.86	1.92
C-C (L)	P/6	7P	42.0	0.00539	6.01	36.06	1.33	0.01173	9.19	55.14	2.01
C-C (R)	7P/6	7P	6.0	0.00195	26.44	22.66	1.05	0.00282	29.12	24.96	1.08
D-D	5P/6	55P/24	2.8	0.00000	30.20	30.2	0.70	0.00090	43.49	43.49	0.96
D-D (M)	5P/6	55P/24	2.8	0.00000	26.32	26.3	0.61	0.00074	42.71	42.71	0.95
E-E	5P/6	35P/12	3.5	0.00000	30.20	30.2	0.81	0.00138	38.42	38.42	0.98
E-E (M)	5P/6	35P/12	3.5	0.00000	26.32	26.3	0.71	0.00122	37.69	37.69	0.96
F-F	5P/6	145P/24	7.3	0.00287	24.81	29.77	1.14	0.00344	26.32	31.58	1.13
F-F (M)	5P/6	145P/24	7.5	0.00226	22.52	27.96	1.07	0.00338	25.39	31.52	1.13
G-G	5P/6	20P/3	8.0	0.00091	18.03	21.64	0.89	0.00379	24.97	29.96	1.17
G-G (M)	5P/6	20P/3	8.0	0.00024	16.86	20.24	0.82	0.00329	24.40	29.28	1.12

*L indicates slightly left of loading point

**R indicates slightly right of loading point

***M indicates concrete area is modified by removing the area of opening

3.2.4 Serviceability Stress Limits

Serviceability can be divided into three stages, first at release (with the beam flatwise), second at lifting, and third in the test. In order to reduce formwork cost, the beam was cast on the side, and this made the stresses at release more critical because the self-weight of the beam was not contributing to reducing the top tension stresses. Therefore, to achieve the stress limit at release, the required concrete strength at release was around 6 ksi. It turned out this relatively high release strength was no problem as Tindall planned to cast the beam on a Friday and release it on Monday. Second, lifting was implemented in two steps starting with rotating the beam upright using all four lifting loops to avoid any extra load on the section at the opening. After that, the beam was oriented on its strong axis, and allowed for lifting from the two end lifting loops only. The last stage was to make sure the required cracking load reached over 20 kips, which led us to monitor the concrete strength over time to make sure it was sufficient before testing the beam. The

prediction of the cracking load was adjusted to 21.5 kips once the day-of-testing concrete strength of 8,170 psi was considered.

3.3 Finite Element Modelling

3.3.1 Modelling in VecTor2

The Big Beam was modelled using a nonlinear finite-element modelling program VecTor2 to validate the hand calculation approaches. The software is based on the Modified Compression Field Theory (MCFT) and includes constitutive models for a variety of second-order effects including compression softening, tension stiffening, and tension softening. The concrete properties used are from the laboratory tests performed at the precast plant and at the laboratory. The rebar and prestressing strand properties used were obtained from mill certificates provided by Tindall. The beam supports and load points were modelled as steel plates. Instead of considering stirrups as smeared reinforcement, all stirrups were individually constructed in the model. For FEM analysis, a mesh size of 30 mm $\times$ 30 mm with a maximum aspect ratio of 1.5 were used. However, due to geometric complexities near the opening, triangular elements were also implemented. VecTor2 predicts failure by crushing of concrete in the compression region at midspan at the ultimate load, as marked by green elements shown in Figure 11 that indicate high compressive stress. VecTor2 predicts a cracking load of 24.7 kips and an ultimate load of 39.6 kips. The ultimate load predicted by FEM is in good agreement with the test results as VecTor2 can accommodate the moment-shear interaction near the mid opening accurately. The applied load vs midspan deflection predicted by VecTor2 is shown in Figure 12. In the prediction submitted, the team predicted a mid-span deflection of 0.5 in. based on the results given by Vector2. However, it was an oversight for the team to not add the magnitude of the camber (shown as negative by Vector2) which would have resulted in a total predicted deflection of 0.66 in., much closer to the actual deflection of 0.89 in. The graphical outputs of the finite-element model is found in Appendix D, along with the deformed shaped and cracking response at critical load levels.

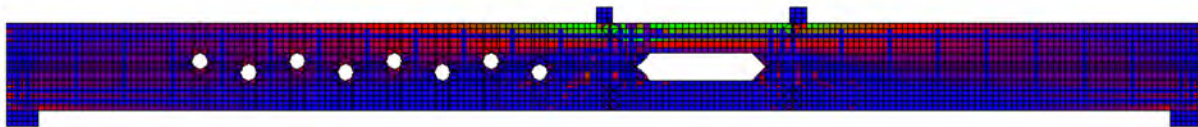


Figure 11: Modelling big beam in VecTor2

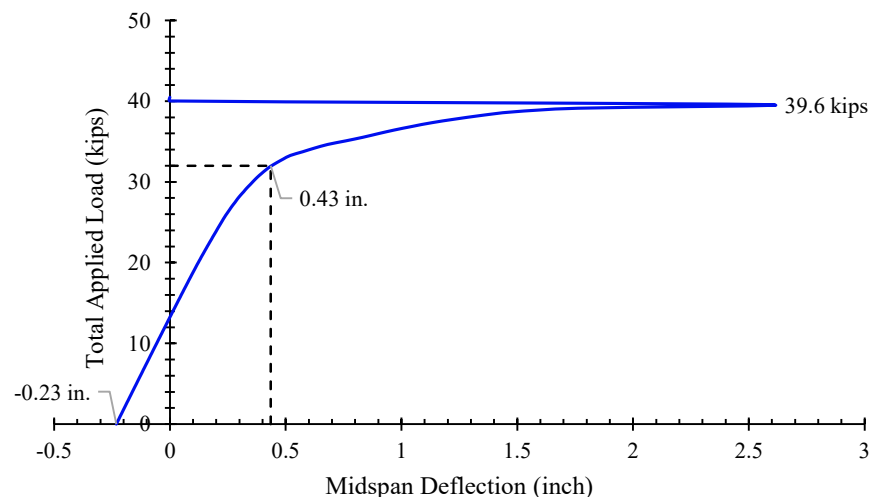


Figure 12: Load-midspan deflection predicted by VecTor2

4 Fabrication

For fabrication and casting of the Big Beam, the team had the opportunity to visit the plant of Tindall Corporation in Spartanburg, South Carolina where we were initially briefed about plant safety regulations, the plan of work, and an overview of the different kinds of precast projects regularly conducted at the plant. Figure 13 shows the NCSU Big Beam Team in their PPE with Tindall ready for Big Beam fabrication.



Figure 13: Site visit to Tindall Corporation

4.1 *Crafting the Formwork*

Formwork is an essential part of precast. The team's objective was to have formwork that can sustain the pressure of fresh concrete at low cost. After discussions with our PCI producer, it was decided that the Big Beam would be cast flatwise in order to reduce the contact surface and therefore reduce formwork cost. Although the beam was rectangular, it had openings along the span. For the openings near midspan, foam was cut to the shape of the opening was used to block the width of the beam. For the smaller openings, 3 in. nominal diameter PVC pipes were cut and placed along the span. All joints in the formwork were sealed with caulking to prevent any leaks. The team finally measured and verified the dimensions of the formwork and opening sizes and locations as part of quality control. The formwork was set on a prestress bed at the Tindall plant as shown in Figure 14.

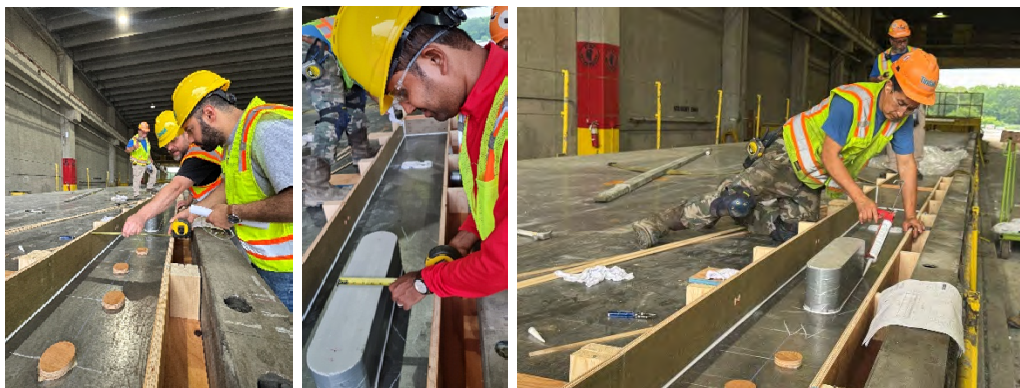


Figure 14: Setting the formwork and verifying dimensions

4.2 Placing Reinforcement

The next step included laying down the stirrups at both ends and the prestressing strands were looped in through the set of stirrups. The ends of the formwork were drilled with holes according to the strand pattern, with the preference from the plant to have a 2 in. spacing between strands which was accommodated in the team's design. The top strands had a cover of 2.25 in. and the bottom most layer of strands had a cover of 1.75 in. It was essential that the strand pattern was symmetrical so that there were no concerns about sweep after destressing for such a small-scale beam. The strands, pulled through and chucked at both ends, were then jacked to full tension after inspection. A hydraulic jack pulled the strands sequentially by reacting against the bulkhead of the prestressing bed and stressed to 75% of f_{pu} . The 3/8 in. and 1/2 in. diameter strands were each stressed to 23 kips and 41.3 kips respectively. During this procedure, as part of plant safety regulations, the team observed the procedure from a distance.

Two legged closed stirrups with a 135° hook were used. The locations of stirrups were marked in the formwork and were placed according to the design spacing. The stirrups were then tied to the strands using a rebar tie gun at their four corners. Both the NCSU team and support staff from Tindall Corporation built the rebar cage together and it was later approved by the quality control manager. In addition to lifting loops at the ends, since the beam had to be rotated to vertical position, two additional lifting loops were added to the ends of the largest opening to prevent torsion during lifting. In order to prevent the PVC pipes from toppling over during concrete pouring, two #3 rebars were temporarily placed above the pipes and tied to the stirrups, to be later removed just before the concrete started to set. Figure 15 shows all the steps involved in placing the reinforcement within the formwork.



Figure 15: Placing reinforcement within the formwork

4.3 Concrete Casting

The concrete was batched at the plant. After performing necessary quality control checks, the concrete was gradually poured into the formwork starting from one end. Due to light shear reinforcement and high workability, the mix flowed into the formwork very smoothly. As the mix was poured, it was lightly vibrated for compaction to release any trapped air bubbles. Finally, a wooden screed was used to level the surface initially, which would later be further smoothed using trowels for the final finish. Figure 16 shows the beam being cast at the plant.



Figure 16: Casting the Big Beam using an architectural mix

4.4 Release and Delivery

The team had to be careful about the destressing sequence at release as the member was cast flatwise, and therefore, was oriented in its weak-axis orientation with a stand pattern designed primarily for the self weight in a strong-axis orientation. The release sequence involved cutting one of the top strands first followed by releasing a strand from the upper layer of bottom strand profile so that the stresses are balanced accordingly. After casting, the beam was tilted vertical, and the surface was sand blasted before the beam was successfully delivered to the NCSU Constructed Facilities Laboratory on May 23rd without any damage. Figure 17 shows the NCSU big beam delivered to the CFL Laboratory.



Figure 17: Handling the Big Beam at the plant (a) and transport to the laboratory (b)

5 Testing

The Big Beam was tested at the North Carolina State University Constructed Facilities Laboratory on Tuesday, June 10th (2025) at 2:15 pm. The age of the beam at test time was 25 days. To measure the cracking load, prescribed deflection, and ultimate flexural capacity, the beam was tested in four-point bending per the guidelines of the PIC Big Beam Competition 2024-2025. This section discusses in detail the test setup, instrumentation, and test results.

5.1 Test Setup

Figure 18 shows the test setup used. The four-point bending setup ensures a relatively constant moment region so that flexural cracks can develop, and this configuration produces a high moment-to-shear force ratio near the midspan. The two applied point loads are equal in magnitude, placed 3 ft apart, with one acting at the midspan and the other acting 3 ft from the midspan. For the NCSU design, this allowed a distance of 6 inches from the edge of the large opening to each load point. This small space is specifically important as there is no shear reinforcement in the opening, and the opening had to be a sufficient distance away from any stress concentration directly under the load points. Additional shear reinforcement near the opening in the form of bursting reinforcement was provided to ensure an adequate factor of safety against a shear failure.

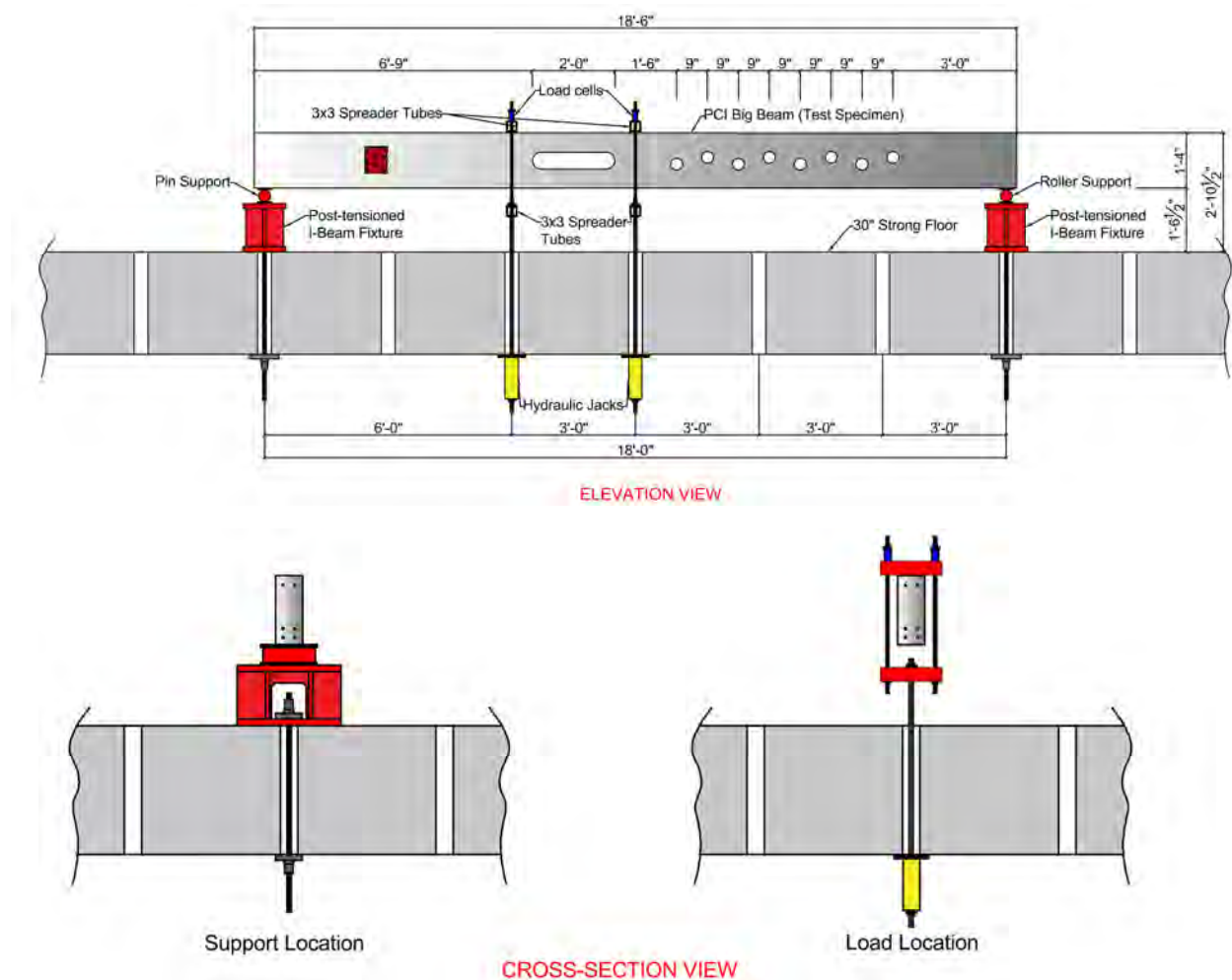


Figure 18: Test setup showing elevation and cross-section

The beam was supported with 6 in. wide, ½ in. thick steel plates on either end, grouted to the bottom surface of the beam to create a span of 18 ft. The beam was intentionally made 1 foot longer than the span to allow a 6" support overhang on each side. Support plates in turn rested on 3.5 in. diameter steel rollers to simulate pin and roller conditions. Both the pin-roller supports rested on a I-shaped steel support stands that were post-tensioned to the laboratory floor to prevent movement. Two hydraulic cylinders, each having a capacity of 60 kips, reacted against the laboratory floor from underneath, and pulled down on the Big Beam using steel spreader tubes placed above and below the beam in the configuration shown in Figure 18. This configuration ensured equal load at both load points. A hydraulic pump was used to pressurize the two cylinders equally.

5.2 Instrumentation

The girder was instrumented to measure the applied load, displacement field for the entire beam using digital image correlation (DIC) and vertical displacement at 4 locations as shown in Figure 19. Four load cells, rated for 50 kips, were placed at each end of both steel tubes that were located above the beam. The combined load cell reading was determined and the total applied load. The entire span of the beam was monitored using three-dimensional DIC equipment. The load cells and DIC system were connected to a data acquisition system (DAQ) that recorded data at 0.5 Hz. Throughout the loading, the DIC system measured the three-dimensional displacement field of the span by capturing high-resolution photographs of a speckle pattern painted on the surface. In addition to the DIC system, four string potentiometers were placed at selected locations along the span, two at the loading points, one at the middle of the large opening, and one at an opening along the longer span.

