Best Practices for Estimating Camber of Bulb T and Florida Girders

Prepared for Mississippi Department of Transportation

State Study No. 288

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Report Date: April 1, 2019

FHWA Technical Report Documentation Page

<table>
<tr><td>1.Report No.
FHWA/MDOT-RD-19-288</td><td>2. Government Accession No.</td><td colspan="2">3. Recipient's Catalog No.</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle
Best Practices for Estimating Camber of Bulb T and Florida Girders</td><td colspan="2">5. Report Date
April 1, 2019</td></tr>
<tr><td colspan="2">6. Performing Organization Code</td></tr>
<tr><td colspan="2">7. Author(s)
David A. Tomley, P.E., MSCE
Senior Structural Engineer</td><td colspan="2">8. Performing Organization Report No.
288</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address
Thompson Engineering, Inc.
100 Business Park Drive, Suite G
Ridgeland, MS 39157</td><td colspan="2">10. Work Unit No. (TRAIS)</td></tr>
<tr><td colspan="2">11. Contract or Grant No.
MS-DOT-RD-19-288</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address
Mississippi Department of Transportation
PO Box 1850
Jackson, MS 39215-1850</td><td colspan="2">13. Type Report and Period Covered
January 29, 2018 – April 1, 2019</td></tr>
<tr><td colspan="2">14. Sponsoring Agency Code</td></tr>
<tr><td colspan="4">15. Supplementary Notes</td></tr>
<tr><td colspan="4">16. Abstract
MDOT has experienced under-camber prestressed concrete girders recently on several projects that have led to construction delays and/or increased construction costs. The need to address current practices for estimating beam camber were addressed through; literature search, survey of other State DOT current practices, historic material and beam camber data provided by the Mississippi Concrete Girder Manufacturers, and camber estimate calculations for items that influence beam camber. The research findings included improvements to; better understanding of beam camber, material property versus strength expectation, ride smoothness, increased Industry awareness, advancing MDOT's current practices, enhancing MDOT's database of historic material and beam camber information, reducing design and/or functional modifications to MDOT projects, minimizing added project and infrastructure costs, and reducing delays during construction.</td></tr>
<tr><td colspan="2">17. Key Words
Precast/Prestressed Concrete, PCI, AASHTO, beam camber, bridges, deflections, concrete compressive strength</td><td colspan="2">18. Distribution Statement
Unclassified</td></tr>
<tr><td>19. Security Classif. (of this report)
Unclassified</td><td>20. Security Classif. (of this page)
Unclassified</td><td>21. No. of Pages</td><td>22. Price</td></tr>
</table>

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Acknowledgments

The author wishes to thank Bill Barstis, MDOT Research Division, for his assistance and coordination throughout the research study. In addition, Justin Walker, MDOT Bridge Design Division, developed the impetus for the research study. The research could not have been completed without the collaboration and input from the Mississippi Concrete Girder Manufacturers in particular; Christy Powell and Renford Lucas (J.J. Ferguson Prestress-Precast), Jim Polk (F-S Prestress), and Ben Spruill (Gulf Coast Pre-Stress).

The following MDOT employees contributed their time and assisted with the development of the research study through participation on the Technical Advisory Committee:

Rhea Vincent, Tucker Stafford, and Seth Roberts - MDOT Research Division

Paul T. Dees, Neal Terry, and Spencer Yates - MDOT Bridge Design Division

Will Crawley – MDOT Materials Division

The contribution put forth by everyone involved with the research study is greatly appreciated.

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Executive Summary

MDOT has experienced under-camber prestressed concrete girders recently on several projects that have led to; construction delays, increased construction costs, design and/or functional modifications, and the need to address current practices to estimate beam camber.

A comprehensive literature search, survey of other State DOT current practices related to beam camber, historic material and beam camber data provided by the Mississippi Concrete Girder Manufacturers, broad range of camber data sets/calculations, conclusions, recommendations, and implementation plan were the basis of the research study.

Improvements made consist of:

1) Better understanding of beam camber
2) Material property versus strength expectation
3) Ride smoothness
4) Increased Industry awareness
5) Advancing MDOT's current practices
6) Enhancing MDOT's database of historic material and beam camber information
7) Reducing design and/or functional modifications to MDOT projects
8) Minimizing added project and infrastructure costs
9) Reducing delays during construction

Background

The push for longer concrete girders has hastened because of reduced cost of construction, reduced cost of maintenance and reduced environmental impacts. With the extensions in length, estimation of the camber within the bridge beam becomes critical for ride, clearance, and performance especially for Bulb T and Florida girders with high design compressive strength.

MDOT’s current practice is to use draped (harped) strands to reduce the accumulated pre-stress force at the bottom of the beams near the ends. Other States use de-bonded straight strands and other stress relieving methods to accomplish the pre-stress force reduction. However, because draping the strands induces a countering pre-stress force in the top of the beam in conjunction with very high compressive strengths, the final cambers are traditionally very low and can be hard to predict. Methods used by other states have produced better final camber values, but have developed cracking and/or end bursting.

The objective of this research study is to create a “best practices” guideline and policy in predicting the camber for Bulb T and Florida girders with high design compressive strength including review of MDOT and Mississippi Concrete Girder Manufacturer’s documentation to compare design concrete strength verses actual concrete strength along with historical measured camber data. The research study will also increase MDOT’s knowledge base related to beam camber and include review of other State DOT practices related to beam camber.

Anticipated benefits carried within the “best practices or guidelines” will be; improved ride due to better camber prediction, improved vertical clearance prediction, improved performance of precast/prestressed concrete girders, improved material property versus strength expectation, minimize differences between the estimated and actual cambers which can cause construction delays and/or add costs to MDOT projects, and reduce design and/or functional modifications to MDOT projects.

The Technical Advisory Committee (TAC) consisted of representatives from MDOT’s Bridge Design Division, Materials Division, and Research Division.

Literature Search

An extensive literature review was conducted on twenty-six publications to capture a broad-range of topics related to the research study. Various publications were reviewed to ascertain what research has been previously performed related to beam camber and/or which literature documents address beam camber and/or contained additional information related to the research study. Items that influence beam camber noted in the various publications were summarized. Not every item that influenced camber was investigated; rather the items that were included in multiple literature documents were carried forward and included in the research study.

The list of the publications reviewed and items related to the research topic along with items that were noted to influence beam camber are included in Appendix B. The literature review document in Appendix B-1 includes quotes from the list of publications along with highlighted information that is of particular interest to the research topic.

Research Approach

The research study consisted of various tasks that addressed the research topic including close collaboration and input by MDOT's Bridge Design, Materials, and Research Divisions, and the Technical Advisory Committee. A kick-off meeting was held at MDOT to discuss the research plan, coordinate information that MDOT would provide throughout the research, and verify contact information. Two Technical Advisory Committee (TAC) meetings took place on August 3, 2018 and November 13, 2018 to further discuss the research progress and go over research findings summarized to date. Copies of the presentation given at the TAC meetings are included in Appendix A.

A literature search was performed to collect all relevant publications and reviewed with MDOT for applicability to the research project. The Precast/Prestressed Concrete Institute (PCI) publications website along with other Industries and Academic Institutes were utilized for the literature search.

MDOT's current practices related to girder camber was reviewed and compared to other State DOT practices. Historic material information related to beam camber and concrete compressive strengths were collected from the Mississippi Concrete Girder Manufacturers that included F-S Prestress, Gulf Coast Pre-Stress, and J.J. Ferguson Prestress-Precast.

The current AASHTO Bridge Design Specifications and the PCI Bridge Design Manual related to the research topic was reviewed.

Various camber estimate calculations were developed for various concrete girder types and span lengths to establish data sets for further analysis. Items that influenced beam camber that were selected to be included in the research study were further evaluated to compare the effect on estimating beam camber.

An interim report was submitted to MDOT and the TAC for review and comment with the final report addressing the interim report review comments.

Research Findings and Applications

LITERATURE REVIEW

The literature review generated the following list of items that are related to the research topic and/or were thought to influence beam camber:

- strength gain > 28 days
- material properties
- camber prediction methods
- camber variability
- section properties
- instrumentation & monitoring
- high strength concrete using local materials (LADOTD)
- temperature effects on camber
- design procedures
- measured camber
- prestress losses
- experimental program
- AASHTO specifications
- Sensitivity Study (TXDOT)
- probabalistic comparison/effect of variability on prestress losses and camber & deflections
- test data
- transportation weight limits
- factors that influence span capabilities (prestress losses, allowable tension, local producer member capabilities f'c)
- camber tolerances
- debonded strands
- anchor zone reinforcing
- QC records (WSDOT)
- humidity
- historical material data
- support conditions
- modification factors for camber estimates
- camber experiences by other State DOT's
- when to measure initial camber
- scheduling pours
- recommendations for practice
- curing
- strand development and transfer lengths

Eight of the twenty-six literature review documents included three or more of the above items within the body of the literature document as shown in the table in Appendix B-2. The items listed above that are related to the research topic that were noted in three or more of the literature review documents included:

- material properties
- camber prediction methods
- temperature effects on camber
- prestress losses
- historical material data
- modification factors for camber estimates

These items provided a starting point for further investigation into their influence on estimating beam camber.

AASHTO LRFD Bridge Design Specifications and PCI Bridge Design Manual

After the literature review was completed, MDOT and other State DOT current practices related to estimating beam camber was reviewed. The most current version of the AASHTO LRFD Bridge Design Specifications (BDS) and the PCI Bridge Design Manual was reviewed for further information and guidelines.

AASHTO's LRFD BDS provide guidelines on estimating prestress losses according to section 5.9.5. Approximate prestress losses are covered in Section 5.9.5.3-Approximate Estimate of Time-Dependent Losses and although not applicable to the research study, section 5.9.5.4 provides guidelines for estimating time-dependent losses along with commentary section C5.9.5.4.1 for additional information on applicability of Section 5.9.5.4.

The PCI Bridge Design Manual section 2.4.7 provides additional information on the density; section 3.4.2.6 discusses camber; section 8.7 discusses camber and deflection; and section 8.7.1 addresses the Multiplier Method for predicting time-dependent camber.

MDOT & Other State DOT Current Practices

Prior to performing camber calculations MDOT's current practices addressing beam camber were reviewed that included MDOT's Bridge Design Manual, bridge design memorandum, Bulb-T design procedure, and Prestressed Beam Camber Deflection spreadsheet. In addition, MDOT's Standard Specifications for Road and Bridge Construction was reviewed. To assist with developing camber data sets, sample plans for Bulb T bridge projects in Leake and Marshall County were provided by MDOT.

Other State DOT current practices related to beam camber were reviewed and included Florida, Nebraska, Texas, Washington, Alabama, and Louisiana.

Current practices were categorized into three areas that related to beam camber:

1. design of precast/prestressed concrete girders
2. camber estimates
3. information that is placed on the contract plans/drawings

Items that were expected to influence camber were listed for the following three categories (i.e., design, camber, and plan drawings) as shown in Table 1.

Table 1 - Items That Were Expected to Influence Camber

Item	Design	Camber	Plan Dwgs
strand profile (draped, straight, debonding, top strand)	X	X	X
fillet/haunch thickness	X	X	
roadway vertical curve ordinate	X	X	
camber estimating method (PCI Multiplier, time-dependent)	X	X	
dead load distribution	X	X	
girder section properties & strand templates	X	X	X
material properties (f'ci, f'c, E, unit weight, aggregate type)	X	X	X
prestress loss data (time, humidity, curing method)	X	X	X
temperature gradient	X	X	
prestressed beam detail plan sheet information	X	X	X
camber		X	

Specific guidelines for the selected State DOT's related to the above items associated with design, camber estimates, manufacture, and construction of precast/prestressed concrete girders were listed and are referenced in Appendix C.

Camber Limits

MDOT includes a camber limit in the Prestress Requirements Table on the contract plan drawings. The camber limit shown on the plans is the acceptable range of camber for the given precast/prestressed concrete girder.

Historic Material Data

It was important to collect historic material information in order to have insight into what the current concrete girder manufacturing capabilities are. The historic material data that was of interest to the research study included the concrete compressive strength at release, concrete compressive strength at twenty-eight (28) days, and actual beam cambers. Mississippi currently has three concrete girder manufacturers; F-S Prestress, Gulf Coast Pre-Stress, and J.J. Ferguson Prestress-Precast. The Mississippi Concrete Girder Manufacturers provided historic material data and are included in the August 3, 2018 TAC Presentation in Appendix A.

Concrete Compressive Strengths

Producer 1 provided three concrete test reports all from the same project with a required design release concrete strength of 5.6 ksi and required design 28-day concrete strength of 6.5 ksi. Camber data was not provided. The age and concrete compressive strength (i.e., break strength) are shown in Table 2 for the three concrete test reports.

Table 2 - Historic Material Data - Producer 1

report 1		report 2		report 3	
age	break strength (psi)	age	break strength (psi)	age	break strength (psi)
1	4706	1	5695	1	4270
1	5305	1	5745	1	4520
1	5435	1	5895	1	4335
1	5845	1	no break	1	6670
1	6050	1	no break	1	6955
1	5995	1	no break	1	6915
2	7675	2	no break	2	no break
2	7190	2	no break	2	no break
2	7040	2	no break	2	no break
3	no break	3	7960	3	no break
3	no break	3	7535	3	no break
3	no break	3	7595	3	no break
28	9755	28	9035	28	10265
28	9455	28	9180	28	9570
28	9370	28	9075	28	9800

The average concrete strengths are shown in Figure 1 for the three concrete test reports. The graph provides insight to the amount of strength gain the concrete achieves from release to 28-days.

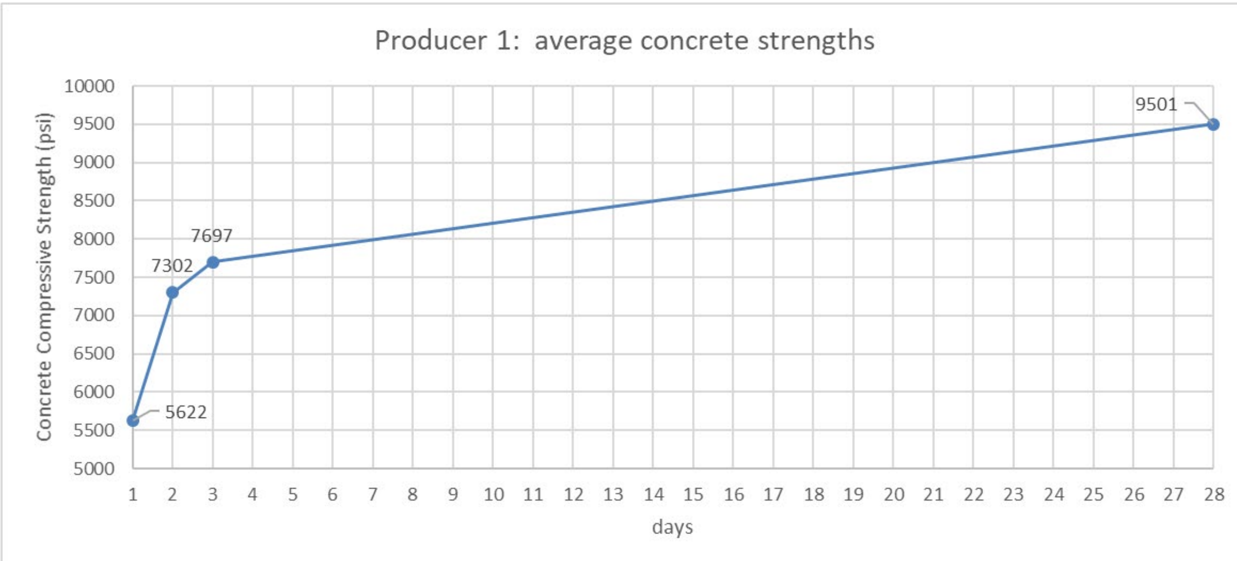

Figure 1 - Average Concrete Strengths (Producer 1)

Producer 2 provided concrete pour reports from five separate projects. Projects included AASHTO Type 4 girders with lengths of 100 and 110 ft. The required design concrete strength at release varied from 4.2 to 5.0 ksi and the required design concrete strength at 28-days varied from 5.0 to 6.0 ksi. Design camber, measured camber data at release, and 28-day measured camber were provided. Project 5 did not include 28-day data. Additional data and graphs of the measured camber are included in the August 3, 2018 TAC presentation in Appendix A-1.

The age and concrete compressive strength (i.e., break strength) are shown in Table 3 for two pours from Project 1. The remainder of the break strength data for all five projects and pours are included in the August 3, 2018 TAC presentation in Appendix A-1.

Table 3 - Partial Historic Material Data (Producer 2)

project 1

pour 1		pour 2	
	break strength		break strength
age	(psi)	Age	(psi)
1	4431	1	8788
1	4205	1	9415
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
2	no break	2	no break
2	no break	2	no break
2	no break	2	no break
3	no break	3	no break
3	no break	3	no break
3	no break	3	no break
28	11124	28	10848
28	12449	28	10358
28	11218	28	10262

Average release concrete compressive strengths were plotted and compared to the design release concrete compressive strengths shown in Figure 2.

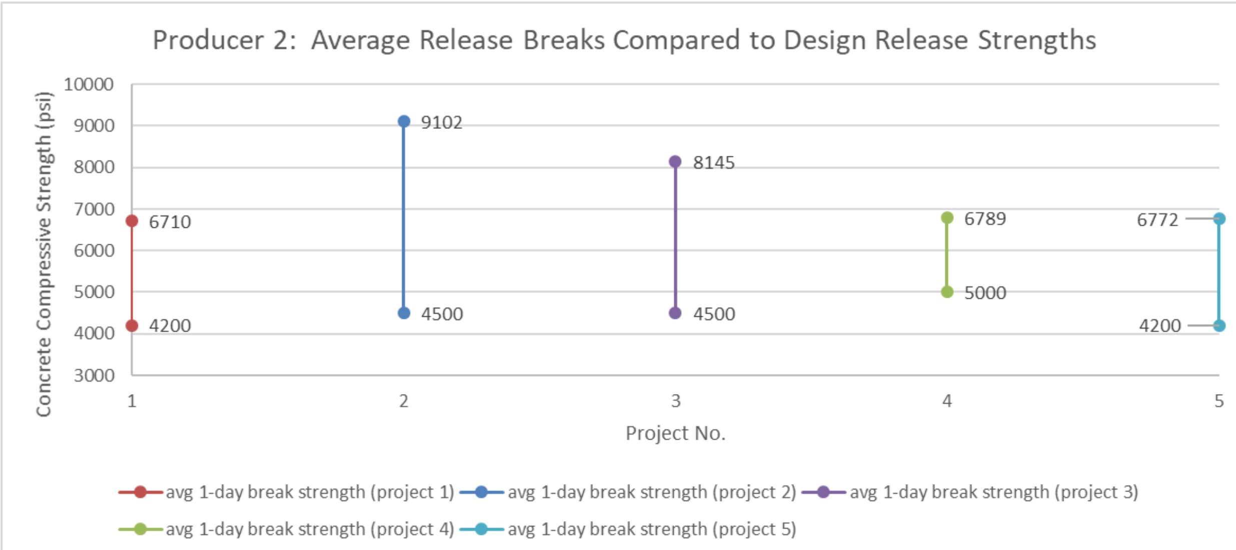

Figure 2 - Average Release Concrete Compressive Strengths Compared to Design Concrete Compressive Strengths (Producer 2)

Average 28-day concrete compressive strengths were plotted and compared to the design release concrete compressive strengths shown in Figure 3.

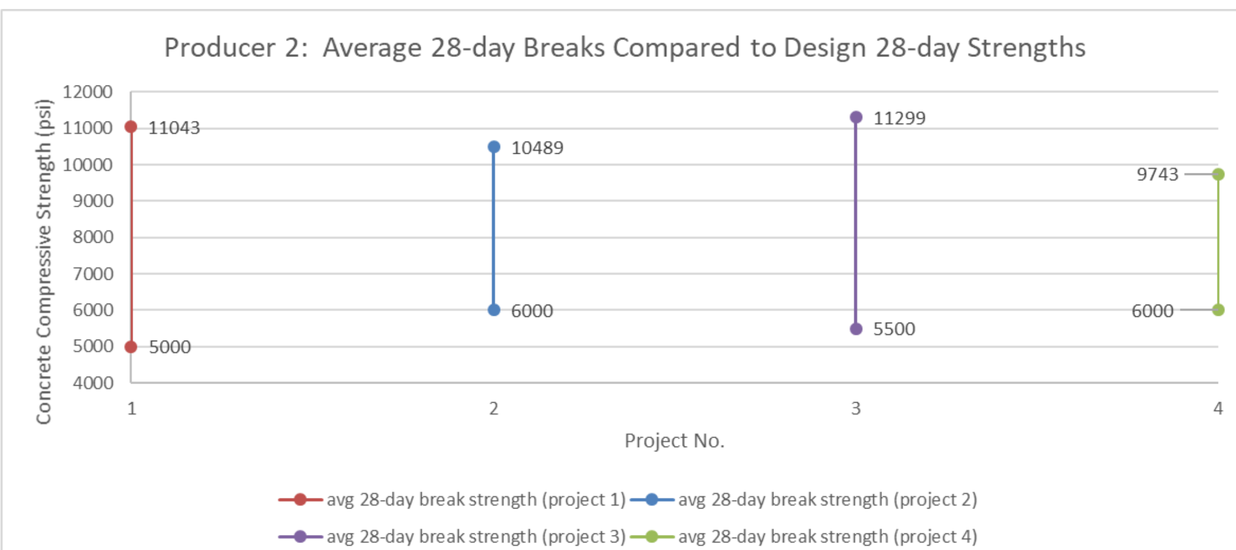

Figure 3 - Average 28-day Concrete Compressive Strengths Compared to Design Concrete Compressive Strengths (Producer 2)

The average concrete strengths are shown in Figure 4 for the four projects. The graph provides insight to the amount of strength gain the concrete achieves from release to 28-days.

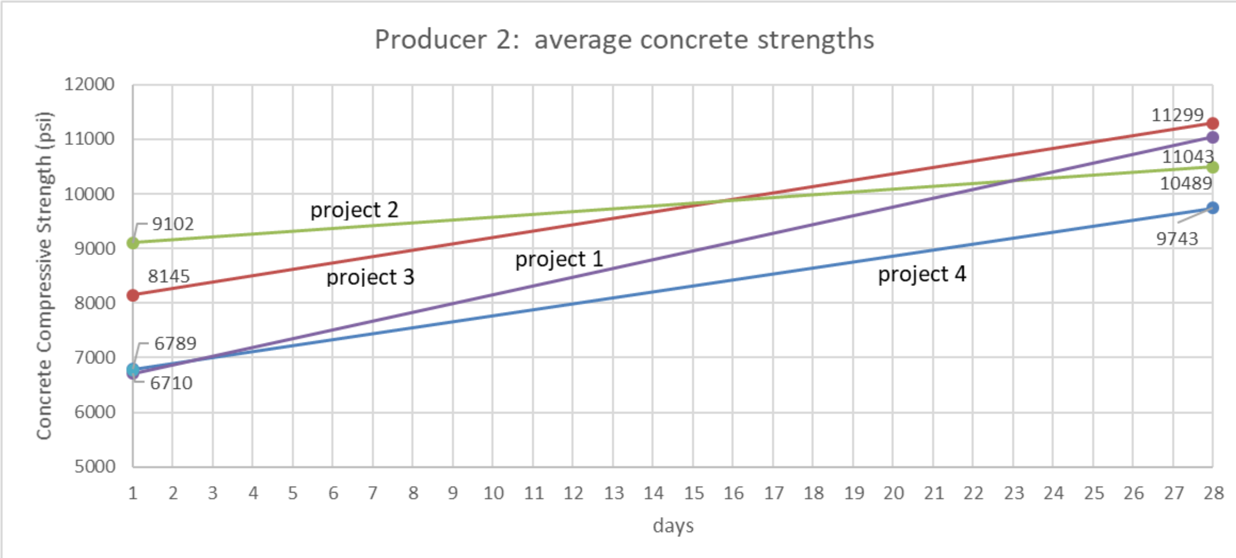

Figure 4 - Average Concrete Strengths (Producer 2)

Producer 3 provided fourteen concrete test reports all from the same project with a required design release concrete strength of 6.8 ksi and required design 28-day concrete strength of 8.5 ksi. Camber data at release was provided. The age and concrete compressive strength (i.e., break strength) are shown in Table 4 for three pours. The remainder of the break strength data from the test reports are included in the August 3, 2018 TAC presentation in Appendix A

-1.

Table 4 - Partial Historic Material Data (Producer 3)

pour 1		pour 2		pour 3	
	break strength		break strength		break strength
age	(psi)	age	(psi)	age	(psi)
3	9015	1	7030	3	6765
3	9180	1	7510	3	8305
12	9666	1	7095	3	7595
12	9522	1	7390	14	9867
28	10203	14	9143	14	9984
28	10237	14	9100	14	10125
		14	9299	14	10106
		14	9344	28	10310
		28	10048	28	10270
		28	10000	28	10440
		28	10488	28	10408
		28	10520		

Average release and 28-day concrete compressive strengths were plotted and compared to the design release and 28-day concrete compressive strengths shown in Figure 5.

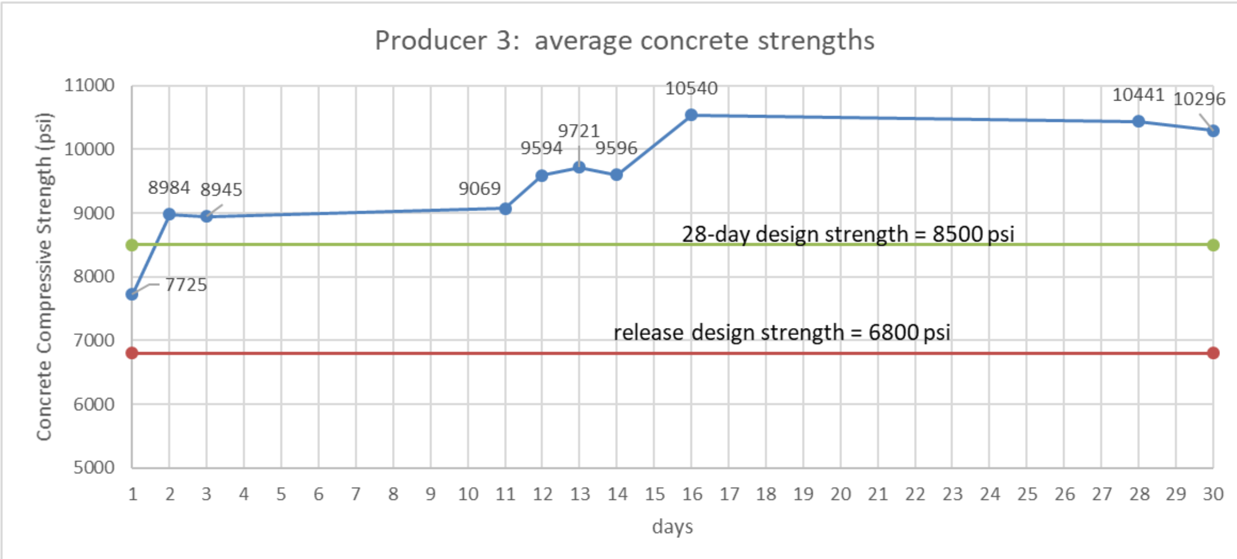

Figure 5 - Average Concrete Strengths (Producer 3)

Camber at release data were plotted for the various beam types shown in Figure 6.

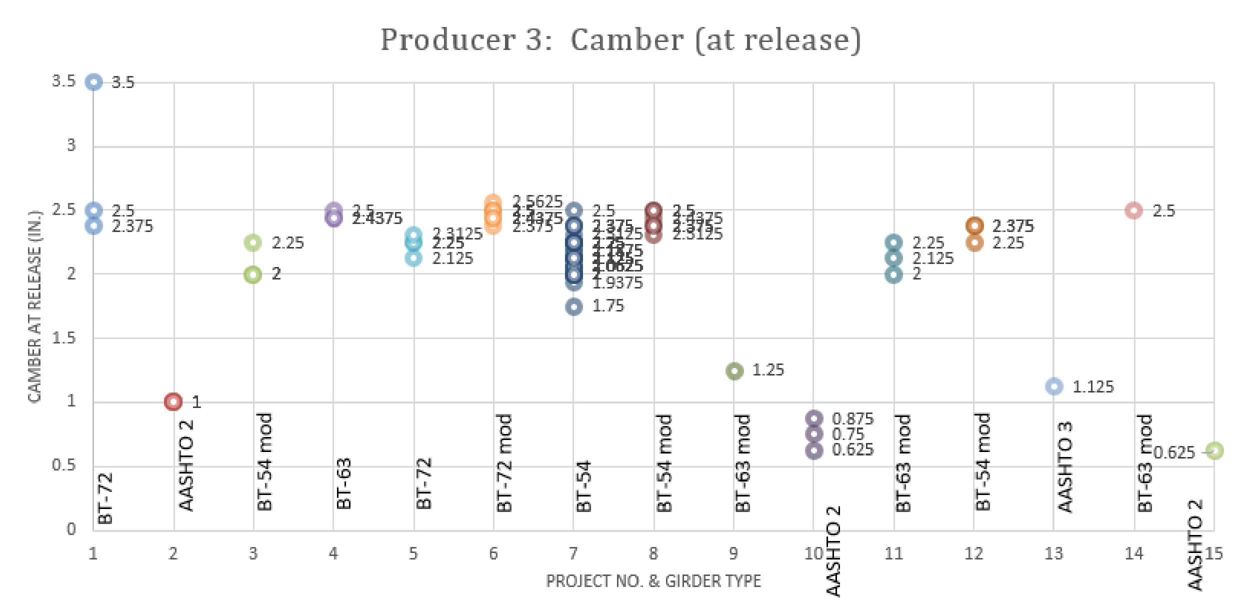

Figure 6 - Camber At Release (Producer 3)

The data indicates that there is variation in camber at release.

A summary of the design and average concrete compressive strengths at release (i.e., f'ci) and at 28-days (i.e., f'c) provided by the three Mississippi Concrete Girder Manufacturers are shown in Table 5.

Table 5 - Historic Material Data - Concrete Strength Summary

	Producer 1	Producer 2	Producer 3
design f'ci	5600 psi	4480 psi	6800 psi
Average actual f'ci	5622 psi	7503 psi	7725 psi
Ratio of average actual f'ci/design f'ci	1.004	1.67	1.14
Design f'c	6500 psi	5625 psi	8500 psi
Average actual f'c	9501 psi	10644 psi	10441 psi
Ratio of average actual f'c/design f'c	1.46	1.89	1.23
Ratio of Average actual f'ci/average actual f'c	0.59	0.71	0.74

Based on the historic material data, several observations with respect to the release concrete compressive strength (f'ci) and 28-day concrete compressive strength (f'c) were made to advance the research:

1. Use the low, high, and average ratios to vary f'ci and f'c to evaluate the effects on camber estimates. Values for f'c include; low = 1.004, average = 1.27, high = 1.66. Values for f'c include; low = 1.23, average = 1.53, high = 1.89.

2. Producer 3 provided 2-day break data (not shown in Table 5) that can be considered in utilizing higher design release strengths. For example, the average 2-day actual concrete compressive strength was 8984 psi which is a ratio increase from the 6800 psi design f'ci of 1.32.

3. The relationship between f'ci/f'c can be used to understand the strength gain during design. The above values based on historic material information can assist in establishing guidelines. For example the lowest ratio was 0.59, the highest was 0.74, and the average ratio was 0.68.

Beam Camber

A summary of the actual (i.e., measured) beam cambers by the Mississippi Concrete Girder Manufacturers are shown in Table 6.

Table 6 - Historic Material Data - Actual Beam Camber Summary

	Producer 1	Producer 2	Producer 3
Estimated camber (at release)	Data not provided	0.94 in.	Data not provided
Average measured camber (at release)	no camber data provided	0.74 in.	Provided data on 15 projects with different beam types
Average measured camber (28-days)	no camber data provided	2.37 in.	no camber data provided
Ratio of Average measured camber (28-days)/average measured camber (at release)	no camber data provided	3.2	no camber data provided

Based on the historic material data, several observations with respect to beam cambers were made to advance the research:

1. For Producer 2; use both the average actual f'ci and design f'ci to estimate camber at release for similar beam type and span length and compare the estimated camber differences at release to see if there is a correlation between the estimated camber and measured camber at release. Compare the 28-day camber data to the estimated camber data sets.

2. For Producer 3; the camber data shows variation in the measured camber at release; therefore calculate the range of variation (i.e., low and high values from the average measured camber). Look for consistencies between various beam types on the spread/magnitude the variations in the measured cambers at release. Vary f'ci and Eci using average actual f'ci values and compare the effects with measured camber to see if there is a correlation to the relationship between; the design f'ci and estimated camber and the actual f'ci and measured camber.

MDOT provided deck calculations during construction from the contractor for the Byhalia Bridge on SR309 located in Marshall County (100299/303000). MDOT through its consultant on the project provided the following data shown in Table 7, which compares the actual beam camber to the estimated beam camber at erection (before added dead load deflection). Actual beam camber at erection (after added dead load deflection) was unkown. For the information provided, the actual beam cambers at erection for Spans 1, 2, and 3 were appreciably less than the estimated beam camber. This project demonstrates that the manufactured concrete girders have less camber at erection than estimated.

Table 7 - Historic Camber Data - MDOT Project

Project	Span Length (ft)	Beam Type & No. Strands	Required concrete strengths (f'ci/f'c)	Estimated beam camber at Erection (before added DL)	Actual beam camber at Erection (before added DL)	Estimated beam camber (after added DL deflection)
SR309 Marshall Co	96 Span 1	BT-54 26 Strands	7000 5500 Release	2-3/8”	Varies 1.08” to 1.56”	13/16”
	138 Span 2	BT-72 34 Strands	7000 5500	2-9/16”	Varies 0.48” to 2.16”	1/8”
	96 Span 3	BT-54 26 Strands	7000 5500	2-3/8”	Varies 0.0” to 1.32”	13/16”

Camber Data Sets

Before camber estimate calculations could be made, a camber data set outline (see Appendix D-1) was developed to provide an overview and plan of information that would be researched to evaluate the effects on beam camber. The information collected from the literature review, historic material information provided by the Mississippi Concrete Girder Manufacturers, MDOT projects that have experienced "under-camber" on girders at erection, and MDOT's current practices for estimating camber were considered in the camber data sets. Two pieces of information were known and were to be evaluated in the camber data sets:

1. Actual concrete compressive strengths both at release and at 28-days are greater than design concrete compressive strengths.

2. Several MDOT projects have experienced under-camber on girders at erection. Data provided by MDOT provided insight to the amount of differences between the design camber compared to actual/measured camber at erection.

The plan for the camber data sets was to evaluate the differences between camber estimates using actual concrete compressive strengths based on the historic material information provided by the Mississippi Concrete Girder Manufacturers compared to design concrete compressive strengths. Various sets of data were developed through example camber estimate calculations that captured the sensitivity of the difference in camber estimates between using actual concrete compressive strengths compared to using the design concrete compressive strengths. For a particular girder type, the minimum and maximum span capabilities were evaluated to capture the range of camber variations by varying the girder lengths. The sample plans provided by MDOT's Bridge Division for the Leake County and Marshall County projects were used along with AASHTO Type 4 girders and FL Bulb-T girders.

Items that Influence Camber

The following items are expected to influence camber and were included in the camber data sets.

- Material Properties
  - Release concrete compressive strength (f'ci)
  - 28-day concrete compressive strength (f'c)
  - Release modulus of elasticity (Eci)
  - 28-day modulus of elasticity (Ec)
  - Unit weight (wc)

- The self weight of the girders were varied by using different units weights (e.g., 150 pcf, 155 pcf, and 160 pcf) to evaluate the effect of unit weight on estimated camber.
- The girder self weight (for a BT-54 and FL Bulb-T FIB-72) accounting for the additional weight of the prestressing strand and girder reinforcing was compared to the girder self weight based on the girder section only. The increase in the unit weight is approximately 5 pcf when considering the additional weight of the prestressing strand and girder reinforcing.

– Aggregate Type

- AASHTO LRFD Equation 5.4.2.4-1 for the Modulus of Elasticity is dependent on; the correction factor for source of aggregate (K1) where K1 is typically taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction; unit weight of concrete; and specified compressive strength of concrete.

- Girder Section Properties

– Transformed vs. Gross section properties

Since MDOT currently uses transformed section properties, gross section properties were evaluated to compare differences in the prestress losses and thus the influence on beam camber. Therefore, the camber data sets compared prestress losses with and without transformed section properties to evaluate whether prestress losses differ; if prestress losses differ between using gross section properties compared to transformed section properties then isolate the moment of inertia effects when using transformed section properties by manually entering the percentage of prestress losses (%) to be the same for gross section properties and transformed section properties.

– Girder Type/Moment of Inertia

Four different girder types were selected to capture the sensitivity of camber estimates.

- AASHTO Type 4
- BT-54 Marshall County
- BT-72 Leake County
- FIB-72

- Span Lengths

  Three span lengths were selected for each girder type to capture the minimum, average, and maximum span ranges; again to capture the sensitivity of camber estimates.

    - 90 ft. AASHTO Type 4
    - 110 ft. BT-54 Marshall County
    - 138 ft. BT-72 Leake County
    - 155 ft. FIB-72

- Prestress losses and data

  As it pertains to prestress loss computations, various parameters are used part of the prestress loss estimates that include; time at release, age of deck placement, final age, and relative humidity.

  PCI's Bridge Design Manual (design examples) assume the following construction schedule; 1-day at transfer, 90-days at erection/deck placement, and 20,000-days at final stage. Section 9.0.1 Service Life discusses the assumed age for the various design examples as related to long-term (i.e., final) prestress losses.

  - Age of girder at erection
    - Time can vary and is dependent on the construction schedule
  - Age of deck placement
    - Time can vary and is dependent on the construction schedule
  - Beam curing time (1, 2, or 3 days)
  - Beam storage age (3, 6 or 12 months)
  - Time-dependent model analysis
    - Final age
    - Time can vary depending on service life assumptions
  - Humidity/seasonal variation
    - Humidity can vary depending on location of Mississippi Concrete Girder Manufacturer and time of year

- Curing method (moist vs. steam)
    - Not included in the research but the curing method can vary depending on selection by Mississippi Concrete Girder Manufacturer
- Jacking force
    - Not included in the research but some amount of variation is expected in the method and sequence the Mississippi Concrete Girder Manufacturers use when jacking and releasing the prestressing strand

- Strand Patterns and Profile
    - Draped strand
    - Straight strand (including debonding)
    - Top Strand
        - Where required to satisfy allowable stresses using straight strand patterns, top strand with and/or without reduced pull
    - Number and size of strand
    - Strand templates
- Haunch/fillet thickness

    The haunch/fillet consists of the additional concrete that is placed on top of precast concrete girder and between the bottom of the deck slab and the top of the precast concrete girder. The thickness of the haunch/fillet is dependent on the beam camber at erection (after all dead load deflection), roadway vertical curve ordinate, deck cross-slope/superelevation, and the as-constructed beam seat elevations. A minimum thickness at the edge of flange is suggested to facilitate forming deck. The thickness of the haunch/fillet varies along the length of the beam due to camber in the prestressed concrete girder. The haunch/fillet thickness can be included in the composite section or not, and if not added as additional dead load on the girder.

    The camber data sets compared deflections at erection with and without the haunch/fillet thickness in the composite section properties.

- Dead load distribution

  AASHTO LRFD 4.6.2.2-Beam-Slab Bridges states "Where bridges meet the conditions specified herein, permanent loads of and on the deck may be distributed uniformly among the beams and/or stringers."