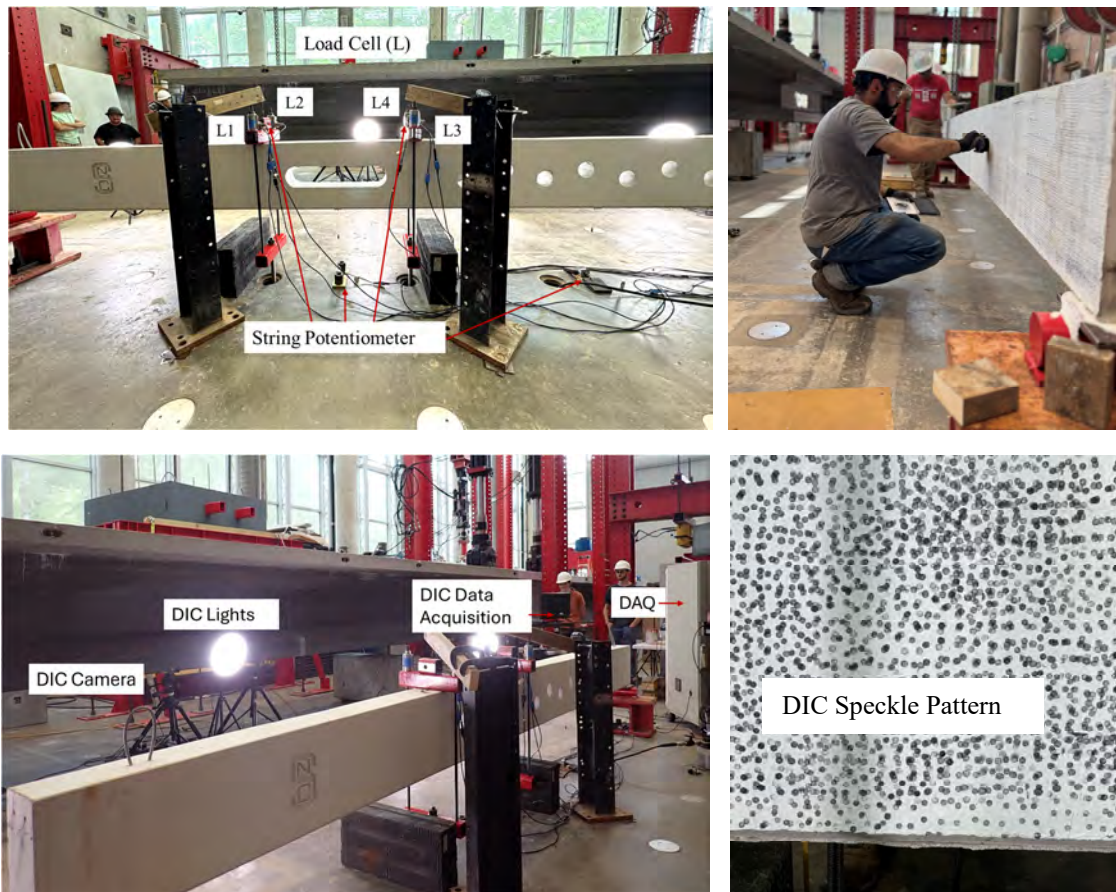


Figure 19: Instrumentation for measuring load and deflection

The combined DIC system used two systems of two cameras. Although it would have sufficed to use one system to cover only the constant moment region, the team desired to monitor the entire span to see how large-scale prestressed girders behave with and without openings since there have been few studies on that topic aspect. Software was used to create a displacement field from calibrated DIC photographs, with the software essentially comparing the images taken during loading pixel-by-pixel to determine the movement of the speckle pattern relative to a reference image taken before any load is applied. The displacement field was used to determine the cracking load and deflection, and also mapped the strain of the entire beam for academic purposes. To synchronize the timing between the conventional instruments and the DIC, a simultaneous voltage pulse is sent to both, with the time of the pulse recorded to enable synchronization of the data.

5.3 Loading Protocol and Structural Response

The beam was loaded gradually and monitored carefully for any cracks. As load increased, tensile flexural stress overcome the precompression stress observed via initial camber. When the applied load generates tensile stresses higher than the tensile capacity of concrete, flexural cracks will develop. The team decided to load the beam until the first crack could be visually observed. Loading was paused at 26 kips when there was clear non-linearity in the load-displacement behavior. Upon observation, multiple cracks were observed. The cracks were marked and the widths were measured to be 0.01 mm. Since the cracks were very small, it wasn't possible to detect them visually earlier. The beam was loaded to 30 kips so that the cracks could grow and then the beam was unloaded to zero. Unloading was completed because the crack reopening moment can be determined later to help calculate prestress losses accurately following the methods developed in Saha et al. (2024). It is noted that the ultimate load is not impacted by unloading and reloading to a level just after cracking as detailed in Saha et al. (2024). Since large scale prestressed concrete beams are difficult to construct for research purposes, the team decided to take this competition as an opportunity to do further analysis using detailed displacement field data obtained from DIC and work towards publishing the findings from the test. Hence the aforementioned approach was followed. After unloading, the beam was again monotonically loaded to 32 kips, and the loading was paused so that the team could mark the growth of cracks. Loading was continued to 35 kips and then the beam was loaded to failure. The beam reached an ultimate load of 39.96 kips and failed abruptly by crushing of the compression region at midspan. The load vs midspan deflection for the entire loading protocol is shown in Figure 20. It's worth highlighting that even with the openings in the left span, the prestressed concrete Big Beam successfully failed in flexure at midspan. This not only allowed us to save material, but is also significant to show that such geometry is possible in a prestressed concrete beam.

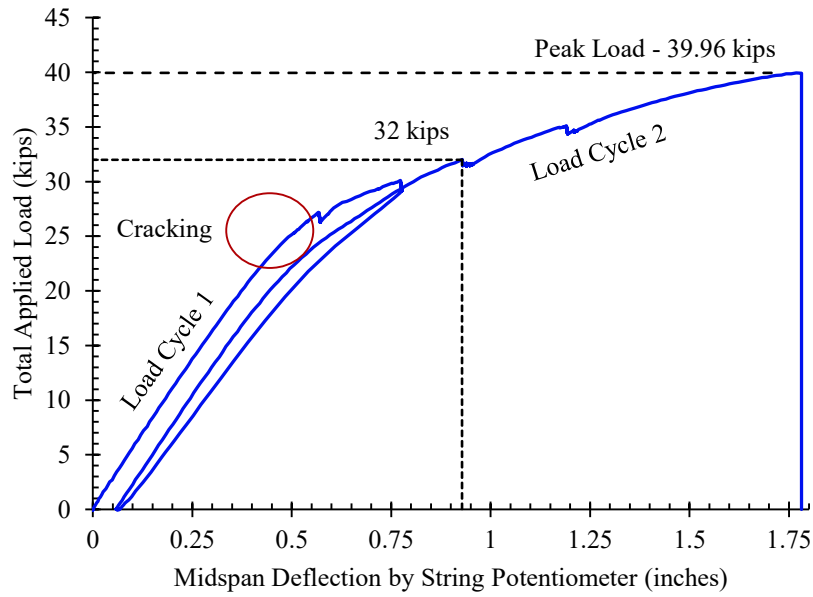


Figure 20: Loading protocol

Figure 21 shows the Big Beam before the test and after failure.

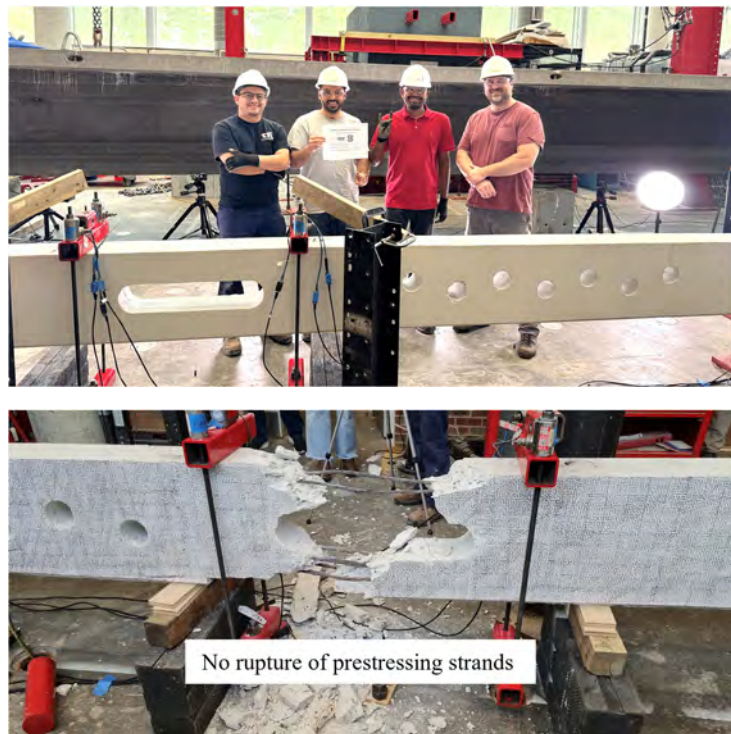


Figure 21: Big Beam: before (above) and after (below) failure

6 Results and Analyses

The cracking load, midspan deflection at 32 kips and the peak load were corroborated using DIC data. The DIC system captures high resolution pictures throughout the loading protocol and converts the images into detailed displacement field data. Since two DIC camera systems were used with overlapping fields of view (one system at each end of the beam), loads and deflections measured from each system were averaged. The load vs midspan deflection plot obtained by post-processing the DIC data from the two spans are shown in Figure 22. The first crack forms when the load-displacement transitions from linear to non-linear behavior. The cracking load was determined by finding the point of intersection of two tangents plotted across the uncracked and cracked stiffness of the measured response. The first phase of the loading protocol was used to determine the cracking load since the crack reopening moment is slightly smaller than the cracking moment. In the second phase of loading, when the beam was loaded above the cracking load, the inelastic cracked response dominated behavior, with the stiffness of the member softening considerably. DIC measurement shows the beam cracked at 21.49 kips and had a deflection of 0.89 inches at midspan at 32 kips. The beam finally reached a peak applied load of 39.96 kips as measured by the load cells and it crushed in the compression region and failed.

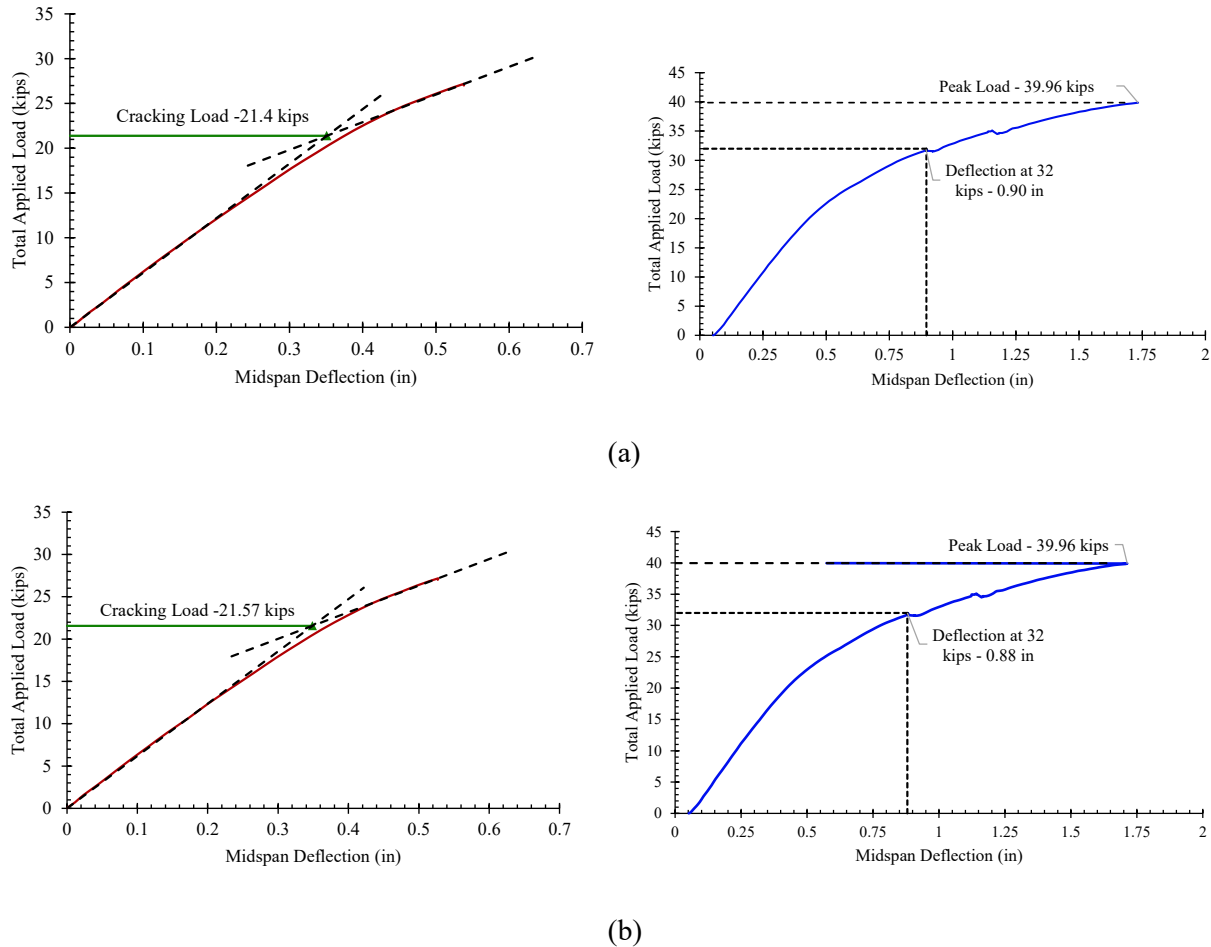
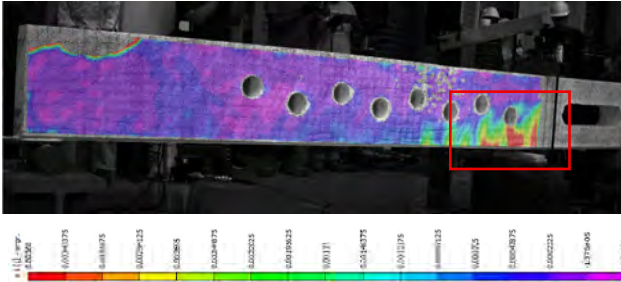
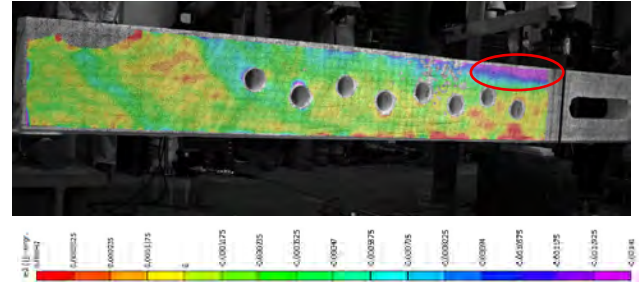


Figure 22: Load-midspan deflection obtained from DIC (a) Span with small openings (b) Span with larger opening between loading points

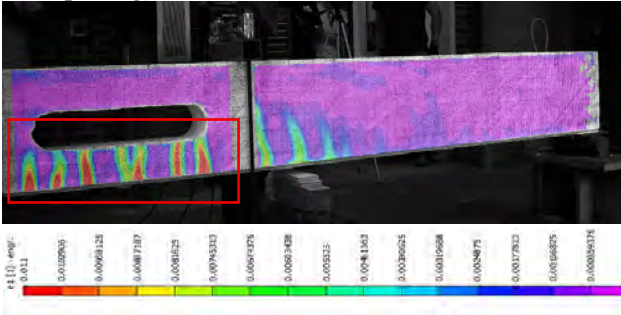
DIC principal strain maps at peak for both spans are shown in Figure 23. The principal tensile strain maps provide visual confirmation of the growth of cracks throughout the loading protocol. The principal strain maps show very high strains between the loading points, mapping the shape of multiple flexural cracks extending from the bottom to the face of opening. On the other hand, the principal compressive strain map indicates high strain concentrations near the top compression zone, as shown by the purple coloration. This suggests that the beam was nearing concrete crushing, which was consistent with the observed failure mode.



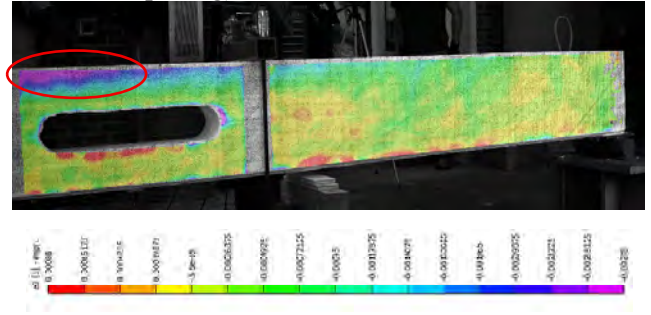
(a) Principal Tensile Strain at Peak (Span with small opening)



(b) Principal Compressive Strain at Peak (Span with small opening)



(c) Principal Tensile Strain at Peak (Span with large opening)



(b) Principal Compressive Strain at Peak (Span with large opening)

Figure 23: Principal strain map obtained from DIC

For small-scale deformations, DIC gives an accurate measurement of deformation and therefore, DIC results were used for comparison with our prediction. Using DIC, it was possible to subtract any support displacements from midspan deflection. The use of traditional instrumentation validates the DIC measurements. Figure 24 shows the measured load-displacement response of the beam at midspan using deflections obtained from the string potentiometer in both phases of the loading. The cracking load was 21.99 kips using this approach, as shown in Figure 24(a). At 32 kips, the member showed a midspan deflection of 0.93 in. from the conventional instrumentation, which included effects of minor support and loading system deflections. A maximum displacement of 1.78 in was recorded by the string potentiometers (see Figure 24b). The aforementioned measurements from traditional instrumentation conform well with the measurements obtained from DIC and therefore validates our approach. Overall, the Big Beam showed excellent agreement with the intended design and pre-test evaluations, with deflection at 32 kips being the lowest accuracy prediction.

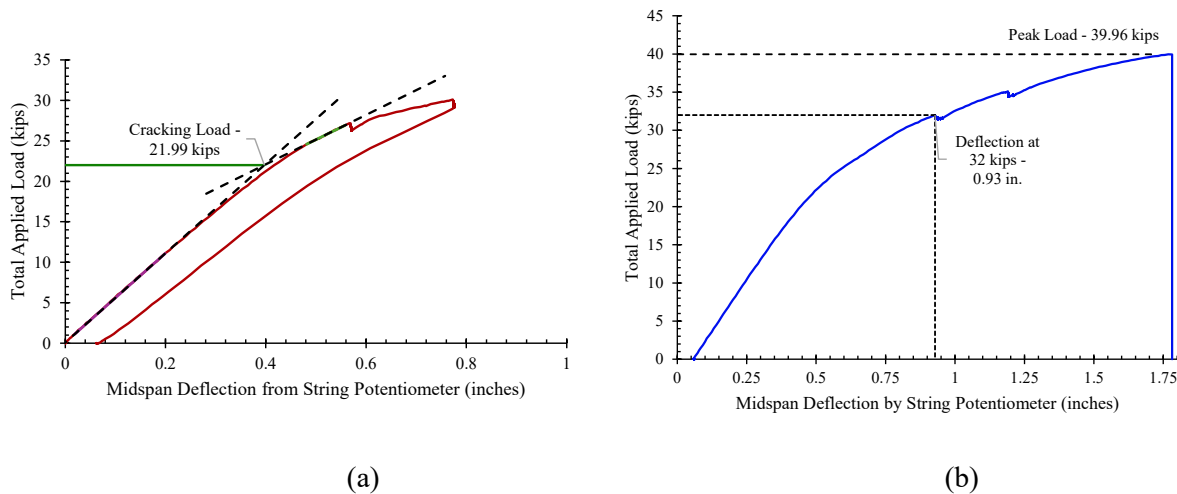


Figure 24: Load-midspan deflection (a) Cracking load measured at first load cycle (b) Deflection and ultimate load measured from second load cycle

A summary of predictions by the various methods, test results, and official predictions for judging is listed in Table 10.

Table 10: Summary of predictions by codes, FEM software, test results and official predictions

<i>Approach</i>	<i>Cracking Load (kips)</i>	<i>Max. Applied Load (kips)</i>	<i>Deflection at 32 kips (in)</i>
ACI 318 & PCI Design Handbook	21.32	39.50	-
AASHTO LRFD	21.40	39.80	-
Layered Sectional Analysis	21.7	40.8	-
VecTor2	24.70	39.60	0.43
Test result with Traditional Instrumentation	21.99	39.96	0.93
DIC Test Results	21.49	39.96	0.89
Official Predictions as Submitted	21.50	39.90	0.50 (submitted prediction mistakenly not considering camber) 0.66 (considering camber)

7 Team Statements:

Anindya Samya Saha –

Participating in the PCI Big Beam Competition was a valuable and rewarding experience that deepened my understanding of precast concrete. I was particularly excited to visit Tindall Corporation's precast plant in Spartanburg, where I had the opportunity to work closely with experienced engineers during the casting of our beam. This hands-on experience highlighted the contrast between theoretical learning, laboratory practices, and real-world field applications—especially in terms of detailing, efficiency, and construction tolerances.

The site visit enhanced my understanding of the types of construction typically performed in a precast facility and the methods used in their execution. It was fascinating to witness our designed beam being fabricated and cast from the ground up. The use of an architectural concrete mix stood out to me due to its exceptional workability and rapid strength gain. This introduced me to the precise role of admixtures such as plasticizers and the importance of accurate mix design. Because our beam incorporated unique architectural features like openings, the competition evolved into a research-driven initiative. This allowed us to go beyond standard code provisions and explore innovative design concepts. One of the most exciting moments was achieving 0% error in our predictions of cracking and ultimate capacity. This reinforced my respect for structural codes and the level of confidence they offer in ensuring the safety and performance of concrete structures.

I am deeply grateful to my teammates and our advisor, Dr. Lucier, whose expertise, guidance, and dedication were instrumental in the success of the Prestressed Wolfpack team. Knowing that this project came with stringent requirements made the challenge even more motivating—and I thoroughly enjoyed taking it on with full commitment, as I thrive on challenges that push me to grow.

Khaled Al-Sakaji

Participating in the PCI Big Beam Competition after years of working in the industry as a structural design engineer made this an exceptionally valuable experience. It bridged the gap between theoretical design and actual structural behavior, encompassing every phase, from schematic design and fabrication to casting and finally testing the beam. Collaborating with the Tindall team was especially rewarding; their expertise and enthusiasm encouraged creative thinking and helped bring our bold ideas to life.

Departing from the conventional I-beam design, we embraced an innovative concept made possible by Dr. Lucier support and guidance along with Tindall's confidence in their architectural white concrete mix, an exceptional material that aligned beautifully with our vision. Visiting their Spartanburg, SC facility and witnessing the quality and precision of their precast production made the experience even more meaningful.

Our design featured eight circular openings and a large central opening between the applied loads, presenting significant structural challenges. Tackling this complex geometry pushed our design to the limits and required reconciling analytical models with real-world performance. It tested our technical skills, deepened our understanding of material behavior, and reinforced the importance of close coordination throughout the design and fabrication process. This competition was far more than a theoretical exercise, it fostered technical growth, strengthened collaboration, and deepened my appreciation for the full lifecycle of structural elements. It ultimately helped shape me into a more thoughtful, well-rounded, and effective engineer.

Paul Acuna

Participating in the PCI Big Beam Competition has been one of the most complete, challenging, and rewarding academic experiences I have ever had. The unique structure of this competition pushes students beyond the classroom by requiring us not just to design for strength, but to accurately predict performance based on real material behavior. This emphasis on precision led us to carefully study concrete and steel properties, refine our prediction tools, and deepen our understanding of structural mechanics and experimental work.

One of the most valuable aspects of the competition was the direct collaboration with precast producers and professional engineers. Working with Tindall Corporation gave us a unique opportunity to engage with industry experts, ask questions, and see how decisions are made on the ground. These insights are incredibly meaningful as we grow into our roles as structural engineers.

The hands-on learning in the lab was equally enriching. From designing the test setup to instrumenting the beam and recording data, every stage demanded technical knowledge, attention to detail, and teamwork. The competition simulates a real-world engineering project, where timelines, collaboration, and performance all matter.

Beyond the technical takeaways, this journey strengthened the bond between teammates. We learned how to navigate differences, support each other, and work together under pressure. I will always remember the moment after the test, when a group of students from across the lab, gathered around our beam to celebrate with us. That energy and sense of community is something I will treasure.

A heartfelt thanks to my teammates for their trust and dedication, to Tindall Corporation for their generous sponsorship and mentorship, and to Dr. Greg Lucier, the best lab manager and advisor we could ask for. Thank you also to the CFL staff and students who supported us along the way. Finally, thank you PCI for creating this one-of-a-kind opportunity for students to grow, lead, and learn.

Acknowledgments

The NC State University team sincerely thanks the entire Tindall Corporation team for their outstanding support throughout this project. We are especially appreciative of Mr. Matt Thorne, Mr. Mike Milkovitz, Farshad Shokrgozar, Mary Ann Griggas-Smith, the entire QC Department, and the entire fabrication team for their expert guidance during the design phase, hands-on assistance with fabrication, and for inspiring us to explore innovative ideas with our Big Beam.

We cordially thank Taylor Brodbeck, Mohammad Qambar, and Johnathan McEntire at the NC State University Constructed Facilities Laboratory for their continuous support throughout the process. A special note of appreciation goes to our faculty advisor, Dr. Greg Lucier, whose enthusiasm, guidance, and timely intervention have been instrumental in helping us tackle this one of a kind Big Beam Test.

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A1: Concrete Mix Properties

Key						
	= SSD Weights					
	=SSD Volume					
	Wet Test Information					
Test Time	Spread	VSI	Concrete Temp	Ambient Temp	Unit Weight	Air
8:45	22	0	78	72	143.95	5.6

Specified Comp. Strength (f'cr) =	6,000 psi @ 28 Days	Slump =	22	+/- 1.5"
Release Compressive Strength =	3,500 psi @ 12-16 hrs.	Air Content =	5.0%	+/- 1.0 %
Max. Initial Set (ASTM C403) =	5 Hours	Unit Weight =	145.7	+/- 2 lbs.

MATERIALS								
Material	Type	Supplier	Specification	SSDSG	Abs%	Free%	Other Data	Item #
Lehigh Alborg White Cemen I/II		Aalborg - Tampa Fla.	ASTM-C150	3.15	0	0	0	6054
Texas #7 Limestone	#7	Martin Marietta - Texas Maryland	ASTM-C33	2.834	0.0036	0.03	101.8 pcf (odv)	1337
Arrowood 1/2 Cleaned	1/2"	Martin Marietta	ASTM-C33	2.92	0.005	0.025	120 pcf (odw)	1302
Sand White Wash	Mortar Sand	American Materials	ASTM-C144	2.608	0.006	0.025	2.40 = FM	5258
RAE-260	Air	Russtech	ASTM-C260	1.2	0	0	8.9	188
Extendflo X90	HRWR	Russtech	ASTM-C494	1.07	0	0	10	161
Waterpel-S	Water Repellant	Russtech	ASTM-C494	1.02	0	0	8.9	226
VMA-758	Viscosity Modifier	Russtech	ASTM-C494	1.02	0	0	8.9	1269
Water	Tap	SWS	Potable Water	1	0	0	0	1

MIX PROPORTIONS								
Material	Dry Weight - lbs/cy	Design Quantities		Automatic Mode	Typical Batch With			Item #
		SSD Weights - lbs/cy	SSD Volume		Mixer Settings	Moist. Adj.		
Lehigh Alborg White Cement	800	800	4.07	800	lbs/cy	800	lbs/cy	6054.00
Texas #7 Limestone	1385	1390	7.86	1390	lbs/cy	1433	lbs/cy	1337.00
Arrowood 1/2 Cleaned	207	208	1.14	208	lbs/cy	213	lbs/cy	1302
Sand White Wash	1210	1217	7.48	1217	lbs/cy	1249	lbs/cy	5258
RAE-260	0.21	0.21	0.00	0.375	oz/cwt	3	oz/cy	188.00
Extendflo X90	5.86	5.86	0.09	9.375	oz/cwt	75	oz/cy	161.00
Waterpel-S	1.32	1.32	0.02	2.375	oz/cwt	19	oz/cy	226.00
VMA-758	2.23	2.23	0.03	4.000	oz/cwt	32	oz/cy	1269.00
Water	310	310	4.97	310	lbs/cy	230	lbs/cy	
Air @ 5%			1.35					
SSD Totals		3935	27.02					
W/C Ratio =		0.388						
Unit Weight =		145.7						

BATCHING REQUIREMENTS	
1.) Batch Time = 130 Seconds	
2.) Set Fine And Coarse Aggregate Free Moistures @ Mixer Using Q.C. Oven Dry Tests.	
3.) Required Concrete Temperature = 90 F, + 10 F, - 20 F	

Tindall

Better Building Through Technology

Concrete Mix Design

Tindall Corporation, Spartanburg Division

CN-A116 AMGEN

Normal Weight, f'c = 6,000 PSI

Designed And Distributed By Thomas Ardito
6/11/2025

Original Design 3/15/22

A2: Rebar Specifications



GERDAU

JS-ML-CARTERSVILLE
184 OLD GRASSDALE ROAD NE
CARTERSVILLE, GA 30121
USA

Page 1 / 1

DOCUMENT ID:
0000705143

CERTIFIED MATERIAL TEST REPORT

CUSTOMER SHIP TO ALLIED CRAWFORD GREENVILLE INC 5211 EDWARDS RD TAYLORS, SC 29687-2813 USA		CUSTOMER BILL TO ALLIED CRAWFORD GREENVILLE INC 5211 EDWARDS RD TAYLORS, SC 29687-2813 USA		GRADE GGMULTI		SHAPE / SIZE Angle / 6X4X1/2	
SALES ORDER 13824689/000230		CUSTOMER MATERIAL N°		LENGTH 20'00"		WEIGHT 9,720 LB	
CUSTOMER PURCHASE ORDER NUMBER 126622-3		BILL OF LADING 1323-0000264686		DATE 05/08/2024		SPECIFICATION / DATE or REVISION ASTM A572-14, A572-15 ASTM A617, A36-14, ASME SA-36 ASTM A709-18, AASHTO M270-15 CSA G40.20-13/G40.21-13	

CHEMICAL COMPOSITION		C (%)	Mn (%)	P (%)	S (%)	Si (%)	Cu (%)	Ni (%)	Cr (%)	Mo (%)	V (%)	Nb (%)	N (%)	Sn (%)
0.12	0.91	0.015	0.020	0.22	0.28	0.12	0.12	0.035	0.002	0.014	0.0105	0.011		

MECHANICAL PROPERTIES		G/L (Inches)	UTS (PSI)	UTS (MPa)	YS 0.2% (PSI)	YS (MPa)
8.000	8.000	74700	515	53200	367	
23.30	23.30	74500	514	53700	370	

COMMENTS / NOTES

This grade meets the requirements for the following grades:
 ASTM Grades: A36; A572-50; A709-36; A709-50
 SA Grades: 44W; 50W
 AASHTO Grades: M270-36; M270-50
 ASME Grades: SA36

Gerdau's steel is 100% recyclable. Support the circular economy through our Metals Recycling Partnership. For details, visit www2.gerdau.com/metals-recycling, or contact metalsrecycling@gerdau.com.

Shankar

SHANKAR YALAMANCHILI
QUALITY DIRECTOR

Phone: (404) 267-1071 Email: Shankar.Yalamanchili@gerdau.com

YAN WANG

QUALITY ASSURANCE MGR

Phone: (770) 587 5718 Email: yan.wang@gerdau.com

The above figures are certified chemical and physical test records as contained in the permanent records of the company. We certify that these data are correct and in compliance with specified requirements. No weld repair was performed on this material. The material has not been in contact with mercury while in Gerdau possession. For all products other than billets or beam blanks, this material was produced (Electric Arc Furnace, Melted, Continuously Cast, Hot Rolled and, if applicable, Cold-Drawn) in the USA. For billets or beam blanks, this material was produced (Electric Arc Furnace, Melted and Continuously Cast) in the USA. CMTR complies with EN 10204 3.1.



EXCELENCIA EN CALIDAD

Acería Ramos Arizpe

Carretera a Mondova Km 4 #2125 Tramo Santa Cruz-Ojo Caliente
C.P./Zip 25900 RAMOS ARIZPE, COAHUILA
Teléfono (+52) 01 818 368 1111
MX 800 831 5700, USA 1800 332 2376.
customerdea@deacero.com

/ CERTIFICATE OF TEST AN ANALYSIS

No. Certificado / Certificate No:	96460 - 25336068 - 22
Fecha / Date:	09/07/2024

Hecho en México / Made in Mexico

DATOS DEL CLIENTE / SOLD TO		CLIENTE CONSIGNADO / SHIP TO	
Cliente / Customer:	ALLIED CRAWFORD STEEL INC / SOUTH EAST	Cliente / Customer:	ALLIED CRAWFORD STEEL - GREENVILLE
Dirección / Address:	7135 BRYANWKE CIRCLE	Dirección / Address:	5211 EDWARDS ROAD
Ciudad / City:	NORTH CHARLESTON	Ciudad / City:	TAYLORS
País / Country:	USA C.P./Zip: 29418	Estado / State:	SC
Teléfono / Phone:	8435626300		
Correo Electrónico / eMail:	cparriss@mcswusa.com		

COMPOSICIÓN QUÍMICA / CHEMICAL COMPOSITION (% PESO / WEIGHT)	
Colada / Heat	Producto / Description of Goods
75168	FLAT BAR 6" x 14" A36/A529-50 20' 2.0T
74233	FLAT BAR 6" x 38" A36/A529-50 20' 2.0T
30029	FLAT BAR 6" x 38" A36/A529-50 20' 2.0T
74777	ANGLE 2" x 2" x 1/8" A36/A529-50 20' 2.0T
74929	ANGLE 2" x 2" x 1/8" A36/A529-50 20' 2.0T
74929	CHANNEL 3" x 4.1 A36/A529-50 40' 4.0T

PROPIEDADES MECÁNICAS / MECHANICAL PROPERTIES	
Colada / Heat	Producto / Description of Goods
75168	FLAT BAR 6" x 14" A36/A529-50 20' 2.0T
74233	FLAT BAR 6" x 38" A36/A529-50 20' 2.0T
30029	FLAT BAR 6" x 38" A36/A529-50 20' 2.0T
74777	ANGLE 2" x 2" x 1/8" A36/A529-50 20' 2.0T
74929	ANGLE 2" x 2" x 1/8" A36/A529-50 20' 2.0T
74929	CHANNEL 3" x 4.1 A36/A529-50 40' 4.0T

OTROS DATOS ADICIONALES / ADDITIONAL INFORMATION

/ %Elongation specimen 8 in [200 mm]	
/ 1 ppm = 0.0001%	
/ Coating Type: Uncoated	
/ Country of Origin and Exporter: Mexico	

Certificamos que este material ha sido producido, inspeccionado y probado de acuerdo a las normas de fabricación del acero aplicables a la ASTM A36-2008, A529-2005 (re aprobada el 2009), A572-2012 y A592-2011 y a las normas dimensionales NMX E252, ASTM A6/A36/A529-50 / The information on the chemical and mechanical properties contained in this certificate are average values obtained from the company's internal records, where we certify that this material has been produced, inspected and tested in accordance with the standard ASTM A6/A36/A529-50



MANUEL MARCELINO GARCIA RAMOS
Gerente de Aseguramiento de Calidad / Quality Assurance Manager

A3: Prestressing Strand Specifications



MATERIAL CERTIFICATION OF COMPLIANCE

630 National Drive
Gallatin, TN 37066
336-719-9000

TINDALL CORPORATION
3076 NORTH BLACKSTOCK ROAD
SPARTANBURG SC 29301

Bill of Lading: 00418734
Order Number: 581050
PO Number: S61178

Insteel Wire Products Company 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 A416-18. The attached test report(s) represent the result of such test(s).