  Therefore, the camber data sets assumed that the dead load of the concrete barriers are equally distributed to all girders in the bridge transverse section.

- PCI multipliers used in the PCI Multiplier Method for estimating camber

  A common method to calculate beam camber is to utilize the PCI Multiplier Method; which consists of calculating the deflections at release independently for the prestressing effect and self-weight effect then superimposing the prestressing effect and self-weight effect to obtain the net camber at release. To obtain the beam camber at erection; apply the PCI multipliers to the prestress deflection and the self-weight deflection separately. The respective multipliers for the prestress effect and self-weight effect are 1.80 and 1.85 respectively. The net camber at erection include the superposition of the prestress and girder self-weight effects.

  Refer to PCI Bridge Design Manual 8.7.1. Deflection (down) multiplier at erection = 1.85 and camber (up) multiplier at erection = 1.80. Apply the deflection and camber multipliers to the deflection due to girder self-weight and camber due to prestressing respectively.

- Temperature gradients

  Refer to AASHTO LRFD 3.12.3-Temperature Gradient and Commentary section 3.12.3 for guidelines.

- Roadway Vertical curve ordinate

  - Although this item is not directly related to estimating camber, it could have an effect on the haunch thickness at the ends of girders and is related to calculating beam seat elevations, therefore procedures are suggested to include the roadway vertical curve ordinate in the calculation of the haunch/fillet thickness at the ends of the girders.
  - Depending on whether the bridge is located within a crest or sag vertical curve, an adjustment to the haunch/fillet thickness is suggested to account for elevation differences between a non-linear profile grade and the linear grade connecting the centerline of girder supports at the end of the girders
  - The research did not consider the roadway vertical curve ordinate.

Girder Types, Bridge Typical Sections, and Span Lengths

Four girder types and associated span lengths were selected to capture the variations in estimating beam camber. Girder span lengths were increased and decreased to further capture the upper and lower bounds of beam camber estimates and variations. Sample plans were provided by MDOT for the BT-54 and BT-72 girder types located in Marshall and Leake Counties respectively. Although the Marshall and Leake County projects contained multiple spans with different girder types and span lengths, only one girder type and span length was selected and included in the camber data sets.

- AASHTO Type 4 [90 ft.]
- BT-54 (Marshall County) [110 ft.]
- BT-72 (Leake County) [138 ft.]
- FIB-72 [155 ft.]

A time dependent model was analyzed for the FIB-72 girder to further evaluate the influence of; humidity, girder fabrication time (e.g., 1, 2, and 3 days), average historical concrete compressive release and 28-day strengths, and extended beam storage time (e.g., 6 and 12 months).

Baseline Camber Data Set Assumptions

All baseline analyses used the following input data. The assumptions used for the baseline analyses attempted to be consistent with MDOT's Bridge Design practices. Two differences of note is the concrete compressive strength for the deck when designing Florida Bulb T beams is 4.5 ksi and the weight of metal stay-in-place deck forms are typically included in the design of the beams.

- Deck thickness = 7.75 inches
- Two Barriers weight = 0.600 klf (equally distributed over all girders)
- 0.6" low-relaxation strands
- f'c deck = 4.0 ksi
- Girder unit weight = 0.150 kcf
- Transformed section properties
- Haunch/fillet thickness added as dead load to girder and not included in composite section
- Approximate prestress losses
- PCI Multiplier Method for camber and deflection calculations at erection

The following LEAP Bridge Concrete software programs (Version 18.00.00.34) by Bentley Systems were used for the camber data set calculations; Precast/Prestressed Girder was utilized for the camber calculations and Spliced-Girder was utilized for the time-dependent model analysis.

AASHTO Type 4 [90 ft.]

An interior AASHTO Type 4 girder from the bridge transverse section shown in Figure 7 was analyzed part of the camber data set calculations.

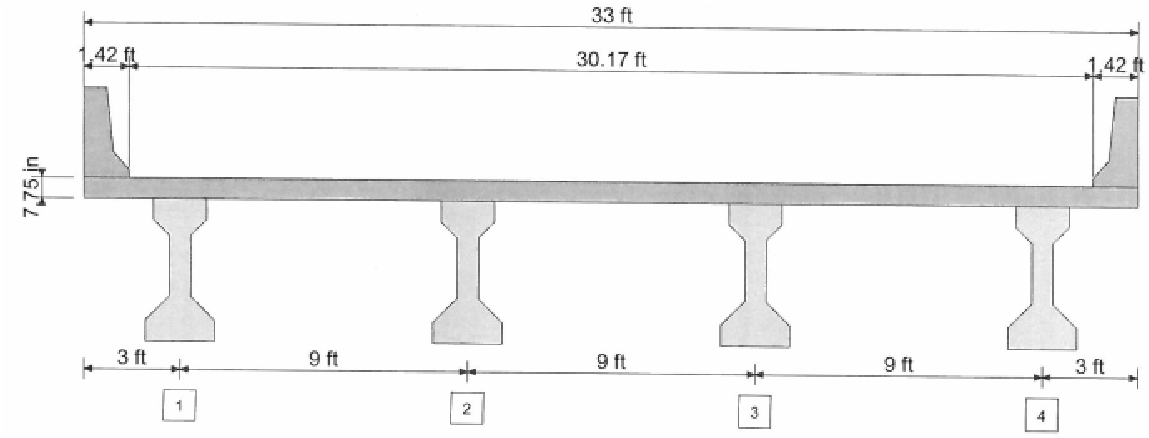

Figure 7 - AASHTO Type 4 Bridge Transverse Section used for Data Sets

A baseline span length of 90 ft. was used with design compressive strengths at release and 28-days of 5.5 ksi and 8.0 ksi respectively. Twenty-eight (28) 0.6-inch diameter draped strands were required. Lower and upper span lengths of 75 ft. and 105 ft. were also analyzed.

Estimated cambers were calculated at release and at erection (before added dead load deflection) and at erection (after added dead load deflection) for the baseline analysis and for the various items that were expected to influence camber. Table 8 summarizes the estimated cambers. A concrete compressive strength of 7.0 ksi was used for the average release strength (f'ci). A concrete compressive strength of 12.2 ksi was used for the average 28-day strength (f'c). An aggregate factor (K1) of 1.10 was used for the limestone aggregate.

Table 8 - Data Sets for Camber Calculations (AASHTO Type 4)

girder camber (inches)		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline analysis	2.437	-0.935	1.502	4.386	-1.729	2.657	-0.987	1.670
2	limestone aggregate	2.237	-0.854	1.383	4.027	-1.580	2.447	-0.905	1.542
3	avg f'ci/f'c, unit wt 155, gross section	2.159	-0.878	1.281	3.886	-1.624	2.262	-0.849	1.413
4	baseline analysis, minimum span length 75 ft.	1.257	-0.456	0.801	2.262	-0.844	1.418	-0.482	0.936
5	baseline analysis, maximum span length 105 ft.	4.195	-1.715	2.480	7.551	-3.172	4.379	-1.813	2.566
6	avg f'ci/f'c, limestone aggregate	2.082	-0.792	1.290	3.747	-1.465	2.282	-0.793	1.489

BT-54 (Marshall County) [110 ft.]

An interior BT-54 girder from the bridge transverse section shown in Figure 8 was analyzed part of the camber data set calculations.

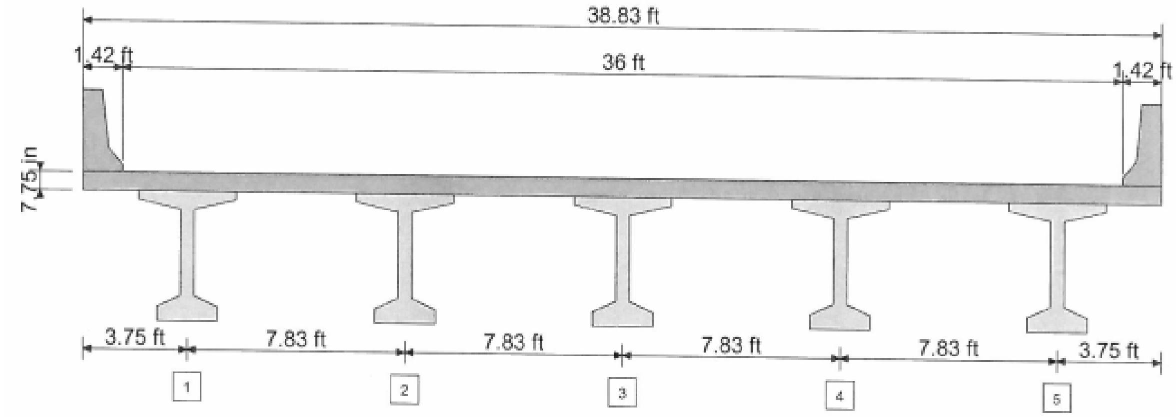

Figure 8 - BT-54 (Marshal County) Bridge Transverse Section used for Data Sets

A baseline span length of 110 ft. was used with design compressive strengths at release and 28-days of 6.3 ksi and 7.7 ksi respectively. Thirty two (32) 0.6-inch diameter draped strands were required. Lower and average span lengths of 80 ft. and 95 ft. were also analyzed.

Estimated cambers were calculated at release and at erection (before added dead load deflection) and at erection (after added dead load deflection) for the baseline analysis and for the various items that were expected to influence camber. Table 9 summarizes the estimated cambers. A concrete compressive strength of 8.0 ksi was used for the average release strength (f'ci). A concrete compressive strength of 11.8 ksi was used for the average 28-day strength (f'c). A concrete compressive strength of 10.46 ksi was used for the high release strength (f'ci). A concrete compressive strength of 14.6 ksi was used for the high 28-day strength (f'c). An aggregate factor (K1) of 1.10 was used for the limestone aggregate.

Table 9 - Data Sets for Camber Calculations (BT-54 Marshall County)

girder camber (inches)		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline analysis	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.931	3.012
2	avg f'ci/f'c	3.998	-1.390	2.608	7.197	-2.571	4.626	-1.691	2.935
3	high f'ci/f'c	3.698	-1.279	2.419	6.657	-2.366	4.291	-1.588	2.703
4	unit wt 155	4.045	-1.454	2.591	7.281	-2.690	4.591	-1.819	2.772
5	unit wt 160	3.826	-1.414	2.412	6.886	-2.616	4.270	-1.717	2.553
6	gross section	4.334	-1.592	2.742	7.801	-2.944	4.857	-2.056	2.801
7	fillet included	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.918	3.025
8	temperature	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.513	3.430
9	PCI mult 1.0	4.283	-1.496	2.787	4.283	-1.496	2.787	-1.931	0.856
10	increase fillet thickness	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.998	2.945
11	top strand	4.205	-1.487	2.718	7.568	-2.752	4.816	-1.921	2.895
12	straight strands with debonding	4.576	-1.496	3.080	8.236	-2.767	5.469	-1.931	3.538
13	2, 4	3.774	-1.350	2.424	6.794	-2.498	4.296	-1.593	2.703
14	2, 5	3.568	-1.313	2.255	6.422	-2.430	3.992	-1.502	2.490
15	2, 7	3.998	-1.390	2.608	7.197	-2.571	4.626	-1.680	2.946
16	2, 4, 6, 11	3.744	-1.423	2.321	6.739	-2.633	4.106	-1.678	2.428
17	2, 4, 6, 10, 11	3.744	-1.423	2.321	6.739	-2.633	4.106	-1.736	2.370
18	baseline analysis (including prestress losses)	3.863	-1.496	2.367	6.953	-2.767	4.186	-1.931	2.255
19	2, 4 (including prestress losses)	3.404	-1.350	2.054	6.126	-2.498	3.628	-1.593	2.035
20	baseline analysis, minimum span length 80 ft.	1.490	-0.458	1.032	2.682	-0.847	1.835	-0.592	1.243
21	baseline analysis, average span length 95 ft.	2.744	-0.898	1.846	4.940	-1.661	3.279	-1.159	2.120
22	limestone aggregate	3.941	-1.368	2.573	7.094	-2.532	4.562	-1.770	2.792
23	avg f'ci/f'c, limestone aggregate	3.676	-1.271	2.405	6.617	-2.351	4.266	-1.550	2.716

To gain insight into which items influenced camber estimates the most and/or combination of several of the items acting together, Table 10 includes the percentage (%) difference between the baseline analysis.

Table 10 - Data Sets for Camber Calculations (BT-54 Marshall County) % Difference

	BT-54 (110 ft.)						
#		camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
1	baseline analysis	2.787	% difference	4.943	% difference	3.012	% difference
2	avg f'ci/f'c	2.608	-6.42%	4.626	-6.41%	2.935	-2.56%
3	high f'ci/f'c	2.419	-13.20%	4.291	-13.19%	2.703	-10.26%
4	unit wt 155	2.591	-7.03%	4.591	-7.12%	2.772	-7.97%
5	unit wt 160	2.412	-13.46%	4.270	-13.62%	2.553	-15.24%
6	gross section	2.742	-1.61%	4.857	-1.74%	2.801	-7.01%
7	fillet included	2.787	0.00%	4.943	0.00%	3.025	0.43%
8	temperature	2.787	0.00%	4.943	0.00%	3.430	13.88%
9	PCI mult 1.0	2.787	0.00%	2.787	-43.62%	0.856	-71.58%
10	increase fillet thickness	2.787	0.00%	4.943	0.00%	2.945	-2.22%
11	top strand	2.718	-2.48%	4.816	-2.57%	2.895	-3.88%
12	straight strands with debonding	3.080	10.51%	5.469	10.64%	3.538	17.46%
13	2, 4	2.424	-13.02%	4.296	-13.09%	2.703	-10.26%
14	2, 5	2.255	-19.09%	3.992	-19.24%	2.490	-17.33%
15	2, 7	2.608	-6.42%	4.626	-6.41%	2.946	-2.19%
16	2, 4, 6, 11	2.321	-16.72%	4.106	-16.93%	2.428	-19.39%
17	2, 4, 6, 10, 11	2.321	-16.72%	4.106	-16.93%	2.370	-21.31%
18	baseline analysis (including prestress losses)	2.367	-15.07%	4.186	-15.31%	2.255	-25.13%
19	2, 4 (including prestress losses)	2.054	-26.30%	3.628	-26.60%	2.035	-32.44%
22	limestone aggregate	2.573	-7.68%	4.562	-7.71%	2.792	-7.30%
23	avg f'ci/f'c, limestone aggregate	2.405	-13.71%	4.266	-13.70%	2.716	-9.83%

Table 11 summarizes the camber at release and camber at erection (before added dead load deflection) and sorts the camber values from largest to smallest.

Table 11 - Data Sets for Camber Calculations Sorted (BT-54 Marshall County)

description	camber at release	description	camber at erection
straight strands with debonding	3.080	straight strands with debonding	5.469
baseline analysis	2.787	baseline analysis	4.943
fillet included	2.787	fillet included	4.943
temperature	2.787	temperature	4.943
PCI mult 1.0	2.787	increase fillet thickness	4.943
increase fillet thickness	2.787	gross section	4.857
gross section	2.742	top strand	4.816
top strand	2.718	avg f'ci/f'c	4.626
avg f'ci/f'c	2.608	avg f'ci/f'c, fillet included	4.626
avg f'ci/f'c, fillet included	2.608	unit wt 155	4.591
unit wt 155	2.591	limestone aggregate	4.562
limestone aggregate	2.573	avg f'ci/f'c, unit wt 155	4.296
avg f'ci/f'c, unit wt 155	2.424	high f'ci/f'c	4.291
high f'ci/f'c	2.419	unit wt 160	4.270
unit wt 160	2.412	avg f'ci/f'c, limestone aggregate	4.266
avg f'ci/f'c, limestone aggregate	2.405	baseline analysis (including prestress losses)	4.186
baseline analysis (including prestress losses)	2.367	avg f'ci/f'c, unit wt 155, gross section, top strand	4.106
avg f'ci/f'c, unit wt 155, gross section, top strand	2.321	avg f'ci/f'c, unit wt 155, gross section, increase filklet thickness, top strand	4.106
avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness, top strand	2.321	avg f'ci/f'c, unit wt 160	3.992
avg f'ci/f'c, unit wt 160	2.255	avg f'ci/f'c, unit wt 155 (including prestress losses)	3.628
avg f'ci/f'c, unit wt 155 (including prestress losses)	2.054	PCI mult 1.0	2.787

Several of the items that influenced camber in Table 11 were plotted as shown in Figure 8. The historic camber at release data measured by the Mississippi Concrete Girder Manufacturers are shown horizontally in the graph to compare with the estimated camber at release data. For the items plotted there is a close comparison to the estimated camber at release to the measured camber at release.

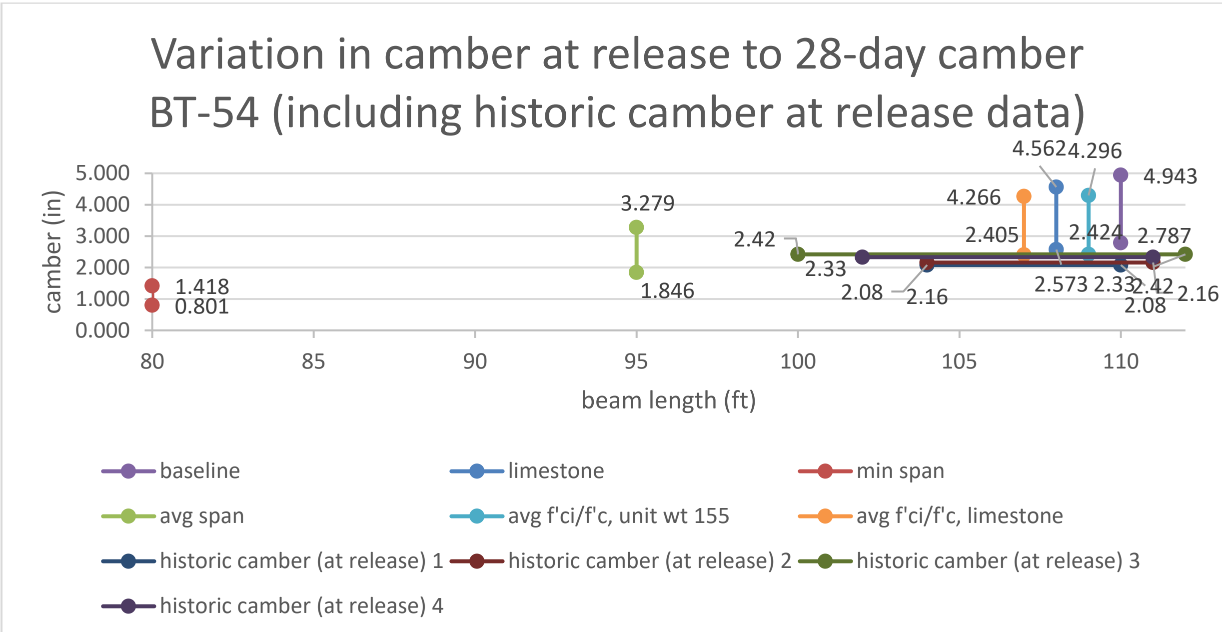

Figure 8 - Variation in Camber at Release and 28-Days (BT-54)

BT-72 (Leake County) [138 ft.]

An interior BT-72 girder from the bridge transverse section shown in Figure 9 was analyzed part of the camber data set calculations.

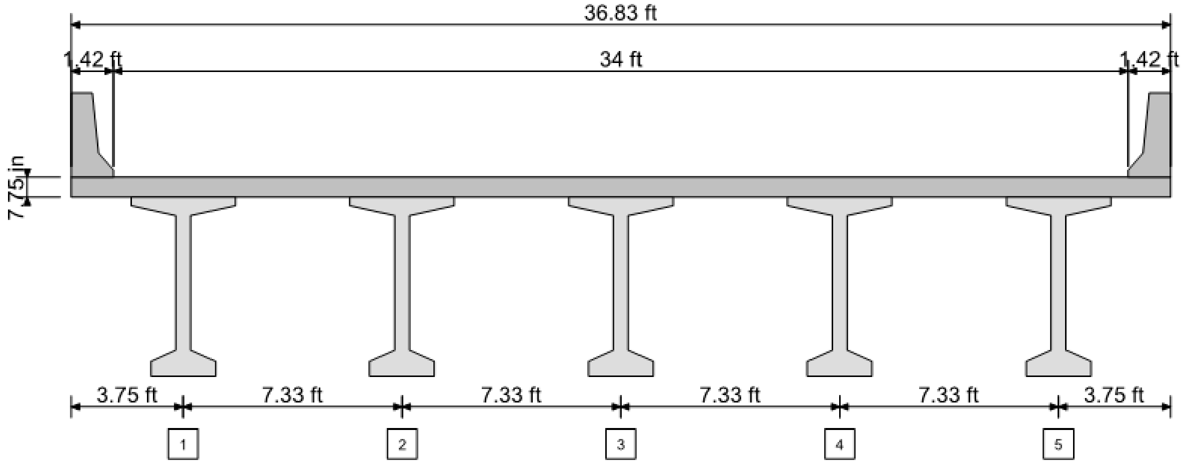

Figure 9 - BT-72 (Leake County) Bridge Transverse Section used for Data Sets

A baseline span length of 138 ft. was used with design compressive strengths at release and 28-days of 6.0 ksi and 7.5 ksi respectively. Thirty-six (36) 0.6-inch diameter draped strands were required. Lower and average span lengths of 100 ft. and 120 ft. were also analyzed.

Estimated cambers were calculated at release and at erection (before added dead load deflection) and at erection (after added dead load deflection) for the baseline analysis and for the various items that were expected to influence camber. Table 12 summarizes the estimated cambers. A concrete compressive strength of 7.62 ksi was used for the average release strength (f'ci). A concrete compressive strength of 11.5 ksi was used for the average 28-day strength (f'c). A concrete compressive strength of 9.96 ksi was used for the high release strength (f'ci). A concrete compressive strength of 14.2 ksi was used for the high 28-day strength (f'c). An aggregate factor (K1) of 1.10 was used for the limestone aggregate.

Table 12 - Data Sets for Camber Calculations (BT-72 Leake County)

girder camber (inches)		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline analysis	4.968	-2.166	2.802	8.943	-4.007	4.936	-2.256	2.680
2	avg f'ci/f'c	4.639	-2.012	2.627	8.351	-3.723	4.628	-1.977	2.651
3	high f'ci/f'c	4.293	-1.853	2.440	7.727	-3.428	4.299	-1.856	2.443
4	unit wt 155	4.694	-2.106	2.588	8.448	-3.895	4.553	-2.124	2.429
5	unit wt 160	4.440	-2.048	2.392	7.992	-3.789	4.203	-2.005	2.198
6	gross section	5.073	-2.310	2.763	9.132	-4.274	4.858	-2.406	2.452
7	fillet included	4.968	-2.166	2.802	8.943	-4.007	4.936	-2.239	2.697
8	temperature	4.968	-2.166	2.802	8.943	-4.007	4.936	-1.745	3.191
9	PCI mult 1.0	4.968	-2.166	2.802	4.968	-2.166	2.802	-2.256	0.546
10	increase fillet thickness	4.968	-2.166	2.802	8.943	-4.007	4.936	-2.364	2.572
11	top strand	4.885	-2.155	2.730	8.793	-3.986	4.807	-2.244	2.563
12	straight strands with debonding	5.327	-2.156	3.171	9.589	-3.988	5.601	-2.256	3.345
13	2, 4	4.384	-1.958	2.426	7.890	-3.621	4.269	-1.860	2.409
14	2, 5	4.144	-1.904	2.240	7.460	-3.522	3.938	-1.756	2.182
15	2, 7	4.639	-2.012	2.627	8.351	-3.723	4.628	-1.962	2.666
16	2, 4, 6, 11	4.390	-2.066	2.324	7.902	-3.822	4.080	-1.967	2.113
17	2, 4, 6, 10, 11	4.390	-2.066	2.324	7.902	-3.822	4.080	-2.061	2.019
18	baseline analysis (including prestress losses)	4.505	-2.166	2.339	8.110	-4.007	4.103	-2.256	1.847
19	2, 4 (including prestress losses)	3.976	-1.958	2.018	7.157	-3.621	3.536	-1.860	1.676
20	baseline analysis, minimum span length 100 ft.	1.585	-0.651	0.934	2.853	-1.204	1.649	-0.677	0.972
21	baseline analysis, average span length 120 ft.	2.940	-1.335	1.605	5.292	-2.469	2.823	-1.391	1.432
22	limestone aggregate	4.573	-1.982	2.591	8.232	-3.666	4.566	-2.067	2.499
23	avg f'ci/f'c, limestone aggregate	4.270	-1.842	2.428	7.686	-3.408	4.278	-1.811	2.467

To gain insight into which items influenced camber estimates the most and/or combination of several of the items acting together, Table 13 includes the percentage (%) difference between the baseline analysis.

Table 13 - BT-72 (Leake County) Bridge Transverse Section used for Data Sets % Difference

	BT-72 (138 ft.)						
#		camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
1	baseline analysis	2.802	% difference	4.936	% difference	2.680	% difference
2	avg f'ci/f'c	2.627	-6.25%	4.628	-6.24%	2.651	-1.08%
3	high f'ci/f'c	2.440	-12.92%	4.299	-12.91%	2.443	-8.84%
4	unit wt 155	2.588	-7.64%	4.553	-7.76%	2.429	-9.37%
5	unit wt 160	2.392	-14.63%	4.203	-14.85%	2.198	-17.99%
6	gross section	2.763	-1.39%	4.858	-1.58%	2.452	-8.51%
7	fillet included	2.802	0.00%	4.936	0.00%	2.697	0.63%
8	temperature	2.802	0.00%	4.936	0.00%	3.191	19.07%
9	PCI mult 1.0	2.802	0.00%	2.802	-43.23%	0.546	-79.63%
10	increase fillet thickness	2.802	0.00%	4.936	0.00%	2.572	-4.03%
11	top strand	2.730	-2.57%	4.807	-2.61%	2.563	-4.37%
12	straight strands with debonding	3.171	13.17%	5.601	13.47%	3.345	24.81%
13	2, 4	2.426	-13.42%	4.269	-13.51%	2.409	-10.11%
14	2, 5	2.240	-20.06%	3.938	-20.22%	2.182	-18.58%
15	2, 7	2.627	-6.25%	4.628	-6.24%	2.666	-0.52%
16	2, 4, 6, 11	2.324	-17.06%	4.080	-17.34%	2.113	-21.16%
17	2, 4, 6, 10, 11	2.324	-17.06%	4.080	-17.34%	2.019	-24.66%
18	baseline analysis (including prestress losses)	2.339	-16.52%	4.103	-16.88%	1.847	-31.08%
19	2, 4 (including prestress losses)	2.018	-27.98%	3.536	-28.36%	1.676	-37.46%
22	limestone aggregate	2.591	-7.53%	4.566	-7.50%	2.499	-6.75%
23	avg f'ci/f'c, limestone aggregate	2.428	-13.35%	4.278	-13.33%	2.467	-7.95%

Table 14 summarizes the camber at release and camber at erection (before added dead load deflection) and sorts the camber values from largest to smallest.

Table 14 - Data Sets for Camber Calculations Sorted (BT-72 Leake County)

description	camber at release	description	camber at erection
straight strands with debonding	3.171	straight strands with debonding	5.601
baseline analysis	2.802	baseline analysis	4.936
fillet included	2.802	fillet included	4.936
temperature	2.802	temperature	4.936
PCI mult 1.0	2.802	increase fillet thickness	4.936
increase fillet thickness	2.802	gross section	4.858
gross section	2.763	top strand	4.807
top strand	2.730	avg f'ci/f'c	4.628
avg f'ci/f'c	2.627	avg f'ci/f'c, fillet included	4.628
avg f'ci/f'c, fillet included	2.627	limestone aggregate	4.566
limestone aggregate	2.591	unit wt 155	4.553
unit wt 155	2.588	high f'ci/f'c	4.299
high f'ci/f'c	2.440	avg f'ci/f'c, limestone aggregate	4.278
avg f'ci/f'c, limestone aggregate	2.428	avg f'ci/f'c, unit wt 155	4.269
avg f'ci/f'c, unit wt 155	2.426	unit wt 160	4.203
unit wt 160	2.392	baseline analysis (including prestress losses)	4.103
baseline analysis (including prestress losses)	2.339	avg f'ci/f'c, unit wt 155, gross section, top strand	4.080
avg f'ci/f'c, unit wt 155, gross section, top strand	2.324	avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness, top strand	4.080
avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness, top strand	2.324	avg f'ci/f'c, unit wt 160	3.938
avg f'ci/f'c, unit wt 160	2.240	avg f'ci/f'c, unit wt 155 (including prestress losses)	3.536
avg f'ci/f'c, unit wt 155 (including prestress losses)	2.018	PCI mult 1.0	2.802

FIB-72 [155 ft.]

An interior FIB-72 girder from the bridge transverse section shown in Figure 10 was analyzed part of the camber data set calculations.

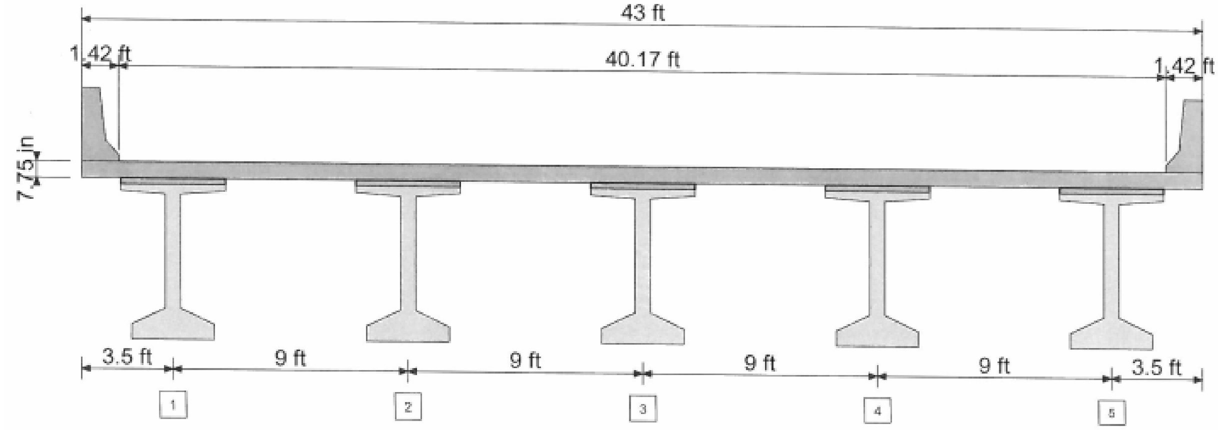

Figure 10 - FIB-72 Bridge Transverse Section used for Data Sets

A baseline span length of 155 ft. was used with design compressive strengths at release and 28-days of 6.6 ksi and 8.5 ksi respectively. Fifty-nine (59) 0.6-inch diameter straight strands with debonding and four (4) top strands with reduced pull were required. Lower, average, and upper span lengths of 120 ft., 140 ft., and 160 ft. were also analyzed.

Estimated cambers were calculated at release and at erection (before added dead load deflection) and at erection (after added dead load deflection) for the baseline analysis and for the various items that were expected to influence camber. Table 15 summarizes the estimated cambers. A concrete compressive strength of 8.4 ksi was used for the average release strength (f'ci). A concrete compressive strength of 13.0 ksi was used for the average 28-day strength (f'c). A concrete compressive strength of 11.0 ksi was used for the high release strength (f'ci). A concrete compressive strength of 16.1 ksi was used for the high 28-day strength (f'c). An aggregate factor (K1) of 1.10 was used for the limestone aggregate.

Table 15 - Data Sets for Camber Calculations (FIB-72)

girder camber (inches)		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline analysis	6.815	-3.507	3.308	12.266	-6.489	5.777	-3.124	2.653
2	avg f'ci/f'c	6.360	-3.256	3.104	11.447	-6.024	5.423	-2.734	2.689
3	high f'ci/f'c	5.880	-2.994	2.886	10.584	-5.539	5.045	-2.566	2.479
4	unit wt 155	6.434	-3.407	3.027	11.581	-6.303	5.278	-2.940	2.338
5	unit wt 160	6.083	-3.312	2.771	10.950	-6.127	4.823	-2.772	2.051
6	gross section	6.933	-3.696	3.237	12.480	-6.838	5.642	-3.295	2.347
7	fillet included	6.815	-3.507	3.308	12.266	-6.489	5.777	-3.107	2.670
8	temperature	6.815	-3.507	3.308	12.266	-6.489	5.777	-2.472	3.305
9	PCI mult 1.0	6.815	-3.507	3.308	6.815	-3.507	3.308	-3.124	0.184
10	increase fillet thickness	6.815	-3.507	3.308	12.266	-6.489	5.777	-3.270	2.507
11	2, 4	5.997	-3.160	2.837	10.795	-5.846	4.949	-2.574	2.375
12	2, 5	5.667	-3.071	2.596	10.201	-5.681	4.520	-2.426	2.094
13	2, 7	6.360	-3.256	3.104	11.447	-6.024	5.423	-2.719	2.704
14	2, 4, 6	6.074	-3.303	2.771	10.933	-6.111	4.822	-2.691	2.131
15	2, 4, 6, 10	6.074	-3.303	2.771	10.933	-6.111	4.822	-2.817	2.005
16	baseline analysis (including prestress losses)	6.206	-3.507	2.699	11.171	-6.489	4.682	-3.124	1.558
17	2, 4 (including prestress losses)	5.463	-3.160	2.303	9.834	-5.846	3.988	-2.574	1.414
18	baseline analysis, minimum span length 120 ft.	2.757	-1.331	1.426	4.963	-2.462	2.501	-1.185	1.316
19	baseline analysis, average span length 140 ft.	4.774	-2.444	2.330	8.592	-4.521	4.071	-2.176	1.895
20	baseline analysis, maximum span length 160 ft.	7.664	-4.141	3.523	13.796	-7.661	6.135	-3.688	2.447
21	limestone aggregate	6.268	-3.206	3.062	11.282	-5.931	5.351	-2.860	2.491
22	avg f'ci/f'c, limestone aggregate	5.355	-3.007	2.348	9.639	-5.564	4.075	-2.532	1.543

To gain insight into which items influenced camber estimates the most and/or combination of several of the items acting together, Table 16 includes the percentage (%) difference between the baseline analysis.

Table 16 - Data Sets for Camber Calculations (FIB-72) % Difference

	FIB-72 (155 ft.)						
#		camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
1	baseline analysis	3.308	% difference	5.777	% difference	2.653	% difference
2	avg f'ci/f'c	3.104	-6.17%	5.423	-6.13%	2.689	1.36%
3	high f'ci/f'c	2.886	-12.76%	5.045	-12.67%	2.479	-6.56%
4	unit wt 155	3.027	-8.49%	5.278	-8.64%	2.338	-11.87%
5	unit wt 160	2.771	-16.23%	4.823	-16.51%	2.051	-22.69%
6	gross section	3.237	-2.15%	5.642	-2.34%	2.347	-11.53%
7	fillet included	3.308	0.00%	5.777	0.00%	2.670	0.64%
8	temperature	3.308	0.00%	5.777	0.00%	3.305	24.58%
9	PCI mult 1.0	3.308	0.00%	3.308	-42.74%	0.184	-93.06%
10	increase fillet thickness	3.308	0.00%	5.777	0.00%	2.507	-5.50%
11	2, 4	2.837	-14.24%	4.949	-14.33%	2.375	-10.48%
12	2, 5	2.596	-21.52%	4.520	-21.76%	2.094	-21.07%
13	2, 7	3.104	-6.17%	5.423	-6.13%	2.704	1.92%
14	2, 4, 6	2.771	-16.23%	4.822	-16.53%	2.131	-19.68%
15	2, 4, 6, 10	2.771	-16.23%	4.822	-16.53%	2.005	-24.43%
16	baseline analysis (including prestress losses)	2.699	-18.41%	4.682	-18.95%	1.558	-41.27%
17	2, 4 (including prestress losses)	2.303	-30.38%	3.988	-30.97%	1.414	-46.70%
21	limestone aggregate	3.062	-7.44%	5.351	-7.37%	2.491	-6.11%
22	avg f'ci/f'c, limestone aggregate	2.348	-29.02%	4.075	-29.46%	1.543	-41.84%

Table 17 summarizes the camber at release and camber at erection (before added dead load deflection) and sorts the camber values from largest to smallest.

Table 17 - Data Sets for Camber Calculations Sorted (FIB-72)

description	camber at release	description	camber at erection
baseline analysis	3.308	baseline analysis	5.777
fillet included	3.308	fillet included	5.777
temperature	3.308	temperature	5.777
PCI mult 1.0	3.308	increase fillet thickness	5.777
increase fillet thickness	3.308	gross section	5.642
gross section	3.237	avg f'ci/f'c	5.423
avg f'ci/f'c	3.104	avg f'ci/f'c, fillet included	5.423
avg f'ci/f'c, fillet included	3.104	limestone aggregate	5.351
limestone aggregate	3.062	unit wt 155	5.278
unit wt 155	3.027	high f'ci/f'c	5.045
high f'ci/f'c	2.886	avg f'ci/f'c, unit wt 155	4.949
avg f'ci/f'c, unit wt 155	2.837	unit wt 160	4.823
unit wt 160	2.771	avg f'ci/f'c, unit wt 155, gross section	4.822
avg f'ci/f'c, unit wt 155, gross section	2.771	avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness	4.822
avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness	2.771	baseline analysis (including prestress losses)	4.682
baseline analysis (including prestress losses)	2.699	avg f'ci/f'c, unit wt 160	4.520
avg f'ci/f'c, unit wt 160	2.596	avg f'ci/f'c, limestone aggregate	4.075
avg f'ci/f'c, limestone aggregate	2.348	avg f'ci/f'c, unit wt 155 (including prestress losses)	3.988
avg f'ci/f'c, unit wt 155 (including prestress losses)	2.303	PCI mult 1.0	3.308

A time dependent model was analyzed for the FIB-72 girder to further evaluate the influence of; humidity, girder fabrication time (e.g., 1, 2, and 3 days), average historical concrete compressive release and 28-day strengths, and extended beam storage time (e.g., 6 and 12 months). Table 18 summarizes the estimated cambers for the baseline analysis. The remainder of the estimated camber data sets are included in the November 13, 2018 TAC presentation in Appendix A-2.

Table 18 - Data Sets for Camber Calculations (FIB-72) Time-Dependent Analysis Baseline

girder camber (inches)				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	1	1	-4.197	7.636	0.000	3.439
2	store beam, transport beam, and erect beam at project site	50	51	-6.854	12.125	-0.183	5.088
3	form deck and place rebar	30	81	-7.304	12.853	-0.231	5.318
4	pour and cure deck	14	95	-8.816	13.038	-0.464	3.758
5	pour and cure barriers	14	109	-9.093	13.186	-0.803	3.290
6	time step	365	474	-10.067	14.147	-1.698	2.382

One item that became apparent in reviewing the camber estimate data for all the items that influence camber for all of the girder types was the ratio of the erection camber (before added dead load deflection) to release camber. Table 19 highlights this information for each of the studied girders. One anomaly in comparing the historic material camber data provided by the only Mississippi Concrete Girder Manufacturer who provided 28-day camber information for AASHTO Type 4 girders was an average measured 28-day to average measured camber at release = 3.20.