3/8 (9.5mm) 7W 270 LR ASTM A416

Test Report Number: 40250310103511

Lot/Serial Numbers

1201405214338
1201405214339
1201405214341
1201405214342
1201405214344
1201405214349
1201405214351

Heat Number

8000033624

The products listed in this certification were manufactured and fabricated in the United States of America.

Insteel Wire Products Company 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 Moustaffa block pull-out test.

All Domestic Prestressing Strand was made from steel rod that was melted, manufactured, and processed completely in the United States of America. The rod was then manufactured into PC Strand in the United States at Insteel Wire Products Company plant in either Sanderson, FL; Houston, TX; or Gallatin, TN. The material meets the "Buy America" requirements of 23 CFR 635.410.

The products listed in this certification were manufactured in the U.S.A. from wire rod which was manufactured in the U.S.A.

Quality Assurance Manager:

A handwritten signature in black ink, appearing to read "Josh D. Scully", written over a horizontal line.

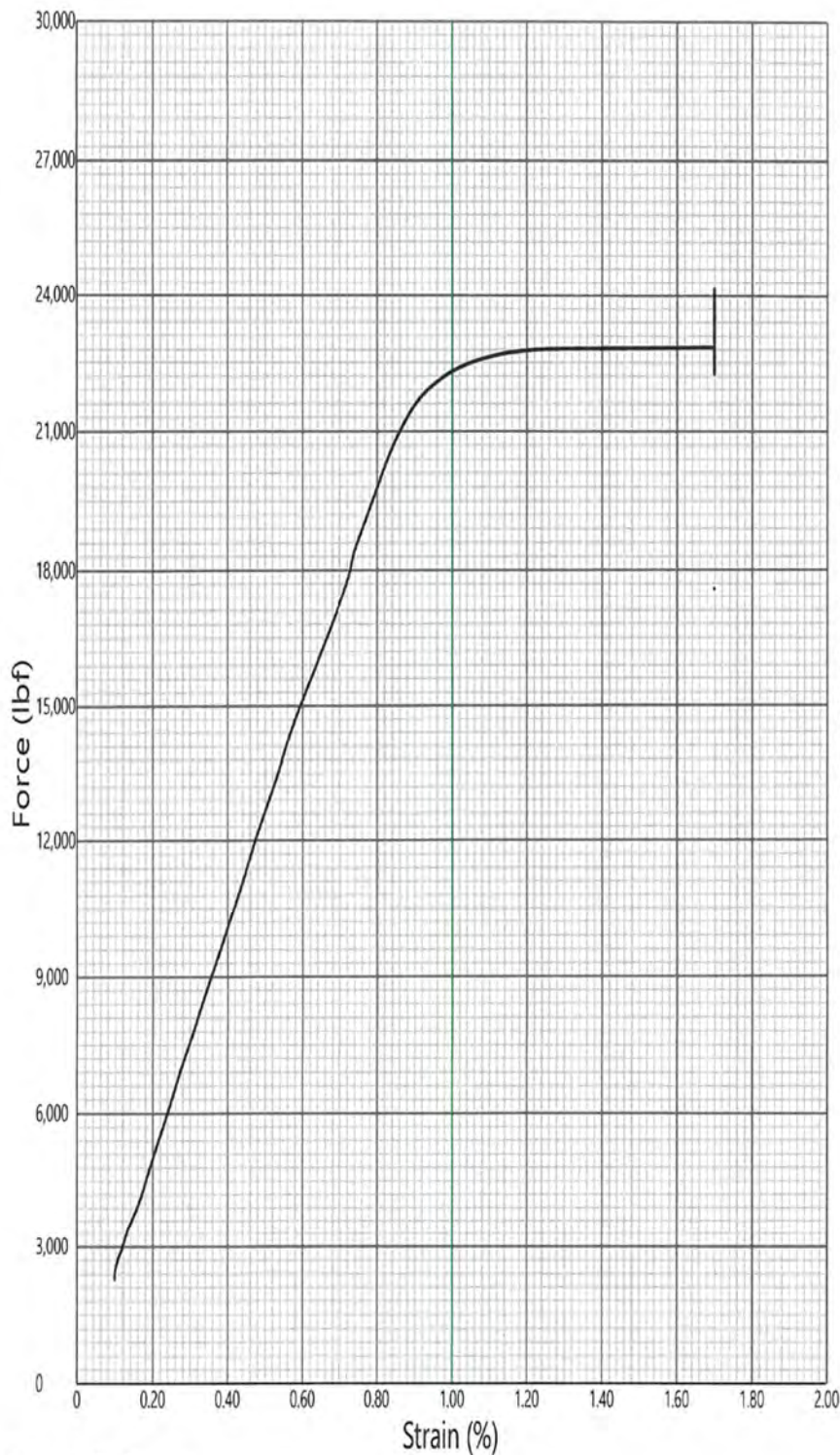
Joshua Scully

Date: 21-MAY-2025



This test performed at
630 National Drive
Gallatin, TN 37066
(800) 791-4108

Test Type: 3/8" 270 7W LOW RELAXATION



Test Number: 40250310103511
Tested By: WRC
Ultimate Breaking Strength: 24,154 lbf
Ultimate Breaking Strength: 107 kN
Load @ 1% Extension: 22,300 lbf
Load @ 1% Extension: 99.2 kN
Ultimate Elongation: 6.3 %
Strand Diameter: 0.375 in
Rep. Area: 0.0850 in²
Rep. Area: 54.8 mm²
Actual Area: 0.08582 in²
Actual Area: 55.37 mm²
Lay Length: 5.25 in
Avg Modulus of Elasticity: 28.8 Mpsi
Avg Modulus of Elasticity: 198,569 MPa
Reference:



MATERIAL CERTIFICATION OF COMPLIANCE

1 Wiremil Road
Sanderson, FL 32087
336-719-9000

TINDALL CORPORATION
3076 NORTH BLACKSTOCK ROAD
SPARTANBURG SC 29301

Bill of Lading: 00407718
Order Number: 574559
PO Number: S60481

Insteel Wire Products Company 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 A416-18. The attached test report(s) represent the result of such test(s).

1/2 (12.7mm) 7W 270 LR ASTM A416

Test Report Number: 46250331110320

Lot/Serial Numbers

1301465210678
1301465210680
1301465210681
1301465210682
1302465245420
1302465245421
1302465245422
1302465245428

Heat Number

8000034426

The products listed in this certification were manufactured and fabricated in the United States of America.

Insteel Wire Products Company 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 Moustaffa block pull-out test.

All Domestic Prestressing Strand was made from steel rod that was melted, manufactured, and processed completely in the United States of America. The rod was then manufactured into PC Strand in the United States at Insteel Wire Products Company plant in either Sanderson, FL; Houston, TX; or Gallatin, TN. The material meets the "Buy America" requirements of 23 CFR 635.410.

The products listed in this certification were manufactured in the U.S.A. from wire rod which was manufactured in the U.S.A.

Quality Assurance Manager:

Charney Rhoden

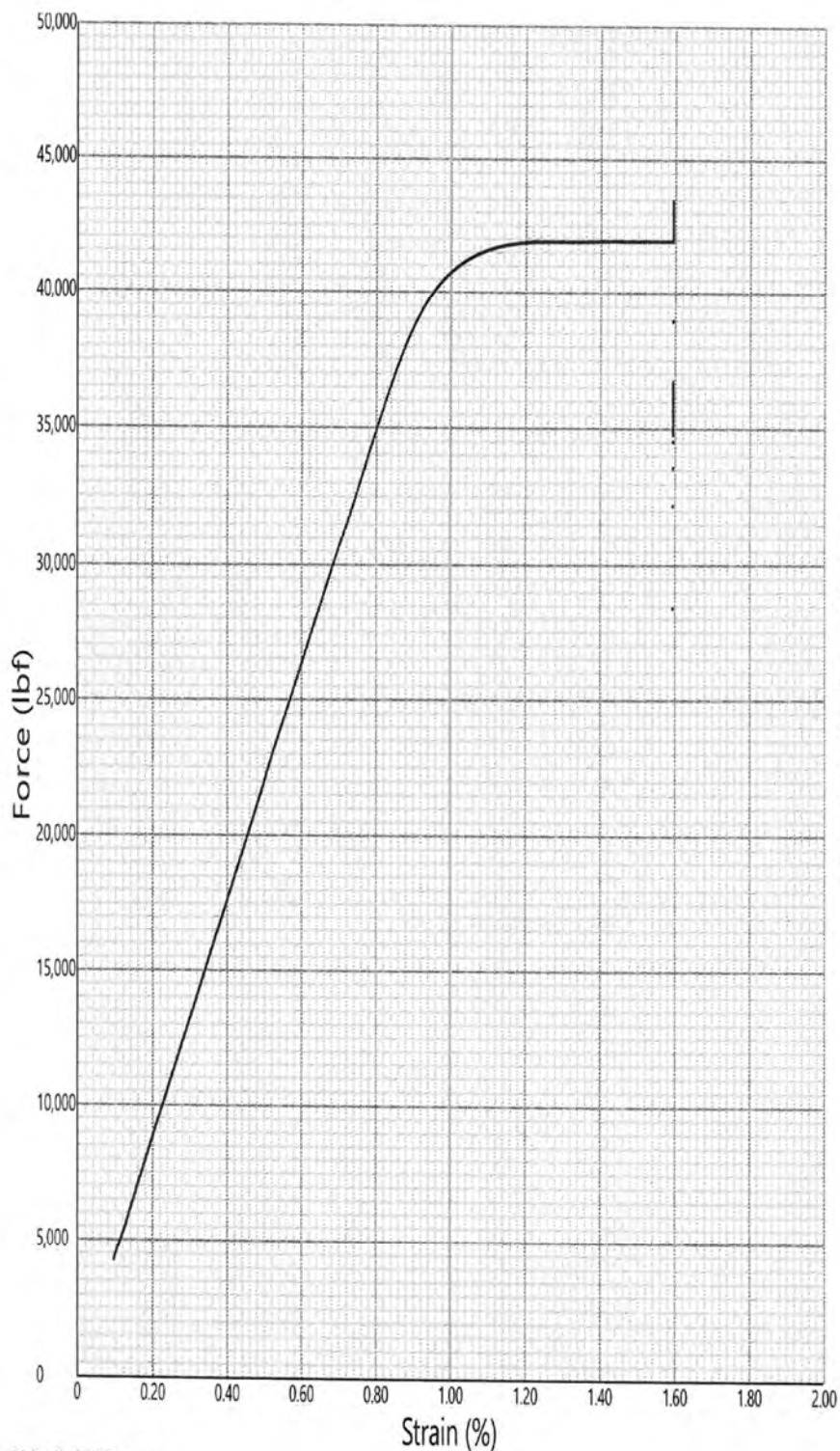
Date: 31-MAR-2025



This test performed at
1 Wiremill Road
Sanderson, FL 32087
(800) 791-4108

Test Type: 1/2" 270 7W LOW RELAXATION

Test Number: 46250331110320
Tested By: CER
Ultimate Breaking Strength: 43,456 lbf
Ultimate Breaking Strength: 193 kN
Load @ 1% Extension: 40,710 lbf
Load @ 1% Extension: 181 kN
Ultimate Elongation: 4.7 %
Strand Diameter: 0.500 in
Rep. Area: 0.153 in²
Rep. Area: 98.7 mm²
Actual Area: 0.1510 in²
Actual Area: 97.43 mm²
Lay Length: 7.60 in
Avg Modulus of Elasticity: 28.8 Mpsi
Avg Modulus of Elasticity: 198,569 MPa
Reference:



Method: Insteel Strand 7W Yield Test (rev. 113)
v10.3.1.10 - 33981SUS - Insteel Wire Products

Output: Insteel Strand 7W Yield Test (rev. 109)
Hydraulic: (398)/1.1. Printed: 3/31/2025 11:03 AM

Appendix B: Design Calculations

B1: Flexural Capacity and Stress Limit Check: ACI 318, PCI Design Handbook 8th Edition

(A) Inputs

Section Dimensions

b	6.5 in
h	16 in
Centroid from top surface, Yt	8 in
Cross-Sectional Area, Ag	104 in ²
Major Moment of Inertia, Ix	2219 in ⁴
Minor Moment of Inertia, Iy	366 in ⁴
Section modulus for the extreme top fiber, S _{gt}	277 in ³
Section modulus for the extreme bottom fiber, S _{gb}	277 in ³

Concrete:

Type	Normal Weight
Unit Weight, W _c	143.5 pcf
λ	1
Initial Compression strength, f _{ci}	6,431.4 ksi
Concrete Strength, f _c	8.17 ksi
modulus of elasticity of concrete at Transfer, E _{ci}	4571 ksi
modulus of elasticity of concrete at Service, E _c	5152 ksi
Modulus of Rupture, f _r	21.4 ksi

Prestressing Strands:

tensile strength, f _{pu}	270
Strand Yield strength, f _{py} = 0.9 f _{pu}	243 ksi
Jacking to Ultimate Stress/Force Ratio	75%
Strand Jacking Stress, f _{pi} = 0.75* f _{pu}	202.50 ksi
Modulus of elasticity, E _p , ksi	28,800 ksi
Jacking Strain, Δε _p	0.0070
Maximum Strain in Strands	0.007
Max Concrete Strain at Strand Level ε _{pc}	0.0000

Location	Layer No	Jacking Ratio	n of Strand	Strand Dia	Ag _s	dpi (from Top)	Jacking Stress, f _{pi}	Jacking Strain, Δε _p	Jacking Force, P _{pi}
Top	6	75%	2	3/8	0.17	2.25	202.5	0.007031	34.43
	5	75%			0		202.5	0.007031	0.00
	4	75%			0		202.5	0.007031	0.00
Bottom	3	75%	0	1/2	0	11.25	202.5	0.007031	0.00
	2	75%	2	1/2	0.306	12.25	202.5	0.007031	61.97
	1	75%	2	1/2	0.306	14.25	202.5	0.007031	61.97
P _{pi} =									158.36

(B) Transformed Section Calculation

Strands

Layer	Area, A, in ²	Centroid, Y, in	A*Y
6	0.17	2.25	0.3825
5	0	0	0
4	0	0	0
3	0	11.25	0
2	0.306	12.25	3.7485
1	0.306	14.25	4.3605
ΣA	0.782	Σ A*Y	8.4915

Strands Centroid (Top of Beam), Y _{p,t} = ΣA*Y/ΣA	10.859
Ecc. (top beam), ep_gross = eg = Y _t - Y _{p,t} = 2.659	2.659
Ecc. (top beam), ep_transformed = eli = Y _{bt} - Y _{p,t} = 2.743	2.743

1- Full Section (No Openings)

Transformed Section Properties From Top of Beam for Major Axis					
Transformed Beam Centroid					
Material	Area, A, in ²	n = E/E _c	Area, n*A, in ²	Centroid, Y _t , in	n*A*Y
RC Section	104	1	104	8.000	832.00
Prestressing	0.782	5.59	4.37	10.859	47.47
		Alt = 3A	108.37	ΣA*Y	879.47
Transformed Section Centroid from top of Section, Y _{b,t}				8.115	in
Transformed Section Centroid from Bottom of Section, Y _{b,bt}				7.885	in
Transformed Beam Major Moment of Inertia, I _{xt}					
Material	I gross, in ⁴	Area, n*A, in ²	ntroid, Y	A * (Y _{rc} - Y _i) ²	gross + A (Y _i - Y _t) ²
RC Section	2,218.67	104	8.000	1.383	2,220.0
Prestressing	0	4.371	10.859	32.899	32.9
				ΣI gross + ΣA*(Y _i - Y _t) ²	2,252.9
Transformed Major Moment of Inertia, I _{xt}				2,253	in ⁴
Section modulus for the extreme top fiber of the transformed section at transfer, S _{xtt}				278	in ³
Section modulus for the extreme bottom fiber of the transformed section at transfer, S _{xtb}				286	in ³

Transformed Section Properties From Top of Beam for Minor Axis					
Transformed Beam Centroid					
Material	Area, A, in ²	n = E/Ec	Area, n*A, in ²	Centroid, X _i , in	n*A*X
RC Section	104	1	104	3.250	338.00
Prestressing	0.782	5.59	4.37	3.250	14.21
		Ati = ΣA	108.37	ΣA*Y	352.21
Transformed Section Centroid from top of Section, Y _{b_tt}				3.250	in
Transformed Section Centroid from Bottom of Section, Y _{b_bt}				3.250	in
Transformed Beam Major Moment of Inertia, I _{yt}					
Material	I gross, in ⁴	Area, n*A, in ²	ntriod, Y _i	A * (Y _{rc} - Y _i) ²	gross + A (Y _i - Y _c) ²
RC Section	366.17	104	3.250	0.000	366.2
Prestressing	0	4.371	3.250	0.000	0.0
				Σ I gross + Σ A*(Y _i -Y _c) ²	366.2
Transformed Major Moment of Inertia, I _{yt}				366	in ⁴
Section modulus for the extreme top fiber of the transformed section at transfer, S _{ytt}				113	in ³
Section modulus for the extreme bottom fiber of the transformed section at transfer, S _{ybt}				113	in ³

1. Section at Opening

Full Width Opening Size

Opening Start from top, Y

5.5 in

Opening Depth, X

5 in

Concrete Thickness below the Opening, Z

5.5

Revised Centroid from top surface, Y_{to}

8.00 in

Revised Centroid from bottom surface, Y_{bo}

8.00 in

Revised Cross-Sectional Area, A_{go}

71.5 in²

Major Moment of Inertia, I_{xo}

2150.958 in⁴

Minor Moment of Inertia, I_{yo}

251.740 in⁴

Section modulus for the extreme top fiber, S_{tgo}

268.87 in³

Section modulus for the extreme bottom fiber, S_{bgo}

268.87 in³

Ecc. (top beam), e_{p_gross} = e_{go} = Y_t - Y_{p_t} = 2.859

2.859

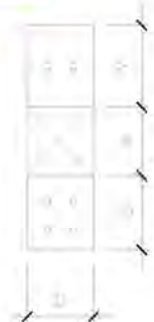
Ecc. (top beam), e_{p_transformed} = e_{to} = Y_{bt} - Y_{p_t} = 2.694

2.694

Beam with Opening Major Moment of Inertia, I _{yo}					
Material	I gross, in ⁴	Area, A, in ²	ntriod, Y _i	A * (Y _{rc} - Y _i) ²	gross + A (Y _i - Y _c) ²
Top Section	90.12	35.75	13.250	985.36	1,075.5
Bottom Section	90.12	35.75	2.750	985.36	1,075.5
				Σ I gross + Σ A*(Y _i -Y _c) ²	2,151.0

Transformed Section with Opening Properties From Top of Beam for Major Axis					
Transformed Beam with Opening Centroid					
Material	Area, A, in ²	n = E/Ec	Area, n*A, in ²	Centroid, Y _i , in	n*A*Y
RC Section with Opening	71.5	1	71.5	8.000	572.00
Prestressing	0.7820	5.59	4.37	10.859	47.47
		Ati = ΣA	75.87	ΣA*Y	619.47
Transformed Section Centroid from top of Section, Y _{b_tto}				8.165	in
Transformed Section Centroid from Bottom of Section, Y _{b_bto}				7.835	in
Transformed Beam with Opening Major Moment of Inertia, I _{xt}					
Material	I gross, in ⁴	Area, n*A, in ²	ntriod, Y _i	A * (Y _{rc} - Y _i) ²	gross + A (Y _i - Y _c) ²
RC Section with Opening	2,150.96	71.5	8.000	1.940	2,152.9
Prestressing	0	4.371	10.859	31.725	31.7
				Σ I gross + Σ A*(Y _i -Y _c) ²	2,184.6
Transformed Major Moment of Inertia, I _{xt}				2,185	in ⁴
Section modulus for the extreme top fiber of the transformed section at transfer, S _{xtto}				268	in ³
Section modulus for the extreme bottom fiber of the transformed section at transfer, S _{xbto}				279	in ³

Transformed Section with Opening Properties From Top of Beam for Minor Axis					
Transformed Beam with Opening Centroid					
Material	Area, A, in ²	n = E/Ec	Area, n*A, in ²	Centroid, X _i , in	n*A*X
RC Section with Opening	71.5	1	71.5	3.250	232.375
Prestressing	0.7820	5.59	4.37	3.250	14.21
		Ati = ΣA	75.87	ΣA*Y	246.58
Transformed Section Centroid from top of Section, Y _{b_tto}				3.250	in
Transformed Section Centroid from Bottom of Section, Y _{b_bto}				3.250	in
Transformed Beam Major Moment of Inertia, I _{yt}					
Material	I gross, in ⁴	Area, n*A, in ²	ntriod, Y _i	A * (Y _{rc} - Y _i) ²	gross + A (Y _i - Y _c) ²
RC Section with Opening	251.74	71.5	3.250	0.000	251.7
Prestressing	0	4.371	3.250	0.000	0.0
				Σ I gross + Σ A*(Y _i -Y _c) ²	251.7
Transformed Major Moment of Inertia, I _{yt}				252	in ⁴
Section modulus for the extreme top fiber of the transformed section at transfer, S _{ytto}				77	in ³
Section modulus for the extreme bottom fiber of the transformed section at transfer, S _{ybt}				77	in ³



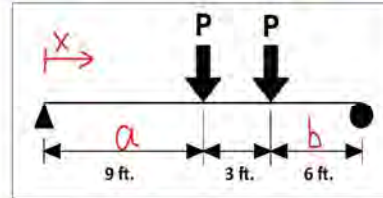
Applied Load Calculation

Beam Length = 18.5 ft
 Span Length = 18 ft
 Spacing Between the Load Cells (P1 & P2) = 3 ft
 Beam Width = 5 in
 Beam Depth = 16.5 in
 Cross-section area = 82.5 in²

Unit Weight = 148 pcf
 Distributed Self-Weight = 0.08 klf

for load cell calculations

a = 9 ft
 b = 6 ft
 R1 = 8.9 kips
 R2 = 12.4 kips



Load Cell Value, P1 = 10.66 kips
 Load Cell Value, P2 = 10.66 kips
 Ultimate Load Factor = 1.85

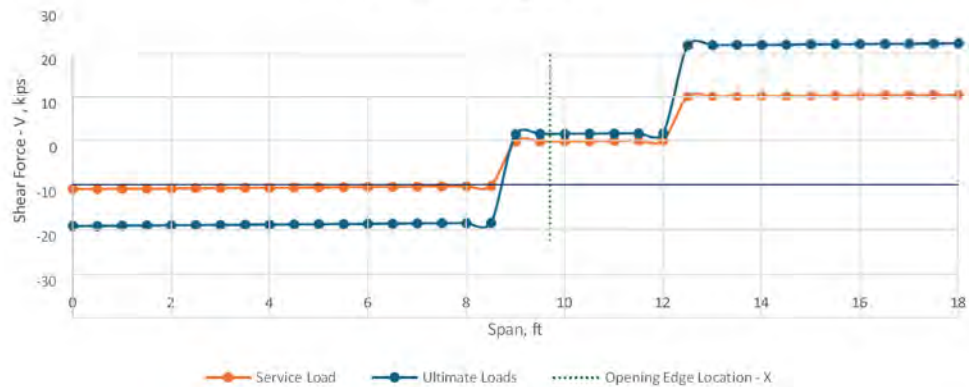
Max Moment location at Opening @ X = 6.7 ft

Shear & Moment @ X

	Total Applied Load, P	V, kips	M, k-ft
Service	21.32	1.836	82.12
Ultimate	39.442	3.346	149.02

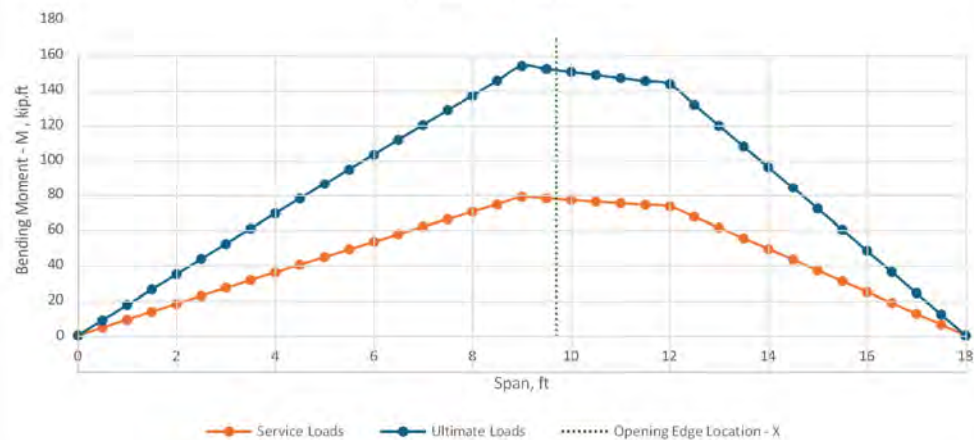
Service Load

Shear Force Diagram - V



Service Load

Bending Moment Diagram - M



(C) Stress Calculation

Section Type

Class U

Stress Limits for Concrete		
	Compressive Stresses, fc, ksi	Tensile Stresses, ft, psi
Temporary Stresses before Losses, at Transfer	$0.60 f_{ci}' = 0.60 * 6.4314 = 3.85884$	240.59
Stresses at Service Limit State after Losses	$0.6 f_c' = 0.6 * 8.17 = 4.902$	$7.5 f_{ci}' = 677.91$

Table R24.5.2.1—Serviceability design requirements

	Class U	Class T	Class C	Nonprestressed
Assumed behavior	Uncracked	Transition between uncracked and cracked	Cracked	Cracked
Section properties for stress calculation at service loads	Gross section 24.5.2.2	Gross section 24.5.2.2	Cracked section 24.5.2.3	No requirement
Allowable stress at transfer	24.5.3	24.5.3	24.5.3	No requirement
Allowable compressive stress based on uncracked section properties	24.5.4	24.5.4	No requirement	No requirement
Tensile stress at service loads 24.5.2.1	$\leq 7.5 \sqrt{f_c'}$	$7.5 \sqrt{f_c'} \leq f_t \leq 12 \sqrt{f_c'}$	No requirement	No requirement
Deflection calculation basis	24.2.3.8, 24.2.4.2 Gross section	24.2.3.9, 24.2.4.2 Cracked section, bilinear	24.2.3.9, 24.2.4.2 Cracked section, bilinear	24.2.3, 24.2.4.1 Effective moment of inertia
Crack control	No requirement	No requirement	24.3	24.3
Computation of Δf_{pe} or f_t for crack control	—	—	Cracked section analysis	$\Delta f(A_s \times \text{lever arm})$, or $2/3 f_t$
Side skin reinforcement	No requirement	No requirement	9.7.2.3	9.7.2.3

24 Serviceability

Stresses At Release (Elastic Shortening)

Beam will be cast on the side, therefore self-weight not reducing stresses on the beam

Stress Limits for Concrete		
	Compressive Stresses, fc, ksi	Tensile Stresses, ft, psi
Temporary Stresses before Losses, at Transfer	3.85884	240.59

Effective pretension force just before release, $P_{pj} = f_{pj} \times A_{ps}$

158.355 kips

Calculations Considering Transformed Section

Elastic Shortening Loss Calculation:

Stress in concrete due to prestressing, f_{cp} 1.461 ksi

Strain in concrete at strands level after release, ϵ_{ci} 0.000319661

Loss in prestressing steel due to elastic shortening, Δf_{pe} 9.21 ksi

Expected Initial Losses 4.5%

Initial Prestressing Stress on the Section, $f_{pi} = f_{pj} - \Delta f_{pe}$ 193.294 ksi

==> Initial Prestressing Force on the Section, $P_{pi} = f_{pi} \times A_{ps}$ 151.16 ksi

1 Stresses At the Side of Beam Full Section

Area of transformed section at transfer, A_{ti} 108.37 in²

Transformed Major Moment of Inertia, I_{xt} 2,252.9 in⁴

Transformed Section Centroid from top of Section, $Y_{h_{tt}}$ 8.115 in

Section modulus for the extreme top fiber of the transformed section at transfer, S_{tt} 277.62 in³

Section modulus for the extreme bottom fiber of the transformed section at transfer, S_{bt} 285.74 in³

Strand Layer No	Jacking Force, P _{pp}	e _{cc} for layer, e _i	P _j /A _g	Moment = P _{pp} * e _i	M _{ppj} *e _i /l _g	f _{cp} , ksi	ε _{ci}	losses, ksi	% loss	Initial Prestressing Force, P _{pi}	Moment = P _{pi} * e _i
6	34.43	5.87	0.32	201.91	-1.13	-0.81	-0.000178	-5.12	-2.53%	35.30	207.02
5	0.00	0.00	0.00	0.00	0.00	0.00	0.000000	0.00	0.00%	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00	0.00	0.000000	0.00	0.00%	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00	0.000000	0.00	0.00%	0.00	0.00
2	61.97	-4.13	0.57	-256.21	0.80	1.37	0.000299	8.63	4.26%	59.33	-245.29
1	61.97	-6.13	0.57	-380.14	1.18	1.75	0.000384	11.06	5.46%	58.58	-359.38
			M _{ppj} =		-434.43	==> Initial Prestressing Force on the Section =				153.20	-397.65
											Initial Prestress Moment

Top Stress, ft

$$f_t = \frac{P_{pi}}{A_{ti}} - \frac{P_{pi}e_{ti}}{S_{tti}}$$

Strands Axial Stress, Ppi/Ati = 1.41 ksi
 Strands Bending Stress, -Ppi*eti/Stti = -1.43 ksi
 Self-weight Bending, Mg/Stti = 0.00 ksi
 ==> **ft = 1.41+1.43+0=** -11.17 ksi

-0.02 ≥ -0.24 ==> Section Top Stresses is under Tensions G within The Limit

Bottom Stress, fb

$$f_b = \frac{P_{pi}}{A_{ti}} + \frac{P_{pi}e_{ti}}{S_{bti}}$$

Axial component, Ppi/Ati = 1.41 ksi
 Strands Bending, Ppi*eti/Sbti = 1.39 ksi
 Self-weight Bending, -Mg/Sbti = 0.00 ksi
 ==> **fb = 1.41+1.39+0=** 2.81 ksi

2.81 < 0.6 fci = 3.85884 ==>Section Bottom Stresses is under Compression G within The

2 Stresses At the Side of Beam with Opening Section

Area of transformed section at transfer, Ati 75.87 in²

Transformed Section Centroid from top of Section, Yb_tto 8.16 in

Section modulus for the extreme top fiber of the transformed section at transfer, Stt 267.57 in³

Section modulus for the extreme bottom fiber of the transformed section at transfer, Sbt 278.82 in³

Strand Layer No	Jacking Force, Ppj, kips	Initial Prestressing Force, Ppi, kips	ecc for layer, ei, in	Moment = Ppi * ei, k-in
6	34.43	32.86	5.91	194.36
5	0.00	0.00	8.16	0.00
4	0.00	0.00	8.16	0.00
3	0.00	0.00	-3.09	0.00
2	61.97	59.15	-4.09	-241.64
1	61.97	59.15	-6.09	-359.93
==> Initial Prestressing Force on the Section =		15	==> Initial Prestress Moment =	

Top Stress, ft

$$f_t = \frac{P_{pi}}{A_{ti}} - \frac{P_{pi}e_{ti}}{S_{tti}}$$

Strands Axial Stress, Ppi/Ati = 1.99 ksi
 Strands Bending Stress, -Ppi*eti/Stti = -1.52 ksi
 Self-weight Bending, Mg/Stti = 0.00 ksi
 ==> **ft = 1.99-1.52+0=** 0.47 ksi

0.47 ≥ -0.24 ==> Section Top Stresses is Under Compression

Bottom Stress, fb

$$f_b = \frac{P_{pi}}{A_{ti}} + \frac{P_{pi}e_{ti}}{S_{bti}}$$

Axial component, Ppi/Ati = 1.99 ksi
 Strands Bending, Ppi*eti/Sbti = 1.46 ksi
 Self-weight Bending, -Mg/Sbti = 0.00 ksi
 ==> **fb = 1.99+1.46+0=** 3.45 ksi

3.45 < 0.60 fci = 3.85884 ==>Section Bottom Stresses is under Compression G within The

Calculations Considering Gross Section

Elastic Shortening Loss Calculation:

Stress in concrete due to prestressing, fcp	1.523 ksi
Strain in concrete at strands level after release, ϵ_{ci}	0.000333097
Loss in prestressing steel due to elastic shortening, Δf_{pe}	9.59 ksi
Expected Initial Losses	4.7%
Initial Prestressing Stress on the Section, $f_{pi} = f_{pj} - \Delta f_{pe}$	192.907 ksi
=> Initial Prestressing Force on the Section, $P_{pi} = f_{pi} \times A_{ps}$	150.85 ksi

Stresses At the Side of Beam Full Section

Area of transformed section at transfer, A_g	104.00 in ²
Major Moment of Inertia, I_x (gross)	2218.666667 in ⁴
Centroid from top surface, Y_t	8.00 in
Section modulus for the extreme top fiber of the Gross section at transfer, S_{tg}	277.33 in ³
Section modulus for the extreme bottom fiber of the Gross section at transfer, S_{bg}	277.33 in ³