Table 19 - Average Ratio of Erection to Release Camber

Girder Type	Average ratio of erection to release camber
AASHTO Type 4 (90 ft.)	1.77
BT-54 (110 ft.)	1.77
BT-72 (138 ft.)	1.76
FIB-72 (155 ft.)	1.74
FIB-72 (155 ft.) time dependent model	1.65 1.73 (after forming deck)

Beam Camber Tolerances

Tolerances for beam camber are included in the PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (Fourth Edition) MNL-116-99, Appendix B-10 I-Beam (Girder) or Bulb-Tee Girder. PCI's Committee on Bridges-Camber FAST Team reviewed the current beam camber tolerances and provided recommendations at the 2012 PCI Committee Days for PCI tolerances with respect to predicted camber at time of prestress transfer.

The FAST Team Mission Statement was to evaluate the current PCI tolerances for camber of bridge girders published in PCI MNL-116 and make recommendations for PCI tolerances with respect to predicted camber at time of prestress transfer. The Team recognized that a database of all variables that affect camber would be ideal to assist with recommendations. However, records for all variables associated with girder camber were not readily available from all plants.

PCI's current tolerances for variation from design camber is +/- 1/8 in. per 10 ft. with a maximum of +/- 1/2 in. up to 80 ft. and a maximum of +/- 1 in. for length greater than 80 ft.

As stated in the PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (Fourth Edition) MNL-116-99

> "Camber is a function of girder cross-section, prestressing force, strand location, concrete properties, girder age, and environmental factors. Each of these attributes have variability both within a plant and among plants. This variability is independent of the camber prediction method used. Prediction of camber is based on empirical formulas. Accuracy of these estimated values decreases with time. Measurement of camber from comparison of predicted design values should be completed within 72 hrs of transfer of prestress. Temperature variation across a member section can have a significant impact on the measured camber. Camber should be evaluated under conditions that minimize the effect of temperature variation due to solar radiation, such as early in the morning.

The FAST Team made the following recommendations:

1. Revise 'g" dimension in Appendix B of PCI MNL-116

   g = Camber Variation from Design Camber Within 72 Hours of Release

   - + 1/8 in. per ten feet, up to a maximum of 1.50 in.
   - - 1/8 inch per ten feet with no lower bound

2. Add a footnote in Appendix B of PCI MNL-116

   Out of tolerance camber should not be a sole cause for rejection.

Effect of Increased Stiffness on Live Load Distribution Factor

The effect of increased stiffness of the girders and live load distribution factor was evaluated when using the average actual 28-day concrete compressive strengths based on the historical material data provided by the Mississippi Concrete Girder Manufacturers. Table 20 presents results for the BT-54 (Marshall County) girder. The 28-day concrete compressive strength used for the baseline analysis was 7.7 ksi. The 28-day concrete compressive strength used for the average f'c analysis was 11.8 ksi.

Table 20 - Effect of Increased Stiffness on Live Load Distribution Factor (BT-54 Marshall County)

Live load distribution factor (shear and moment)	Baseline analysis	Avg f'c	Avg f'c and with haunch/fillet included in composite section
LLV 1-lane	0.674	0.674	0.674
LLV 2-lanes	0.813	0.813	0.813
LLM 1-lane	0.441	0.446 (1% increase)	0.452 (2% increase)
LLM 2-lanes	0.633	0.641 (1% increase)	0.648 (2% increase)

Table 21 presents results for the BT-72 (Leake County) girder. The 28-day concrete compressive strength used for the baseline analysis was 7.5 ksi. The 28-day concrete compressive strength used for the average f'c analysis was 11.5 ksi.

Table 21 - Effect of Increased Stiffness on Live Load Distribution Factor (BT-72 Leake County)

Live load distribution factor (shear and moment)	Baseline analysis	Avg f'c	Avg f'c and with haunch/fillet included in composite section
LLV 1-lane	0.654	0.654	0.654
LLV 2-lanes	0.777	0.777	0.777
LLM 1-lane	0.415	0.420 (1% increase)	0.424 (2% increase)
LLM 2-lanes	0.604	0.611 (1% increase)	0.616 (2% increase)

For both the BT-54 (Marshall County) and the BT-72 (Leake County) girders, the effect on the live load distribution factor for moment by using an increased f'c based on the historical material data provided by the Mississippi Concrete Girder Manufacturers is minimal with only a small (1%) increase in the live load distribution factor for moment and 2% increase when including the haunch/fillet in the composite section. The live load distribution factor for shear did not change.

Conclusions

1. The literature review revealed items that influence camber, increased MDOT's knowledge-base related to the research topic, and provided insight to how other research agencies and/or State DOT's have addressed beam camber or other related topics. The documents researched emphasized how variable the items are that influence camber and thus camber calculations should be considered an estimate due to the variability in the items that influence beam camber.
2. The historic material data provided by the three Mississippi Concrete Girder Manufacturers provided actual data specific to concrete compressive strengths and beam camber. From the historic material data, the average actual concrete compressive strength at release (f'ci) and 28-days (f'c) were computed and used in the camber data sets.
    a. Referring to Table 5, the ratio of the average actual concrete compressive strength at release to the design concrete compressive strength was 1.004, 1.14, and 1.67 for the three Mississippi Concrete Girder Manufacturers with an average of the three to be 1.27.
    b. Referring to Table 5, the ratio of the average actual concrete compressive strength at 28-days to the design concrete compressive strength was 1.23, 1.46, and 1.89 for the three Mississippi Concrete Girder Manufacturers with an average of the three to be 1.53.
    c. Referring to Table 6, several of the Mississippi Concrete Girder Manufacturers provided beam camber data, which was used to compare against the estimated camber. Table 6 contains a summary of the estimated camber at release, average measured camber at release, and average measured camber at 28-days.
3. The various items that were included in the camber data sets did change the values of the camber estimates both individually and more so in combination with each other.
4. The under-camber girders that MDOT has experienced on several projects was validated by the majority of the items included in the camber data sets; which indicated the estimated camber using the research items that influence camber to be less than the current method used to estimate camber (i.e., baseline estimated camber).
5. The Camber data sets provided insight to the range in variability in the camber estimates.
    a. The range in variability increases when comparing camber at erection compared to camber at release.
    b. The range in variability increases as the girder length and/or number of prestressing strands increases.

6. The Camber data sets provided insight to the variation and magnitude of camber estimates for various concrete girder types and girder length ranges selected in comparison to the current (i.e., baseline) estimates for camber.
    i. Actual average values for the concrete compressive strengths at release and 28-days provided by the Mississippi Concrete Girder Manufacturers were included in the camber data sets and assisted with developing recommendations for estimating camber.
7. Other State DOT current practices provided a comparison with Florida, Nebraska, Texas, Washington, Alabama, and Louisiana current practices related to beam camber. For the most part, the selected other State DOT's have similar practices as MDOT's with a few minor differences as noted in Appendix C.
8. The effect on the live load moment distribution factor when using the average historic concrete compressive strength was minimal with a 1-2% increase. The increase in the concrete compressive strength when using the average historic concrete compressive strength did not change the live load shear distribution factor.
9. Based on the research findings and recommendations, the following benefits are realized:
    a. improved material property versus strength expectation
    b. minimize the difference between the estimated and actual beam cambers which will; reduce construction delays and reduce added construction costs to MDOT projects, reduce design and/or functional modifications to MDOT projects, and improve the ride

Recommendations

Design Table for f'ci/f'c

Based on the historic material data and ratio of f'ci/f'c, a design table is recommended for various values of f'c and f'ci accounting for upper and lower bounds. Table 22 uses a ratio of the average actual release concrete strength to average 28-day concrete strength provided by the Mississippi Concrete Girder Manufacturers of 0.59, 0.71, and 0.74 with the average of the three ratios being 0.68. A 10% and 15% lower and upper bound was used and the concrete compressive strength values were rounded to the nearest 100 psi. The recommended design table for f'ci and f'c concrete compressive strengths result in a range of the f'ci/f'c ratio between 0.59 to 0.80.

Table 22 - Recommended Design Table for f'ci/f'c

concrete release strength f'ci (psi)	concrete 28-day strength f'c (psi)	15% lower bound	10% lower bound	10% upper bound	15% upper bound
4000	5900	5000	5300	6500	6800
4500	6600	5600	6000	7300	7600
5000	7400	6300	6600	8100	8500
5500	8100	6900	7300	8900	9300
6000	8800	7500	7900	9700	10100
6500	9600	8100	8600	10500	11000
7000	10300	8800	9300	11300	11800
7500	11000	9400	9900	12100	12700
8000	11800	10000	10600	12900	13500
8500	12500	10600	11300	13800	14400
9000	13200	11300	11900	14600	15200
9500	14000	11900	12600	15400	16100
10000	14700	12500	13200	16200	16900

To use the design table for f'ci/f'c:

1. For a given required 28-day design concrete strength (e.g., 7500 psi) find the row or rows that include 7500 psi in the lower and upper bounds column(s) and select the respective concrete design strength at release (e.g., 4500, 5000, 5500, and 6000 psi). This would indicate that the available manufactured release strength could range between 4500 to 6000 psi.
2. For a given required release design concrete strength (e.g., 5500 psi) move across the same row to find the lower and upper bound available manufactured 28-day strength; which could range between 6900 to 9300 psi.

Minimum Haunch/Fillet Thickness

A minimum thickness for the haunch/fillet is recommended at the edge of the top flange of the prestressed concrete girder. The minimum thickness should be able to accommodate deck forming material and construction tolerances. The minimum thickness at the edge of the top flange will also influence the thickness of the haunch/fillet at the centerline of the girder; which is used for design and bridge geometry/beam seat elevation computations.

Estimated Camber at Release

Recommend adding the estimated camber at release to the Prestress Requirements Table that is included with the bridge girder detail construction plan drawings. The estimated camber at release values will provide MDOT with comparison to the actual/measured beam camber at release that the Mississippi Concrete Girder Manufacturers are currently required to provide. The estimated camber at release values will add to MDOT’s historical data related to beam camber and also provide an early notice or indicator of any differences between the estimated and actual/measured beam camber at release. An early notice or indicator of differences between the estimated and actual/measured beam camber will allow advanced time for possible bridge geometry/beam seat elevation adjustments through coordination with the Contractor and Concrete Beam Manufacture in preparation for the beam erection and deck forming. This advanced notice and early coordination between the Concrete Beam Manufacturer and the Contractor will minimize project delays and reduce added construction costs.

Temperature Gradient

Although the temperature gradient does influence beam camber; it is NOT recommended to include the temperature effects when estimating beam camber. Refer to AASHTO LRFD C3.12.3 for commentary.

Prestress Loss Data

It is recommended to include design guidelines that address the various parameters that are used in the computation of prestress losses which influence beam camber when using the AASHTO LRFD 5.9.4.4-Refined Estimates of Time-Dependent Losses. Refer to PCI’s Bridge Design Manual for suggested time at release, age of deck placement, final age, and relative humidity values. These parameters do not apply and are not used when calculating prestress losses when using the PCI Multiplier Method together with the AASHTO LRFD 5.9.5.3-Approximate Estimate of Time-Dependent Losses.

When estimating beam camber at release and at erection when using the PCI Multiplier Method together with the AASHTO LRFD 5.9.5.3-Approximate Estimate of Time-Dependent Losses, only the prestress losses at release are used to compute deflections and thus beam cambers so therefore the above prestress loss data are not utilized.

Florida Bulb-T Beam Section Properties

Recommend adding the Florida Bulb-T beam section properties and available strand templates to MDOT's Bridge Design Manual and/or Design Standards.

Transformed Section Properties

MDOT currently uses transformed section properties along with several other State DOTs. The AASHTO LRFD bridge design specifications allow the use of transformed section properties so the use of transformed section properties for the design of prestressed concrete girders is recommended.

With the premise that the prestress losses and jacking stress are equal for either a transformed or gross beam section; and as transformed section properties pertain to beam camber, a transformed section will have a larger moment of inertia compared to a gross section. Therefore, the computation of deflections for a transformed section should also be less than a gross section since the moment of inertia is considered part of the stiffness effect which when combined with the modulus of elasticity are used in the denominator of the deflection calculations. Since the deflection is inversely proportional to the stiffness, a greater stiffness will result in a smaller deflection. Under the premise that the prestress losses and jacking stress are equal for either a transformed or gross beam section, the resulting deflections should be similar and will become increasingly different as the ratio between the transformed to gross moment of inertia increases.

Roadway Vertical Curve Ordinate

The haunch/fillet thickness at the ends of the prestressed concrete girders and thus the beam seat elevations should account for the roadway vertical curve ordinate. Therefore, it is recommended to include the roadway vertical curve ordinate in the beam camber computations used to establish bridge geometry and beam seat elevations. The roadway vertical curve ordinate, beam camber, minimum haunch thickness, and deck cross-slope/superelevation are used to calculate the haunch/fillet thickness at the end of the beam used to set beam seat elevations.

Debonding Increments

Recommend updating MDOT's Bridge Design Manual to include MDOT's current practice of using two (2) ft. debond increments when using straight strand.

Draped vs. Straight Strands

According to MDOT Bridge Division Design Manual (Version 6.1), the current policy is to drape strands instead of debonding. However, debonding may be used with permission from the Bridge Engineer on straight strands in certain situations to reduce stresses in the beam. For designs that use either draped and/or straight strands that require top strands to satisfy stresses, the camber data sets indicated the estimated camber is only slightly reduced when top strands are added. Therefore, recommend including top strand when estimating camber when top strand are required to satisfy stresses on a design-by-design basis.

Camber Measurements

Recommend requiring the Mississippi Concrete Girder Manufacturers to measure the beam camber and record the age of the girders prior to shipping the prestressed concrete girder to the project site. This will provide MDOT additional historical beam camber data to compare with the estimated camber measurements at release and at erection.

Since the 28-day concrete compressive strength is currently recorded by the Mississippi Concrete Girder Manufacturers; it is recommended to require the Mississippi Concrete Girder Manufacturers to also measure the beam camber at 28-days to begin collecting historical data related to beam camber.

On select MDOT projects, require the Contractor to measure beam camber at erection after added dead load deflection to begin collecting historical data related to beam camber.

Recommend when measuring beam camber to minimize the effects of temperature variations and/or temperature gradients by measuring camber in the early morning and then again in the early morning for subsequent camber measurements. Also record the time of day and temperature.

Girder Shipping Weight

Recommend requiring the Mississippi Concrete Girder Manufacturers to record the weight of the prestressed concrete girder prior to and/or during shipping. This requirement will provide historical data related to the unit weight of the prestressed concrete girders which influences beam camber estimates.

Additional Concrete Cylinder Breaks

In addition to the current release and 28-day concrete cylinder break requirements; and on select MDOT projects, recommend obtaining additional concrete cylinder breaks at a later timeframe (e.g., 90, 120, 200, 365 days) to begin to collect historical data related to strength gain of the prestressed girder concrete. Acquiring knowledge related to strength gain on the prestressed girder concrete can assist with future load ratings and decisions related to service life of bridge structures.

Aggregate Types

Recommend requiring the Mississippi Concrete Girder Manufacturers to record the type of aggregate and include with the concrete test pour documentation records. The type of aggregate is a consideration in the AASHTO LRFD equation for calculating the Modulus of Elasticity, which influences beam camber. Having historical data for the type of aggregate can aid with computations used for estimating beam camber.

Actual/Measured Modulus of Elasticity

On select MDOT projects, recommend obtaining the actual/measured modulus of elasticity at release and at 28-days to begin collecting historical data related to the modulus of elasticity. The modulus of elasticity influences the beam camber estimate and the actual/measured modulus of elasticity can be compared with values obtained from the current AASHTO LRFD Equation 5.4.2.4-1.

Increased Concrete Strengths

Consider allowing increased concrete strengths on select MDOT projects to optimize superstructure designs and/or eliminate a girder line, which can reduce construction costs. MDOT to coordinate new FX mix design approvals with Mississippi Concrete Girder Manufacturers to take advantage of current production capabilities and increased concrete strengths.

According to MDOT Bridge Division Design Manual (Version 6.1) current design parameters, the 28-day compressive strength for beam concrete shall be 5,000 psi. Strengths of 5,500 psi and 6,000 psi can be used as required by design. Recommend updating MDOT's Bridge Design Manual to include MDOT's current practice of designing Florida I-Beams (FIBs) using 8,500 psi for the 28-day compressive strength.

As shown in Table 5 Historic Material Data – Concrete Strength Summary; for design 28-day concrete strengths of 6,500 psi, 5,625 psi, and 8,500 psi, the average actual 28-day concrete strengths manufactured by the Mississippi Concrete Girder Manufacturers were 9,501 psi, 10,644 psi, and 10,441 psi. The average actual 28-day concrete strengths were 10,195 psi; therefore, an upper-bound of 10,000 psi for the 28-day design concrete compressive strength is recommended on select MDOT projects.

With a new FX mix design for the recommended 10,000 psi 28-day design concrete compressive strength, the PCI multipliers will need to be revisited and revised accordingly based on project data/records.

Estimating Camber

Both approaches are recommended to supplement MDOT's current practices for estimating camber of Bulb-T and Florida girders.

1. Continue using the PCI Multiplier Method; and based on the historic material data provided by the Mississippi Concrete Girder Manufacturers, recommend using adjusted concrete strengths and adjusted modulus of elasticity values based on the average actual strengths, and a unit weight of 155 pcf when calculating camber estimates.

   The research data sets used an increase value of 1.27 times the design release concrete compressive strength and an increase value of 1.53 times the design 28-day concrete compressive strength for the various analyses designated as average f'ci and f'c. For implementation, suggest rounding the adjusted values to 1.25 for the release and 1.50 for the 28-day concrete compressive strengths. MDOT to continue to collect historic material and beam camber data from the Mississippi Concrete Girder Manufacturers to compare with the historic average values for f'ci and f'c and update the f'ci and f'c average values accordingly.

2. Modify the PCI Multipliers to match the camber estimates using the average actual strengths provided by the Mississippi Concrete Girder Manufacturers and a unit weight of 155 pcf. An adjusted multiplier of 1.65 applied to both the deflection and camber components at release provided comparable erection camber estimates (after added dead load deflections) for the BT-54 (Marshall County) and BT-72 (Leake County) girders.

Recommend using both approaches initially to compare with actual/measured field data on future MDOT projects before deciding which approach correlates best with the actual/measured cambers.

Implementation Plan

The following items are for MDOT's consideration part of an Implementation Plan based on the research findings.

1. Disseminate the research findings within MDOT including the Bridge Design Division, Construction Division, Materials Division, and District staff. Update MDOT research library to include final report.
2. Share research findings with contractors, concrete girder manufacturers, consultants, and industry and discuss whether MDOT will change their current methods for estimating camber.
3. Update MDOT's Bridge Design Manual with reference to the research for best practices for estimating camber of Bulb T and Florida girders.
4. Coordinate with Mississippi Concrete Girder Manufacturers and Contractors any new information required to be provided and/or collected in reference to the research.
5. Continue to collect historical material and beam camber data and update MDOT's knowledge-base/database accordingly.
6. Develop Technical Brief that summarizes the research.
7. Pursue publications.

References

AASHTO LRFD Bridge Design Specifications

ALDOT Structural Design Manual-Prestressed Concrete Girder Design Policy, and Bridge Plan Detailing Manual

FDOT Structures Design Guidelines, Structures Detailing Manual, and Index 20010 Series Prestressed Florida I-beams (Rev. 01/16)

LADOTD Bridge Design and Evaluation Manual

NDOR Bridge Division-Bridge Office Policies and Procedures, Section 3.3.1-General Prestressed Girder Policy

PCI Bridge Design Manual

PCI's Committee on Bridges-Camber FAST Team Tolerance Recommendations

PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (Fourth Edition) MNL-116-99 (Appendix B-10 I-Beam (Girder) or Bulb-Tee Girder

TXDOT Bridge Design Manual-LRFD, Section 4-Pretensioned Concrete I Girders

WSDOT Bridge Design Manual, Standard Specifications, and Design Memorandums

Appendices

A. TAC Presentations
   - A-1 August 3, 2018
   - A-2 November 11, 2018
   - A-3 February 11, 2019

B. Literature Review
   - B-1 Literature Review Document
   - B-2 Literature Review Items Related to Research Topic Table

C. Other State DOT Guidelines and Practices
   - C-1 Strand profile (draped, straight, debonding, top strand)
   - C-2 Fillet/haunch thickness
   - C-3 Roadway vertical curve ordinate
   - C-4 Camber Estimating Method (PCI Multiplier, time-dependent)
   - C-5 Dead Load Distribution
   - C-6 Girder section properties & strand templates
   - C-7 Material Properties (f'ci, f'c, E, unit weight, aggregate type)
   - C-8 Prestress Loss Data (time, humidity, curing method)
   - C-9 Temperature Gradient
   - C-10 Prestressed Beam Detail Plan Sheet Information
   - C-11 Camber

D. Camber Data Sets
   - D-1 Camber Data Sets Outline
   - D-2 Sample Plans for MDOT Project in Marshall County
   - D-3 Sample Plans for MDOT Project in Leake County

APPENDIX A

TAC Presentations

A-1

August 3, 2018

MDOT
Best Practices for Estimating Camber of Bulb T and Florida Girders
State Study No. 288

Technical Advisory Committee (TAC) meeting

August 3, 2018

Jackson, MS

David Tomley, P.E. (Assistant Project Manager & Senior Structural Engineer)

Thompson Engineering

Mobile, AL

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Technical Advisory Committee (TAC) meeting agenda

- Provide an update on the status of the research project
  - Project progress schedule
- Work completed
  - Literature Search
  - Review MDOT Documentation and MS Concrete Girder Manufacturer documentation (design vs. actual material information)/Historic Material Data
- Work In-Progress
  - Review MDOT Current Practices
  - Survey other State DOT Current Practices
  - Review AASHTO Bridge Design Specs
  - Develop research data sets for camber calculations

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update on the status of the research project

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Project Progress Schedule

Task	Sub-Task	Work	1	2	3	4	5	6	7	8	9	10	11	12	13	14	Estimated % Completion
	Research		Months from Notice to Proceed (NTP) Date														
			Feb-2018	Mar-2018	Apr-2018	May-2018	Jun-2018	Jul-2018	Aug-2018	Sep-2018	Oct-2018	Nov-2018	Dec-2018	Jan-2019	Feb-2019	Mar-2019	
C1	Kick-Off meeting	2%	100														100
M1	MDOT attend																
C2	a) TAC meetings	3%															0
M2	MDOT attend																
	b) QPRs	2%															0
	c) APR	1%															0
	d) Supporting Documents for Invoices	7%	7.14	7.14	7.14	7.14	7.14										36
C3	Literature Search	8%		87	13												100
C4	Review MDOT Current Practices	10%		27	5	32	7										71
M3	MDOT input																
C5	Survey other State DOT Current Practices	5%					83										83
C6	Review MDOT Documentation (design versus actual concrete strengths)	10%				18	82										100
M4	MDOT input																
C7	Review Mississippi concrete girder manufacturer Documentation (design versus actual concrete strengths)	8%			50	50											100
C8	Review current AASHTO Bridge Design Specifications and other Reference Publication Guidelines	8%															0
C9	Develop Research Data Sets for Camber Calculations	12%															0
C10	Interim Report	12%															0
M5	MDOT Review																
C11	Final Report	12%															0
M6	MDOT Review																
	Total Work	100%															
	Monthly Overall Progress, %		3	10	6	10	14	0	0	0	0	0	0	0	0	0	
	Planned Overall Progress, %		3	26	45	54	65	68	73	75	80	83	89	91	97	100	

Work Completed

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Literature Search

26 documents were reviewed to gain insight into the various aspects associated with the research topic and draw from the previous knowledge-base of information related to estimating camber

- Refer to copies of the literature review documents and summary of items related to the research

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Literature Search

1. Study of Prestress Losses Conducted by Lehigh University
2. High Strength Prestressed Concrete Bridge Girder Performance
3. Prestress Losses In Pretensioned High-Strength Concrete Bridge Girders
4. Precast, prestressed girder camber variability
5. Commercial Software
6. Commercial Software
7. Evaluating the early-age behavior of full-scale prestressed concrete beams using distributed and discrete fibre optic sensors
8. Use of High Performance, High Strength Concrete (HPC) Bulb-Tee Girders Saves Millions on I-10 Twin Span Bridge in New Orleans District
9. The effect of temperature variations on the camber of precast, prestressed concrete girders
10. Improving the Accuracy of Camber Predictions for Precast Pretensioned Concrete Beams
11. Prestress losses and camber growth in wing-shaped structural members
12. Camber and Prestress Losses in Alabama HPC Bridge Girders
13. Prestress loss calculations: Another perspective
14. A Probabilistic Comparison of Prestress Loss Methods in Prestressed Concrete Beams

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Literature Search

15. Effects of production practices on camber of prestressed concrete bridge girders
16. New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders
17. camber tolerances
18. Predicting the Bond Behavior of Prestressed Concrete Beams Containing Debonded Strands
19. A Review of Strand Development Length for Pretensioned Concrete Members
20. Design of Anchorage-Zone Reinforcement in Prestressed Concrete Beams
21. Control of Horizontal Cracking in the Ends of Pretensioned Prestressed Concrete Girders
22. A Rational Method for Estimating Camber and Deflection of Precast Prestressed Members
23. Recommendations for Estimating Prestress Losses
24. Improving Predictions for Camber in Precast, Prestressed Concrete Bridge Girders
25. Predicting Camber, Deflection, and Prestress Losses in Prestressed Concrete Members
26. Estimating Camber, Deflection, and Prestress Losses in Precast, Prestressed Bridge Girders

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Literature Search
Topics related to the Research

A	strength gain > 28 days
B	material properties
C	camber prediction methods
D	camber variability
E	section properties
F	instrumentation & monitoring
G	high strength concrete using local materials (LADOTD)
H	temerature effects on camber
I	design procedures
J	measured camber
K	prestress losses
L	experimental program
M	AASHTO specifications
N	Sensitivity Study (TXDOT)
O	probabalistic comparison/effect of variability on prestress losses and camber & deflections
P	test data
Q	transportation weight limits
R	factors that influence span capabilities (prestress losses, allowable tension, local producer member capabilities f'c)
S	camber tolerances
T	debonded strands
U	anchor zone reinforcing
V	QC records (WSDOT)
W	humidity
X	historical material data
Y	support conditions
Z	modification factors for camber estimates
AA	camber experiences by other State DOT's
BB	when to measure initial camber
CC	scheduling pours
DD	recommendations for practice
EE	curing
FF	strand development and transfer lengths

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Literature Search

topics related to research study

literature doc	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	BB	CC	DD	EE	FF
1																															X	
2	X																															
3		X																														
4		X	X	X	X																											
5			X																													
6			X																													
7						X																										
8							X																									
9								X																								
10		X	X					X	X	X																						
11								X			X	X																				
12		X											X																			
13											X			X									X									
14															X																	
15			X	X												X								X		X						
16																	X	X														
17																			X													
18																				X												
19																																X
20																					X											
21																					X											
22			X																													
23											X																					
24								X														X	X	X	X	X						
25			X																					X		X	X	X	X	X		
26		X	X			X																				X				X		

Literature documents in blue contain 3 or more topics related to the research

Topics in yellow show up in 3 or more of the literature documents reviewed

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Review MDOT Documentation and MS Concrete Girder Manufacturer documentation (design vs. actual material information)

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Historic Material Data

The following information was provided by the three (3) MS Concrete Girder Manufacturers

- ❑ concrete break reports
- ❑ camber data

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Historic Material Data

- Producer 1
  - provided three concrete test reports all from the same project with a required design release concrete strength of 5,600 psi and required design 28-day concrete strength of 6,500 psi
  - No camber data was provided
- Producer 2
  - provided concrete pour reports including cambers at release from five separate projects, projects included AASHTO Type 4 girders with lengths of 100 and 110 ft
  - required design concrete strength at release varied from 4200 to 5000 psi
  - required design concrete strength at 28-days varied from 5000 to 6000 psi
- Producer 3
  - provided fourteen concrete test reports all from the same project with a required design release concrete strength of 6,800 psi and required design 28-day concrete strength of 8,500 psi
  - camber data at release was provided

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Historic Material Data
Producer 1

report 1		report 2		report 3	
	break strength		break strength		break strength
age	(psi)	age	(psi)	age	(psi)
1	4706	1	5695	1	4270
1	5305	1	5745	1	4520
1	5435	1	5895	1	4335
1	5845	1	no break	1	6670
1	6050	1	no break	1	6955
1	5995	1	no break	1	6915
2	7675	2	no break	2	no break
2	7190	2	no break	2	no break
2	7040	2	no break	2	no break
3	no break	3	7960	3	no break
3	no break	3	7535	3	no break
3	no break	3	7595	3	no break
28	9755	28	9035	28	10265
28	9455	28	9180	28	9570
28	9370	28	9075	28	9800

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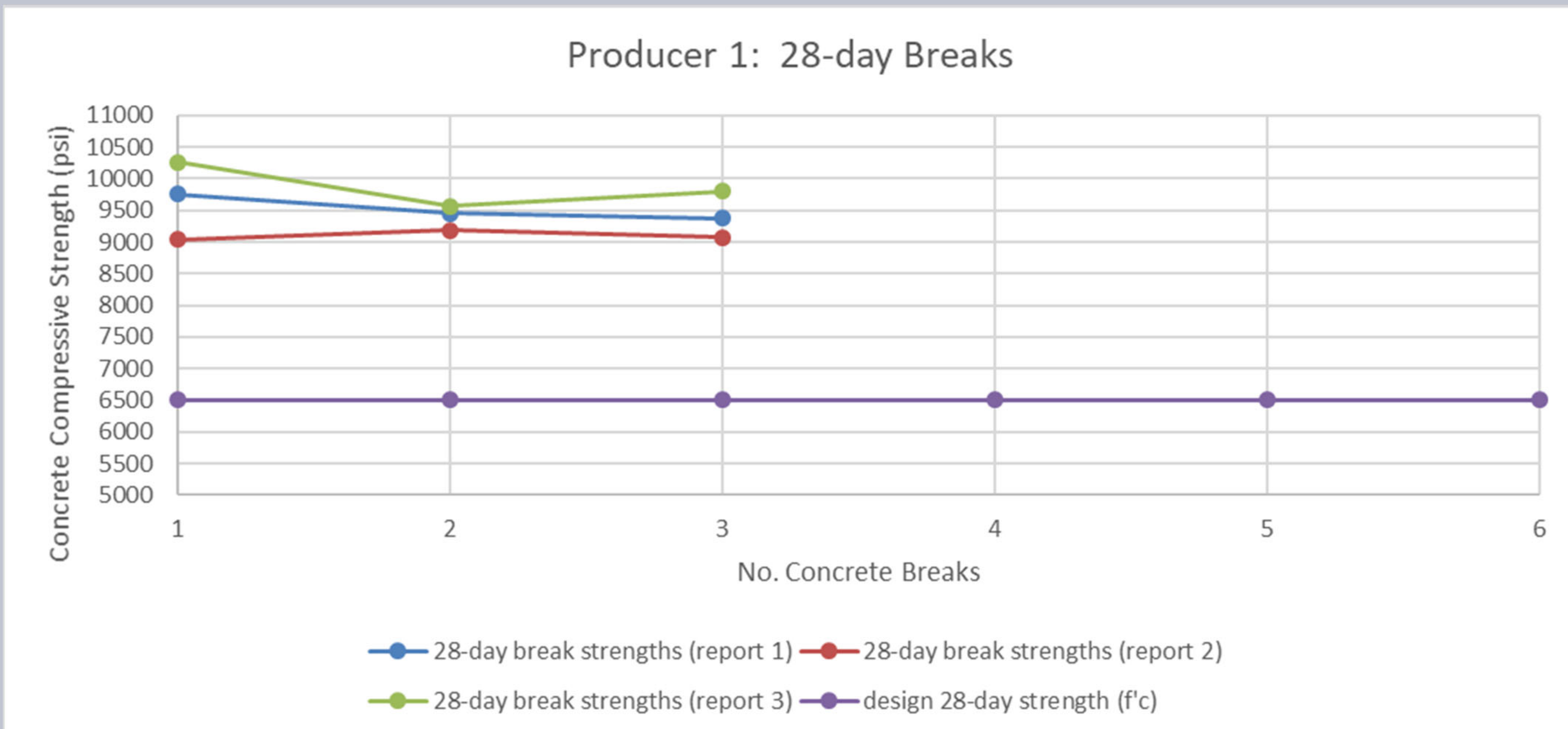
Historic Material Data
Producer 1



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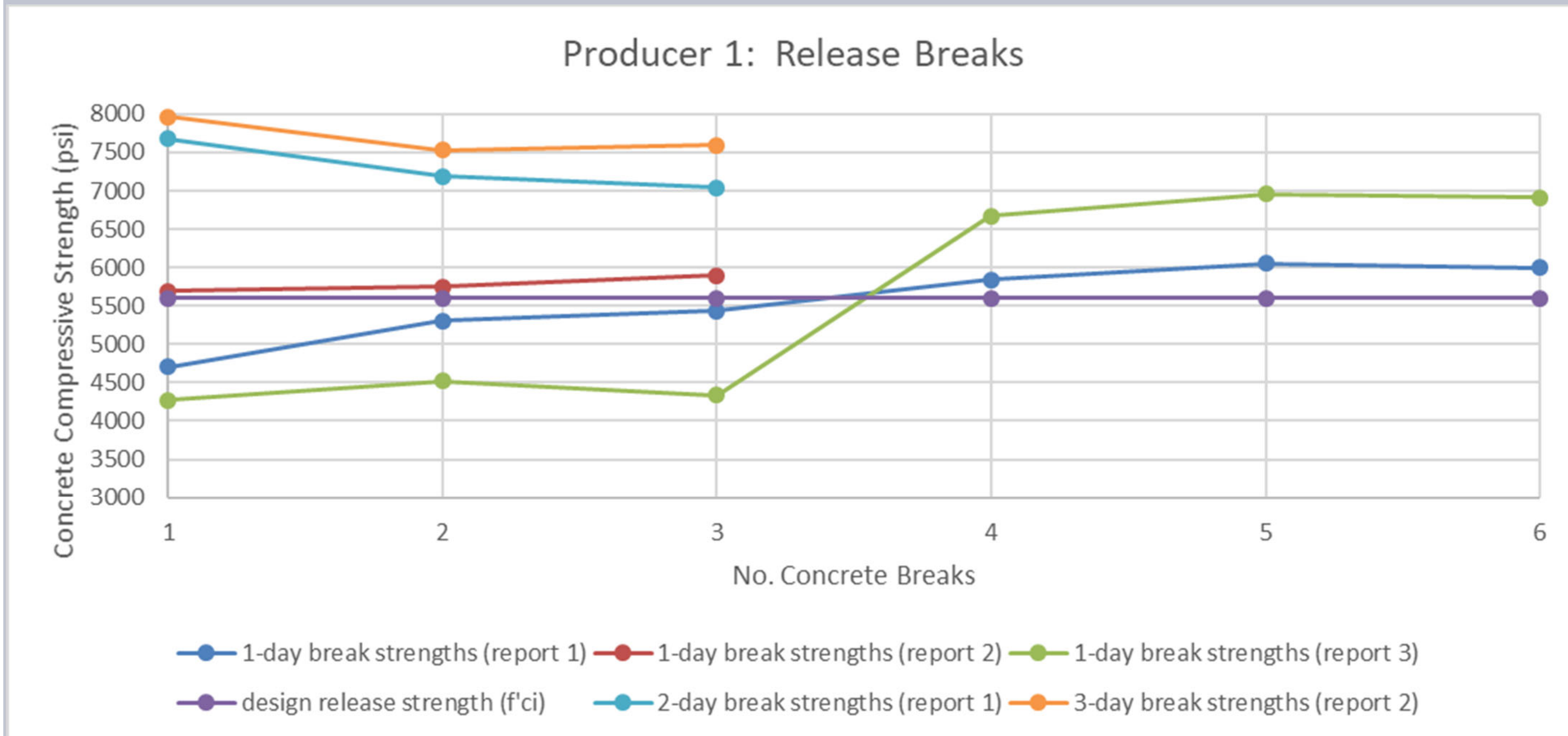
Historic Material Data
Producer 1



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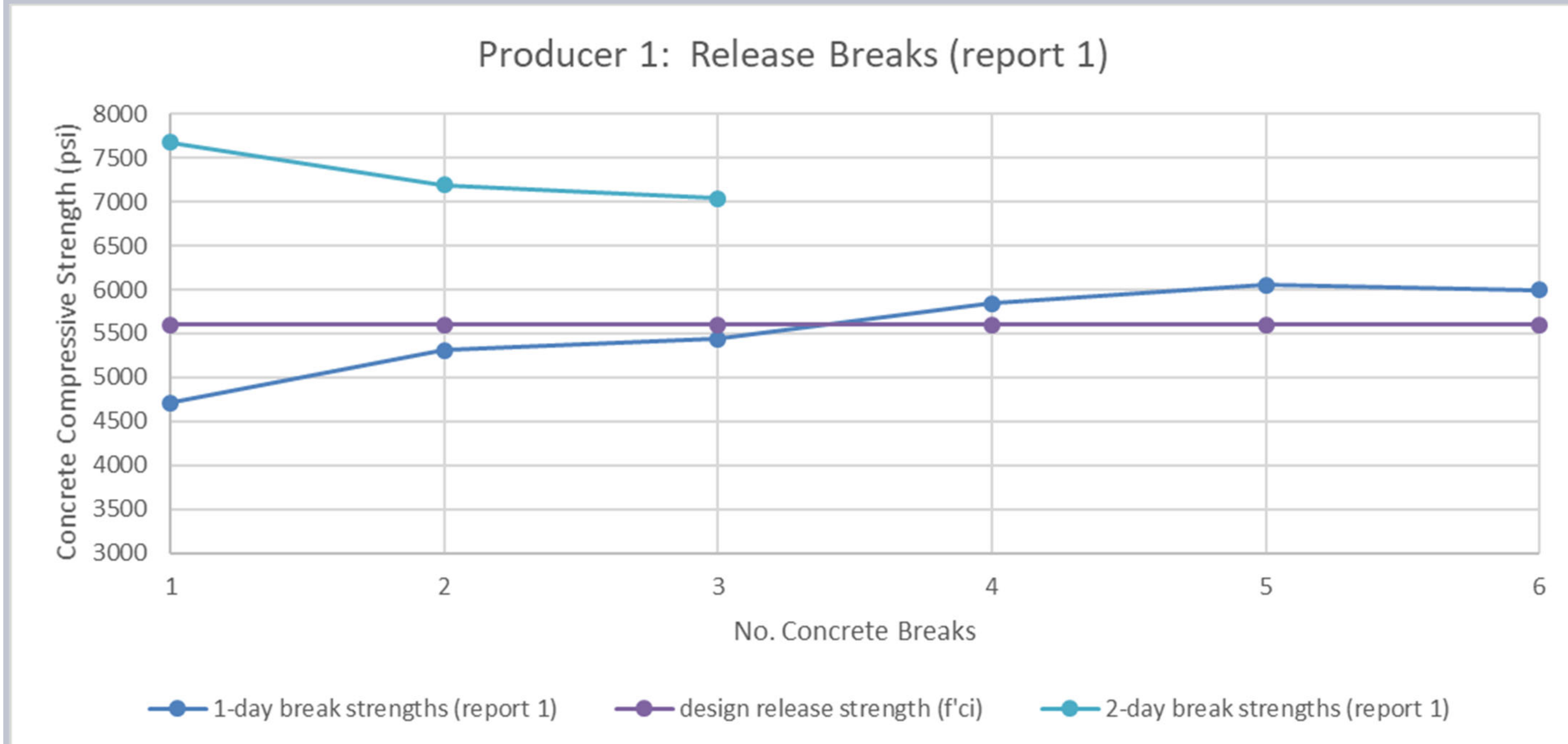
Historic Material Data
Producer 1