Strand Layer No	Jacking Force, P _{pj}	ecc for layer, e _i	P _j /A _g	Moment = P _{pj} * e _i	M _{pj} *e _i /I _g	f _{cp} , ksi	ε _{ci}	losses, ksi	% loss	Initial Prestressing Force, P _{pi}	Moment = P _{pi} * e _i
6	34.43	5.75	0.33	197.94	-1.17	-0.84	-0.000184	-5.31	-2.62%	35.33	203.13
5	0.00	0.00	0.00	0.00	0.00	0.00	0.000000	0.00	0.00%	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00	0.00	0.000000	0.00	0.00%	0.00	0.00
3	0.00	0.00	0.00	0.00	0.00	0.00	0.000000	0.00	0.00%	0.00	0.00
2	61.97	-4.25	0.60	-263.35	0.87	1.46	0.000320	9.22	4.55%	59.14	-251.36
1	61.97	-6.25	0.60	-387.28	1.28	1.87	0.000409	11.79	5.82%	58.36	-364.74
				M _{pj} =	-452.69	=> Initial Prestressing Force on the Section =152.83					-412.97
											Initial Prestress Moment

Top Stress, f_t

$$f_t = \frac{P_{pi}}{A_{ti}} - \frac{P_{pi} e_{ti}}{S_{tti}}$$

Strands Axial Stress, $P_{pi}/A_{ti} =$	1.47 ksi
Strands Bending Stress, $-P_{pi} e_{ti}/S_{tti} =$	-1.49 ksi
Self-weight Bending, $M_g/S_{tti} =$	0.00 ksi
=> f_t = 1.47+1.4G+0=	-0.02 ksi

-0.02 ≥ -0.24 => Section Top Stresses is under Tensions & within The Limit

Bottom Stress, f_b

$$f_b = \frac{P_{pi}}{A_{ti}} + \frac{P_{pi} e_{ti}}{S_{bti}}$$

Axial component, $P_{pi}/A_{ti} =$	1.47 ksi
Strands Bending, $P_{pi} e_{ti}/S_{bti} =$	1.49 ksi
Self-weight Bending, $-M_g/S_{bti} =$	0.00 ksi
=> f_b = 1.47+1.4G+0=	2.96 ksi

2.96 < 0.6 f_{ci} = 3.85884 => Section Bottom Stresses is under Compression & within The

Stresses At the Side of Beam with Opening Section

Area of transformed section at transfer, A_{go}	71.50 in ²
Revised Centroid from top surface, Y_{to}	8.00 in
Section modulus for the extreme top fiber of the Gross section at transfer, S_{tgo}	268.87 in ³
Section modulus for the extreme bottom fiber of the Gross section at transfer, S_{bgo}	268.87 in ³

Strand Layer No	Jacking Force, Ppj, kips	Initial Prestressing Force, Ppi, kips	ecc for layer, ei, in	Moment = Ppi * ei , k-in
6	34.43	32.79	5.75	188.57
5	0.00	0.00	8.00	0.00
4	0.00	0.00	8.00	0.00
3	0.00	0.00	-3.25	0.00
2	61.97	59.03	-4.25	-250.88
1	61.97	59.03	-6.25	-368.93
	==> Initial Prestressing Force on the Section = 150.85		==> Initial Prestress Moment = -431.24	

Top Stress, ft

$$f_t = \frac{P_{pi}}{A_{ti}} - \frac{P_{pi}e_{ti}}{S_{tti}}$$

Strands Axial Stress, Ppi/Ati = 2.11 ksi
 Strands Bending Stress, -Ppi*eti/Stti = -1.60 ksi
 Self-weight Bending, Mg/Stti = 0.00 ksi
 ==> ft = 2.11+(-1.6)+0= **0.51 ksi**

0.51 > -0.24 ==> Section Top Stresses is Under Compression

Bottom Stress, fb

$$f_b = \frac{P_{pi}}{A_{ti}} + \frac{P_{pi}e_{ti}}{S_{bti}}$$

Axial component, Ppi/Ati = 2.11 ksi
 Strands Bending, Ppi*eti/Sbti = 1.60 ksi
 Self-weight Bending, -Mg/Sbti = 0.00 ksi
 ==> fb = 2.11+1.6+0= **3.71 ksi**

3.71 < 0.60 fci = 3.85884 ==>Section Bottom Stresses is under Compression & within the acceptable stress limit

Stresses At Final Time (Before Testing)

Strands Relaxation losses at 21 days (approximate) 2.50%
 Expected Total Losses Percent 7.05%
 effective pretension force after allowing for all losses and gains at service, Ppe = Ppj x (1-Losses) 147.20 kins
 Expected Applied Load before cracking 21.32 kips
 Minimum Applied Service Moment, Ms 42.12 k-ft
 985.44 k-in

Stress Limits for Concrete		
	Compressive Stresses, fc, ksi	Tensile Stresses, ft, psi
Stresses at Service Limit State after Losses	4.902	677.91

1 Stresses At the Side of Beam Full Section

Area of transformed section at transfer, Ati 108.37 in²
 Transformed Section Centroid from top of Section, Yb,tt 8.115 in
 Section modulus for the extreme top fiber of the transformed section at transfer, Stti 278 in³
 Section modulus for the extreme bottom fiber of the transformed section at transfer, Sbti 286 in³

Strand Layer No	Jacking Force, Ppj, kips	Initial Prestressing Force, Ppi, kips	ecc for layer, ei, in	Moment = Ppi * ei, k-in
6	34.43	32.00	5.87	187.69
5	0.00	0.00	8.12	0.00
4	0.00	0.00	8.12	0.00
3	0.00	0.00	-3.13	0.00
2	61.97	57.60	-4.13	-238.15
1	61.97	57.60	-6.13	-353.35
==> effective Prestressing Force on the Section =		147.20	==> Total Prestress Moment =	

Top Stress, ft

$$f_t = \frac{P_{pe}}{A_{ti}} - \frac{P_{pe}e_{ti}}{S_{tti}} + \frac{M_g}{S_{tti}}$$

Axial component, Ppe/Ati = 1.36 ksi
 Strands Bending, -Ppe*eti/Stti = -1.45 ksi
 Self-weight Bending, Mg/Stti = 3.55 ksi
 ==> ft = 1.36+(-1.45)+3.55= **3.46 ksi**

Section Top Stresses is under Compression & within The Compression Stress Limit

Bottom Stress, fb

$$f_b = \frac{P_{pe}}{A_{ti}} + \frac{P_{pe}e_{ti}}{S_{bti}} - \frac{M_g}{S_{bti}}$$

Axial component, Ppe/Ati = 1.36 ksi
 Strands Bending, Ppi*eti/Sbti = 1.41 ksi
 Self-weight Bending, -Mg/Sbti = -3.45 ksi
 ==> fb = 1.36+1.41+(-3.45)= **-0.6772 ksi**

Section Bottom Stresses is under Tensions & within The Limit

2 Stresses At the Side of Beam with Opening Section

Area of transformed section at transfer, A_{ti}	75.87 in ³
Transformed Section Centroid from top of Section, Y_{b_tto}	8.16 in
Section modulus for the extreme top fiber of the transformed section at transfer, S_{tt}	267.57 in ³
Section modulus for the extreme bottom fiber of the transformed section at transfer, S_{bt}	278.82 in ³

Strand Layer No	Jacking Force, Ppj, kips	Initial Prestressing Force, Ppi, kips	ecc for layer, ei, in	Moment = Ppi * ei, k-in
6	34.43	32.00	5.91	189.27
5	0.00	0.00	8.16	0.00
4	0.00	0.00	8.16	0.00
3	0.00	0.00	-3.09	0.00
2	61.97	57.60	-4.09	-235.31
1	61.97	57.60	-6.09	-350.51
==> effective Prestressing Force on the Section = 147.20			==> Total Prestress Moment = -396.55	

Top Stress, ft

$$f_t = \frac{P_{pe}}{A_{ti}} - \frac{P_{pe}e_{ti}}{S_{tti}} + \frac{M_g}{S_{tti}}$$

Axial component, P_{pe}/A_{ti} =	1.94 ksi
Strands Bending, $-P_{pe}e_{ti}/S_{tti}$ =	-1.48 ksi
Self-weight Bending, M_g/S_{tti} =	3.68 ksi
==> $f_t = 1.94 - 1.48 + 3.68 =$	4.14 ksi

Section Top Stresses is under Compression & within The Compression Stress Limit

Bottom Stress, fb

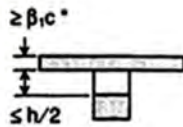
$$f_b = \frac{P_{pe}}{A_{ti}} + \frac{P_{pe}e_{ti}}{S_{bti}} - \frac{M_g}{S_{bti}}$$

Axial component, P_{pe}/A_{ti} =	1.94 ksi
Strands Bending, $P_{pe}e_{ti}/S_{bti}$ =	1.42 ksi
Self-weight Bending, $-M_g/S_{bti}$ =	-3.53 ksi
==> $f_b = 1.94 + 1.42 - 3.53 =$	-0.17 ksi

Section Bottom Stresses is under Tensions & within The Limit

(D) Ultimate Capacity Calculation

Maximum Concrete Strain, ϵ_c	0.003
β_1	0.7232
Concrete Compression Depth, C	5.655 in
Whitney Block Depth, a	4.090 in
Force & Moment in Concrete:	
Whitney Block Area = a x b	26.585 in ²
Force = $0.85 \times f_c \times a \times b$	184.620 kips
Moment Arm = C - a/2	3.610 in-k
Moment = Force x Moment Arm	666.554 in-k



Summation of Forces, kips	184.69
Summation of Moments, in-k	1,788.6
Nominal Capacity, Mn, ft-k	149.0

Forces Equilibrium Ratio	100%
Strength Reduction Factor, ϕ	0.65
Nominal Capacity, ϕM_n , ft-k	96.9

Prestressing Strands

Location	Layer No	No of Strands	Strand Dia	Aps	dpi (from Top)	Strain ϵ_{pc}	Total Strand Strain ϵ_p	Stress, ksi	Force, kips	Moment Arm, in	Moment, in-k
Top	6	2	3/8	0.17	2.25	-0.001806455	0.0052	150.5	25.58	-3.41	-87.11
	5	0	0	0	0	0	0.0000	0.0	0.00	-5.66	0.00
	4	0	0	0	0	0	0.0000	0.0	0.00	-5.66	0.00
Bottom	3	0	1/2	0	11.25	0	0.0000	0.0	0.00	5.59	0.00
	2	2	1/2	0.306	12.25	0.003498191	0.0105	258.7	79.15	6.59	521.97
	1	2	1/2	0.306	14.25	0.0046	0.01159037	261.3	79.95	6.59	687.17
Sum Forces (kips)									184.69	Sum M (in-k)	1,122.0

Design of Bursting Reinforcement

<i>Total Jacking Force, P_o</i>	158.4	kips
<i>Beam Depth, h</i>	16	in
<i>Allowable stress in the stirrups, f_s</i>	30	ksi
<i>f_{se}</i>	170	ksi
<i>d_b</i>	0.50	in
<i>Transfer Length, l_t</i>	28.33	in
Bursting Transverse RFT, A_{vt}	0.063	in²
<i>Provided length = $h/5$</i>	3.20	in
<i>bar size</i>	#3	
<i>bar area</i>	0.11	in ²
<i>No. of Legs</i>	2	
<i>No. of Stirrups</i>	2	
<i>Total Provided Area</i>	0.44	in ²

B2: Flexural Capacity: AASHTO LRFD Specifications

Material Properties	
f'_c (psi)	8170
E_c (ksi)	4616
E_s (ksi)	28717
n	6.22
f_y (ksi)	53.50
f_{pu} (ksi)	287
f_{po} (ksi)	196
L (ft)	18.50
w (k/ft)	0.097

Cracking	
$\Delta\epsilon_p$	0.00705
N_o (kips)	158.4
M_o (kip-in)	436
ϵ_{cen} (in/in)	-0.00032
f_{cr} (ksi)	555
ϵ_{cr} (in/in)	0.00012
ϕ_{cr} (1/in)	0.00006
M_{cr} (kip-in)	1012
M_d (kip-in)	49.9
$M_{cr, Service Load}$	962
P_{cr} (kip)	10.69
$2P_{cr}$ (kip)	21.37

Cross-Section Properties at Midspan	
Cover bottom (in)	1.75
Cover top (in)	2.25
b (in)	6.5
h (in)	16
A_g (in ²)	104
y_{bg} (in)	8
A_{p1} (in ²)	0.306
A_{p2} (in ²)	0.306
A_{p3} (in ²)	0.17
A_p (in ²)	0.782
y_{bs1} (in)	1.75
y_{bs2} (in)	3.75
y_{bs3} (in)	13.75
y_{bs} (in)	5.14
A_t (in ²)	108.1
y_{bt} (in)	7.89
I_g (in ⁴)	2219
I_t (in ⁴)	2251

Flexural Capacity

Choosing top strain at ultimate	
n	4.07
ϵ'_c (in/in)	0.00235
$\epsilon_{ct}/\epsilon'_c$ (in/in)	1
ϵ_{ct} (in/in)	0.00235

Stress Block Factors	0.5	0.75	1	1.25	1.5	1.75
α	0.495	0.712	0.859	0.863	0.744	0.61
β	0.669	0.679	0.704	0.763	0.859	0.963

Iterating on depth of neutral axis

c (in)	ϵ_{cp1} (in/in)	ϵ_{p1} (in/in)	f_{p1} (ksi)	ϵ_{cp2} (in/in)	ϵ_{p2} (in/in)	f_{p2} (ksi)	T (kips)	Cc (kips)	Cp (kips)	C (kips)	$C-T$ (kips)
1	0.0431	0.0502	273.3	-0.0044	0.0027	76.1	167.3	32.1	12.9	45.1	-122.2
2	0.0198	0.0269	256.6	-0.0004	0.0066	188.4	157.0	64.2	32.0	96.3	-60.8
3	0.0120	0.0191	251.0	0.0009	0.0079	218.7	153.6	96.3	37.2	133.5	-20.1
4	0.0081	0.0152	248.1	0.0015	0.0086	229.0	151.8	128.5	38.9	167.4	15.5
3.56	0.0096	0.0167	249.2	0.0013	0.0083	225.6	152.5	114.2	38.4	152.5	0.0

M_u (kip-in)	1792
M_u (kip-ft)	149.3
P_u (kips)	19.9
$2P_u$ (kips)	39.8

B3: Flexural Capacity: Layered Sectional Analysis

% ===== %

% LAYERED-SECTIONAL ANALYSIS %

% %

% Written by: Paul Acuna %

% Spring 2023 %

% %

% ***** %

% %

% Modified for %

% PCI Big Beam Competition %

% Summer 2025 %

%===== %

close all;clear;clc;

format long

%% DATA INPUT %%

%Concrete Properties

fc=8; % Concrete strength [ksi]

eprimac=0.002; %Concrete strain at rupture (Cylinder crushing)

Ec=(57000*sqrt(fc*1000))/1000; %Concrete modulus of elasticity [ksi]

sigmat=(7.5*sqrt(fc*1000))/1000; %Maximum tensile stress [ksi]

%Passive Steel Reinforcement Properties

Es=29000; %Mild steel modulus of elasticity [ksi]

fy=60; % Steel yielding stress [ksi]

As=[

0 0 0

```

]; %% bars, diameter [in], position from the top [in]

%Use [0 1 1] if NO reinforcement


%Prestressing Steel Reinforcement Properties
Ep=28500; %Prestressing steel modulus of elasticity [ksi]
fpu=270; %Prestressing ultimate stress [ksi]
losses=15;%Prestressing losses [%]

%Used to calculate fse as (1-%losses)(fpo)


Ap=[
    2 0.085 2.25 17.1
    2 0.153 12.25 31
    2 0.153 14.25 31
]; %% strands, area [in2], position from the top [in], Po (kips)

%Use [0 1 1 1] if NO prestressing


%Dimensions
dim=[
    6.25 5.5
    0 5
    6.25 5.5
]; %%b,h from the top [mm]


%Analysis criteria

elayers=0.025; %Layers thickness [in] %Should not be changed unless strictly necessary


%% ===== %

```

```

%      VALIDATION, PRELIMINARY CALCS AND PLOT      %

% ===== %

height=sum(dim(:,2)); %Total height of the beam [mm]

% VALIDATION OF THE LAYER THICKNESS

while mod(height,elayers)~=0
    elayers=input('Choose another layer thickness\n');
end
fprintf('Layer thickness is OK when compared to total height\n');

for i=1:size(dim,1)
    while mod(dim(i,2),elayers)~=0
        elayers=input('Choose another layer thickness\n');
    end
end
fprintf('Layer thickness is OK when compared to partial heights\n');

% LAYERS WIDTHS (Matrix with the width per layer)
nlayers=height/elayers;
layers=zeros(nlayers,3);%width, thickness, lever arm to top of the beam

leverarm=elayers/2;
cont=1;
for i=1:size(dim,1)
    k=1;
    while k<=dim(i,2)/elayers
        layers(cont,1)=dim(i,1);

```

```

layers(cont,2)=elayers;
layers(cont,3)=leverarm;
leverarm=leverarm+elayers;

k=k+1;
cont=cont+1;
end
end

% SECTION PLOT
auxy=height;
auxx=max(dim(:,1))/2;
figure
for i=1:size(dim,1)
    insertiony=auxy-dim(i,2); %insertiony point for the plot
    rectangle('Position',[auxx-dim(i,1)/2 insertiony dim(i,1) dim(i,2)],'FaceColor',[0.3010 0.7450
0.9330])%coordenadas de posicion en plano y dimensiones
    auxy=insertiony;
end

%% ===== %

%           ANALYSIS

% ===== %

data=zeros(3,4); %Moment and curvature for the complete response
    %[ec,c,M,Phi]

```

```

M=0; %Calculated moment [kN-m]
Phi=0; %Calculated curvature [kN-m]
cont=1; %Counter of calculated points
menores=1; %Contador de los 4 momentos descendientes in a row
aux=0; %Booleano to verify that the moment has not dropped 5 times in a row
e=zeros(nlayers,1); %Vector where the strains of each layer will be stored
sigma=zeros(nlayers,1); %Vector where the stresses of each layer will be stored
force=zeros(nlayers,1); %Vector where the forces of each layer will be stored
moment=zeros(nlayers,1); %Vector where the moments of each layer will be stored

es=zeros(size(As,1),1); %Vector where the strains of steel layers will be stored
sigmas=zeros(size(As,1),1); %Vector where the stresses of steel layers will be stored
forcesteel=zeros(size(As,1),1); %Vector where the forces of steel layers will be stored
momentsteel=zeros(size(As,1),1); %Vector where the forces of steel layers will be stored

ese=zeros(size(Ap,1),1); %Vector where the initial strains of steel prestressing layers will be stored
esa=zeros(size(Ap,1),1); %Vector where the effective strains of steel prestressing layers will be stored
ep=zeros(size(Ap,1),1); %Vector where the total strains of steel prestressing layers will be stored
sigmap=zeros(size(Ap,1),1); %Vector where the stresses of prestressing steel layers will be stored
forcep=zeros(size(Ap,1),1); %Vector where the forces of prestressing steel layers will be stored
momentp=zeros(size(Ap,1),1); %Vector where the moments of prestressing steel layers will be stored

aux2=0; %Booleano to verify equilibrium at each calculated point

c=elayers; %First initial guess for c [mm]
*****

% for ec=0.0001:0.0001:0.004 %Range of strains used to compute the moment-curvature diagram
for ec=0.00043:0.0001:0.004 %Range of strains used to compute the moment-curvature diagram

```

```

while aux2==0

    for i=1:nlayers

        %STRAIN, STRESSES, FORCES, and MOMENTS in CONCRETE

        %Strain Profile

        Phi=ec/c;

        e(i)=Phi*layers(i,3)-ec; %Strain at mid height of each layer

        %Stress

        if Phi*layers(i,3)-ec <=0 %This factor let us know if we are in C or T

            sigma(i)=-fc*(2*abs(e(i))/eprimac-(e(i)/eprimac)^2);%Stress for each layer in COMPRESSION[ksi]

        else

            sigma(i)=e(i)*Ec; % Stress for each layer in TENSION [ksi]

            if abs(sigma(i))>sigmat

                sigma(i)=0; %NOT considering tension stiffening

            end

        end

    end

    %FORCE

    force(i)=layers(i,1)*layers(i,2)*sigma(i);%Forces in concrete [kips]

    %MOMENTS

    moment(i)=force(i)*layers(i,3)/12; %Moments produced by concrete [kip-ft]

end

%STRAIN, STRESSES, FORCES, and MOMENTS in STEEL REINFORCEMENT

```

```

for w=1:size(As,1)

    %Strain

    es(w)=Phi*As(w,3)-ec;

    %Stress

    if es(w)<=0 %To distinguish between steel in tension and compression

        sigmas(w)=-es(w)*Es; %Stress developed in the steel reinforcement [ksi]

        if abs(sigmas(w))>fy

            sigmas(w)=-fy; %Cap it to the yielding stress

        end

    else

        sigmas(w)=es(w)*Es; %Stress developed in the steel reinforcement [ksi]

        if sigmas(w)>fy

            sigmas(w)=fy; %Cap it to the yielding stress

        end

    end

    %Force

    forcesteel(w)=As(w,1)*pi()/4*(As(w,2))^2*sigmas(w); %Force at each row of steel reinforcement
[kips]

    %Moments in steel rebars

    momentsteel(w)=forcesteel(w)*As(w,3)/12; %Moments produced by each row of rebars [kip-ft]

end

%STRAIN, STRESSES, FORCES, and MOMENTS in PRESTRESSING REINFORCEMENT

for q=1:size(Ap,1)

    %Strain

    ese(q)=((1-losses/100)*Ap(q,4)/Ap(q,2))/Ep;

```

```

esa(q)=Phi*Ap(q,3)-ec;
ep(q)=ese(q)+abs(esa(q));

%Stress
if esa(q)<=0 %To distinguish between prestressing steel in tension and compression
    if ep(q)<=0.0085
        sigmap(q)=-Ep*ep(q); %Stress developed in the prestressing steel reinforcement before
yielding [ksi]
    else
        sigmap(q)=-fpu-0.04/(ep(q)-0.007); %Stress developed in the prestressing steel
reinforcement after yielding [ksi]
    end
    if abs(sigmap(q))>fpu
        sigmap(q)=-fpu; %Cap it to the ultimate stress
    end
else
    if ep(q)<=0.0085
        sigmap(q)=Ep*ep(q); %Stress developed in the prestressing steel reinforcement before
yielding [ksi]
    else
        sigmap(q)=fpu-0.04/(ep(q)-0.007); %Stress developed in the prestressing steel reinforcement
after yielding [ksi]
    end
    if sigmap(q)>fpu
        sigmap(q)=fpu; %Cap it to the ultimate stress
    end
end
end

```

```

%Force

forcep(q)=Ap(q,1)*Ap(q,2)*sigmap(q);%Force at each row of steel reinforcement [kips]

%Moments in steel rebars

momentp(q)=(forcep(q)*Ap(q,3))/12; %Moments produced by each row of rebars [kip-ft]

end

%Equilibrium Verification

sumfuerzas=sum(force)+sum(forcesteel)+sum(forcep); %Sum of forces [kips]
summomentos=sum(moment)+sum(momentsteel)+sum(momentp); %Sum of moments [kips-ft]

if sumfuerzas>0 && abs(sumfuerzas)>0.0001 %aka more force in tension
    if sumfuerzas>200
        c=c+1;
    elseif sumfuerzas<=200 && sumfuerzas>100
        c=c+.25;
    elseif sumfuerzas<=100 && sumfuerzas>50
        c=c+0.1;
    elseif sumfuerzas<=50 && sumfuerzas>1
        c=c+0.01;
    elseif sumfuerzas<=1 && sumfuerzas>0.001
        c=c+0.00001;
    else
        c=c+0.0000001;
    end
elseif sumfuerzas<0 && abs(sumfuerzas)>0.0001 %aka more force in compression
    if sumfuerzas<-200
        c=c-1;
    end
end

```

```

elseif sumfuerzas>=-200 && sumfuerzas<-100
    c=c-0.25;
elseif sumfuerzas>=-100 && sumfuerzas<-50
    c=c-0.1;
elseif sumfuerzas>=-50 && sumfuerzas<-1
    c=c-0.01;
elseif sumfuerzas>=-1 && sumfuerzas<-0.001
    c=c-0.00001;
else
    c=c-0.0000001;
end
else
    aux2=1;
end

end

% summomentos;

%Storing the data for each calculated point
data(cont,1)=ec;
data(cont,2)=c;
data(cont,3)=summomentos;
data(cont,4)=Phi;

cont=cont+1;
aux2=0;

```

```
end
```

```
%% RESULTS (Moment-curvature diagram)
```

```
data=[0 0 0 0;data]; %To add a "zero" start to the data
```

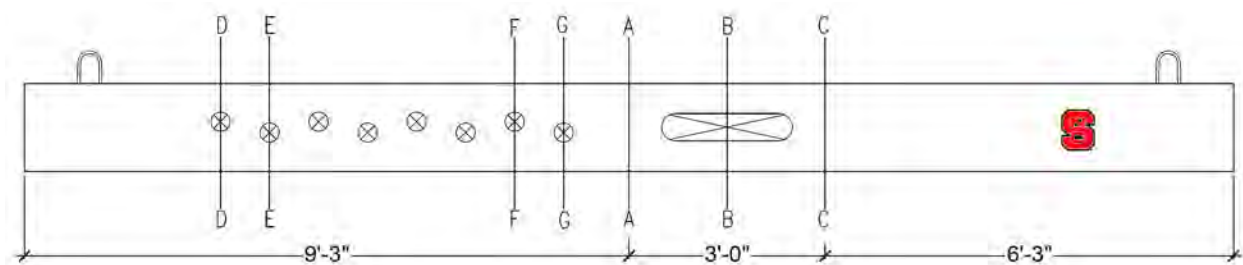
```
figure
```

```
plot(data(:,4),data(:,3));
```

```
format short
```

```
fprintf('Nominal moment of the section = %d [kN-m]\n',max(data(:,3)));
```

Appendix C: Shear Design

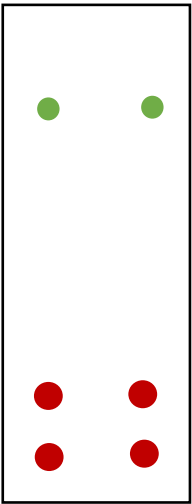


Material Properties

f'_c (psi)	8170
E_c (ksi)	4616
E_s (ksi)	28717
n	6.22
f_y (ksi)	53.50
f_{pu} (ksi)	287.00
f_{po} (ksi)	196.00
L (ft)	18.50
w (k/ft)	0.097

Section A-A (left)

b (in)	6.5
h (in)	16
A_g (in ²)	104
A_p (in ²)	0.782
y_{bg} (in)	8
A_t (in ²)	108.1
y_{bt} (in)	7.88
d_v (in)	11.52
s_x (in)	11.52
a_g (in)	0.375
s_{xe} (in)	15.82



Point load	P
$V(LL)$ (kip)	5P/6
$M(LL)$ (k-ft)	7.5P
M/V	9

No Shear Reinforcement

	ϵ_s	θ	β	$V_c = V$	ϵ_s
	0.0010	32.50	2.55	17.26	0.0014
	0.0012	33.19	2.35	15.92	0.0008
	0.0045	44.73	1.02	6.91	0.0000
	0.0011	32.85	2.45	16.55	0.0011

Check tensile capacity

$A_p f_{ps}$ (kips)	224
M_u (k-ft)	148.9
P (kips)	19.9
Shear Resistance (kips)	200.9
Failure in Shear	Yes
F.S.	0.90

Shear Reinforcement Requirement

V_u	16.55
$0.5\phi V_c$	7.45
Check	Needed

Choose Shear Reinforcement

Bar	#3
A_v (in ²)	0.22
s_{min} (in)	20.0
v_u (ksi)	0.245571
$0.125f'_c$ (ksi)	1.02125
s_{max} (in)	9.216
s (in)	9.0

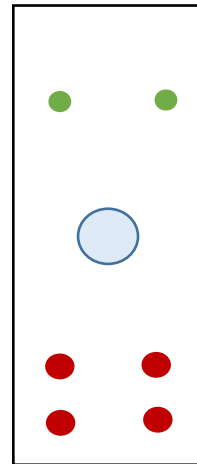
Revised Shear Capacity

ϵ_s	θ	β	V_c	V_s	V	ϵ_s
0.0010	32.50	2.74	18.55	23.65	42.20	0.0129
0.0070	53.35	0.77	5.22	11.21	16.43	0.0010
0.0040	42.94	1.20	8.14	16.19	24.33	0.0047
0.0043	44.12	1.13	7.65	15.53	23.19	0.0041
0.0042	43.79	1.15	7.79	15.71	23.50	0.0043
0.0043	43.88	1.15	7.75	15.67	23.42	0.0042
0.0042	43.86	1.15	7.76	15.68	23.44	0.0042

Check tensile capacity	
$A_p f_{ps}$ (kips)	224
M_u (k-ft)	210.9
P (kips)	28.1
Shear Resistance (kips)	271.2
Failure in Shear	No
F.S.	1.21

Section G-G @8.25 ft (Modified)

b (in)	5
h (in)	16.5
Depth of Void (in)	3.5
A_g (in ²)	72.9
A_p (in ²)	0.782
y_{bg} (in)	8.42
A_t (in ²)	77.0
y_{bt} (in)	8.22
d_v (in)	11.88
s_x (in)	11.88
a_g (in)	0.375
s_{xc} (in)	16.31



Point load	P
$V(LL)$ (kip)	$5P/6$
$M(LL)$ (k-ft)	$20P/3$
M/V	8

No Shear Reinforcement

ϵ_s	θ	β	$V_c = V$	ϵ_s
0.0010	32.50	2.53	11.37	0.0000
0.0005	30.75	3.22	14.47	0.0000
0.0006	31.10	3.05	13.72	0.0000
0.0002	29.84	3.75	16.86	0.0002

Check tensile capacity	
$A_p f_{ps}$ (kips)	224
M_u (k-ft)	134.9
P (kips)	20.2
Shear Resistance (kips)	184.1
Failure in Shear	Yes

F.S. 0.82

Shear Reinforcement Requirement

V_u 16.86
 $0.5\phi V_c$ 7.59
Check Needed

Choose Shear Reinforcement

Bar #3
 A_v (in²) 0.22
 s_{min} (in) 26.1
 v_u (ksi) 0.315
 $0.125f_c$ (ksi) 1.02125
 s_{max} (in) 9.504
 s (in) 9.0

Revised Shear Capacity

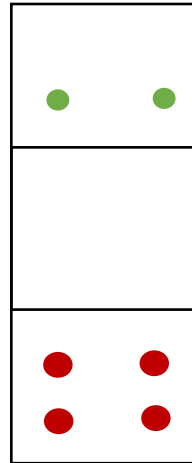
ϵ_s	θ	β	V_c	V_s	V	ϵ_s
0.0010	32.50	2.74	12.33	24.39	36.72	0.0083
0.0046	45.22	1.07	4.82	15.42	20.24	0.0016
0.0031	39.91	1.44	6.46	18.57	25.04	0.0035
0.0033	40.66	1.37	6.17	18.09	24.26	0.0032
0.0033	40.48	1.39	6.24	18.20	24.44	0.0033
0.0033	40.52	1.38	6.22	18.18	24.40	0.0033

Check tensile capacity

$A_p f_{ps}$ (kips) 224
 M_u (k-ft) 195.2
 P (kips) 29.3
Shear Resistance (kips) 250.8
Failure in Shear No
F.S. 1.12

**Section B-B @10.75 ft
(Modified)**

b (in)	6.5
h (in)	16
Depth of Void (in)	5
A_g (in ²)	71.5
A_p (in ²)	0.782
y_{bg} (in)	8.00
A_t (in ²)	75.6
y_{bt} (in)	7.82
d_v (in)	11.52
s_x (in)	11.52
a_g (in)	0.375
s_{xe} (in)	15.82



Point load	P
V(LL) (kip)	P/6
M(LL) (k-ft)	7.25P
M/V	43.5

No Shear Reinforcement

	ϵ_s	θ	β	$V_c = V$	ϵ_s
	0.0010	32.50	2.55	9.77	0.0136
	0.0073	54.49	0.69	2.65	0.0000
	0.0006	31.10	3.08	11.79	0.0177
	0.0034	40.83	1.26	4.84	0.0034

Check tensile capacity

$A_p f_{ps}$ (kips)	224
M_u (k-ft)	210.4
P (kips)	29.0
Shear Resistance (kips)	249.7
Failure in Shear	No
F.S.	1.11

**Shear Reinforcement
Requirement**

V_u	4.84
$0.5\phi V_c$	2.18
Check	Needed

Choose Shear Reinforcement

Bar	#3
A_v (in ²)	0.22
s_{min} (in)	20.0
v_u (ksi)	0.072
$0.125f_c$ (ksi)	1.02125
s_{max} (in)	9.216
s (in)	9.0

Revised Shear Capacity

ϵ_s	θ	β	V_c	V_s	V	ϵ_s
0.0010	32.50	2.74	10.50	23.65	34.15	0.0638
0.0324	142.47	0.19	0.73	-19.61	-18.89	0.0000
0.0162	85.74	0.36	1.40	1.12	2.52	0.0000
0.0081	57.37	0.68	2.60	9.65	12.24	0.0187
0.0134	75.85	0.43	1.66	3.80	5.46	0.0047
0.0108	66.80	0.53	2.02	6.46	8.48	0.0109

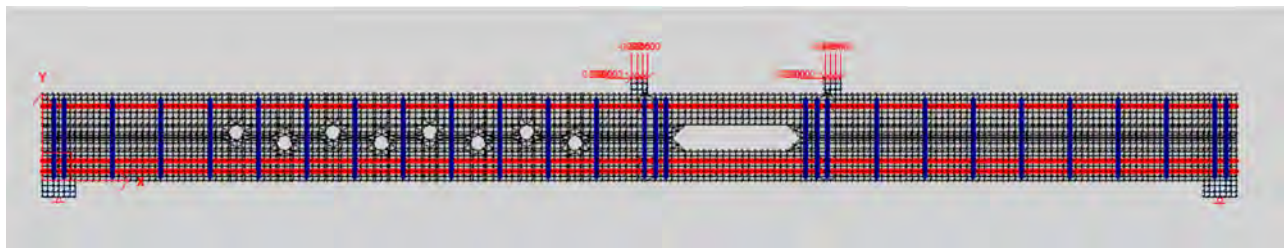
Check tensile capacity

$A_p f_{ps}$ (kips)	224
M_u (k-ft)	368.7
P (kips)	50.9
Shear Resistance (kips)	430.8
Failure in Shear	No
F.S.	1.92