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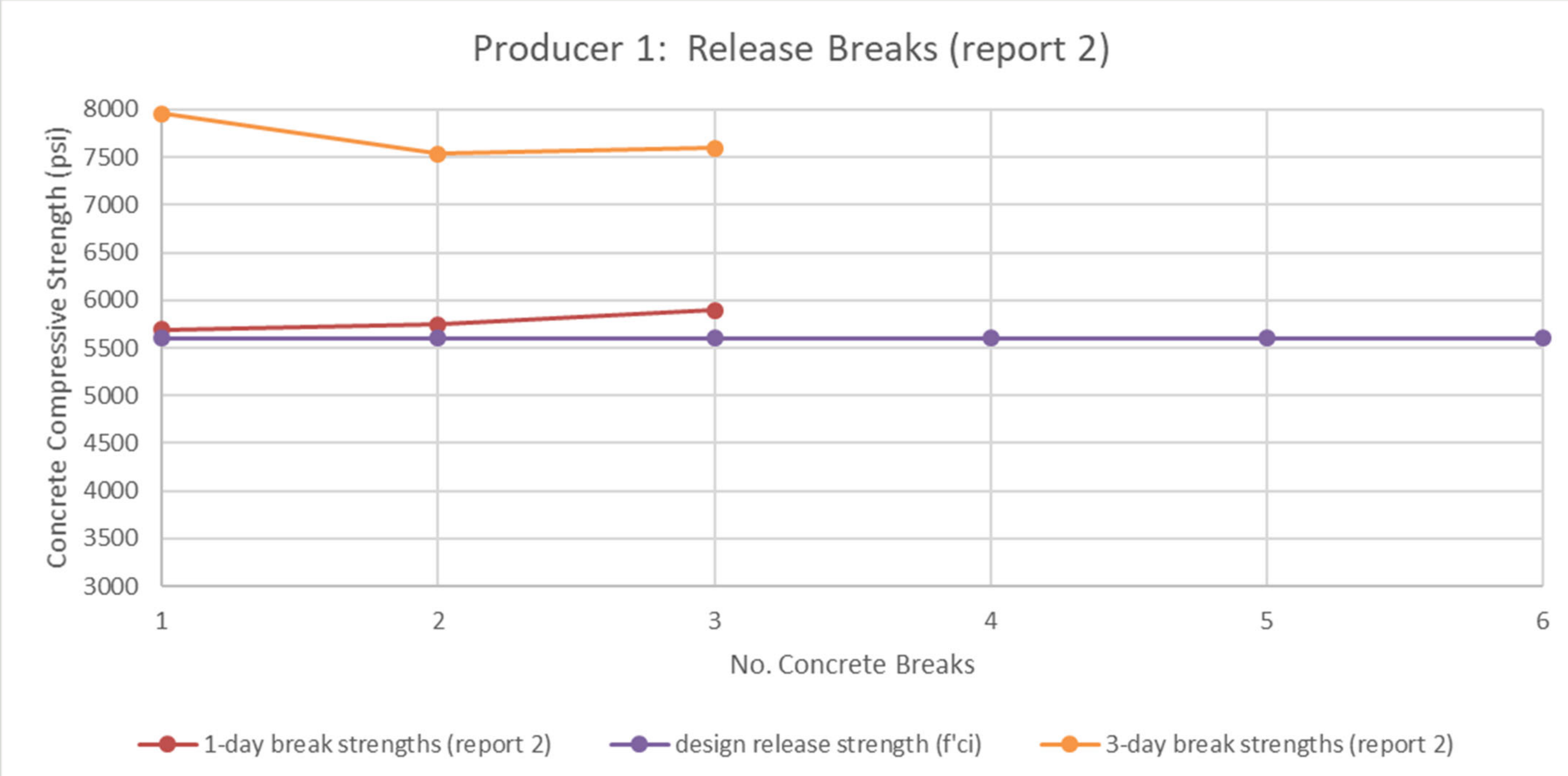
Historic Material Data
Producer 1



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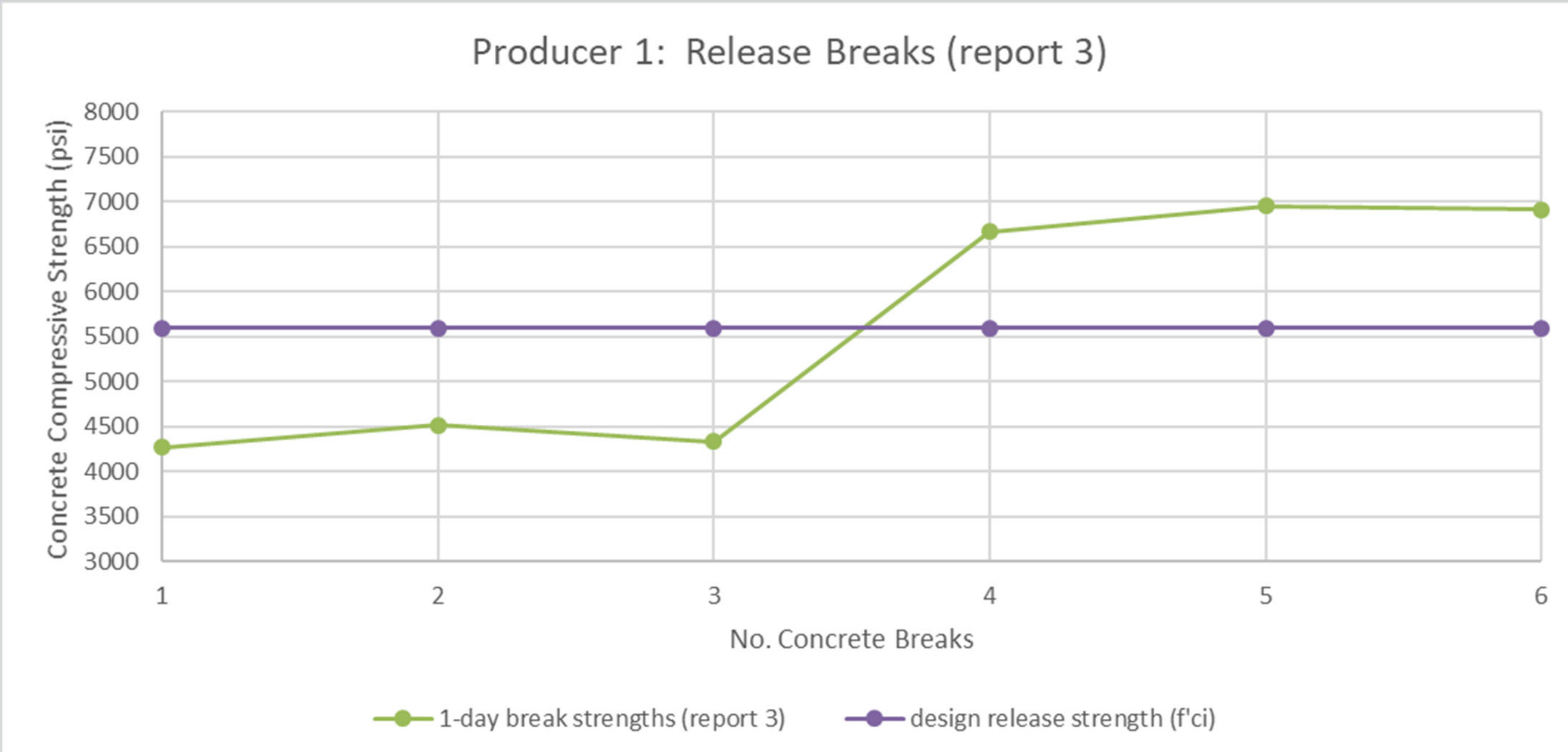
Historic Material Data
Producer 1



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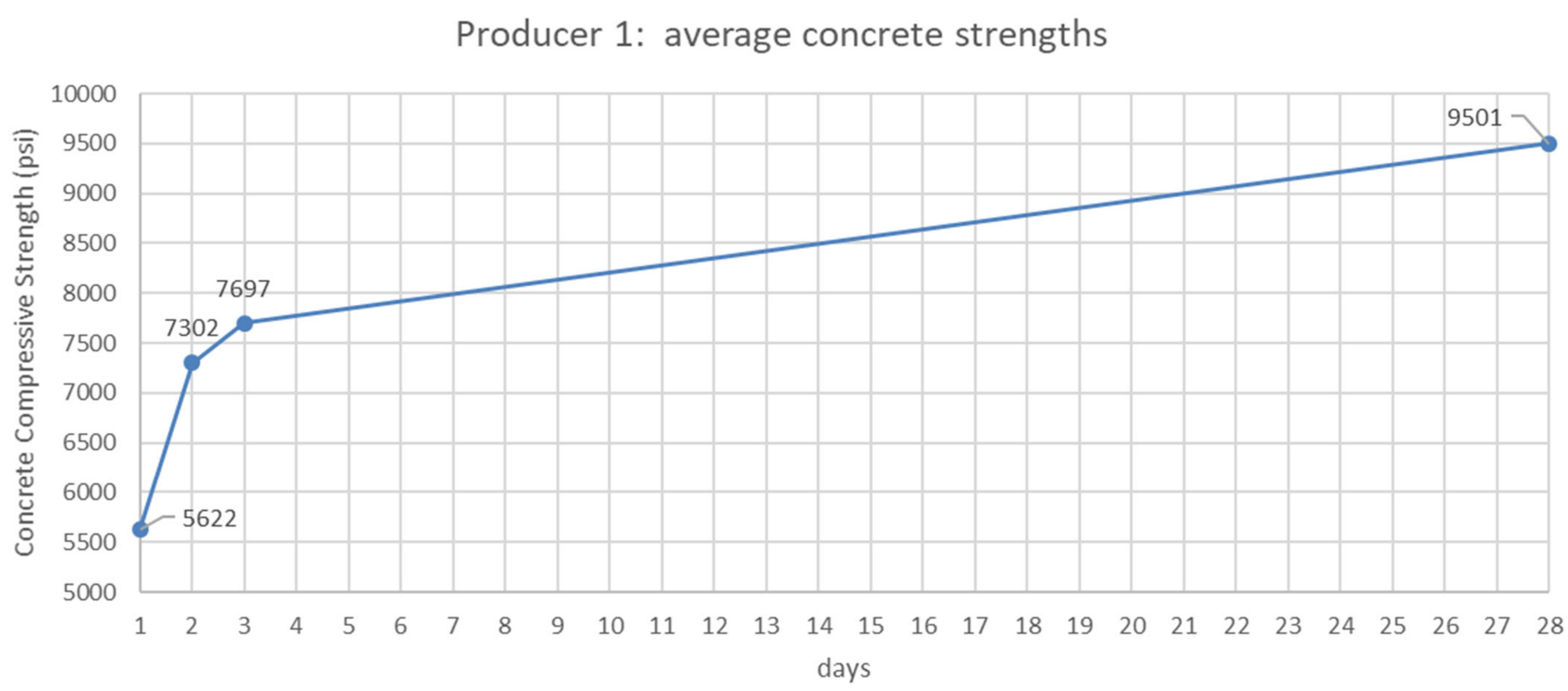
Historic Material Data
Producer 1



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Historic Material Data Producer 1

<u>Observations:</u>

1. average release break strength (5622 psi) exceeded the design release concrete strength (5600 psi) by 0.004%
2. average 28-day break strength (9501 psi) exceeded the design 28-day concrete strength (6500 psi) by 46%
3. ratio of average 1-day break strength (5622 psi) to average 28-day break strength (9501 psi) was 0.59

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Historic Material Data
Producer 2

project 1

pour 1		pour 2	
	break strength		break strength
age	(psi)	age	(psi)
1	4431	1	8788
1	4205	1	9415
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
2	no break	2	no break
2	no break	2	no break
2	no break	2	no break
3	no break	3	no break
3	no break	3	no break
3	no break	3	no break
28	11124	28	10848
28	12449	28	10358
28	11218	28	10262

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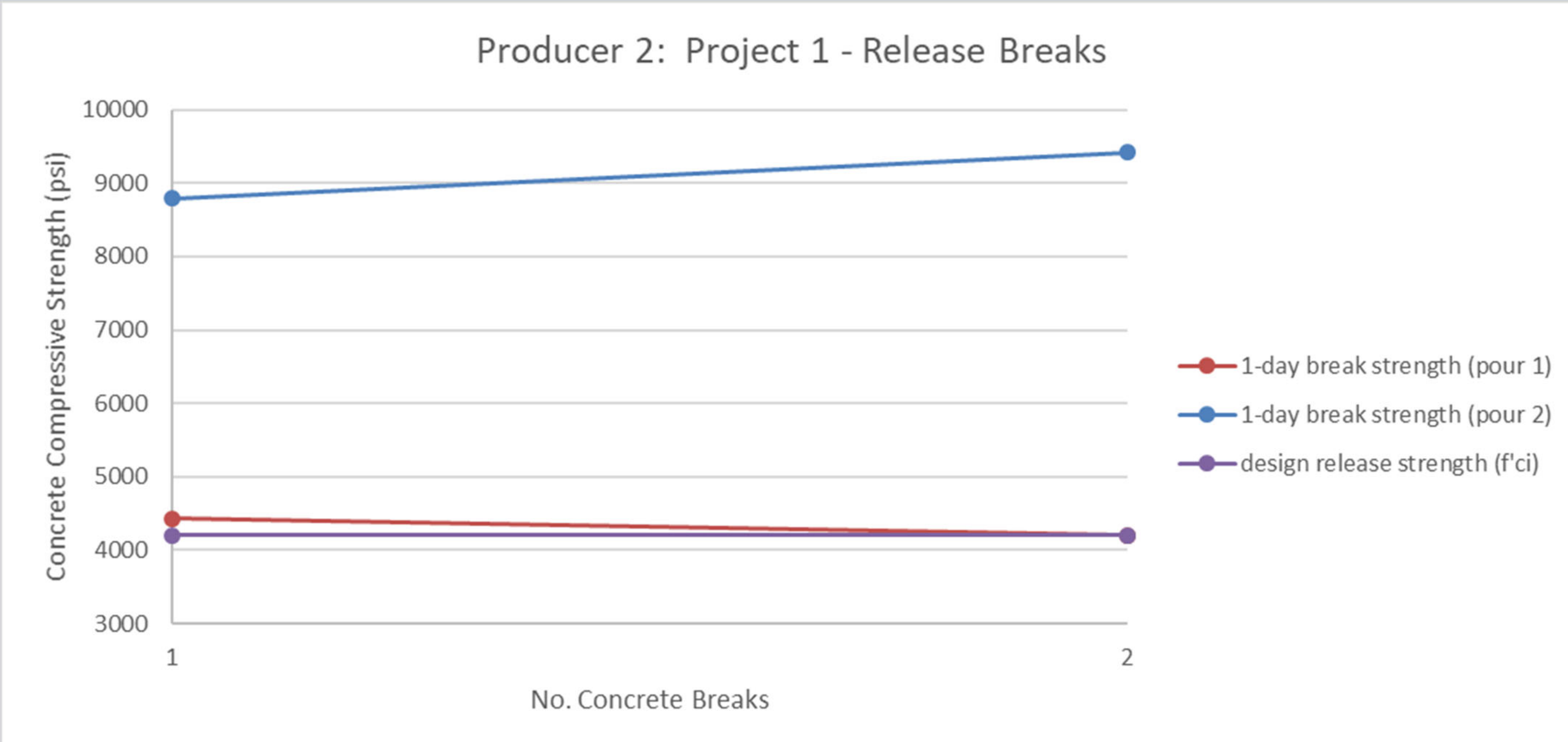
Historic Material Data
Producer 2



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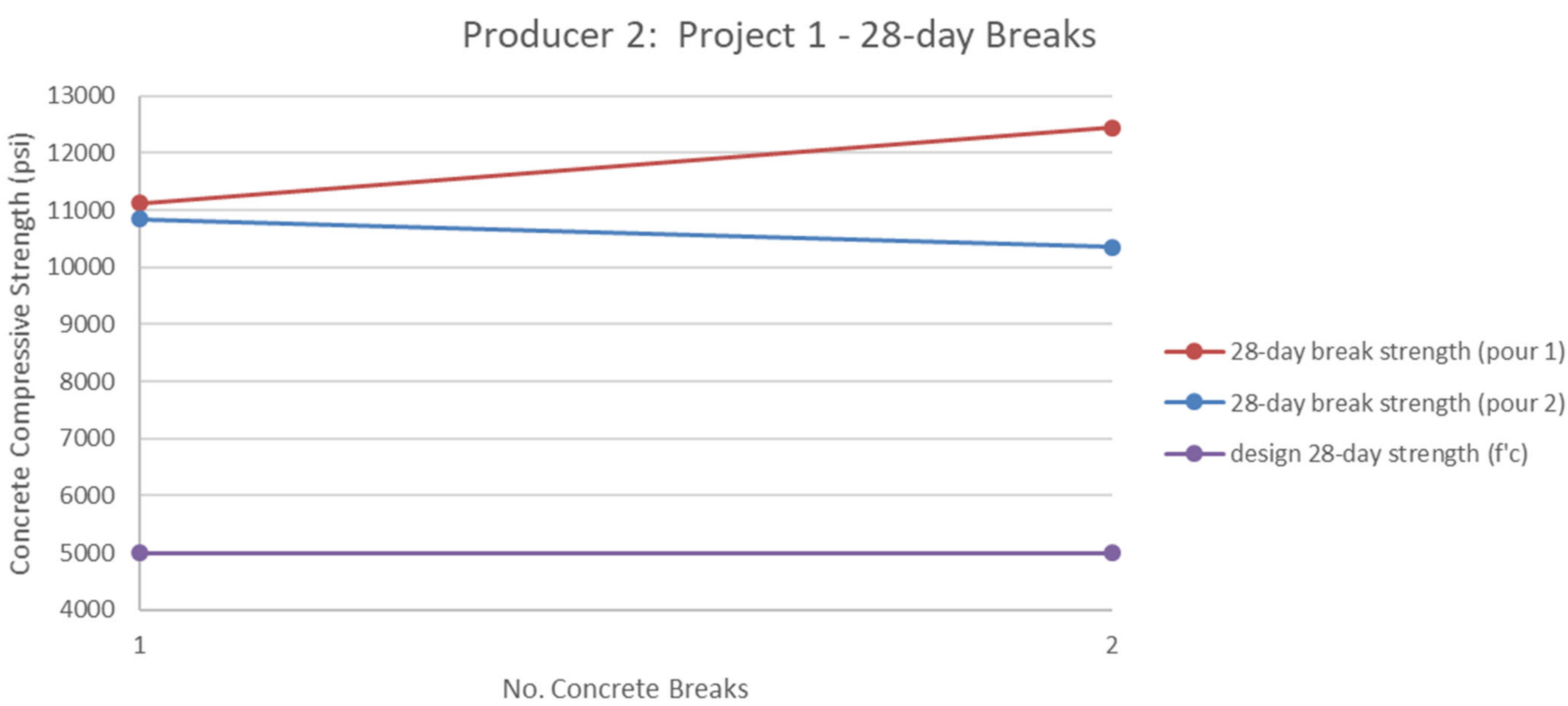
Historic Material Data
Producer 2



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Historic Material Data
Producer 2

project 2

pour 1	
	break strength
age	(psi)
1	8788
1	9415
1	no break
1	no break
1	no break
1	no break
2	no break
2	no break
2	no break
3	no break
3	no break
3	no break
28	10848
28	10358
28	10262

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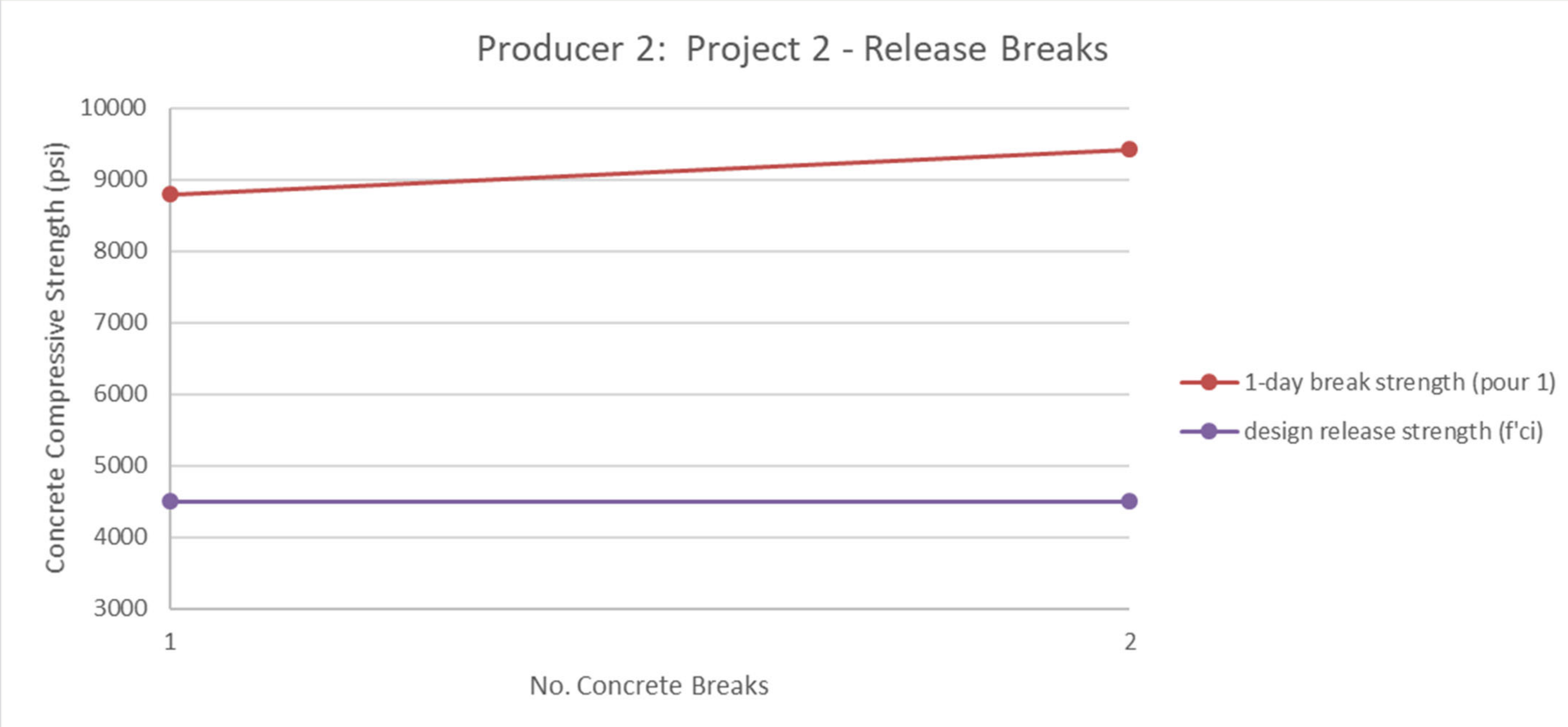
Historic Material Data
Producer 2



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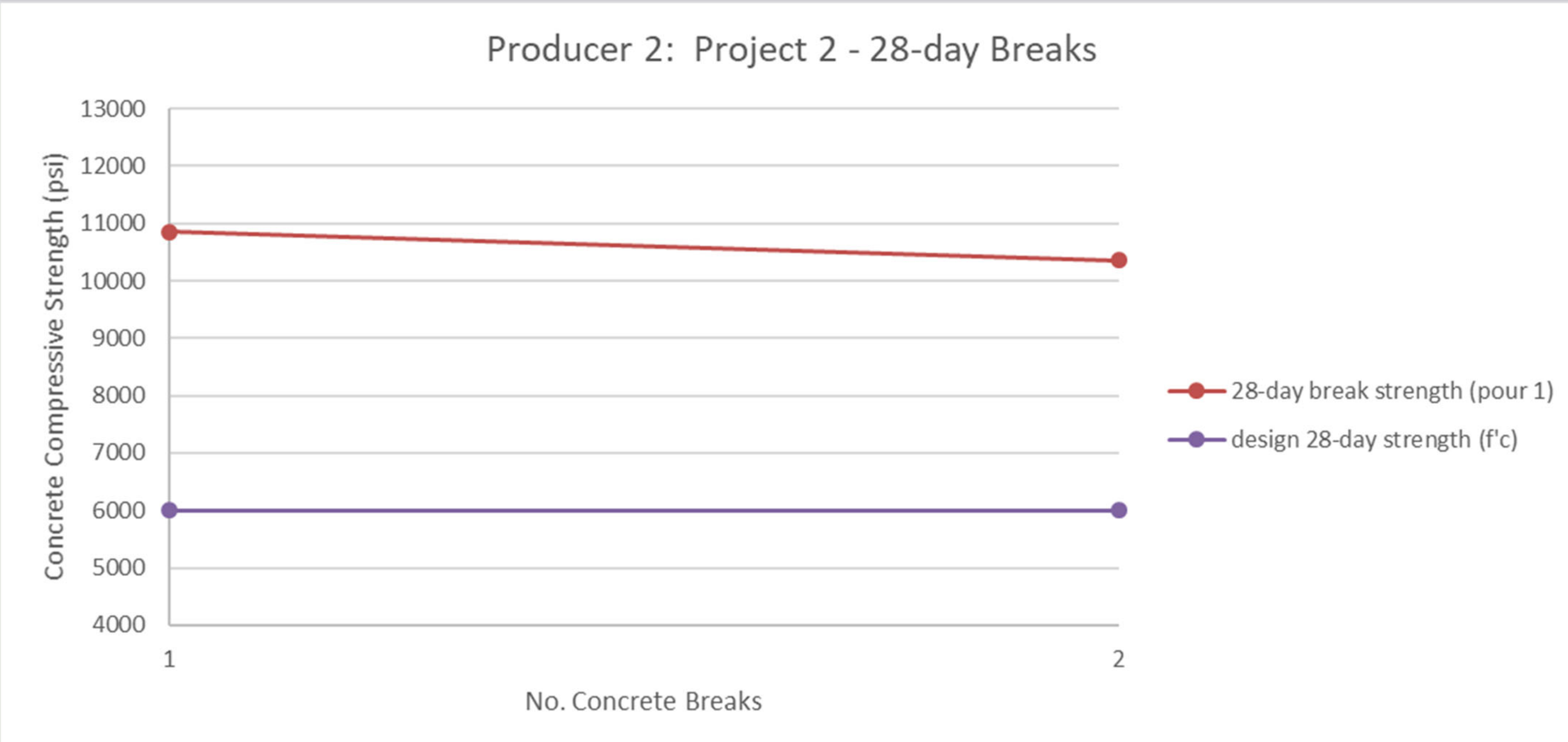
Historic Material Data
Producer 2



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Historic Material Data
Producer 2

project 3

pour 1		pour 2	
	break strength		break strength
age	(psi)	age	(psi)
1	10183	1	5646
1	9837	1	6913
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
2	no break	2	no break
2	no break	2	no break
2	no break	2	no break
3	no break	3	no break
3	no break	3	no break
3	no break	3	no break
28	12746	28	10213
28	12284	28	10352
28	12128	28	10072

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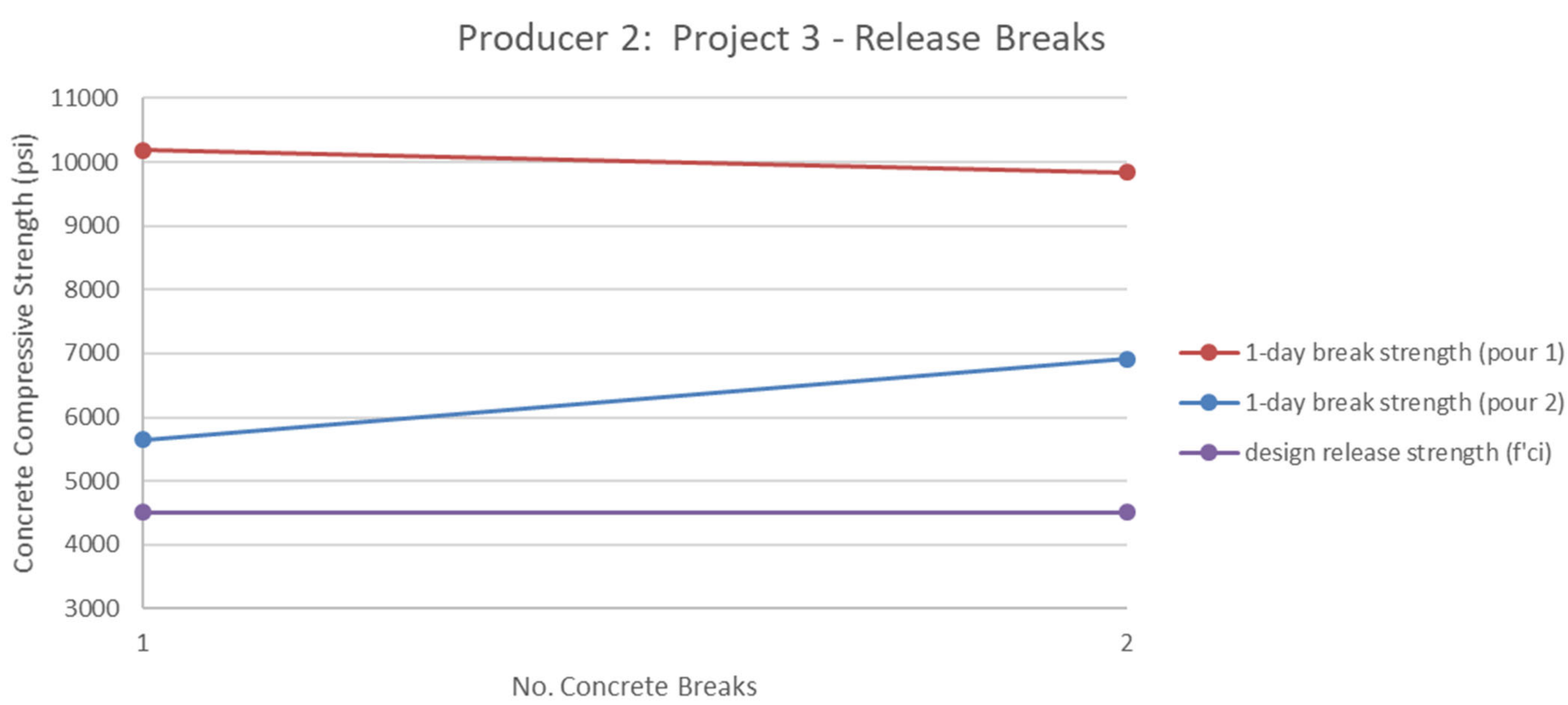
Historic Material Data
Producer 2



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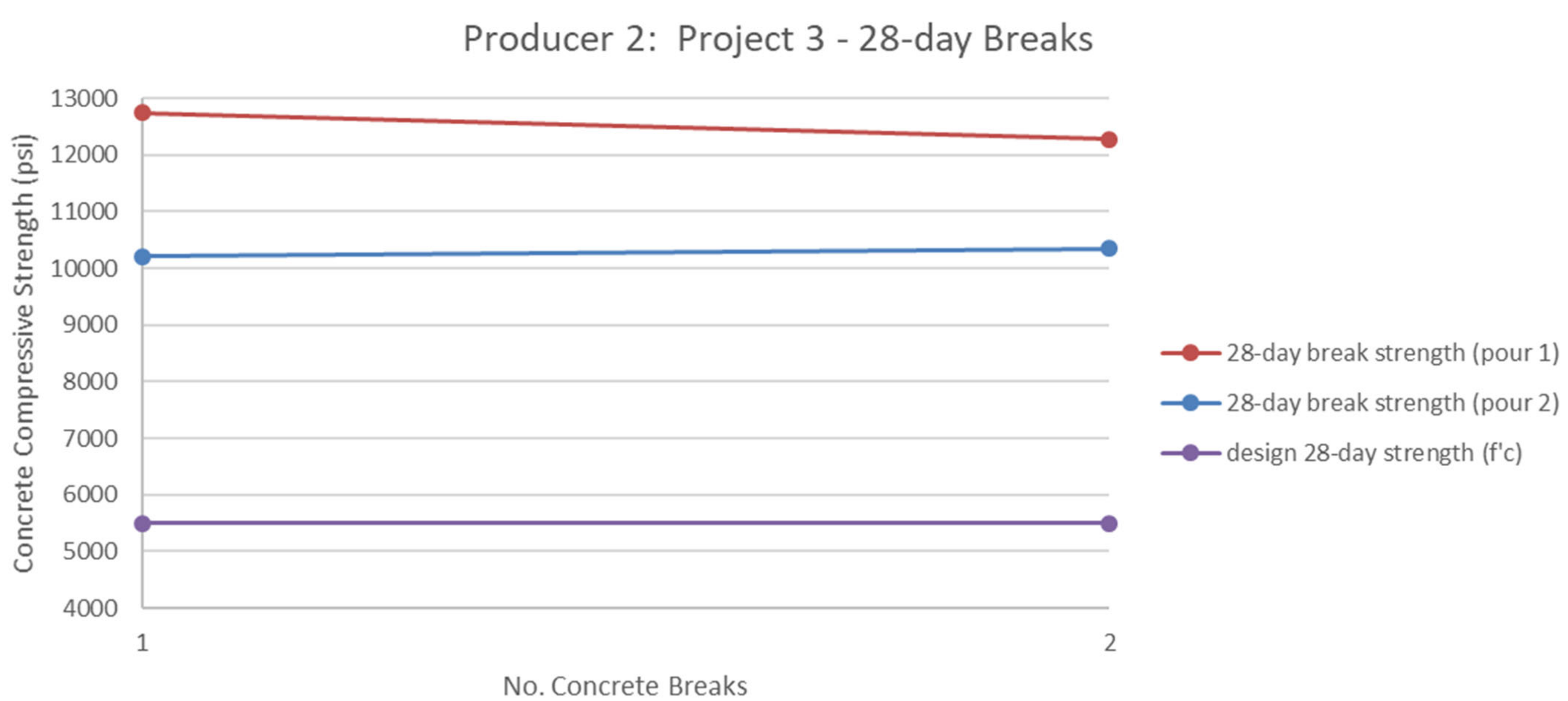
Historic Material Data
Producer 2



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Historic Material Data
Producer 2

project 4

pour 1		pour 2	
	break strength		break strength
age	(psi)	age	(psi)
1	7503	1	5732
1	7856	1	6063
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
2	no break	2	no break
2	no break	2	no break
2	no break	2	no break
3	no break	3	no break
3	no break	3	no break
3	no break	3	no break
28	10159	28	9139
28	10670	28	9002
28	no break	28	no break

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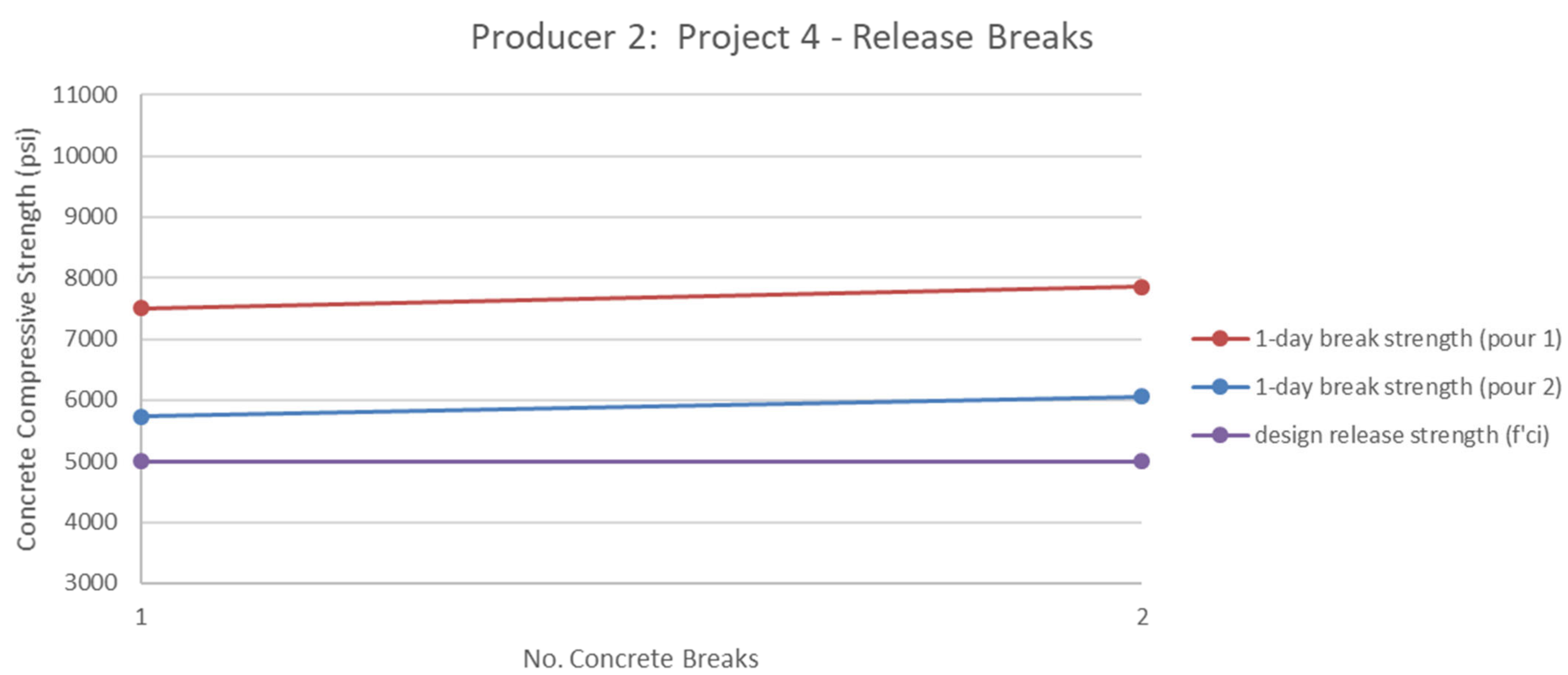
Historic Material Data
Producer 2



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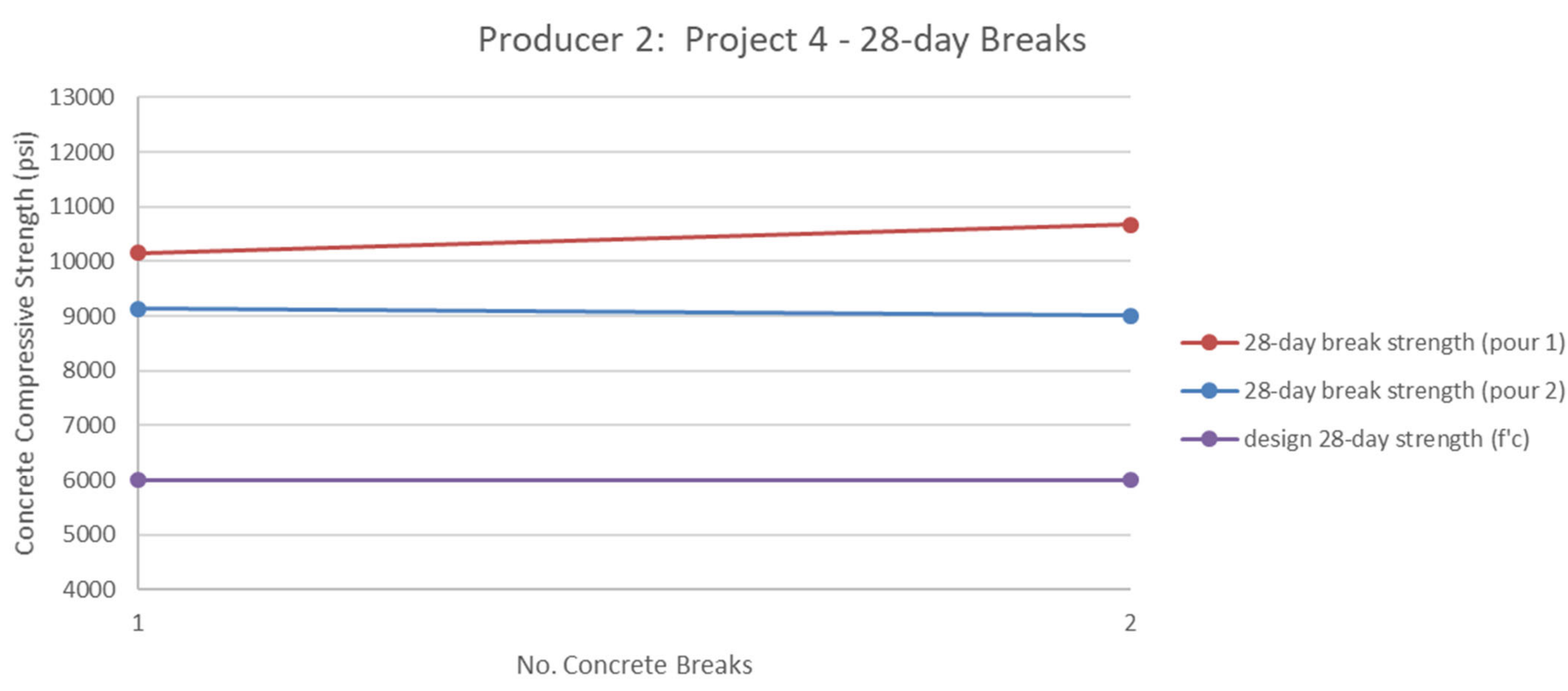
Historic Material Data
Producer 2