Appendix D: VecTor2 Modelling, Results and Analysis

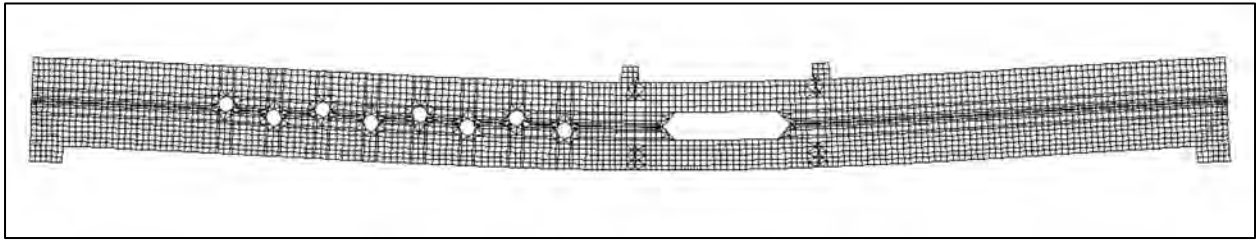
Analysis Details		Material Types	
Total Number of Nodes:	4241		
Total Number of Elements:	5342		
Number of Triangular elements:	506		
Number of Rectangular Elements:	3680		
Number of Truss Elements:	1156		
Number of Linkage Elements:	0		
Number of Contact Elements:	0		
Number of Concrete Types:	2		
Number of Load Stages:	91		
Analysis Type:	VecTor2 V.4		
Analysis Parameters			
Convergence Criteria	Displacements - Weighted	Enter Job Title Structure Def'n Material Types Displacement Factor = 44.00 All dimensions in millimetres	
Compression Base Curve	Popovics (NSC)		
Compression Post-Peak	Modified Park-Kent		
Compression Softening	Vecchio 1992-A		
Tension Stiffening	Modified Bertz 2003		
Tension Softening	Bilinear		
FRC Post-Crack Tension	SDEM (Monotonic)		
Confinement Strength	Kupfer / Richart		
Concrete Dilatation	Variable - Orthotropic		
Cracking Criterion	Mohr-Coulomb (Stress)		
Crack Stress Calc	Basic (DSFMM/CFT)		
Crack Width Check	Crack Limit (Agg/2.5)		
Concrete Bond	Eligehausen Model		
Concrete Creep / Relax	Not Considered		
Concrete Hysteresis	Nonlinear w/ Offsets		
Steel Hysteresis	Seckin (w/ Bauschinger)		
Rebar Dowel Action	Tassios (Crack Slip)		
Rebar Buckling	Modified Dnskat-Maekawa		

Modelling in FormWorks

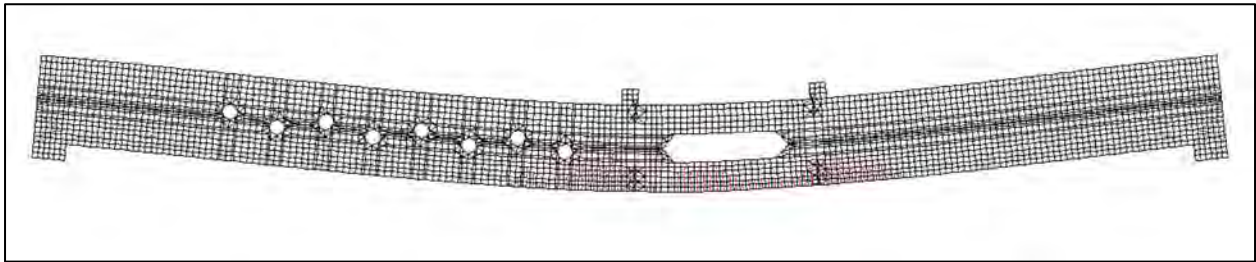


Deformed shape and cracks predicted

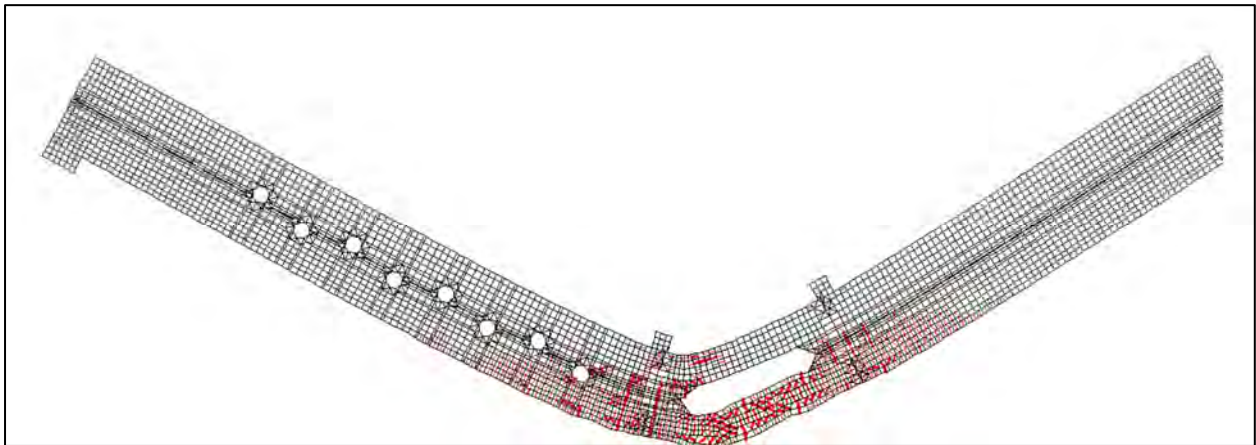
At first crack (24.7 kips)



At 32 kips



At Ultimate Load (39.6 kips)



Appendix E: Cost Calculation

Rectangula Beam

Concrete:

Concrete Type	Conventional Concrete
Beam Width (web)	6.5 in
Beam Depth	16 in
Cross-Sectional Area	104 in ²
Beam Length	18.5 ft
Opening Volume	0.720732541 ft ³
Concrete Volume	12.640 ft ³
	0.468 yd ³
Concrete Strength (f'c)	8.5 ksi
Concrete Unit Cost	145 \$/yd ³
Concrete Cost	67.88 \$

Prestressing Strands:

Strand Type	Number of Strands	Strand Length, ft	Strand Cost, \$/ft	Strand Cost, \$
1/2in Dia	4	18.5	0.3	22.2
3/8in Dia	2	18.5	0.27	9.99

Steel

Total Length of one Stirrups	3.1 ft		
Number of Stirrups	28		
Stirrups Size	#3		
Rebar Diameter	0.375 in		
Rebar Nominal area	0.11 in ²		
Rebar weight	0.376 lb/ft		
Type of Steel	Weight, lb	Unit Cost, \$/lb	Cost, \$
A615/A706	32.6368	0.45	14.69

Forming

Big Opening

Overall Opening Length	2 ft
Overall Opening Width	5 in
Edge Type	Rounded Edge
Big Opening Perimeters	4.476 ft
Total Opening Area	0.796 ft ²

Small Openings

Edge Type	Circular
Width (Diameter)	3.5 in
Length	4 in
Number of Openings	8
Small Openings Perimeters	7.330382858 ft
Small Openings Total Area	0.534507083 ft ²

Opening Summary

Total Opening Perimeters	11.806 ft
Total Opening Area	1.331 ft ²
Total Openings Volume	0.720732541 ft ³

Concrete in Contact with Formwork area Calculations:

1- Beam Formwork Side faces

Perimeter = 39.667 ft
Depth = 0.542 ft
Formwork Area 1, A1 = 21.486 ft²

2- Casting Bed Supporting the beam

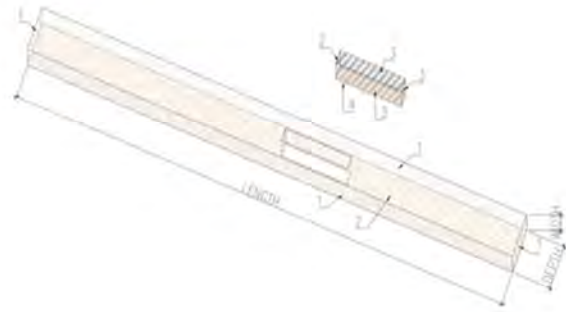
Casting Bed Area = 24.667 ft²
Opening Casting bed Area 4= 1.331 ft²
Formwork Area 2, A2= 23.336 ft²

3- Opening Side faces Area

Perimeter = 11.806 ft
Depth = 0.542 ft
Formwork Area 3, A3 = 6.395 ft²

Total Formwork Areas=A1 + A2 + A3 = 51.217 ft²

Formwork cost , \$/ft² 1.25
Formwork Cost 64.02 \$



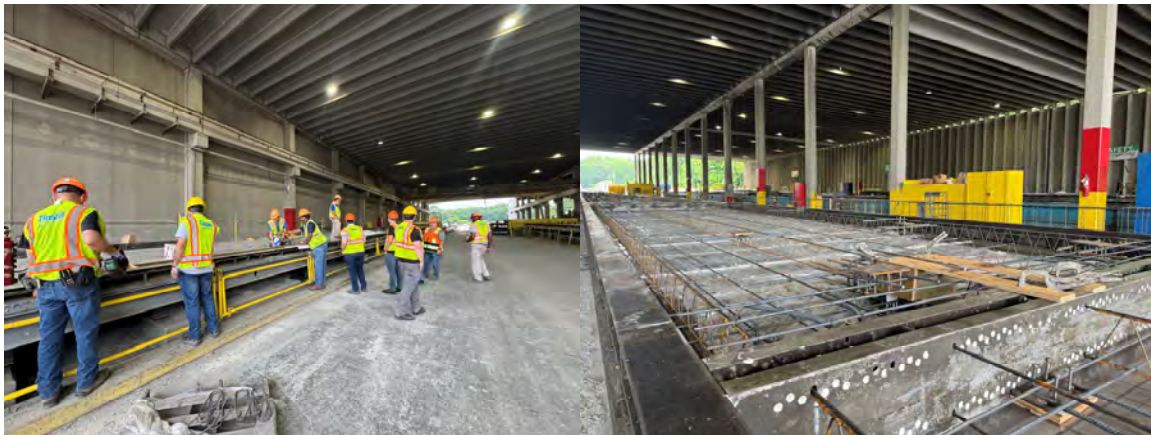
Total Cost = \$ 178.78

Small Opening Optimization					
Number of Openings	Cost	Cost Change	Volume, ft ³	Weight, lb	Weight Change
0	155.85	0.00%	10.27	1,519.6	0.00%
2	156.33	0.31%	10.19	1,508.8	-0.71%
4	156.82	0.62%	10.12	1,498.0	-1.42%
6	157.30	0.93%	10.05	1,487.3	-2.12%
8	157.78	1.24%	9.98	1,476.5	-2.83%

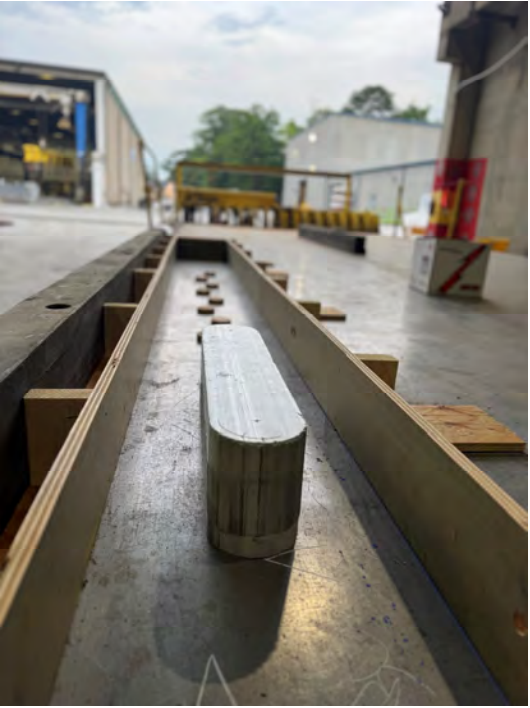
Weight Calculation

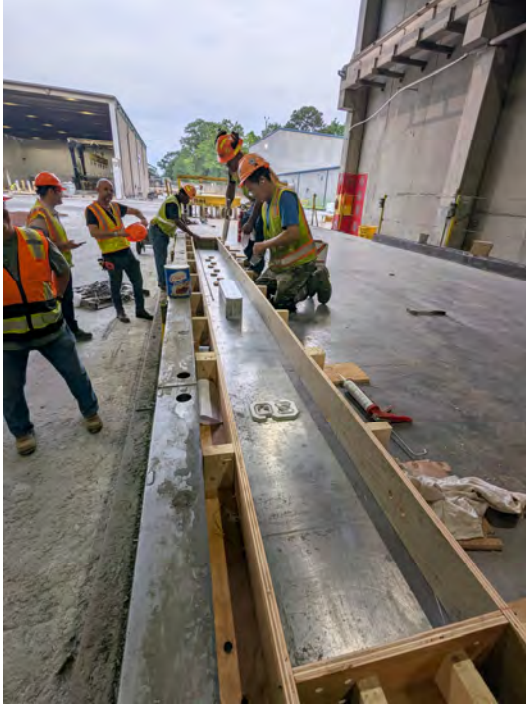
Total Volume of Concrete 12.64 ft³
Unit Weight of Concrete 143.95 lb/ft³
Beam Weight 1819.6 lb

Appendix F: Gallery



Site Visit at Tindall Corporation and Photo of a Building Casted with the New Architectural Mix



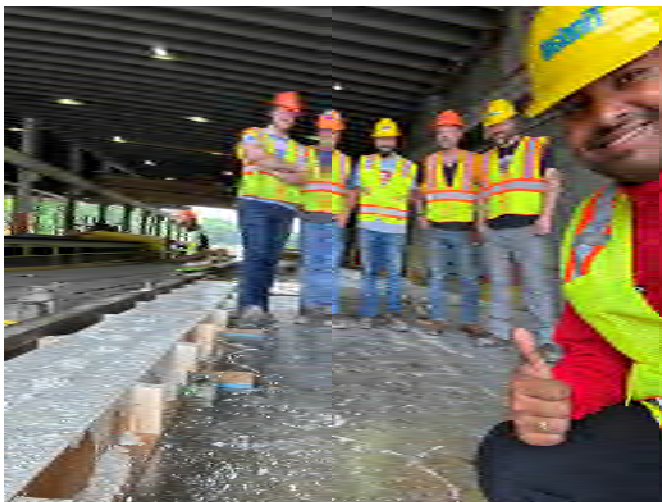


Building the Formwork and Placing Reinforcement

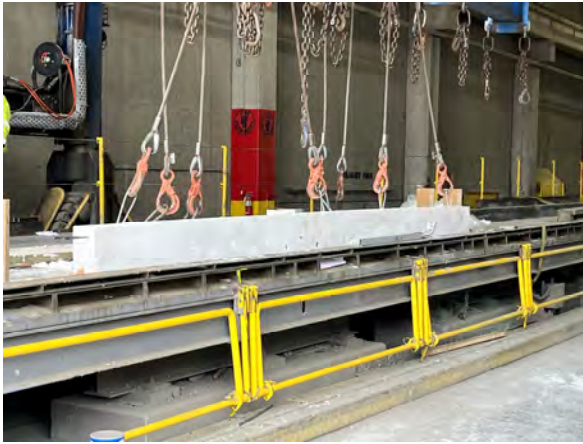


Prestressing Strands





Batching of Concrete Mix and Casting at Site



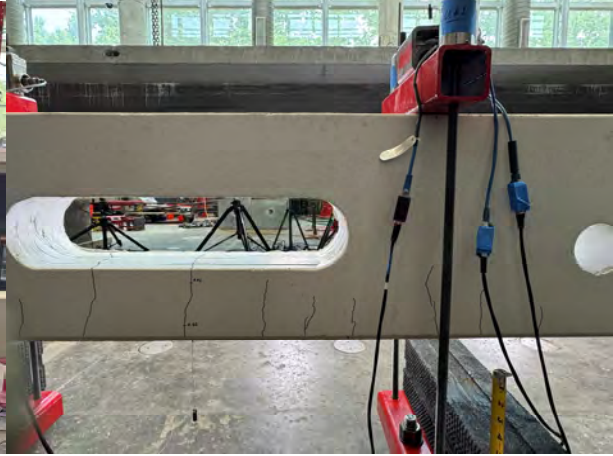
Lifting and Big Beam Transported to CFL



Compressive and Splitting Tensile Strength Tests

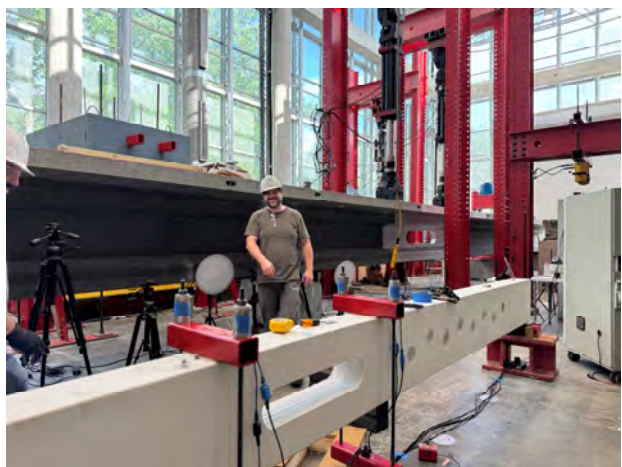


Instrumentation and Test Setup



Test Day





Photography Sessions



Team Meetings