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Historic Material Data
Producer 2

project 5

pour 1		pour 2	
	break strength		break strength
age	(psi)	age	(psi)
1	9049	1	4563
1	8708	1	4766
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
1	no break	1	no break
2	no break	2	no break
2	no break	2	no break
2	no break	2	no break
3	no break	3	no break
3	no break	3	no break
3	no break	3	no break
28	no break	28	no break
28	no break	28	no break
28	no break	28	no break

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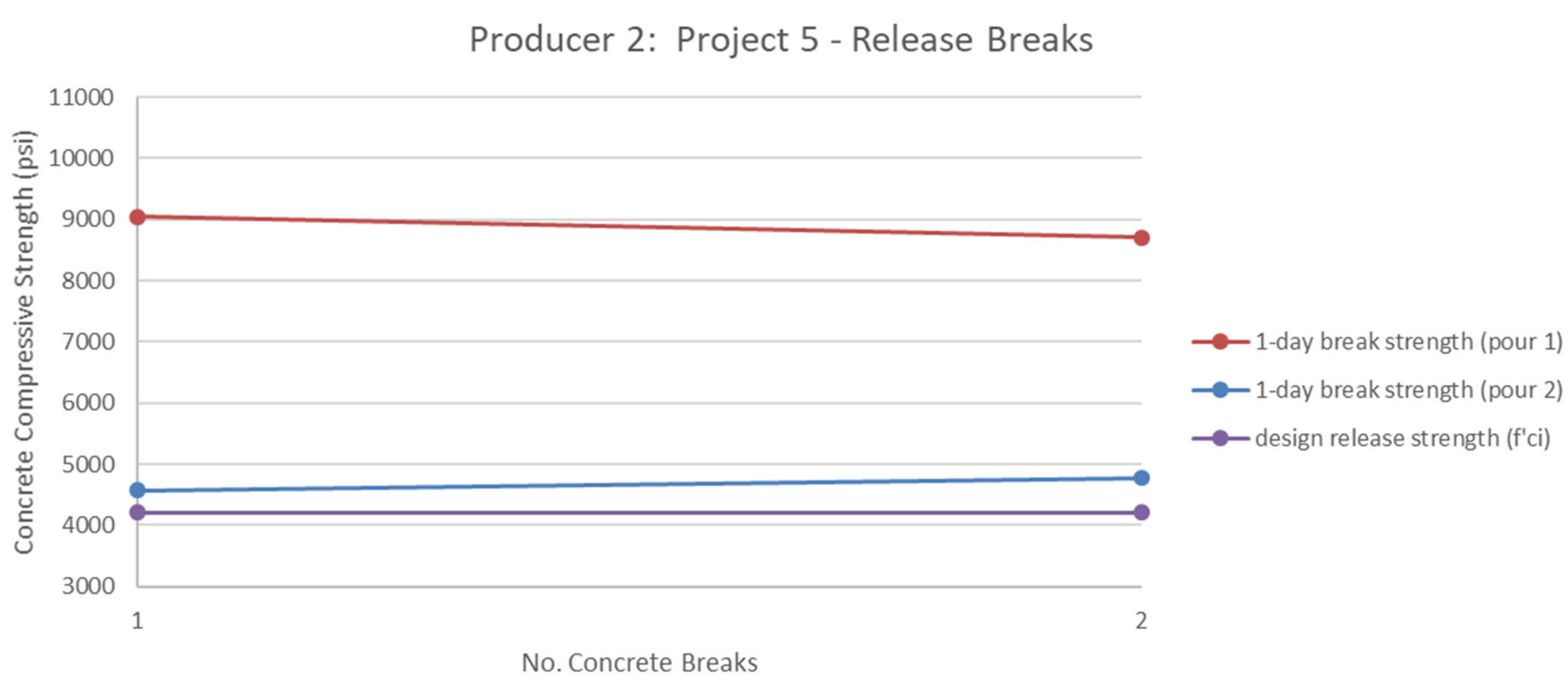
Historic Material Data
Producer 2



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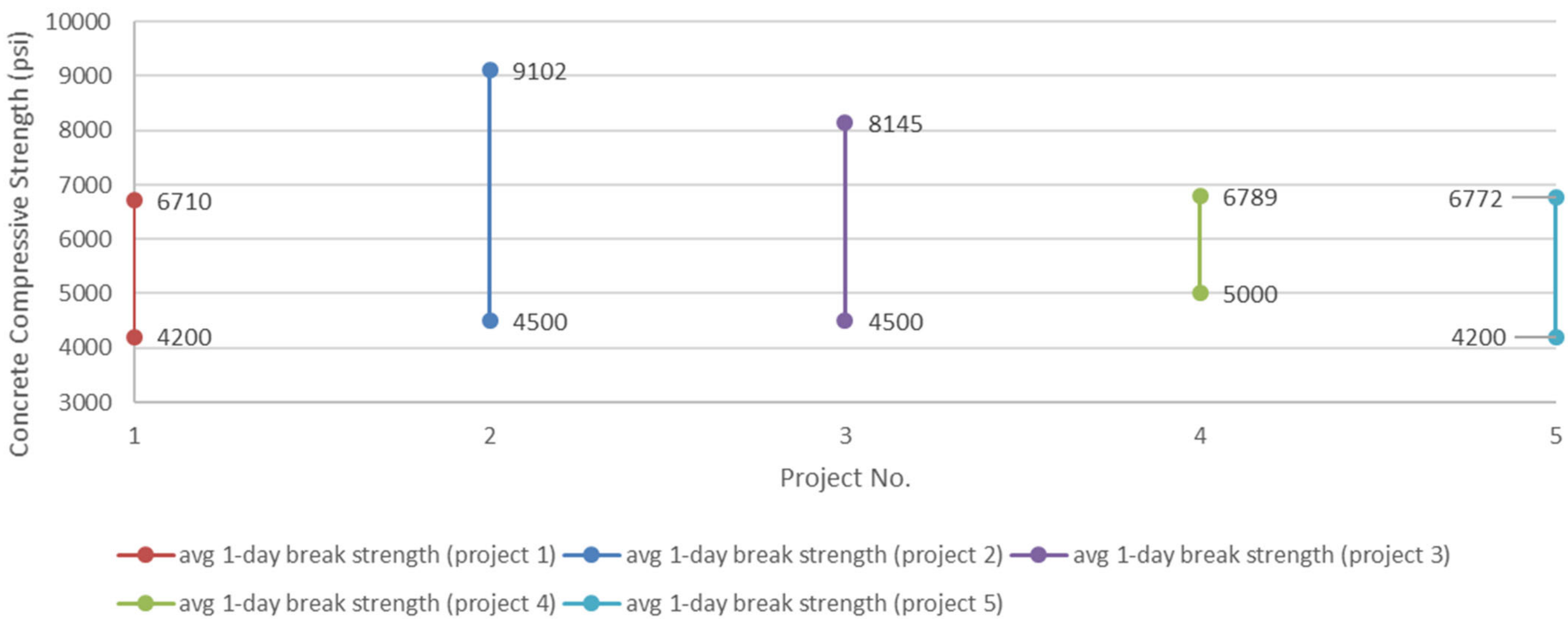
Historic Material Data
Producer 2

Producer 2: Average Release Breaks Compared to Design Release Strengths



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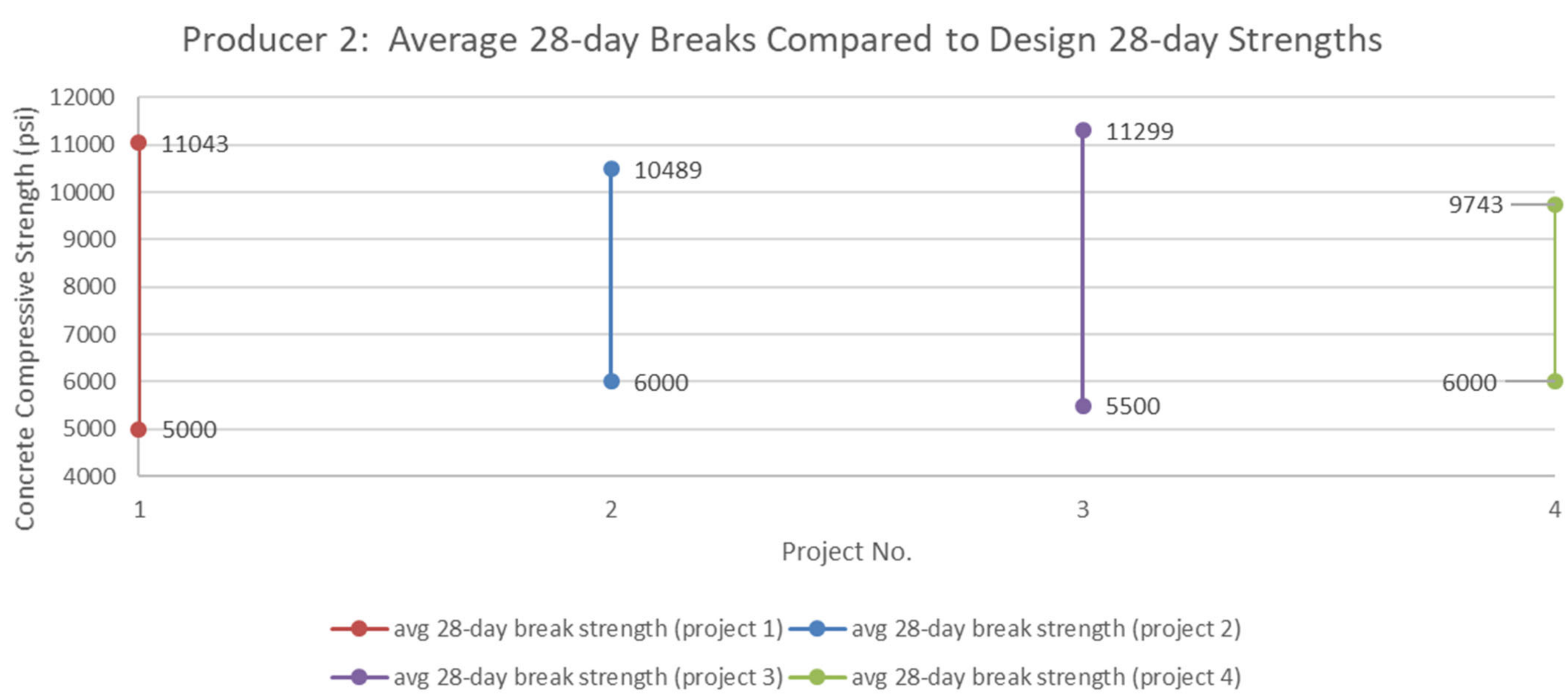
Historic Material Data
Producer 2



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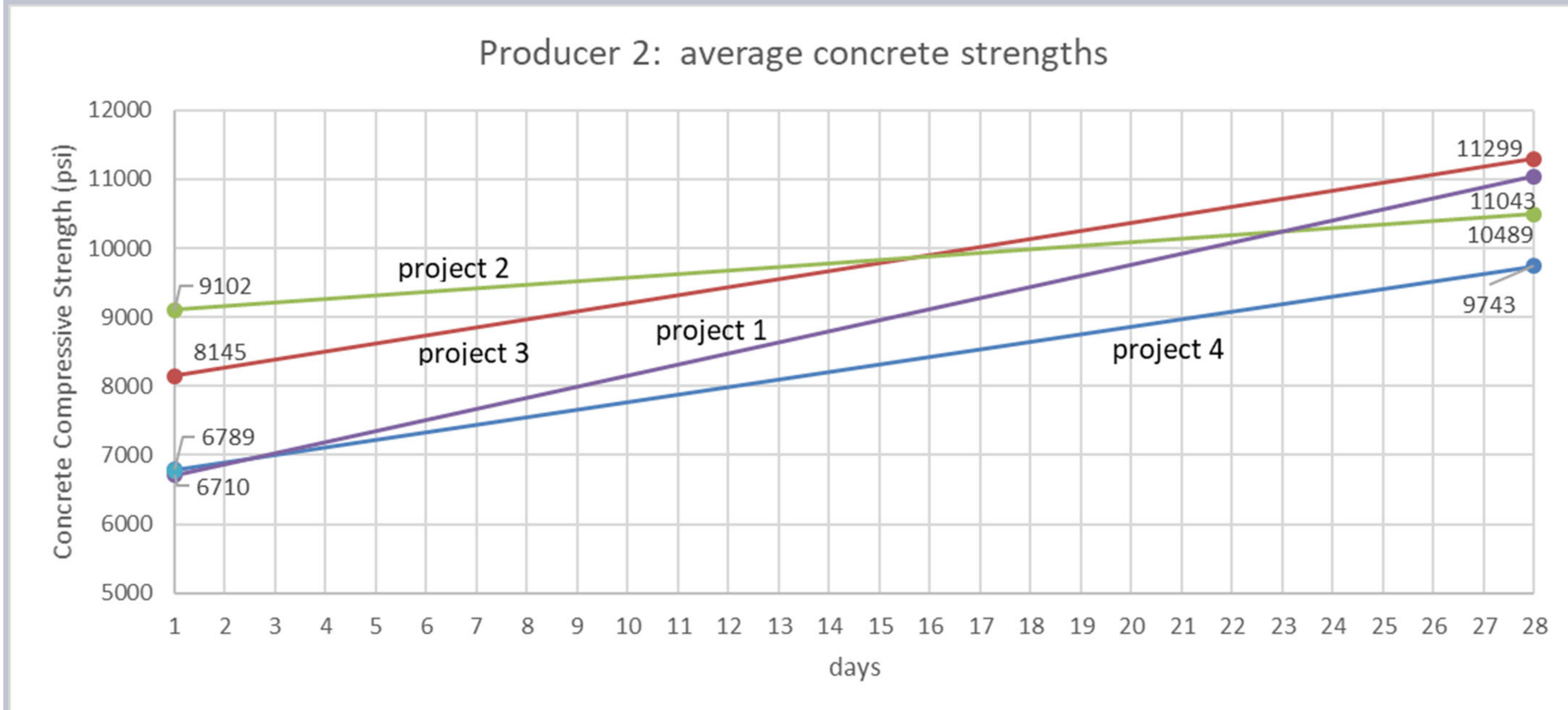
Historic Material Data
Producer 2



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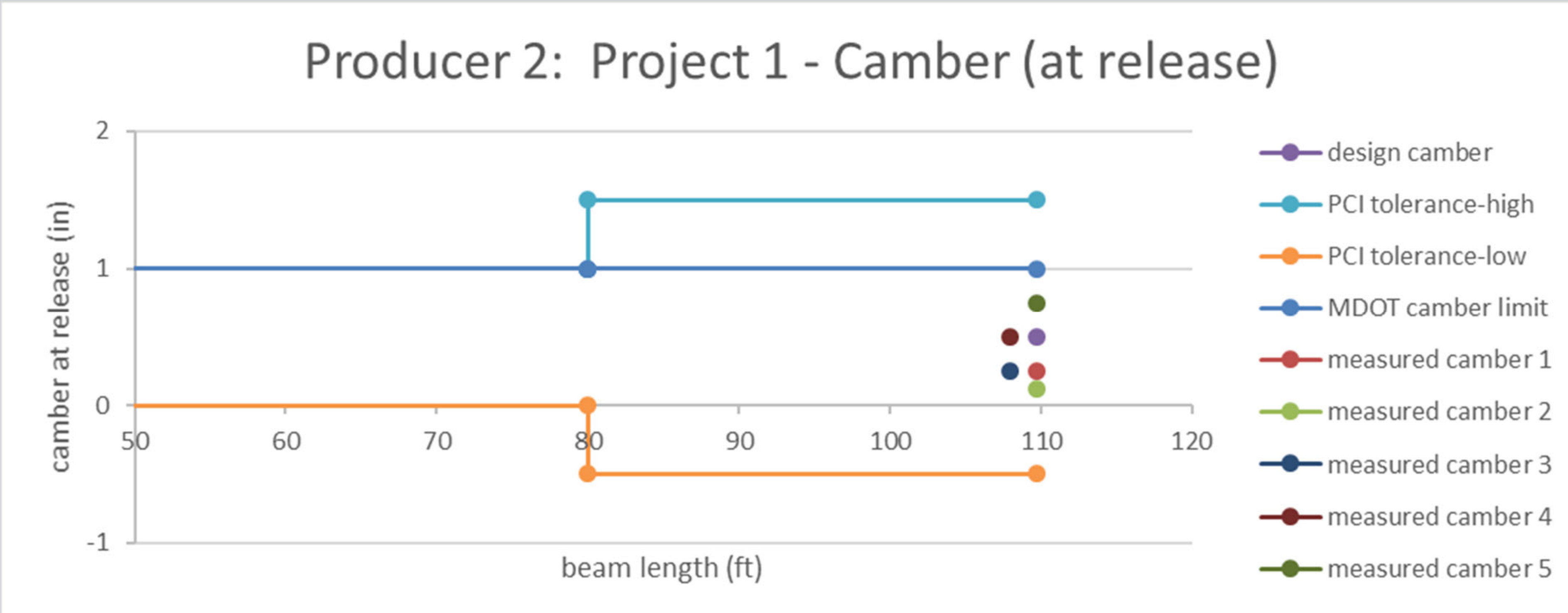
Historic Material Data
Producer 2



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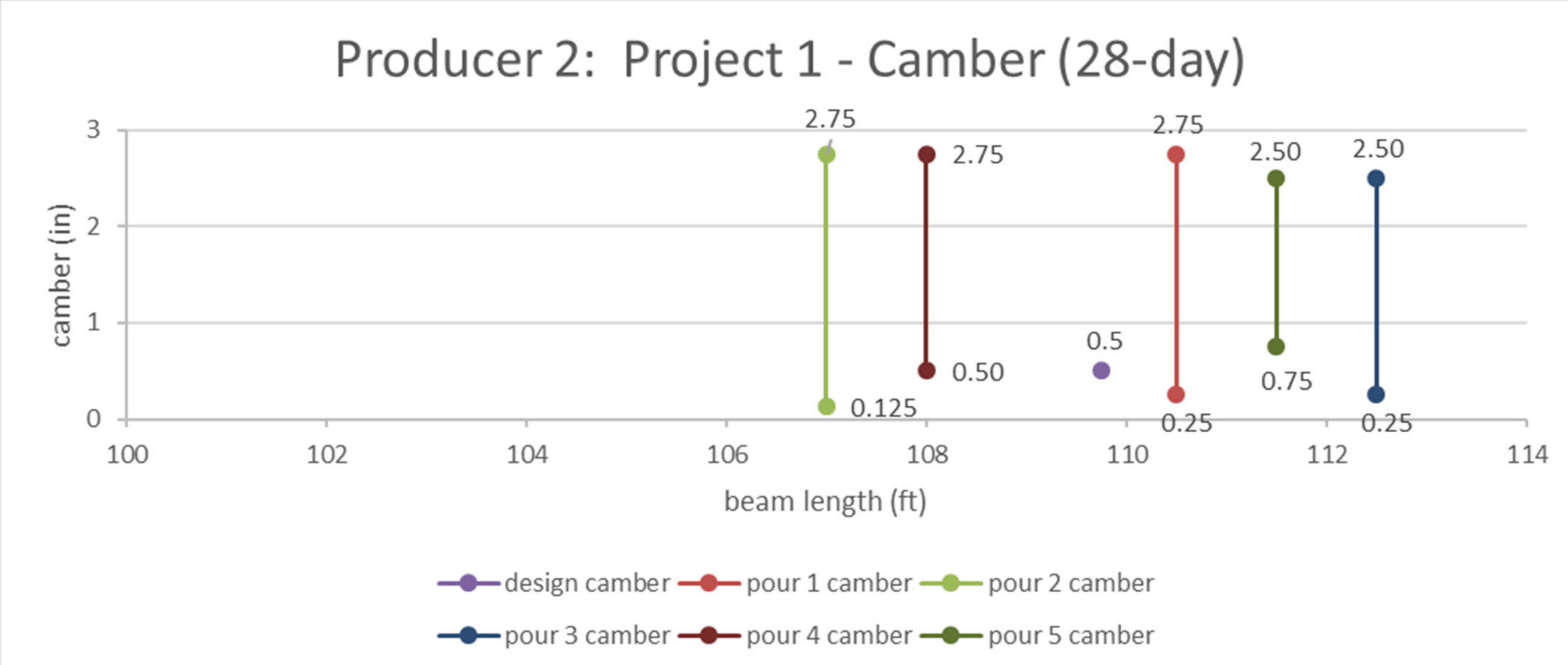
Historic Material Data
Producer 2



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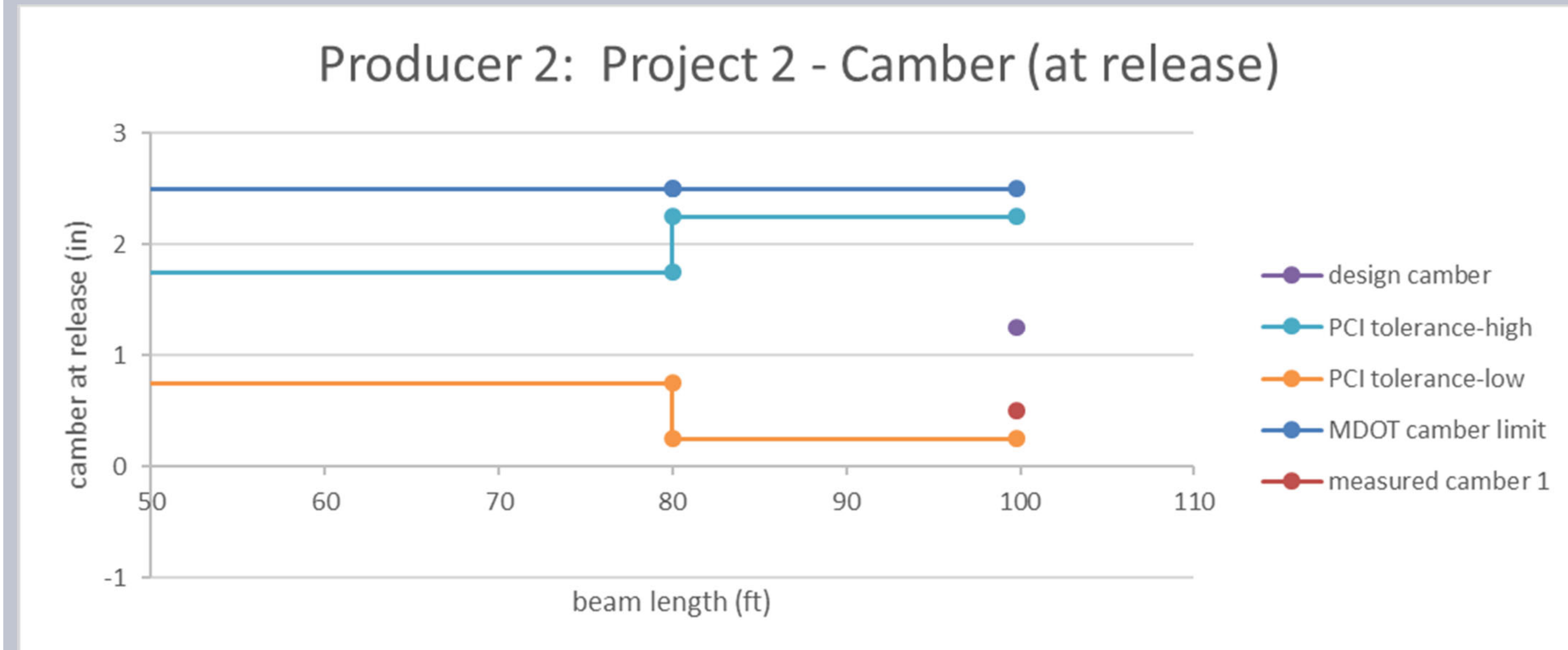
Historic Material Data
Producer 2



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Historic Material Data
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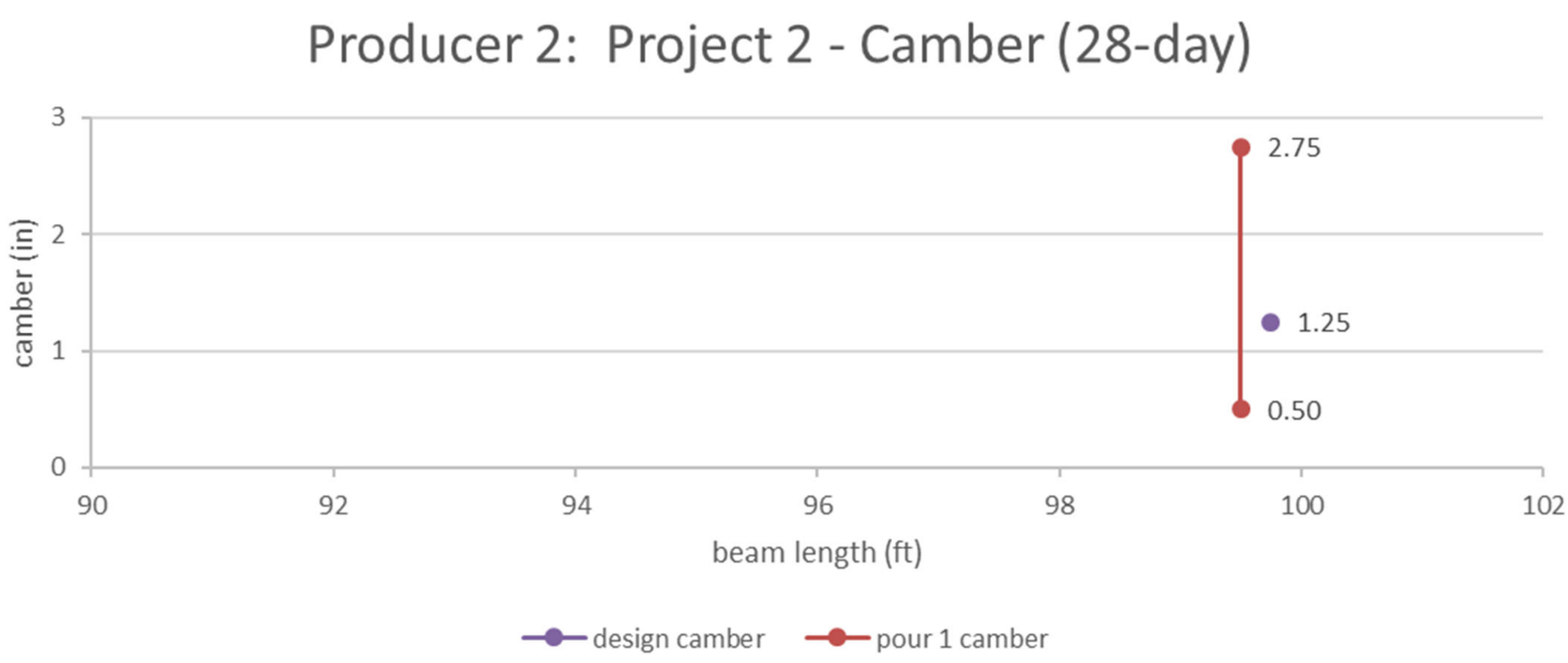

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Historic Material Data
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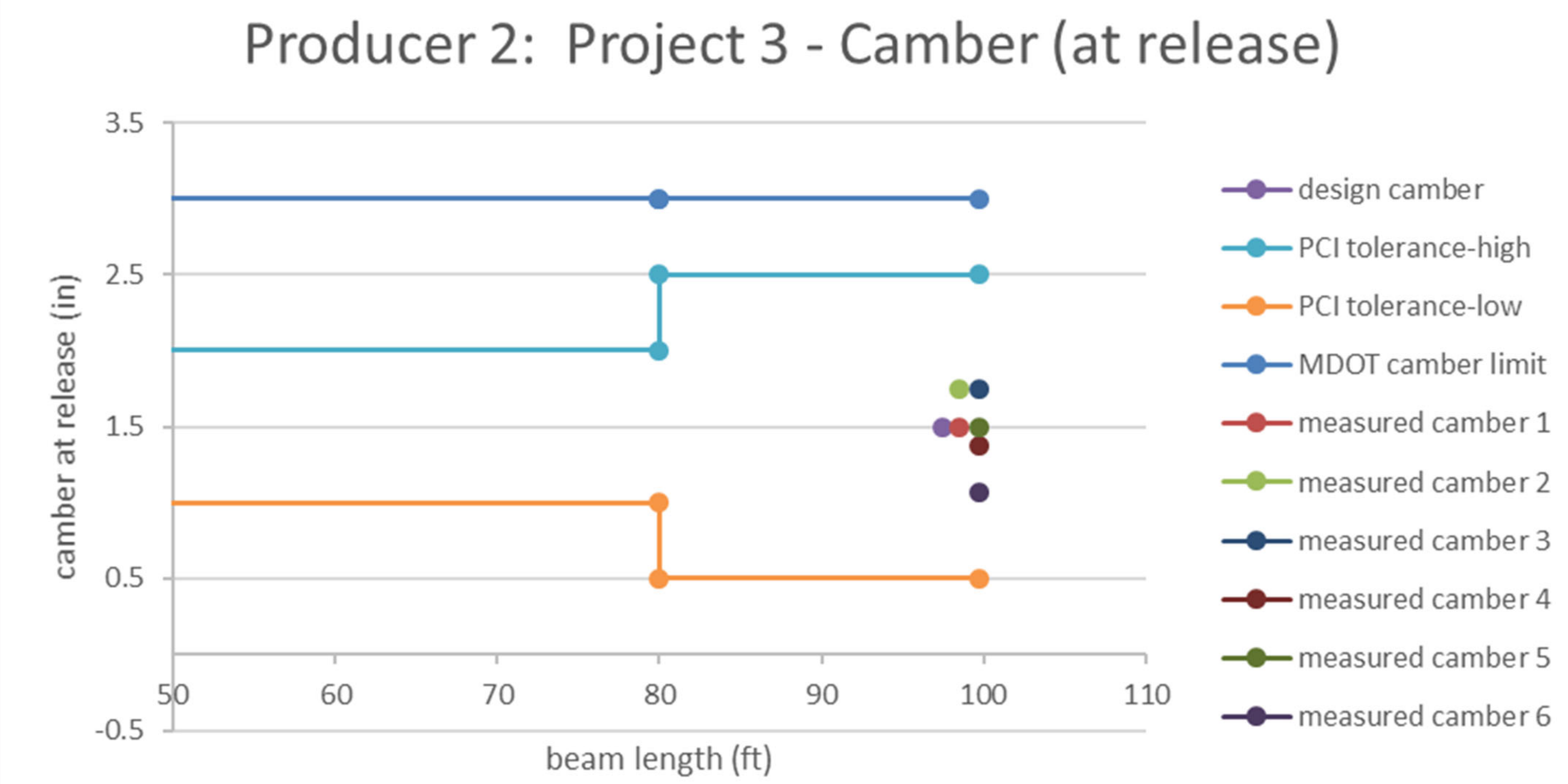

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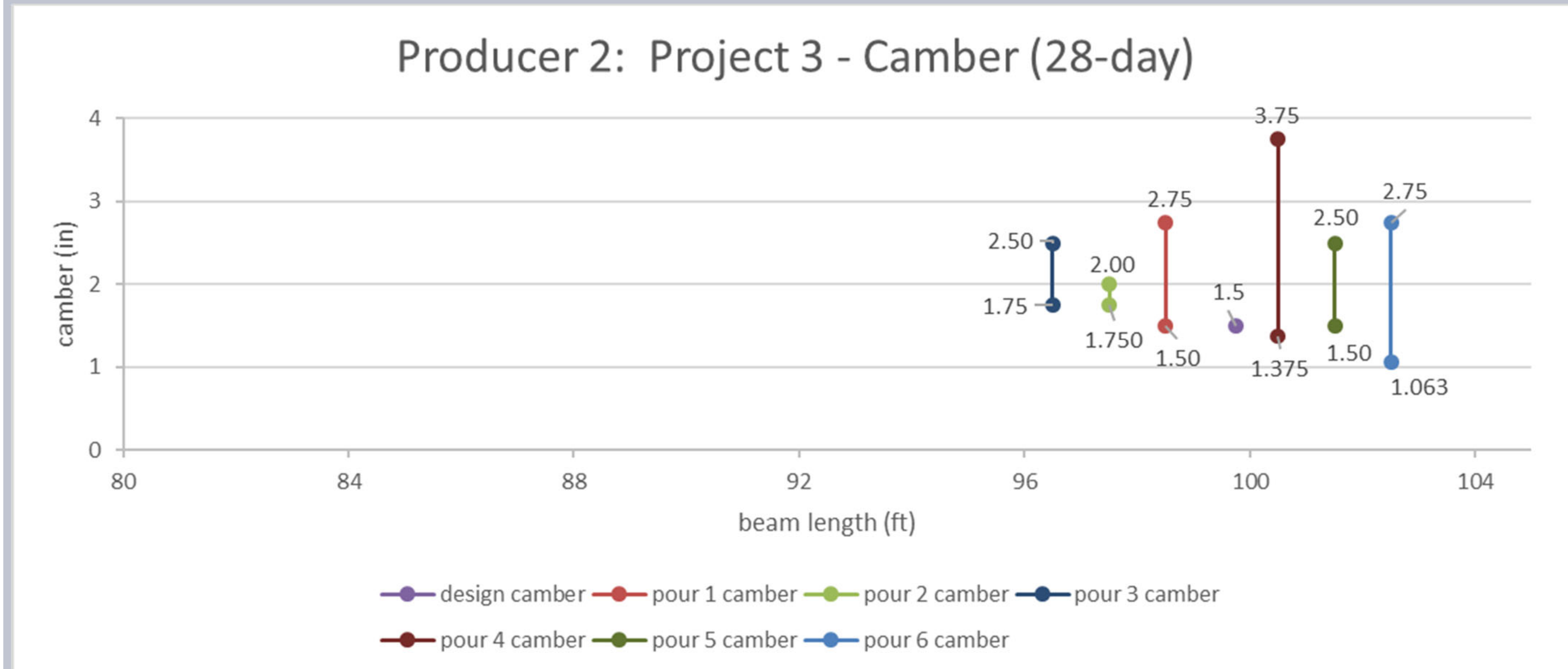
Historic Material Data
Producer 2



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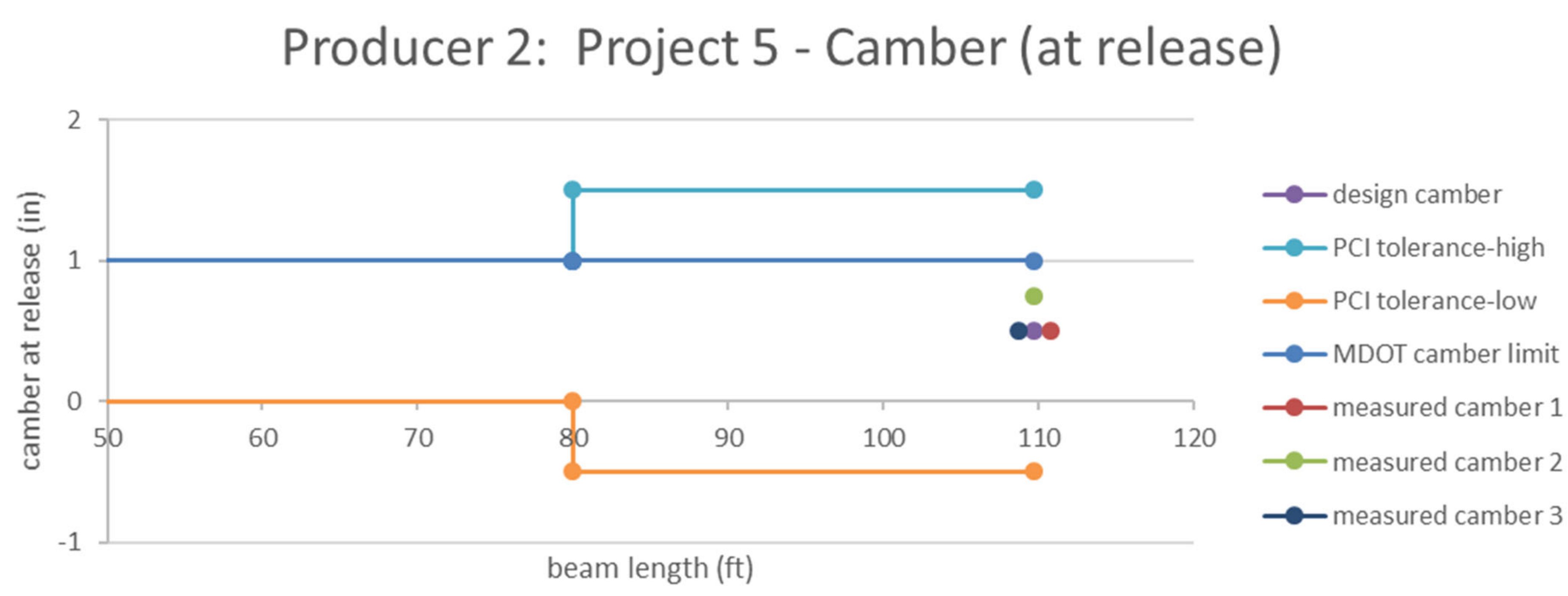
Historic Material Data
Producer 2



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Historic Material Data Producer 2

Observations:

1. average release break strength (7503 psi) exceeded the average design release concrete strength (4480 psi) by 67%
2. average 28-day break strength (10644 psi) exceeded the average design 28-day concrete strength (5625 psi) by 89%
3. ratio of average 1-day break strength (7503 psi) to average 28-day break strength (10644 psi) was 0.71
4. average measured camber at release (0.74") was 21% less than the average design camber (0.94") and was within both MDOT's camber limit requirements and PCI tolerances
5. average measured camber at 28-days (2.37") had a ratio of 3.2 greater than the average measured camber at release (0.74")

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Historic Material Data
Producer 3

pour 1		pour 2		pour 3	
	break strength		break strength		break strength
age	(psi)	age	(psi)	age	(psi)
3	9015	1	7030	3	6765
3	9180	1	7510	3	8305
12	9666	1	7095	3	7595
12	9522	1	7390	14	9867
28	10203	14	9143	14	9984
28	10237	14	9100	14	10125
		14	9299	14	10106
		14	9344	28	10310
		28	10048	28	10270
		28	10000	28	10440
		28	10488	28	10408
		28	10520		

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Historic Material Data
Producer 3

pour 4		pour 5		pour 6	
	break strength		break strength		break strength
age	(psi)	age	(psi)	age	(psi)
3	9675	1	7365	1	7730
3	9275	1	7605	1	7890
3	8575	14	9389	13	8851
13	10787	14	9410	13	8769
13	10705	28	10142	28	9650
13	9615	28	10100	28	9587
13	9597				
28	10787				
28	10705				
28	11000				
28	10896				

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Historic Material Data
Producer 3

pour 7		pour 8		pour 9	
	break strength		break strength		break strength
age	(psi)	age	(psi)	age	(psi)
3	8800	1	7260	1	8349
3	8830	1	7090	1	8285
14	9432	14	9287	1	8320
14	9518	14	9221	1	8225
28	9987	14	9307	14	9898
28	9969	14	9336	14	9879
		28	10204	14	9810
		28	10189	14	9789
		28	10118	28	11856
		28	10132	28	11934
				28	11228
				28	11443

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Historic Material Data
Producer 3

pour 10		pour 11		pour 12	
	break strength		break strength		break strength
age	(psi)	age	(psi)	age	(psi)
3	9550	2	9349	1	8510
3	9675	2	9155	1	8300
3	9948	2	9400	1	8030
3	10035	2	9290	1	8320
14	9474	11	9101	28	9917
14	9525	11	9086	28	9854
14	10017	11	9061	28	10270
14	10046	11	9027	28	10240
28	11120	28	10205		
28	10957	28	10229		
28	11676	28	10197		
28	11491	28	10182		

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Historic Material Data
Producer 3

pour 13		pour 14	
	break strength		break strength
age	(psi)	age	(psi)
1	7580	2	8287
1	7760	2	9110
1	7200	2	9065
1	7115	2	8215
28	10085	16	10612
28	10145	16	10587
28	10025	16	10460
28	9948	16	10500
		30	10202
		30	10237
		30	10343
		30	10400

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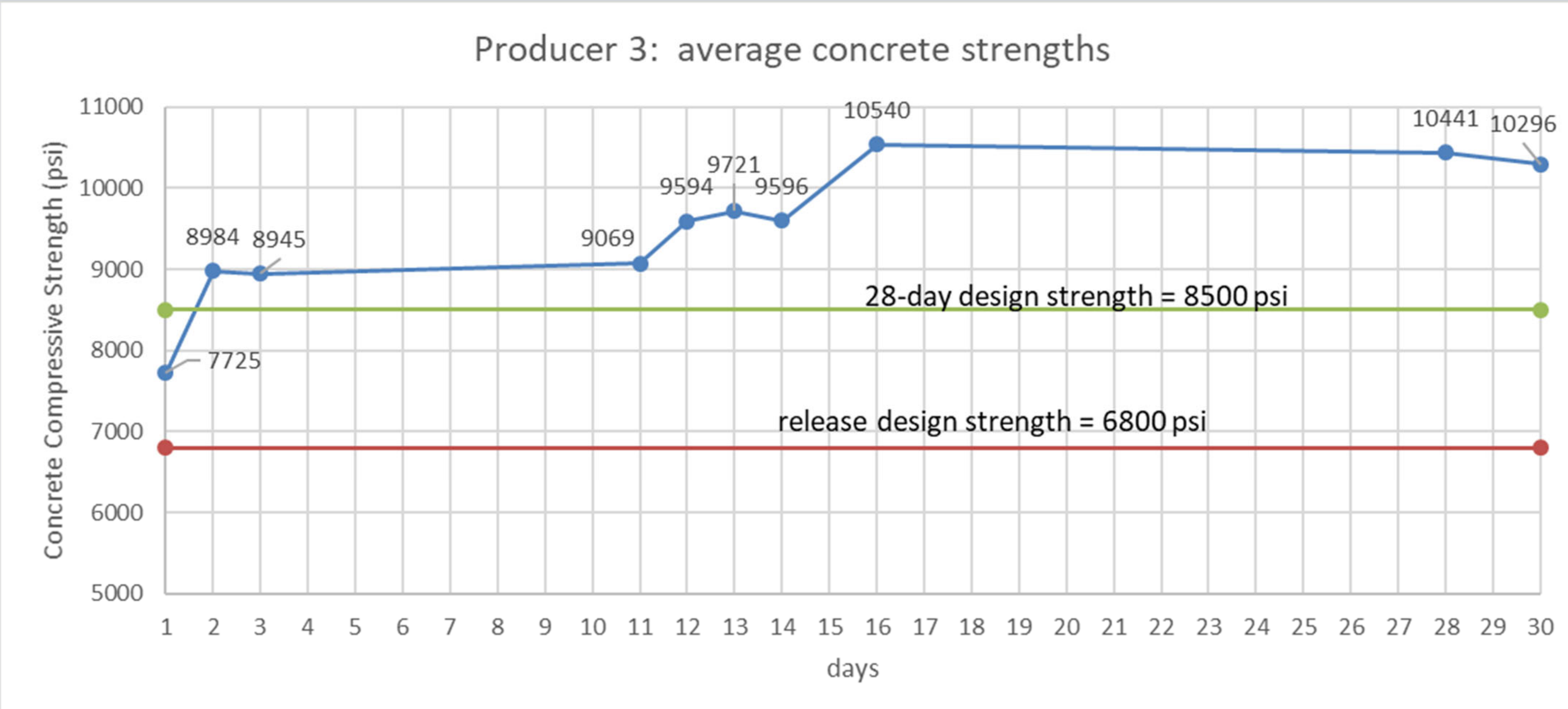
Historic Material Data
Producer 3



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Historic Material Data
Producer 3

camber (at release) was provided by producer 3 on 15 different projects with unique girder types as noted								
project no.	girder type	camber (at release), inches						average
1	BT-72	2.375	3.5	2.5				2.79
2	AASHTO 2	1	1	1	1			1.00
3	BT-54 mod	2	2.25	2				2.08
4	BT-63	2.4375	2.5	2.4375				2.46
5	BT-72	2.25	2.25	2.125	2.3125			2.23
6	BT-72 mod	2.5	2.5625	2.5	2.375	2.5	2.4375	2.47
		2.4375						
7	BT-54		2.125	2	2	2.25	2.125	2.16
		2.25	2.25	2.125	2.3125	2.25	2.125	
		2.1875	2.125	1.9375	2	2.0625	2.0625	
		2.125	1.75	2	2.25	2.1875	2.25	
		2.25	2.5	2.375	2.25	2	2.0625	
		2.0625	2.375	2.25	2.125	2	2.25	
		2.375						
8	BT-54 mod	2.375	2.3125	2.5	2.4375	2.375	2.5	2.42
9	BT-63 mod	1.25						1.25
10	AASHTO 2	0.625	0.875	0.75				0.75
11	BT-63 mod	2.25	2	2.125				2.13
12	BT-54 mod	2.375	2.25	2.375				2.33
13	AASHTO 3	1.125						1.13
14	BT-63 mod	2.5						2.50
15	AASHTO 2	0.625	0.625					0.63

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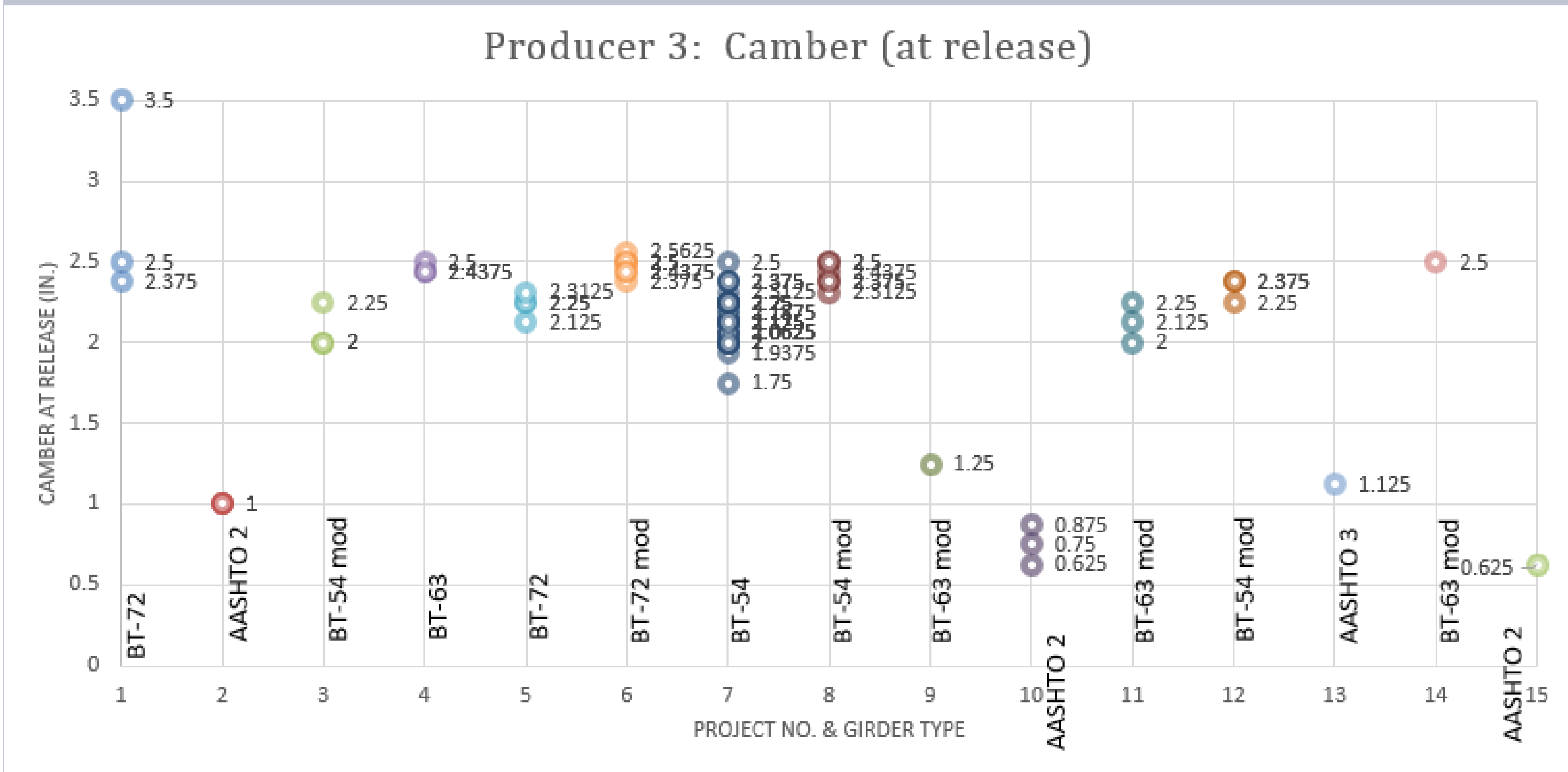
Historic Material Data
Producer 3



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Historic Material Data
Producer 3

Observations:

1. average 1-day release break strength (7725 psi) exceeded the design release concrete strength (6800 psi) by 14%
2. average 2-day release break strength (8984 psi) exceeded the design release concrete strength (6800 psi) by 32%
3. average 28-day break strength (10441 psi) exceeded the design 28-day concrete strength (8500 psi) by 23%
4. ratio of average 1-day break strength (7725 psi) to average 28-day break strength (10441 psi) was 0.74
5. camber data at release for 15 projects provides for additional insight into the amount of variation within a specific girder type and project for further statistical analysis

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Work In Progress

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Review MDOT Current Practices

- In follow up to the project kick-off meeting on March 1 2018, MDOT provided additional documents on their current practices related to design, fabrication, construction, and inspection of precast/prestressed concrete girders.
- Sample plans were provided for a bulb T bridge project located in Leake and Marshall Counties.

- MDOT Bridge Design Manual including bridge design memorandum, Bulb-T design procedure, and Prestressed Beam Camber Deflection spreadsheet
- MDOT Standard Specifications for Road and Bridge Construction including SOPs for inspection of prestressed concrete bridge members
- Sample plans for Bulb T bridge projects in Leake and Marshall County

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Survey other State DOT Current Practices

- This task includes review of Florida, Texas, Washington, and Nebraska DOTs current practices for estimating beam camber for girder types and lengths related to the MDOT research project.
- For comparison with local regional practices, Louisiana and Alabama DOTs current practices will be compared.

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Review AASHTO Bridge Design Specifications

- This task includes review of the current AASHTO LRFD Bridge Design Specifications and the PCI Bridge Design Manual as related to guidelines for estimating camber.

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Develop Research Data Sets for Camber Calculations

- This task includes developing research data sets for further calculations and evaluation of estimating camber.
- Insights gained from the literature review and historical material and camber data provided by the MS Concrete Girder Manufacturers will be included.

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A-2

November 11, 2018

MDOT
Best Practices for Estimating Camber of Bulb T and Florida Girders
State Study No. 288

Technical Advisory Committee (TAC) meeting

November 13, 2018

Jackson, MS

David Tomley, P.E. (Assistant Project Manager
& Senior Structural Engineer)
Thompson Engineering
Mobile, AL

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Technical Advisory Committee (TAC) meeting agenda

Provide an update on the status of the research project

- Project progress schedule

Previous Tasks completed (recap)

- Literature Search
- Review MDOT Documentation and MS Concrete Girder Manufacturer documentation (design vs. actual material information)/Historic Material Data

Tasks completed

- Review MDOT Current Practices
- Survey other State DOT Current Practices
- Review AASHTO Bridge Design Specs and PCI Bridge Design Manual
- Research data sets for camber calculations
- Effect of Increased stiffness on Live Load Distribution Factor
- Compare other State DOT Current Camber Estimating Practices to MDOT

Remaining Tasks

- Interim Report
- Last TAC meeting
- Final Report

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update on the status of the research project

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Project Progress Schedule

			Feb-2018	Mar-2018	Apr-2018	May-2018	Jun-2018	Jul-2018	Aug-2018	Sep-2018	Oct-2018	Nov-2018	Dec-2018	Jan-2019	Feb-2019	Mar-2019	
Research			Months from Notice to Proceed (NTP) Date														Estimated %
Task	Sub-Task	Work	1	2	3	4	5	6	7	8	9	10	11	12	13	14	Completion
C1	Kick-Off meeting	2%	100														100
M1	MDOT attend																
C2	a) TAC meetings	3%						100									33
M2	MDOT attend																
	b) QPRs	2%		100			100										50
	c) APR	1%															0
	d) Supporting Documents for Invoices	7%	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14	7.14						64
C3	Literature Search	8%		87	13												100
C4	Review MDOT Current Practices	10%		27	5	32	7	0	4	9	16						100
M3	MDOT input																
C5	Survey other State DOT Current Practices	5%					83	0	17								100
C6	Review MDOT Documentation (design versus actual concrete strengths)	10%				18	82										100
M4	MDOT input																
C7	Review Mississippi concrete girder manufacturer Documentation (design versus actual concrete strengths)	8%			50	50											100
C8	Review current AASHTO Bridge Design Specifications and other Reference Publication Guidelines	8%							20	65	15						100
C9	Develop Research Data Sets for Camber Calculations	12%							8	30	62						100
C10	Interim Report	12%									10						10
M5	MDOT Review																
C11	Final Report	12%															0
M6	MDOT Review																
Total Work		100%															
Monthly Overall Progress, %			3	12	6	10	16	4	4	10	12	0	0	0	0	0	
Planned Overall Progress, %			3	26	45	54	65	68	73	75	80	83	89	91	97	100	

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Previous Tasks Completed (recap)

Literature Search

26 documents were reviewed to gain insight into the various aspects associated with the research topic and draw from the previous knowledge-base of information related to estimating camber

- Refer to copies of the literature review documents and summary of items related to the research

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Literature Search
Topics related to the Research

A	strength gain > 28 days
B	material properties
C	camber prediction methods
D	camber variability
E	section properties
F	instrumentation & monitoring
G	high strength concrete using local materials (LADOTD)
H	temerature effects on camber
I	design procedures
J	measured camber
K	prestress losses
L	experimental program
M	AASHTO specifications
N	Sensitivity Study (TXDOT)
O	probabalistic comparison/effect of variability on prestress losses and camber & deflections
P	test data
Q	transportation weight limits
R	factors that influence span capabilities (prestress losses, allowable tension, local producer member capabilities f'c)
S	camber tolerances
T	debonded strands
U	anchor zone reinforcing
V	QC records (WSDOT)
W	humidity
X	historical material data
Y	support conditions
Z	modification factors for camber estimates
AA	camber experiences by other State DOT's
BB	when to measure initial camber
CC	scheduling pours
DD	recommendations for practice
EE	curing
FF	strand development and transfer lengths

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Review MDOT Documentation and MS Concrete Girder Manufacturer documentation (design vs. actual material information)

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Historic Material Data

- Producer 1
  - provided three concrete test reports all from the same project with a required design release concrete strength of 5,600 psi and required design 28-day concrete strength of 6,500 psi
  - No camber data was provided
- Producer 2
  - provided concrete pour reports including cambers at release and at 28-days from five separate projects, projects included AASHTO Type 4 girders with lengths of 100 and 110 ft
  - required design concrete strength at release varied from 4200 to 5000 psi
  - required design concrete strength at 28-days varied from 5000 to 6000 psi
- Producer 3
  - provided fourteen concrete test reports all from the same project with a required design release concrete strength of 6,800 psi and required design 28-day concrete strength of 8,500 psi
  - camber data at release was provided

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Historic Material Data
Producer 1

<u>Observations:</u>

1. average release break strength (5622 psi) exceeded the design release concrete strength (5600 psi) by 0.004%
2. average 28-day break strength (9501 psi) exceeded the design 28-day concrete strength (6500 psi) by 46%
3. ratio of average 1-day break strength (5622 psi) to average 28-day break strength (9501 psi) was 0.59

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Historic Material Data Producer 2

Observations:

1. average release break strength (7503 psi) exceeded the average design release concrete strength (4480 psi) by 67%
2. average 28-day break strength (10644 psi) exceeded the average design 28-day concrete strength (5625 psi) by 89%
3. ratio of average 1-day break strength (7503 psi) to average 28-day break strength (10644 psi) was 0.71
4. average measured camber at release (0.74") was 21% less than the average design camber (0.94") and was within both MDOT's camber limit requirements and PCI tolerances
5. average measured camber at 28-days (2.37") had a ratio of 3.2 greater than the average measured camber at release (0.74")

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Historic Material Data Producer 3

Observations:

1. average 1-day release break strength (7725 psi) exceeded the design release concrete strength (6800 psi) by 14%
2. average 2-day release break strength (8984 psi) exceeded the design release concrete strength (6800 psi) by 32%
3. average 28-day break strength (10441 psi) exceeded the design 28-day concrete strength (8500 psi) by 23%
4. ratio of average 1-day break strength (7725 psi) to average 28-day break strength (10441 psi) was 0.74
5. camber data at release for 15 projects provides for additional insight into the amount of variation within a specific girder type and project for further statistical analysis

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Tasks Completed

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Review MDOT Current Practices

- MDOT Bridge Design Manual including bridge design memorandum, Bulb-T design procedure, and Prestressed Beam Camber Deflection spreadsheet
- MDOT Standard Specifications for Road and Bridge Construction including SOPs for inspection of prestressed concrete bridge members
- Sample plans for Bulb T bridge projects in Leake and Marshall County

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Survey other State DOT Current Practices

FL	Ref. FDOT Structures Design Guidelines, Structures Detailing Manual, and Index 20010 Series Prestressed Florida I-beams (Rev. 01/16)
NE	Ref. NDOR Bridge Division-Bridge Office Policies and Procedures, Section 3.3.1-General Prestressed Girder Policy
TX	Ref. TXDOT Bridge Design Manual-LRFD, Section 4-Pretensioned Concrete I Girders
WA	Ref. WSDOT Bridge Design Manual, Standard Specifications, and Design Memorandums
AL	Ref. ALDOT Structural Design Manual-Prestressed Concrete Girder Design Policy, and Bridge Plan Detailing Manual
LA	Ref. LADOTD Bridge Design and Evaluation Manual

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Review AASHTO Bridge Design Specifications

Loss of Prestress covered under AASHTO LRFD Section 5.9.5

5.9.5.1-Total Loss of Prestress (f'c up to 15.0 ksi)

Eq. 5.9.5.1-1 pretensioned members immediately before transfer

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT}$$

Δf_{pES} = sum of all losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads (ksi)

Δf_{pLT} = losses due to long-term shrinkage and creep of concrete, and relaxation of the steel (ksi)

Loss due to elastic shortening Eq. 5.9.5.2.3a-1

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp}$$

f_{cgp} = the concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi).

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

Loss due to elastic shortening Alternative Eq. C5.9.5.2.3a-1

$$\Delta f_{pES} = \frac{A_{ps} f_{pbt} (I_g + e_m^2 A_g) - e_m M_g A_g}{A_{ps} (I_g + e_m^2 A_g) + \frac{A_g I_g E_{ci}}{E_p}}$$

A_{ps} = area of prestressing steel (in.^{2})

A_g = gross area of section (in.^{2})

E_{ci} = modulus of elasticity of concrete at transfer (ksi)

E_p = modulus of elasticity of prestressing tendons (ksi)

e_m = average prestressing steel eccentricity at midspan (in.)

f_{pbt} = stress in prestressing steel immediately prior to transfer (ksi)

I_g = moment of inertia of the gross concrete section (in.^{4})

M_g = midspan moment due to member self-weight (kip-in.)

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Review AASHTO Bridge Design Specifications

- C5.9.5.2.3a

In calculating f_{cgp}, using gross (or net) cross-section properties, it may be necessary to perform a separate calculation for each different elastic deformation to be included. For the combined effects of initial prestress and member weight, an initial estimate of prestress after transfer is used. The prestress may be assumed to be 90 percent of the initial prestress before transfer and the analysis iterated until acceptable accuracy is achieved. To avoid iteration altogether, Eq. C5.9.5.2.3a-1 may be used for the initial section. If the inclusion of an elastic gain due to the application of the deck weight is desired, the change in prestress force can be directly calculated. The same is true for all other elastic gains with appropriate consideration for composite sections.

When calculating concrete stresses using transformed section properties, the effects of losses and gains due to elastic deformations are implicitly accounted for and Δf_{pES} should not be included in the prestressing force applied to the transformed section at transfer. Nevertheless, the effective prestress in the strands can be determined by subtracting losses (elastic and time-dependent) from the jacking stress. In other words, when using transformed section properties, the prestressing strand and the concrete are treated together as a composite section in which both the concrete and the prestressing strand are equally strained in compression by a prestressing force conceived as a fictitious external load applied at the level of the strands. To determine the effective stress in the prestressing strands (neglecting time-dependent losses for simplicity) the sum of the Δf_{pES} values considered must be included. In contrast, analysis with gross (or net) section properties involves using the effective stress in the strands at any given stage of loading to determine the prestress force and resulting concrete stresses.

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Review AASHTO Bridge Design Specifications

5.9.5.3-Approximate Estimate of Time-Dependent Losses

For standard precast, pretensioned members subject to normal loading and environmental conditions, where:

- members are made from normal-weight concrete,
- the concrete is either steam- or moist-cured,
- prestressing is by bars or strands with normal and low relaxation properties, and
- average exposure conditions and temperatures characterize the site,

the long-term prestress loss, Δf_{pLT}, due to creep of concrete, shrinkage of concrete, and relaxation of steel shall be estimated using the following formula:

Eq. 5.9.5.3-1

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR}$$

$$\gamma_h = 1.7 - 0.01H \qquad (5.9.5.3\text{-}2)$$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})} \qquad (5.9.5.3\text{-}3)$$

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)

H = the average annual ambient relative humidity (%)

γ_h = correction factor for relative humidity of the ambient air

γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member

Δf_{pR} = an estimate of relaxation loss taken as 2.4 ksi for low relaxation strand, ~~10.0 ksi for stress relieved strand,~~ and in accordance with manufacturers recommendation for other types of strand (ksi)

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Review AASHTO Bridge Design Specifications

5.9.4.4-Refined Estimates of Time-Dependent Losses

Not applicable to research.

Refer to commentary section C5.9.5.4.1 for additional information on applicability of Section 5.9.4.4.

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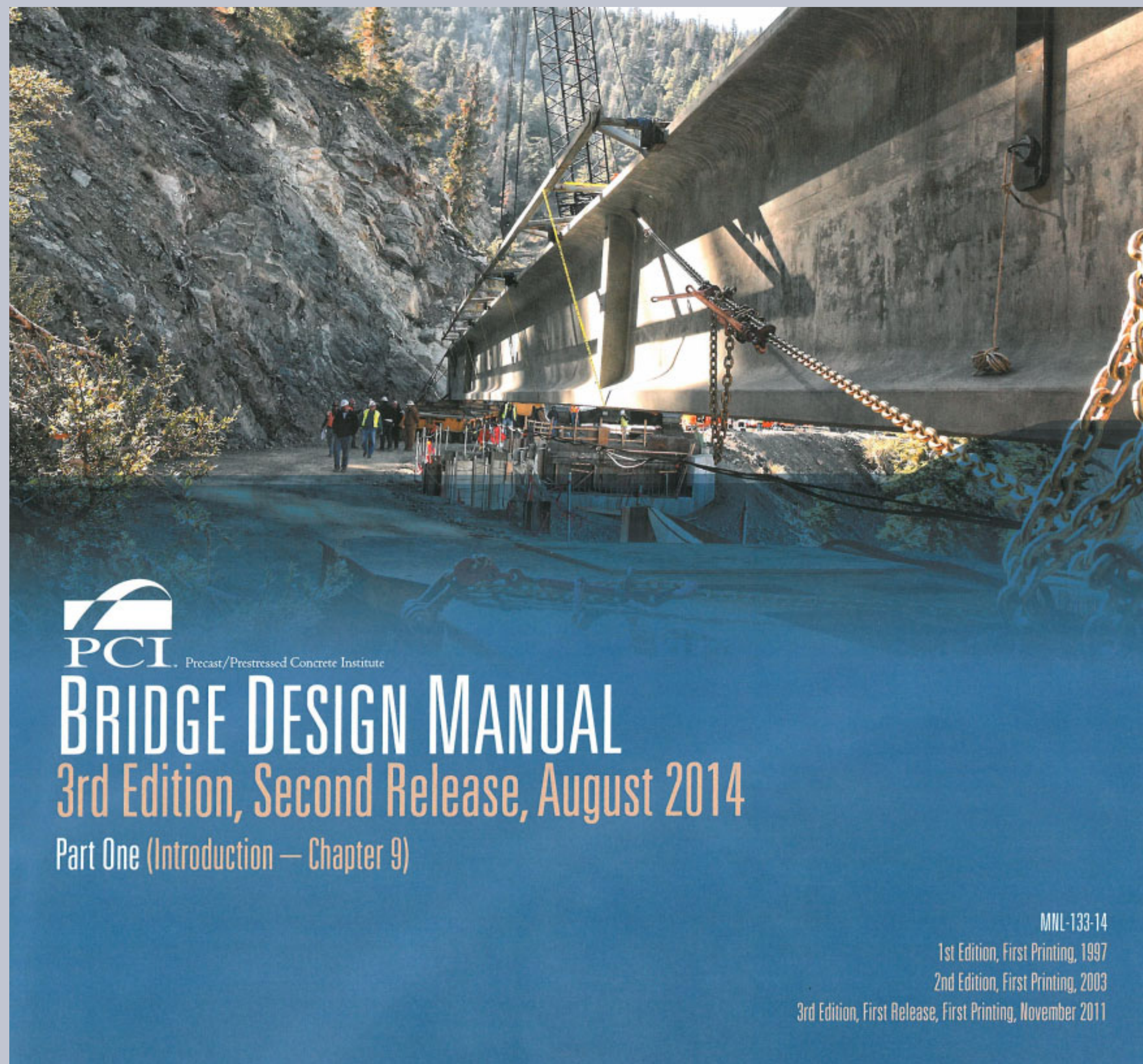
PCI Bridge Design Manual



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2.4.7 Density

2.4.7.1 Normal Weight Concrete

The density of plain normal weight concrete is generally in the range of 0.140 to 0.150 kip/ft^3. The density varies depending on the amount and density of the aggregate and the air, water, and cement contents. The *LRFD Specifications* provides the following densities for plain concrete:

Table 2.4.7.1-
Plain Concrete Densities [LRFD Table 3.5.1-1]

Concrete	Density, kip/ft^3
Lightweight	0.110
Sand-Lightweight	0.120
Normal Weight with $f'_c < 5.0$ ksi	0.145
Normal Weight with 5.0 ksi $\leq f'_c \leq$ 15.0 ksi	$0.140 + 0.001\, f'_c$

where f'_c = specified concrete compressive strength

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PCI Bridge Design Manual

8.7 CAMBER AND DEFLECTION

Generally, there are three sets of beam deformations of interest to the designer:

- vertical deflections (typically at midspan)
- end rotations
- axial shortening

Of these, midspan deflection, or camber, is usually of greatest interest. Unexpected camber at the time of erection may require adjustment of bridge grades to prevent intrusion of the beam top flange into the deck. Additionally, estimates of the final midspan deflections under the action of permanent dead load and live load may be required to ensure serviceability of the bridge.

End rotations are of importance when continuity is introduced at the time of casting the deck. When these rotations are restrained or partially restrained by adjacent spans, secondary time-dependent stresses are introduced in the structure. These stresses must be considered in the design of connections and detailing of the end regions of beams.

Finally, axial shortening of precast, prestressed bridge members must be considered when designing bearings and expansion devices. This information is also helpful in assessing the impact of superstructure restraint against shortening in jointless bridge systems.

This section discusses the computations of camber and deflection including the changes that occur in these quantities with time. The methods that are available to estimate long-term cambers and other deflections of precast, prestressed members fall into three categories, listed in order of increasing complexity and accuracy:

- multiplier methods
- improved multiplier methods, based on estimates of loss of prestress
- detailed analytical methods

Camber in a prestressed beam occurs immediately upon the transfer of the prestressing force. The magnitude of the initial camber is dependent on the length, weight, and moment of inertia of the member; the modulus of elasticity of the concrete; and the arrangement and amount of prestressing. Values for several prestressing arrangements are given in **Table 8.7-1**. The modulus of elasticity of the concrete usually cannot be predicted with precision at the time of the design of the member. The standard prediction formulas are based on values assumed by the designer for concrete unit weight and strength at the time of prestress transfer. These assumed values do not include actual material properties, nor account for such important factors as type of aggregates and ratio of coarse-to-fine aggregate. For these reasons, initial camber predictions using assumed material properties must be regarded as estimates and the designer is cautioned against placing a high degree of confidence in calculated initial cambers (Tadros et al., 2011).

After transfer, camber generally increases with time. Creep of the concrete is primarily responsible for this camber growth. Simultaneously, the gradual loss of prestress due to creep, shrinkage, and strand relaxation has the effect of reducing the initial rate of growth of camber. The magnitude and rates of both creep and shrinkage, and therefore changes in camber, are affected by environmental conditions such as ambient relative humidity and temperature.

From the preceding discussion, it should be obvious that the task of predicting both initial camber and the growth of camber with time is difficult because the large number of random variables that affect this behavior are beyond the designer's control. Estimates of these effects should be recognized as being approximations only.

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8.7.1 Multiplier Method

Perhaps the most used method for predicting time-dependent camber of precast, prestressed members is the set of multipliers given in **Table 8.7.1-1** (Martin, 1977). This method is fairly straightforward. First, elastic deflections caused by the effects of prestressing, beam self-weight, and other dead loads are calculated using conventional elastic analysis techniques. These are multiplied by the appropriate factors selected from **Table 8.7.1-1** to determine the deflections that occur as a result of time-dependent behavior.

Table 8.7.1-1
Suggested Multipliers to be Used as a Guide in
Estimating Long-Term Cambers and Deflections for Typical Members

		Without Composite Topping	With Composite Topping
	At erection:		
(1)	Deflection (↓) component – apply to the elastic deflection due to the member weight at transfer of prestress	1.85	1.85
(2)	Camber (↑) component – apply to the elastic camber due to prestress at the time of transfer of prestress	1.80	1.80
	Final:		
(3)	Deflection (↓) component – apply to the elastic deflection due to the member weight at transfer of prestress	2.70	2.40
(4)	Camber (↑) component – apply to the elastic camber due to prestress at the time of transfer of prestress	2.45	2.20
(5)	Deflection (↓) component – apply to elastic deflection due to superimposed dead load only	3.00	3.00
(6)	Deflection (↓) component – apply to elastic deflection caused by the composite topping	---	2.30

This method gives reasonable estimates for cambers at the time of erection. The method does not, however, properly account for the significant effects of a large cast-in-place deck. The presence of a deck, once cured, drastically changes the stiffness of a typical bridge member. This has the effect of restraining the beam creep strains that are the result of prestressing, member self weight, and the dead load of the deck itself. Also, differential creep and shrinkage between the precast beam and the cast-in-place concrete can produce changes in member deformation. The multipliers for long-term deflection suggested by this method, therefore, should not be used for bridge beams with structurally composite cast-in-place decks.

In addition, it is not recommended that prestressing levels be increased in order to reduce or eliminate long-term downward deflection that might be predicted if the multipliers in **Table 8.7.1-1** are used.

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PCI Bridge Design Manual

9.0.1 Service Life

Design calculations for prestress losses are based on a final age of 20,000 days or 54.8 years to be consistent with previous editions of the manual. These losses, however, are applicable to longer service lives such as 75 or 100 years because the time development factor only changes by 0.2% after 20,000 days.

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Research Data Sets for Camber Calculations

- This task included developing research data sets for further calculations and evaluation of estimating camber.
- Insights gained from the literature review and historical material and camber data provided by the MS Concrete Girder Manufacturers were included.

- A data set outline was developed to provide a basis for the camber calculations.
- Known Information:
  1. Actual concrete compressive strengths at release and at 28-days are greater than design concrete compressive strengths
  2. Several MDOT projects have experienced under-camber on girders at erection
- Plan
  - Evaluate the effects of various items that influence camber
  - Capture the range of variation in camber for various beam types and span lengths

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Research Data Sets for Camber Calculations

Items that influence camber:

- Material Properties
  - Release concrete compressive strength (f'ci)
  - 28-day concrete compressive strength (f'c)
  - Release modulus of elasticity (Eci)
  - 28-day modulus of elasticity (Ec)
  - Unit weight (wc)
  - Aggregate Type
- Girder Section Properties
  - Transformed vs. Gross section properties
  - Girder Type/Moment of Inertia
  - Span Lengths
- Prestress losses and data
  - Age of girder at erection
  - Age of deck placement
  - Beam curing time (1, 2, or 3 days)
  - Beam storage age (3, 6 or 12 months)
  - Time-dependent analysis
  - Humidity/seasonal variation
- Strand Patterns and Profile
  - Draped strand
  - Straight strand (including debonding)
  - Top Strand
  - Number and size of strand
  - Strand templates
- Haunch/fillet thickness
- Dead load distribution
- PCI multipliers used in the PCI Multiplier Method for estimating camber
- Temperature gradients
- Roadway Vertical curve ordinate (indirectly related)

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Research Data Sets for Camber Calculations

Historic Material Information Provided by MS Concrete Girder Manufacturers:

	Producer 1	Producer 2	Producer 3
design f'ci	5600 psi	4480 psi	6800 psi
Average actual f'ci	5622 psi	7503 psi	7725 psi
Ratio of average actual f'ci/design f'ci	1.004	1.67	1.14
Design f'c	6500 psi	5625 psi	8500 psi
Average actual f'c	9501 psi	10644 psi	10441 psi
Ratio of average actual f'c/design f'c	1.46	1.89	1.23
Ratio of Average actual f'ci/average actual f'c	0.59	0.71	0.74

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Research Data Sets for Camber Calculations

Observations with respect to f'ci and f'c:

- Use the low, high, and average ratios to vary f'ci and f'c to evaluate the effects on camber estimates.
  - Values for f'ci include; low = 1.004, average = 1.27, high = 1.66
  - Values for f'c include; low = 1.23, average = 1.53, high = 1.89
- Producer 3 provided 2-day break data (not shown in the above table) that can be considered in utilizing higher design release strengths. For example, the average 2-day actual concrete compressive strength was 8984 psi which is a ratio increase from the 6800 psi design f'ci of 1.32.
- The relationship between f'ci/f'c can be used to understand the strength gain during design. The above values based on historic material information can assist in establishing guidelines. For example the lowest ratio was 0.59, the highest was 0.74, and the average ratio was 0.68.

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Research Data Sets for Camber Calculations

Historic Camber Data Provided by MS Concrete Girder Manufacturers:

	Producer 1	Producer 2	Producer 3
Estimated camber (at release)	Data not provided	0.94 in.	Data not provided
Average measured camber (at release)	no camber data provided	0.74 in.	Provided data on 15 projects with difference beam types
Average measured camber (28-days)	no camber data provided	2.37 in.	no camber data provided
Ratio of Average measured camber (28-days)/average measured camber (at release)	no camber data provided	3.2	no camber data provided

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Research Data Sets for Camber Calculations

Observations with respect to camber:

- For Producer 2; use both the average actual f'ci and design f'ci to estimate camber at release for similar beam type and span length and compare the estimated camber differences at release to see if there is a correlation between the estimated camber and measured camber at release.
  - Compare 28-day camber data to estimated camber data sets
- For Producer 3; the camber data shows variation in the measured camber; therefore calculate the range of variation (i.e., low and high values from the average measured camber).
  - Look for consistencies between various beam types on the spread/magnitude the variations in the measured cambers.
  - Vary f'ci and Eci using average actual f'ci values and compare effects with measured camber to see if there is a correlation to the relationship between 1) the design f'ci and estimated camber and 2) the actual f'ci and measured camber.
- Compare results with estimated camber using PCI multiplier method (at release only).

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Research Data Sets for Camber Calculations

Unit Weight (wc):

- Vary the self weight of the girders by using different units weights (e.g., 150 pcf, 155 pcf, and 160 pcf) to evaluate the effect of unit weight on estimated camber.
- Sample calculation for the girder self weight (for the BT-54 girder in Marshall Co.) has a fabricated girder unit weight of 155 pcf accounting for the additional weight of the strand and reinforcing.
  - Recommend MS concrete girder manufactures collect all girder shipping self weights to be able to have a historical database of unit weights produced. Girder weights are required for shipping/hauling permits also and the girder weights may already be documented by the MS concrete girder manufactures.
- Recommend further research be performed to obtain unit weights and modulus of elasticity information from the MS concrete girder manufactures.
  - Also note the type of aggregate used in the mix-design to further understand whether the AASHTO LRFD formula to compute the modulus of elasticity for concrete can be adjusted based on the type of aggregate and historic information. Since historical information does not exist related to the types of aggregates used to manufacture the girders, the research will not utilize the adjustment factor in the AASHTO LRFD Bridge Design Specifications but the effect of adjusting the aggregate factor (K1) will be included.
  - The modulus of elasticity formula in the AASHTO LRFD Bridge Design Specifications will be used for both the design values of f’ci and f’c and the average actual values of f’ci and f’c to evaluate the effects on estimating camber.

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Research Data Sets for Camber Calculations

Aggregate Type:

AASHTO LRFD Section 5.4.2.4-Modulus of Elasticity

In the absence of measured data, the modulus of elasticity, E_c, for concretes with unit weights between 0.090 and 0.155 kcf and for normal weight concrete with specified compressive strengths up to 15.0 ksi may be taken as:

$$E_c = 120{,}000\, K_1\, w_c^{2.0}\, f_c'^{\,0.33}$$

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the authority of jurisdiction

w_c = unit weight of concrete (kcf); refer to Table 3.5.1-1 or Article C5.4.2.4

f'_{ci} = specified compressive strength of concrete (ksi)

Commentary:

Test data show that the modulus of elasticity of concrete is influenced by the stiffness of the aggregate. The factor K_1 is included to allow the calculated modulus to be adjusted for different types of aggregate and local materials. Unless a value has been determined by physical tests, K_1 should be taken as 1.0. Use of a measured K_1 factor permits a more accurate prediction of modulus of elasticity and other values that utilize it.

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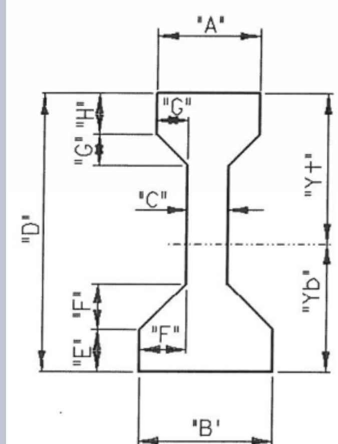
Research Data Sets for Camber Calculations

Girder Section Properties

- Transformed vs. Gross section properties
- Girder Type/Moment of Inertia
  - AASHTO Type 4
  - BT-54 Marshall Co.
  - BT-72 Leake Co.
  - FIB-72
- Baseline span lengths
  - 90'-AASHTO Type 4
  - 110'-BT-54 Marshall Co.
  - 138'-BT-72 Leake Co.
  - 155'-FIB-72



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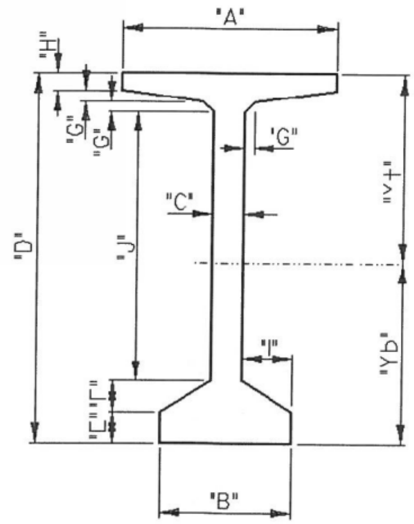

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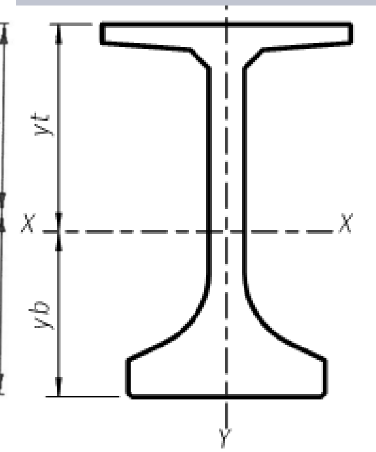

FIB-72
(INDEX 20072)

Type	"A"	"B"	"C"	"D"	"E"	"F"	"G"	"H"	"I"	"J"
IV	1'-8"	2'-2"	8"	4'-6"	8"	9"	6"	8"		
BT-54	3'-6"	2'-2"	6"	4'-6"	6"	4 1/2"	2"	3 1/2"	10"	3'-0"
BT-72	3'-6"	2'-2"	6"	6'-0"	6"	4 1/2"	2"	3 1/2"	10"	4'-6"
FIB-72	4'-0"	3'-2"	7"	6'-0"	7"	7 1/2"		3 1/2"	1'-3 1/2"	4'-1"

Type	Area ($in.^2$)	Wt. (#/ft.)	I ($in.^4$)	Yb (in.)	Yt (in.)	St ($in.^3$)	Sb ($in.^3$)
IV	789	822	260,730	24.73	29.27	8,908	10,543
BT-54	659	686	268,045	27.63	26.37	10,165	9,701
BT-72	767	799	545,850	36.60	35.40	15,419	14,914

FIB-72 SECTION PROPERTIES	
Area ($in.^2$)	1,058.58
Perimeter (in.)	278.57
Ixx ($in.^4$)	740,416
Iyy ($in.^4$)	82,099
yt (in.)	40.06
yb (in.)	31.94

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Research Data Sets for Camber Calculations

Prestress Losses and Data:

- AASHTO LRFD 5.9.5.3-Approximate Estimate of Time-Dependent Losses
- Time at release
  - Girder fabrication time can vary (1-day, 2-day, 3-days, etc.) depending on the precast manufacture work production schedule
- Age of girder erection
  - Time can vary and is dependent on Construction schedule
- Age of deck placement
  - Time can vary and is dependent on Construction schedule
- Final age
  - Time can vary depending on service life assumptions
- Time dependent analysis
- Relative humidity
  - Humidity can vary depending on location of MS concrete girder manufacturer and time of year
- Curing method (moist vs. steam)
  - Not included in the research but the curing method can vary depending on selection by MS concrete girder manufacturer
- Jacking force
  - Not included in the research but some amount of variation is expected in the method and sequence the MS concrete girder manufacturer's use when jacking and releasing the prestressing strand

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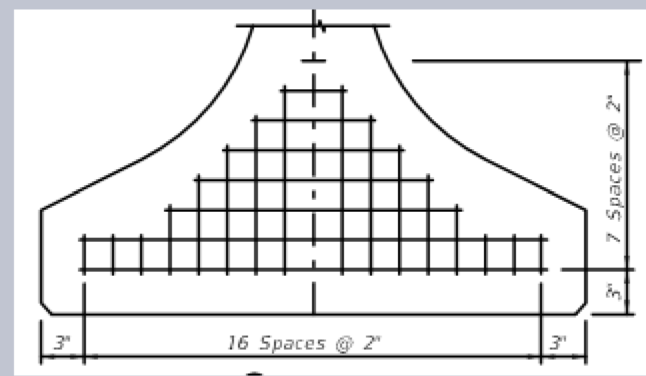
Research Data Sets for Camber Calculations

Strand Patterns and Profile:

- Draped strand
- Straight strand (including debonding)
- Top Strand
  - With or without reduced pull

    Note: Where required to satisfy allowable stresses using straight top strand patterns or used to facilitate girder fabrication
- Number and size of strand

  Note: Influences the prestress force applied to girder section
- Strand templates
  - Recommend including in design standards



strand height (in)	at beam end			at centerline span		
51	0 0	2	102		0	0
49	0 0	2	98		0	0
47	0 0	2	94		0	0
		0	0		0	0
		0	0		0	0
10.5			0		0	0
8.5			0		0	0
6.5	0 0 0 0 0 0	6	39	0 0 0 0 0 0 0 0	8	52
4.5	0 0 0 0 0 0 0 0 0 0	10	45	0 0 0 0 0 0 0 0 0 0 0 0	12	54
2.5	0 0 0 0 0 0 0 0 0 0	10	25	0 0 0 0 0 0 0 0 0 0 0 0	12	30
		32	403		32	136
	strand centroid =	**12.59**	in.	strand centroid =	**4.25**	in.

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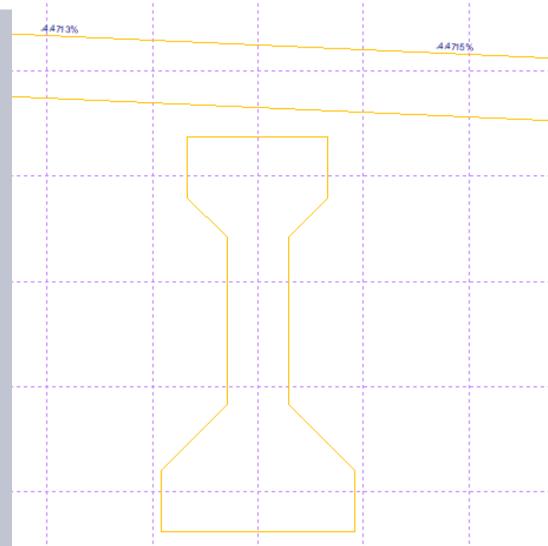
Note: A spreadsheet was developed that calculates the center of gravity of the prestress strand depending on the number of strands and strand location relative to the bottom of the girder.

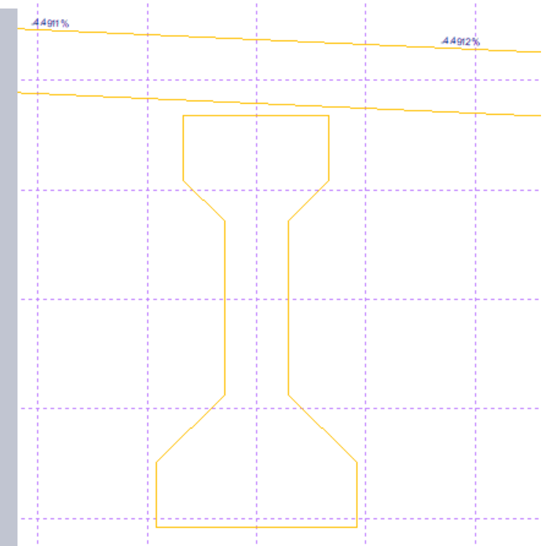
Research Data Sets for Camber Calculations

Haunch/fillet thickness:

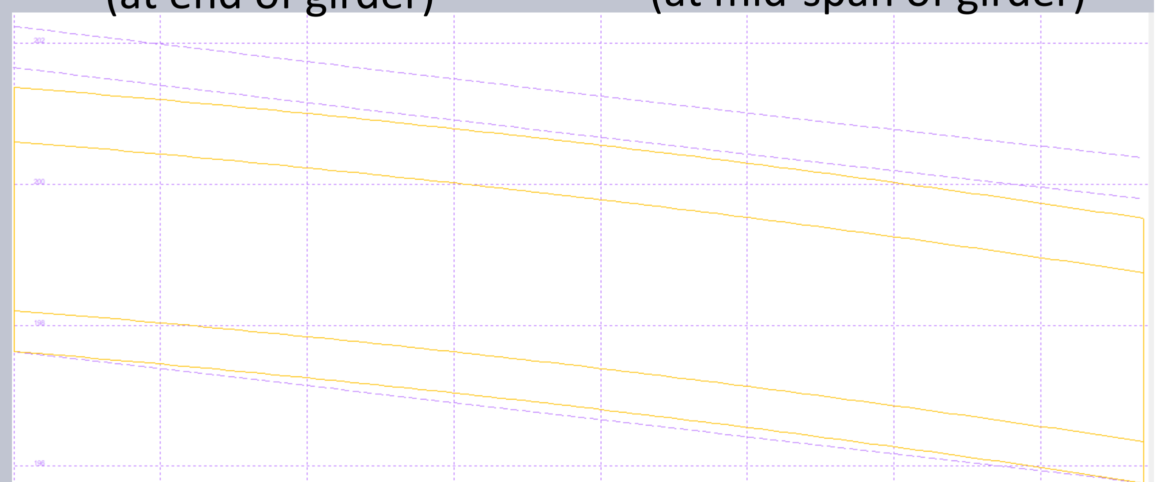
- Consists of the additional concrete that is placed on top of precast concrete girder and between the bottom of the deck slab and the top of the precast concrete girder.
- The thickness of the haunch/fillet is dependent on the beam camber at erection (after all dead load deflection), roadway vertical curve ordinate, deck cross-slope/superelevation, and the as-constructed beam seat elevations.
- A minimum thickness at the edge of flange is recommended to facilitate forming deck
- Thickness varies along the length of the beam
- Can be included in the composite section or not, and if not added as additional dead load on the girder



Girder Section
(at end of girder)



Girder Section
(at mid-span of girder)



Girder Elevation

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Research Data Sets for Camber Calculations

Dead Load Distribution:

- AASHTO LRFD 4.6.2.2-Beam-Slab Bridges

Where bridges meet the conditions specified herein, permanent loads of and on the deck may be distributed uniformly among the beams and/or stringers.

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Research Data Sets for Camber Calculations

Multiplier Method for estimating camber:

- Refer to PCI Bridge Design Manual 8.7.1
  - Deflection (down) multiplier at erection = 1.85
  - Camber (up) multiplier at erection = 1.80
- Apply the deflection and camber multipliers to the deflection due to girder self weight and camber due to prestressing respectively
- Example calculation for camber at erection (prior to added dead load) at mid-span
  - Prestress camber at release = 3.0 inches
  - Girder self weight deflection at release = 1.0 inches
  - Prestress camber at erection = 1.80 x (3.0 inches) = 5.4 inches
  - Girder self weight deflection at erection = 1.85 x (1.0 inches) = 1.85 inches
  - Camber at erection (prior to added dead load) = 5.4 – 1.85 = <u>3.55 inches</u>
  - Ratio of camber at erection (prior to added dead load) to camber at release = 3.55/2.0 = 1.78

    Note: Deflection due to added dead load after erection gets subtracted to the 3.55 inches for camber (at mid-span)

    Note: Camber at mid-span along with minimum haunch thickness, roadway vertical curve ordinate, and deck cross-slope/superelevation are included to calculate the haunch/fillet thickness at the end of the beam used to set beam seat elevations

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Research Data Sets for Camber Calculations

Temperature Gradient:

- AASHTO LRFD 3.12.3-Temperature Gradient

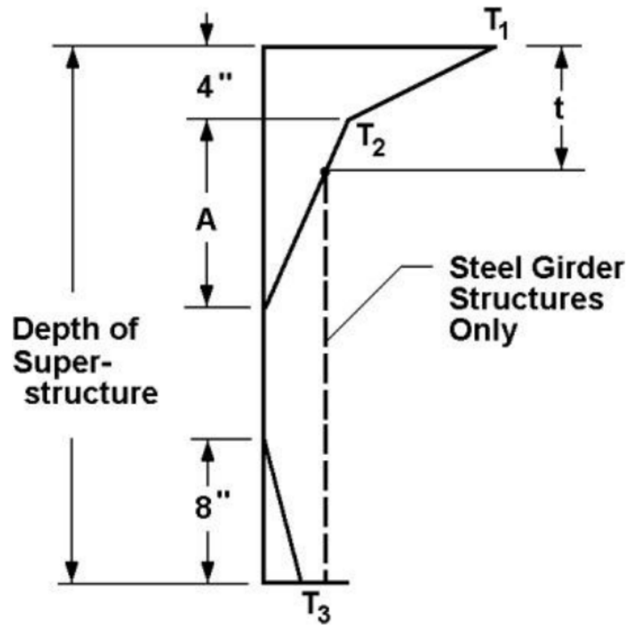

Figure 3.12.3-2—Positive Vertical Temperature Gradient in Concrete and Steel Superstructures

Table 3.12.3-1—Basis for Temperature Gradients

Zone	T_1 (°F)	T_2 (°F)
1	54	14
2	46	12
3	41	11
4	38	9

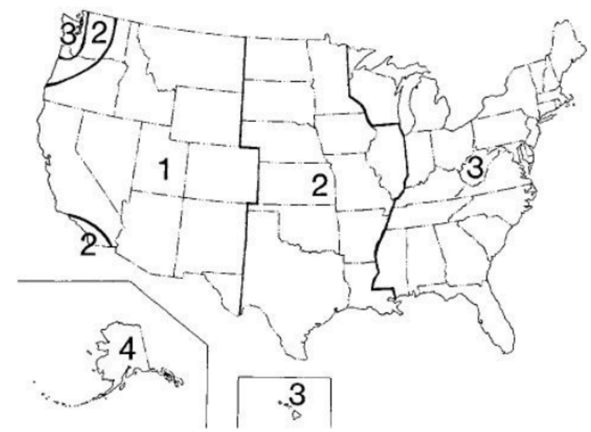

Figure 3.12.3-1—Solar Radiation Zones for the United States

C3.12.3

Temperature gradient is included in various load combinations in Table 3.4.1-1. This does not mean that it need be investigated for all types of structures. If experience has shown that neglecting temperature gradient in the design of a given type of structure has not lead to structural distress, the Owner may choose to exclude temperature gradient. Multibeam bridges are an example of a type of structure for which judgment and past experience should be considered.

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Research Data Sets for Camber Calculations

Roadway Vertical Curve Ordinate:

- Depending on whether the bridge is located within a crest or sag vertical curve, an adjustment to the haunch/fillet thickness is recommended to account for elevation differences between a non-linear profile grade and the linear grade connecting the centerline of girder supports at the end of the girders

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Research Data Sets for Camber Calculations

Using the various items that influence camber, develop baseline and subsequent camber estimates for the following girder types, bridge typical sections, and span lengths:

- AASHTO Type 4 [90 ft.]
- BT-54 (Marshall Co.) [110 ft.]
- BT-72 (Leake Co.) [138 ft.]
- FIB-72 [155 ft.]

A time dependent analysis was run for the FIB-72 girder to further evaluate the influence of:

- Humidity
- Girder fabrication time (1, 2, and 3 days)
- Average historical f'ci and f'c
- Extended beam storage time (6 and 12 months)

All baseline runs use the following input data:

- Deck thickness = 7.75 inches
- Two Barriers weight = 0.600 klf (equally distributed over all girders)
- 0.6" low-relaxation strands
- f'c deck = 4.0 ksi
- Girder unit weight = 0.150 kcf
- Transformed section properties
- Haunch/fillet thickness added as dead load to girder (not included in composite section)
- Approximate prestress losses
- PCI Multiplier Method for camber & deflection calculations at erection

The following LEAP Bridge Concrete software programs (Version 18.00.00.34) by Bentley Systems were used for the camber calculations:

- Precast/Prestressed Girder was utilized for the camber calculations and Spliced-Girder was utilized for the time-dependent analysis.

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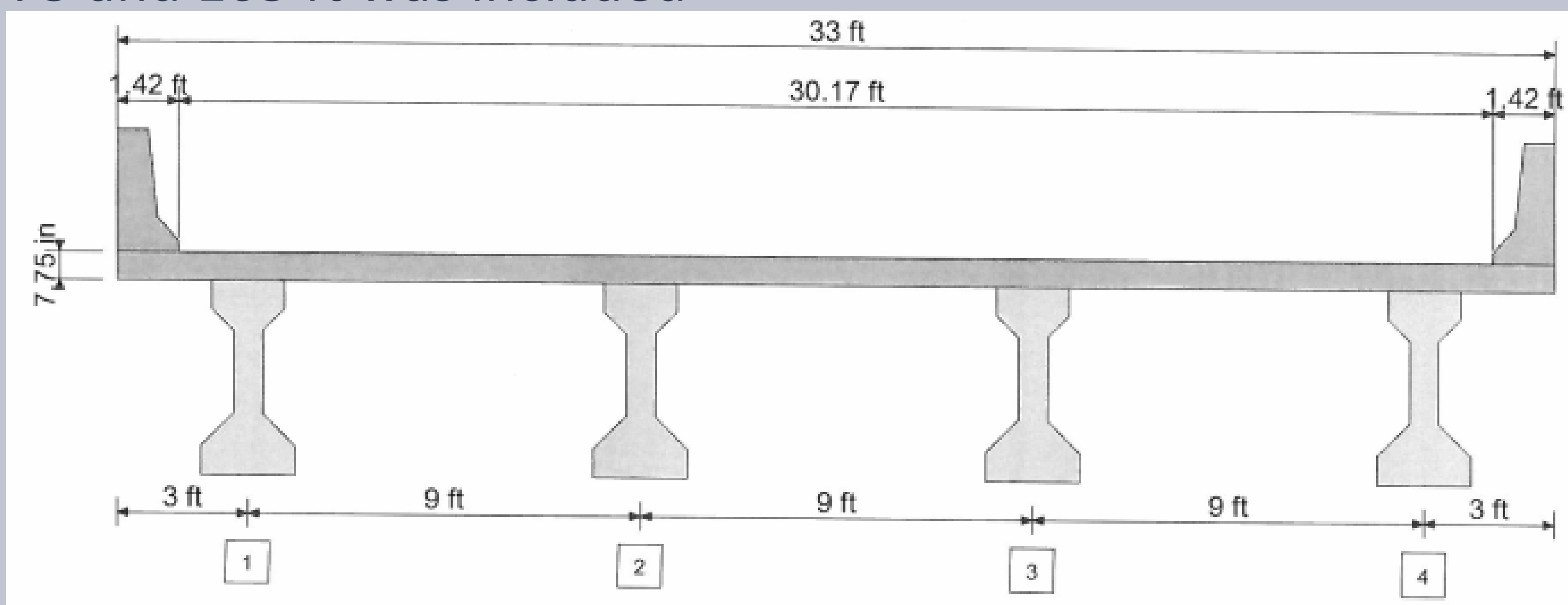
Research Data Sets for Camber Calculations

- AASHTO Type 4
- f'ci = 5.5 ksi, f'c = 8.0 ksi
- 28-0.6" diameter draped strands
- Baseline span length = 90 ft, a lower and upper span length of 75 and 105 ft was included



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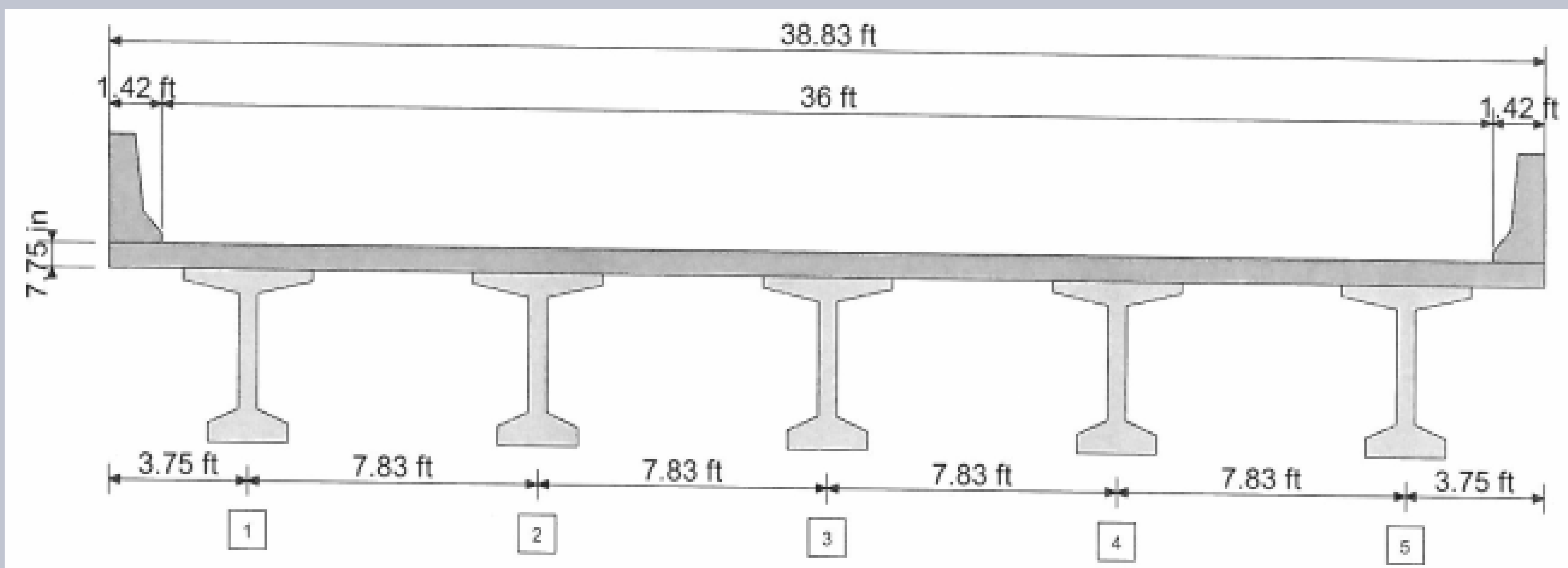
Research Data Sets for Camber Calculations

- BT-54 (Marshall Co.)
- f'ci = 6.3 ksi, f'c = 7.7 ksi
- 32-0.6" diameter draped strands
- Baseline span length = 110 ft, a lower and average span length of 80 and 95 ft was included



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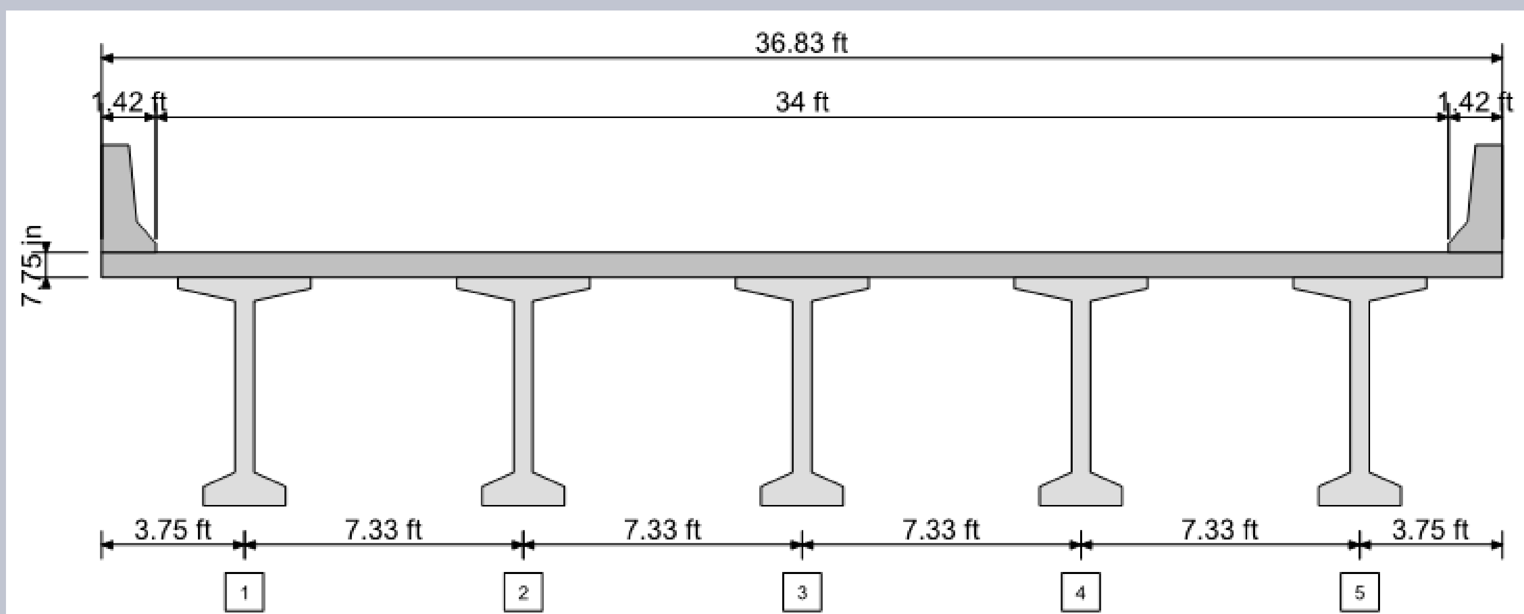
Research Data Sets for Camber Calculations

- BT-72 (Leake Co.)
- f’ci = 6.0 ksi, f’c = 7.5 ksi
- 36-0.6” diameter draped strands
- Baseline span length = 138 ft, a lower and average span length of 100 and 120 ft was included



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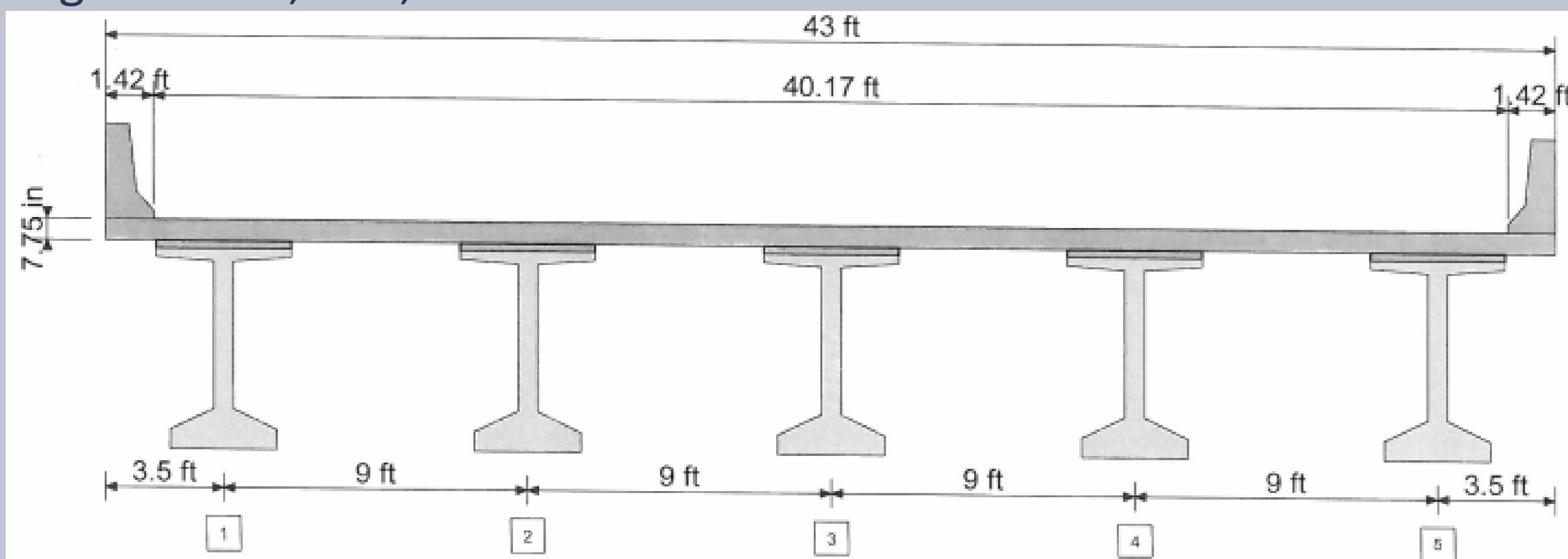
Research Data Sets for Camber Calculations

- FIB-72
- f'ci = 6.6 ksi, f'c = 8.5 ksi
- 59-0.6" diameter straight strands with debonding and 4-top strands with reduced pull
- Baseline span length = 155 ft, a lower, average, and upper span length of 120, 140, and 160 ft was included



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Research Data Sets for Camber Calculations

The next sets of tables include the camber estimates at release and at erection for the various items previously noted that influence camber

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Research Data Sets for Camber Calculations

- AASHTO Type 4

girder camber (inches)		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline run	2.437	-0.935	1.502	4.386	-1.729	2.657	-0.987	1.670
2	limestone aggregate	2.237	-0.854	1.383	4.027	-1.580	2.447	-0.905	1.542
3	avg f'ci/f'c, unit wt 155, gross section	2.159	-0.878	1.281	3.886	-1.624	2.262	-0.849	1.413
4	baseline run, minimum span length 75 ft.	1.257	-0.456	0.801	2.262	-0.844	1.418	-0.482	0.936
5	baseline run, maximum span length 105 ft.	4.195	-1.715	2.480	7.551	-3.172	4.379	-1.813	2.566
6	avg f'ci/f'c, limestone aggregate	2.082	-0.792	1.290	3.747	-1.465	2.282	-0.793	1.489

Note:

Average f’ci = 7.0 ksi

Average f’c = 12.2 ksi

Limestone aggregate used an aggregate factor K1 = 1.10

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Research Data Sets for Camber Calculations

- BT-54
  Marshall Co.

Note:
Average f'ci = 8.0 ksi
Average f'c = 11.8 ksi
High f'ci = 10.46 ksi
High f'c = 14.6 ksi
Limestone aggregate used an aggregate factor K1 = 1.10

girder camber (inches)

#	description	at release prestress	at release self weight	at release camber	at erection prestress	at erection self weight	at erection prestress + self weight	at erection added dead load	at erection camber
1	baseline run	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.931	3.012
2	avg f'ci/f'c	3.998	-1.390	2.608	7.197	-2.571	4.626	-1.691	2.935
3	high f'ci/f'c	3.698	-1.279	2.419	6.657	-2.366	4.291	-1.588	2.703
4	unit wt 155	4.045	-1.454	2.591	7.281	-2.690	4.591	-1.819	2.772
5	unit wt 160	3.826	-1.414	2.412	6.886	-2.616	4.270	-1.717	2.553
6	gross section	4.334	-1.592	2.742	7.801	-2.944	4.857	-2.056	2.801
7	fillet included	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.918	3.025
8	temperature	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.513	3.430
9	PCI mult 1.0	4.283	-1.496	2.787	4.283	-1.496	2.787	-1.931	0.856
10	increase fillet thickness	4.283	-1.496	2.787	7.710	-2.767	4.943	-1.998	2.945
11	top strand	4.205	-1.487	2.718	7.568	-2.752	4.816	-1.921	2.895
12	straight strands with debonding	4.576	-1.496	3.080	8.236	-2.767	5.469	-1.931	3.538
13	2, 4	3.774	-1.350	2.424	6.794	-2.498	4.296	-1.593	2.703
14	2, 5	3.568	-1.313	2.255	6.422	-2.430	3.992	-1.502	2.490
15	2, 7	3.998	-1.390	2.608	7.197	-2.571	4.626	-1.680	2.946
16	2, 4, 6, 11	3.744	-1.423	2.321	6.739	-2.633	4.106	-1.678	2.428
17	2, 4, 6, 10, 11	3.744	-1.423	2.321	6.739	-2.633	4.106	-1.736	2.370
18	baseline run (including prestress losses)	3.863	-1.496	2.367	6.953	-2.767	4.186	-1.931	2.255
19	2, 4 (including prestress losses)	3.404	-1.350	2.054	6.126	-2.498	3.628	-1.593	2.035
20	baseline run, minimum span length 80 ft.	1.490	-0.458	1.032	2.682	-0.847	1.835	-0.592	1.243
21	baseline run, average span length 95 ft.	2.744	-0.898	1.846	4.940	-1.661	3.279	-1.159	2.120
22	limestone aggregate	3.941	-1.368	2.573	7.094	-2.532	4.562	-1.770	2.792
23	avg f'ci/f'c, limestone aggregate	3.676	-1.271	2.405	6.617	-2.351	4.266	-1.550	2.716

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Research Data Sets for Camber Calculations

- BT-72 Leake Co.

Note:
Average f'ci = 7.62 ksi
Average f'c = 11.5 ksi
High f'ci = 9.96 ksi
High f'c = 14.2 ksi
Limestone aggregate used an aggregate factor K1 = 1.10

girder camber (inches)

		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline run	4.968	-2.166	2.802	8.943	-4.007	4.936	-2.256	2.680
2	avg f'ci/f'c	4.639	-2.012	2.627	8.351	-3.723	4.628	-1.977	2.651
3	high f'ci/f'c	4.293	-1.853	2.440	7.727	-3.428	4.299	-1.856	2.443
4	unit wt 155	4.694	-2.106	2.588	8.448	-3.895	4.553	-2.124	2.429
5	unit wt 160	4.440	-2.048	2.392	7.992	-3.789	4.203	-2.005	2.198
6	gross section	5.073	-2.310	2.763	9.132	-4.274	4.858	-2.406	2.452
7	fillet included	4.968	-2.166	2.802	8.943	-4.007	4.936	-2.239	2.697
8	temperature	4.968	-2.166	2.802	8.943	-4.007	4.936	-1.745	3.191
9	PCI mult 1.0	4.968	-2.166	2.802	4.968	-2.166	2.802	-2.256	0.546
10	increase fillet thickness	4.968	-2.166	2.802	8.943	-4.007	4.936	-2.364	2.572
11	top strand	4.885	-2.155	2.730	8.793	-3.986	4.807	-2.244	2.563
12	straight strands with debonding	5.327	-2.156	3.171	9.589	-3.988	5.601	-2.256	3.345
13	2, 4	4.384	-1.958	2.426	7.890	-3.621	4.269	-1.860	2.409
14	2, 5	4.144	-1.904	2.240	7.460	-3.522	3.938	-1.756	2.182
15	2, 7	4.639	-2.012	2.627	8.351	-3.723	4.628	-1.962	2.666
16	2, 4, 6, 11	4.390	-2.066	2.324	7.902	-3.822	4.080	-1.967	2.113
17	2, 4, 6, 10, 11	4.390	-2.066	2.324	7.902	-3.822	4.080	-2.061	2.019
18	baseline run (including prestress losses)	4.505	-2.166	2.339	8.110	-4.007	4.103	-2.256	1.847
19	2, 4 (including prestress losses)	3.976	-1.958	2.018	7.157	-3.621	3.536	-1.860	1.676
20	baseline run, minimum span length 100 ft.	1.585	-0.651	0.934	2.853	-1.204	1.649	-0.677	0.972
21	baseline run, average span length 120 ft.	2.940	-1.335	1.605	5.292	-2.469	2.823	-1.391	1.432
22	limestone aggregate	4.573	-1.982	2.591	8.232	-3.666	4.566	-2.067	2.499
23	avg f'ci/f'c, limestone aggregate	4.270	-1.842	2.428	7.686	-3.408	4.278	-1.811	2.467

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Research Data Sets for Camber Calculations

- FIB-72

Note:
Average f'ci = 8.4 ksi
Average f'c = 13.0 ksi
High f'ci = 11.0 ksi
High f'c = 16.1 ksi
Limestone aggregate used an aggregate factor K1 = 1.10

girder camber (inches)

		at release			at erection				
#	description	prestress	self weight	camber	prestress	self weight	prestress + self weight	added dead load	camber
1	baseline run	6.815	-3.507	3.308	12.266	-6.489	5.777	-3.124	2.653
2	avg f'ci/f'c	6.360	-3.256	3.104	11.447	-6.024	5.423	-2.734	2.689
3	high f'ci/f'c	5.880	-2.994	2.886	10.584	-5.539	5.045	-2.566	2.479
4	unit wt 155	6.434	-3.407	3.027	11.581	-6.303	5.278	-2.940	2.338
5	unit wt 160	6.083	-3.312	2.771	10.950	-6.127	4.823	-2.772	2.051
6	gross section	6.933	-3.696	3.237	12.480	-6.838	5.642	-3.295	2.347
7	fillet included	6.815	-3.507	3.308	12.266	-6.489	5.777	-3.107	2.670
8	temperature	6.815	-3.507	3.308	12.266	-6.489	5.777	-2.472	3.305
9	PCI mult 1.0	6.815	-3.507	3.308	6.815	-3.507	3.308	-3.124	0.184
10	increase fillet thickness	6.815	-3.507	3.308	12.266	-6.489	5.777	-3.270	2.507
11	2, 4	5.997	-3.160	2.837	10.795	-5.846	4.949	-2.574	2.375
12	2, 5	5.667	-3.071	2.596	10.201	-5.681	4.520	-2.426	2.094
13	2, 7	6.360	-3.256	3.104	11.447	-6.024	5.423	-2.719	2.704
14	2, 4, 6	6.074	-3.303	2.771	10.933	-6.111	4.822	-2.691	2.131
15	2, 4, 6, 10	6.074	-3.303	2.771	10.933	-6.111	4.822	-2.817	2.005
16	baseline run (including prestress losses)	6.206	-3.507	2.699	11.171	-6.489	4.682	-3.124	1.558
17	2, 4 (including prestress losses)	5.463	-3.160	2.303	9.834	-5.846	3.988	-2.574	1.414
18	baseline run, minimum span length 120 ft.	2.757	-1.331	1.426	4.963	-2.462	2.501	-1.185	1.316
19	baseline run, average span length 140 ft.	4.774	-2.444	2.330	8.592	-4.521	4.071	-2.176	1.895
20	baseline run, maximum span length 160 ft.	7.664	-4.141	3.523	13.796	-7.661	6.135	-3.688	2.447
21	limestone aggregate	6.268	-3.206	3.062	11.282	-5.931	5.351	-2.860	2.491
22	avg f'ci/f'c, limestone aggregate	5.355	-3.007	2.348	9.639	-5.564	4.075	-2.532	1.543

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Research Data Sets for Camber Calculations

The next sets of tables include the camber estimates using a time-dependent analysis for the FIB-72 girder for several of the various items previously noted that influence camber

Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - baseline

girder camber (inches)				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	1	1	-4.197	7.636	0.000	3.439
2	store beam, transport beam, and erect beam at project site	50	51	-6.854	12.125	-0.183	5.088
3	form deck and place rebar	30	81	-7.304	12.853	-0.231	5.318
4	pour and cure deck	14	95	-8.816	13.038	-0.464	3.758
5	pour and cure barriers	14	109	-9.093	13.186	-0.803	3.290
6	time step	365	474	-10.067	14.147	-1.698	2.382

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Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - Low humidity

girder camber (inches)							
				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	1	1	-4.197	7.636	0.000	3.439
2	store beam, transport beam, and erect beam at project site	50	51	-8.094	14.160	-0.378	5.688
3	form deck and place rebar	30	81	-8.743	15.177	-0.488	5.946
4	pour and cure deck	14	95	-10.330	15.433	-0.709	4.394
5	pour and cure barriers	14	109	-10.682	15.636	-1.044	3.910
6	time step	365	474	-11.918	16.945	-1.923	3.104

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Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - 2-day cure

girder camber (inches)							
				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	2	2	-3.550	6.538	-0.016	2.972
2	store beam, transport beam, and erect beam at project site	50	52	-5.388	9.685	-0.192	4.105
3	form deck and place rebar	30	82	-5.706	10.208	-0.238	4.264
4	pour and cure deck	14	96	-7.196	10.342	-0.473	2.673
5	pour and cure barriers	14	110	-7.455	10.449	-0.812	2.182
6	time step	365	475	-8.316	11.147	-1.710	1.121

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Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - 3-day cure

girder camber (inches)				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	3	3	-3.440	6.349	-0.022	2.887
2	store beam, transport beam, and erect beam at project site	50	53	-5.084	9.172	-0.194	3.894
3	form deck and place rebar	30	83	-5.372	9.646	-0.239	4.035
4	pour and cure deck	14	97	-6.856	9.768	-0.474	2.438
5	pour and cure barriers	14	111	-7.112	9.865	-0.813	1.940
6	time step	365	476	-7.946	10.502	-1.712	0.844

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Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - Average historical f'ci and f'c

Note:
Average f'ci = 8.4 ksi
Average f'c = 13.0 ksi

girder camber (inches)				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	1	1	-3.397	6.084	0.000	2.687
2	store beam, transport beam, and erect beam at project site	50	51	-5.280	9.280	-0.122	3.878
3	form deck and place rebar	30	81	-5.565	9.750	-0.151	4.034
4	pour and cure deck	14	95	-6.823	9.870	-0.362	2.685
5	pour and cure barriers	14	109	-7.034	9.965	-0.665	2.266
6	time step	365	474	-7.728	10.572	-1.446	1.398

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Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - Extended beam storage (6 months)

girder camber (inches)							
				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	1	1	-4.197	7.636	0.000	3.439
2	store beam, transport beam, and erect beam at project site	200	201	-7.962	13.947	-0.287	5.698

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Research Data Sets for Camber Calculations

- FIB-72
- Time-dependent analysis
  - Extended beam storage (12 months)

girder camber (inches)				deflections (at center/mid-span of girder)			
stage #	description	duration (days)	age (days)	self weight	prestress	shrinkage	total deflection/camber
1	pour beam & stress strands	1	1	-4.197	7.636	0.000	3.439
2	store beam, transport beam, and erect beam at project site	364	365	-8.217	14.366	-0.310	5.839

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Research Data Sets for Camber Calculations

The next sets of graphs depict the variation in the camber estimates at release and at erection for several of the various items previously noted that influence camber and includes the historic camber data provided by the MS Concrete Girder Manufacturers

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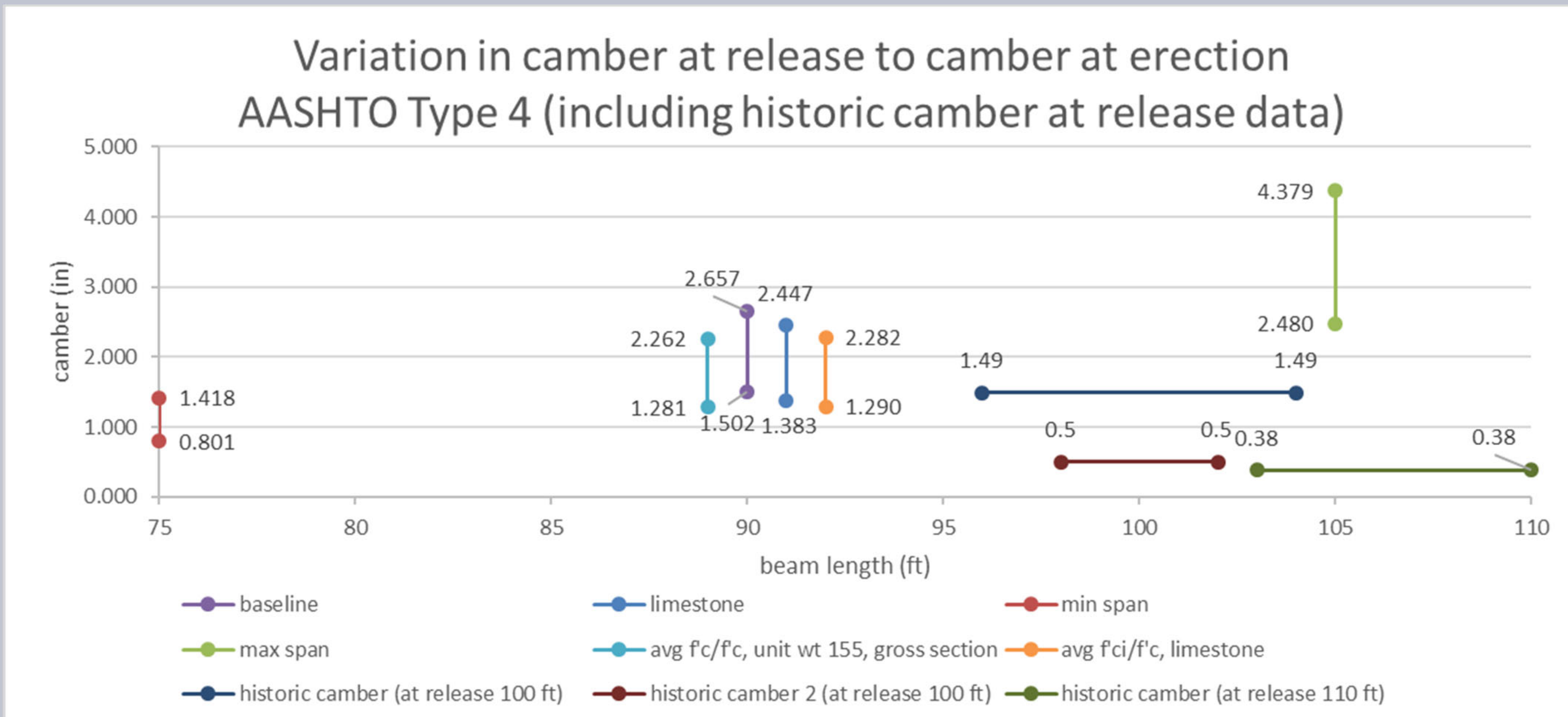
Research Data Sets for Camber Calculations



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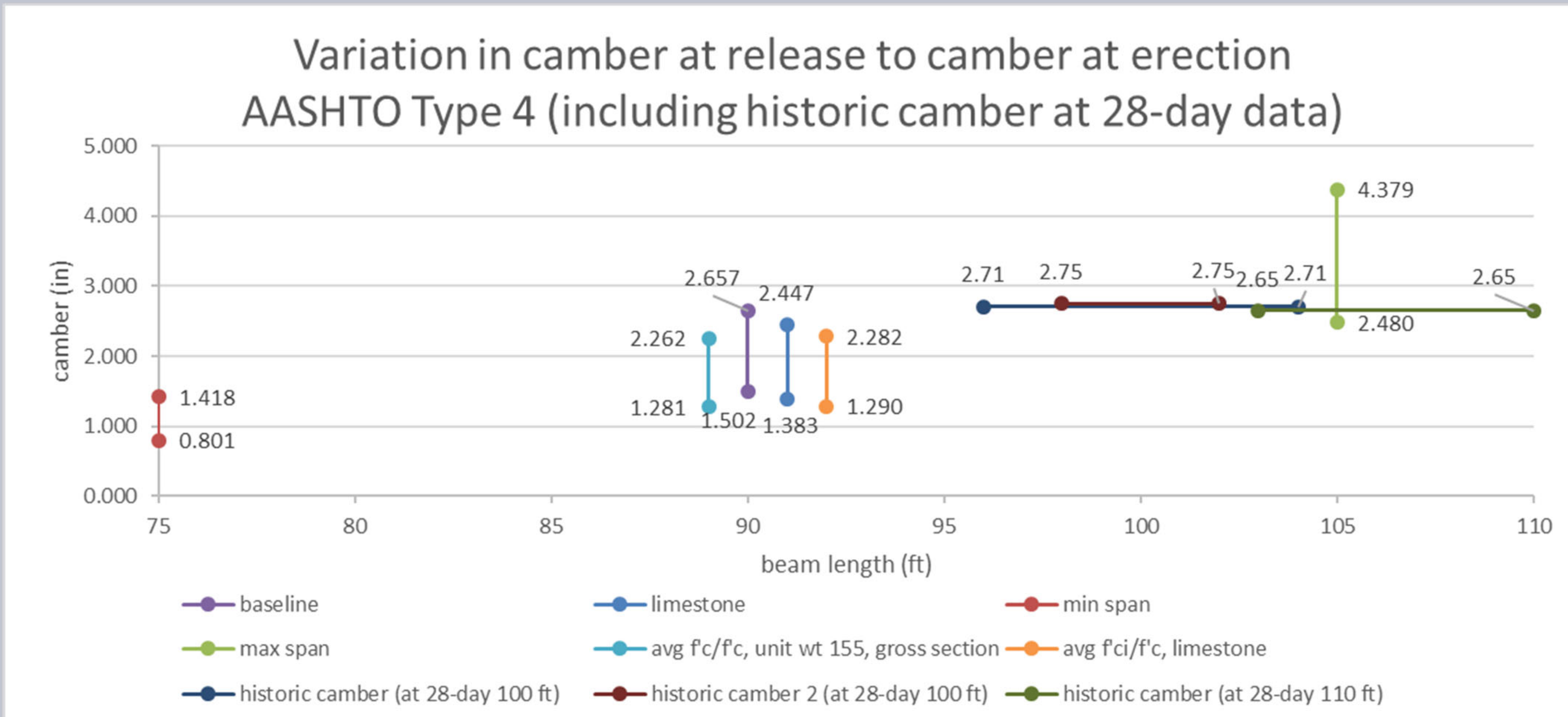
Research Data Sets for Camber Calculations



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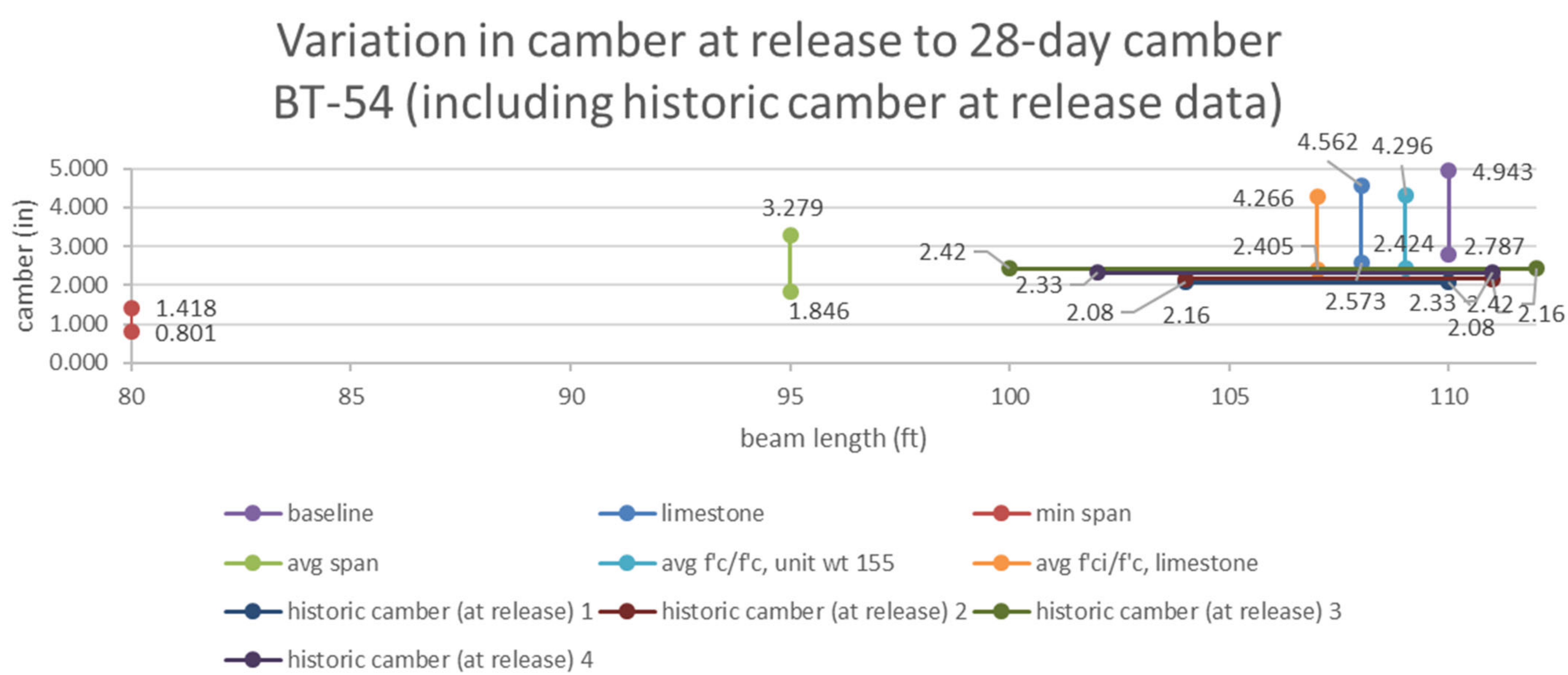
Research Data Sets for Camber Calculations



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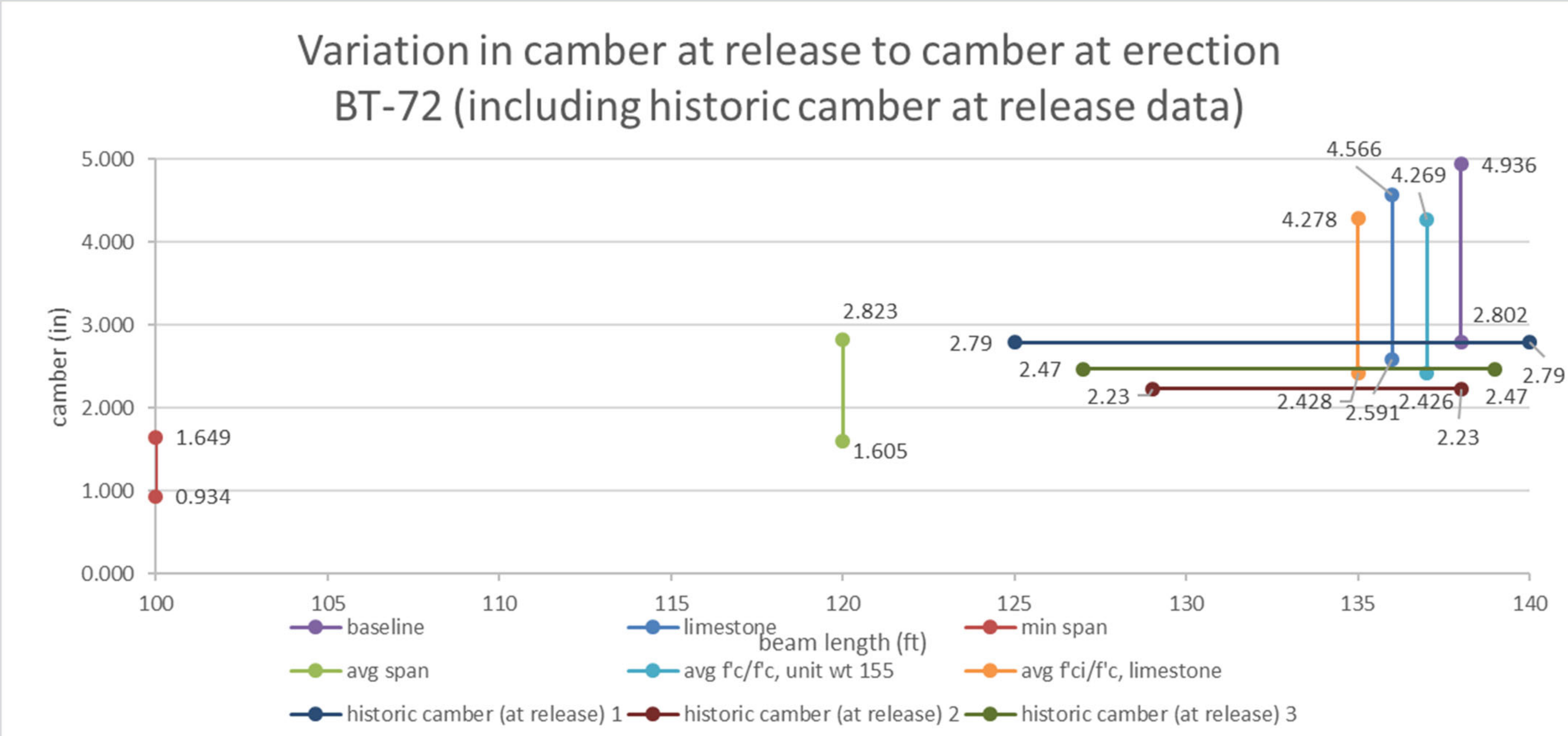
Research Data Sets for Camber Calculations



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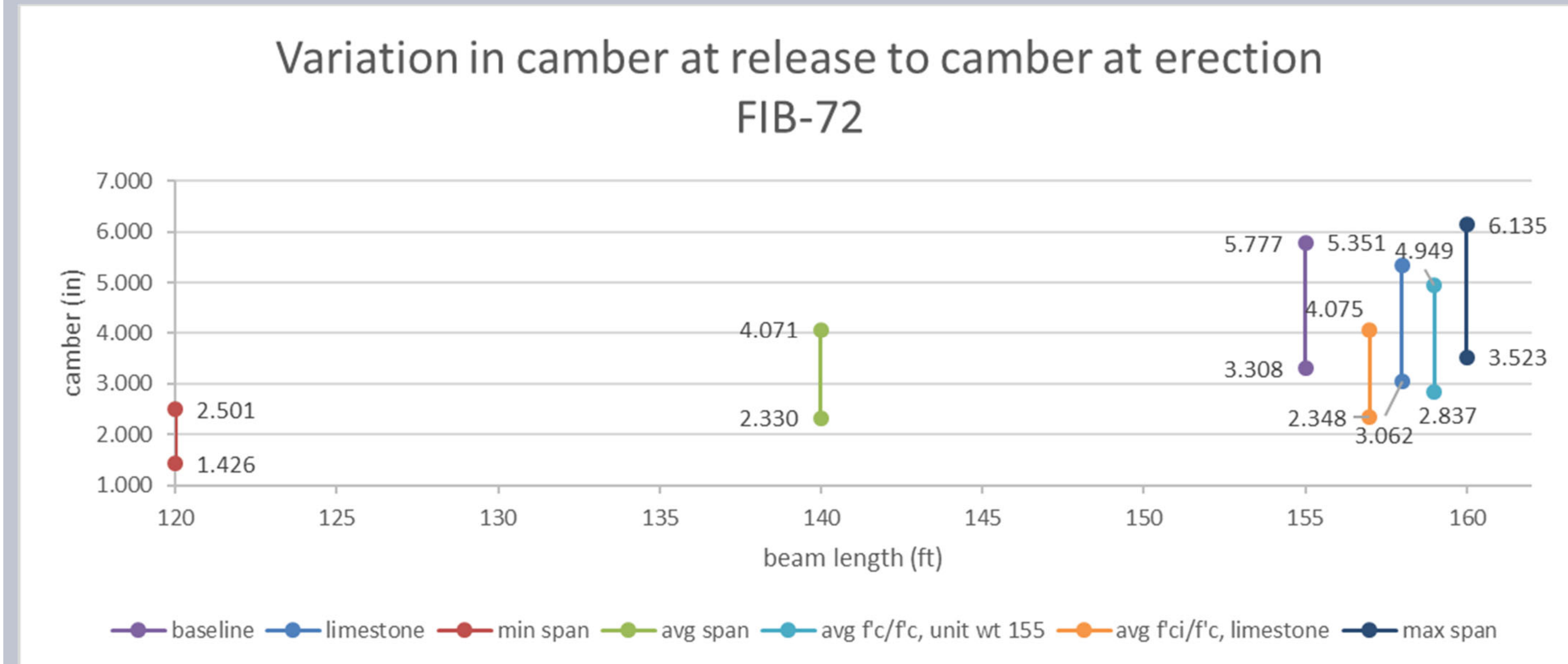

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Research Data Sets for Camber Calculations

The next sets of slides show graphs of the various items that influence camber using a time-dependent analysis

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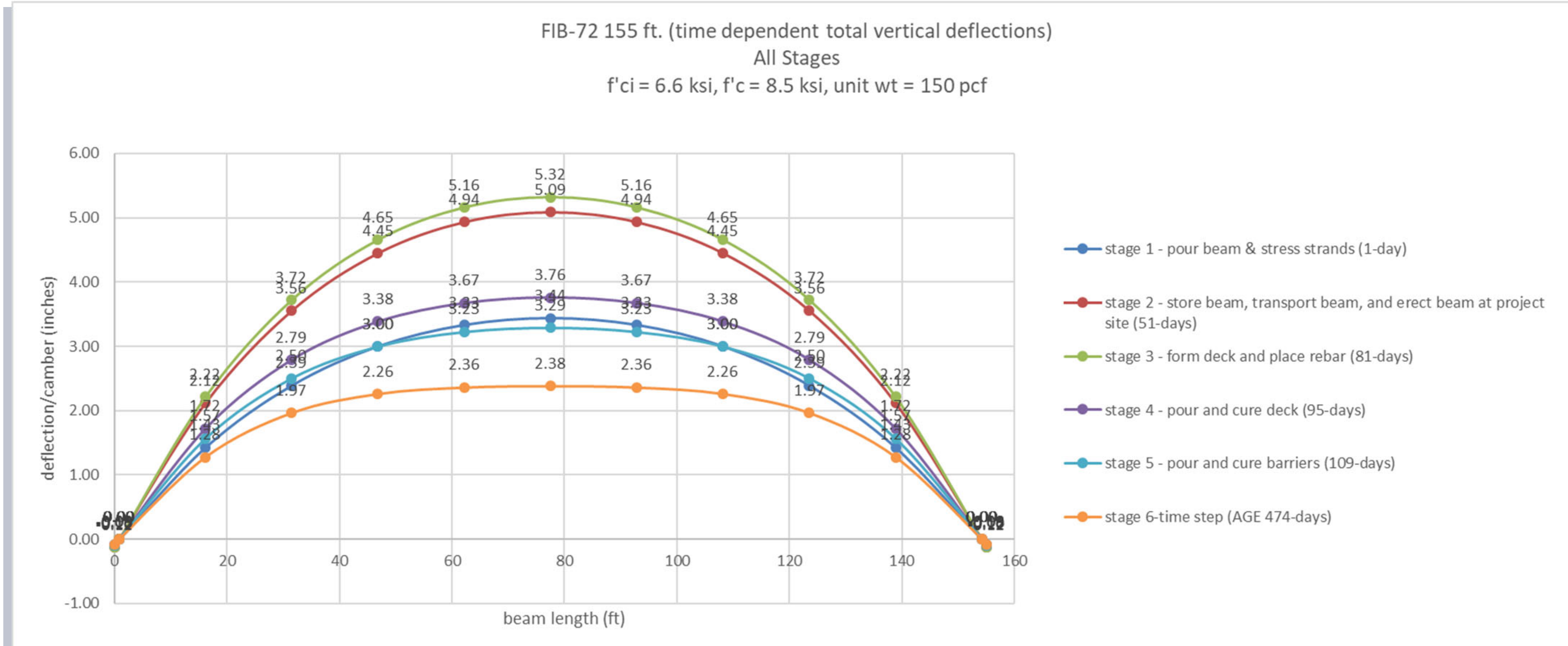
Research Data Sets for Camber Calculations



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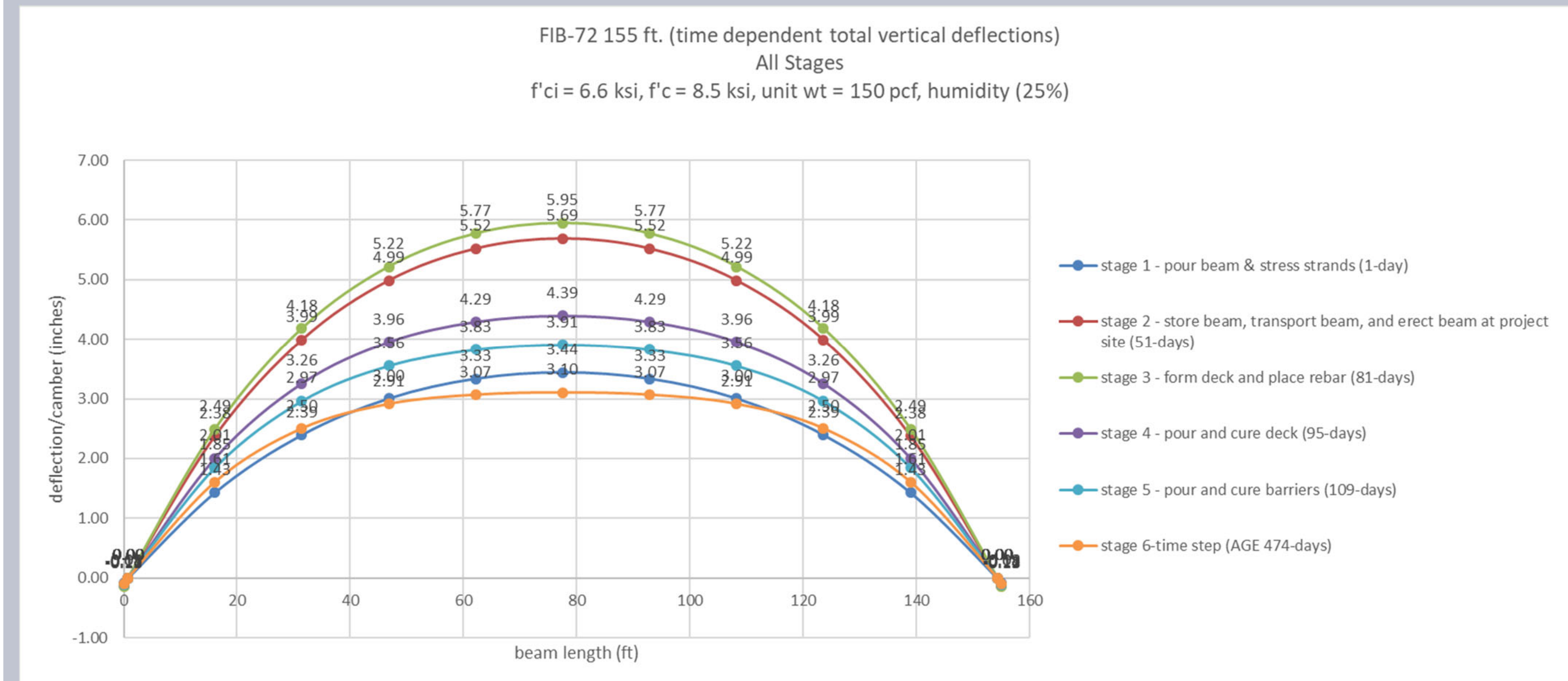

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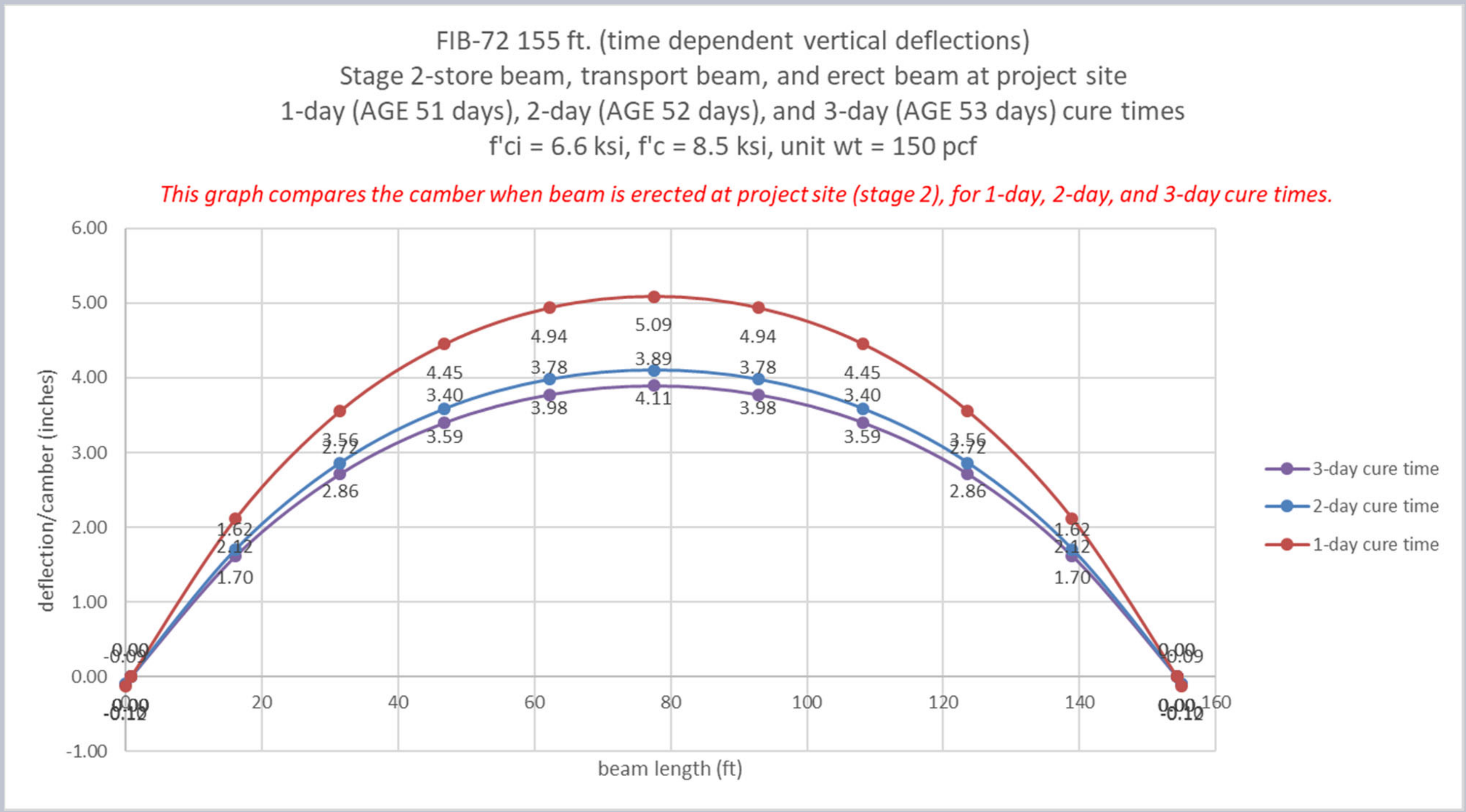
Research Data Sets for Camber Calculations



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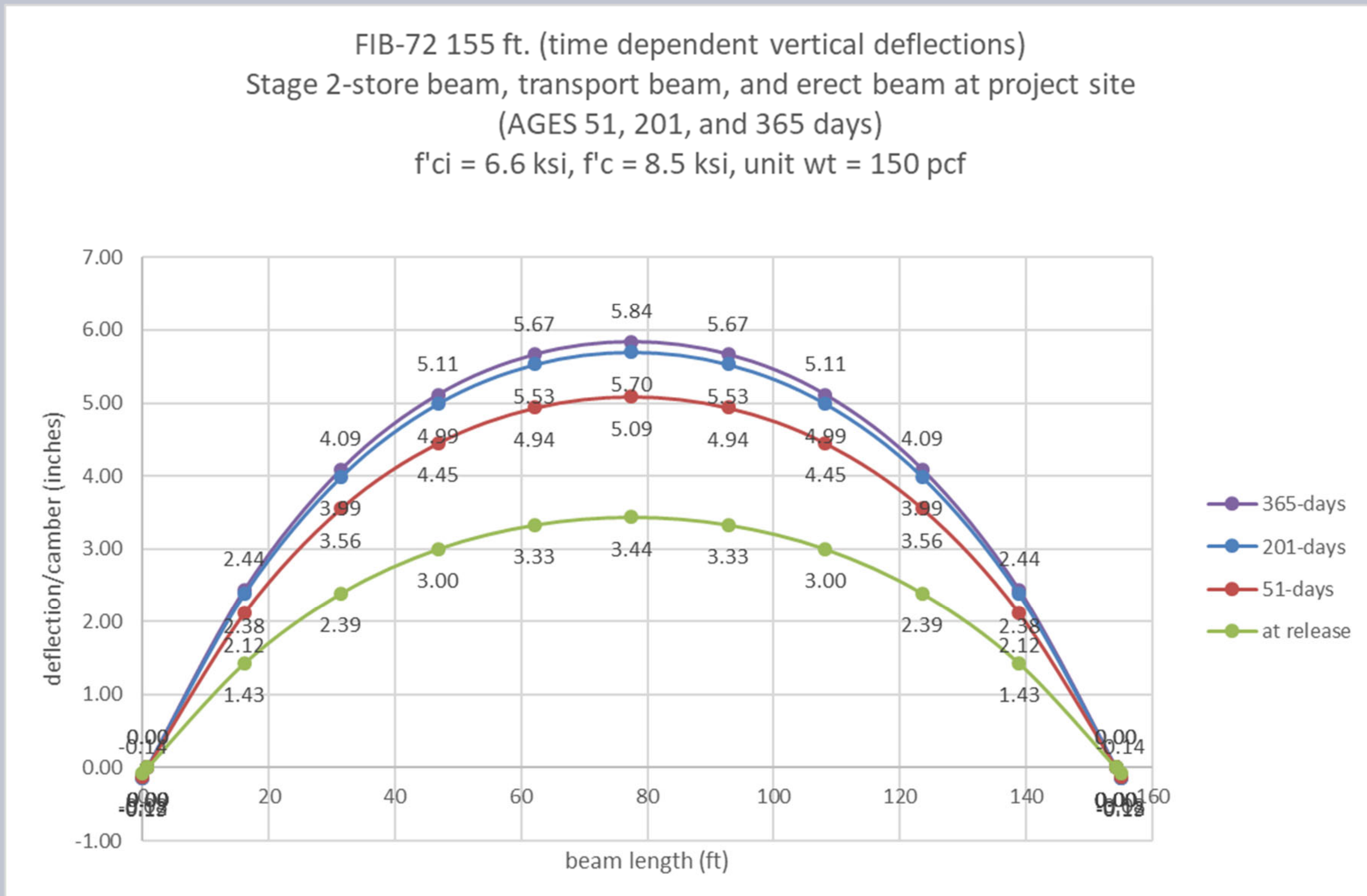
Research Data Sets for Camber Calculations



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Research Data Sets for Camber Calculations

The next sets of graphs depict the variation in the camber estimates at release for the various items previously noted that influence camber

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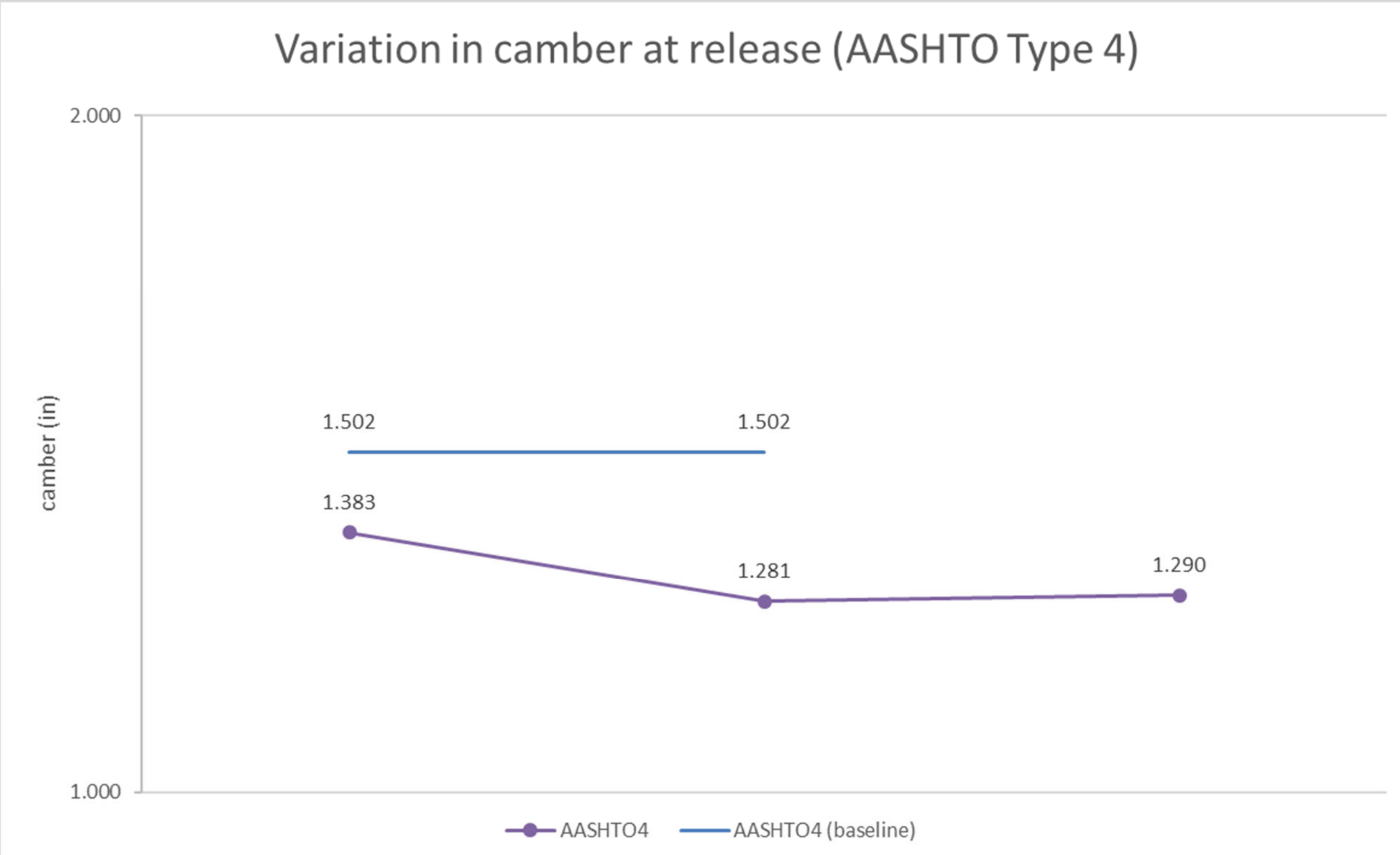
Research Data Sets for Camber Calculations



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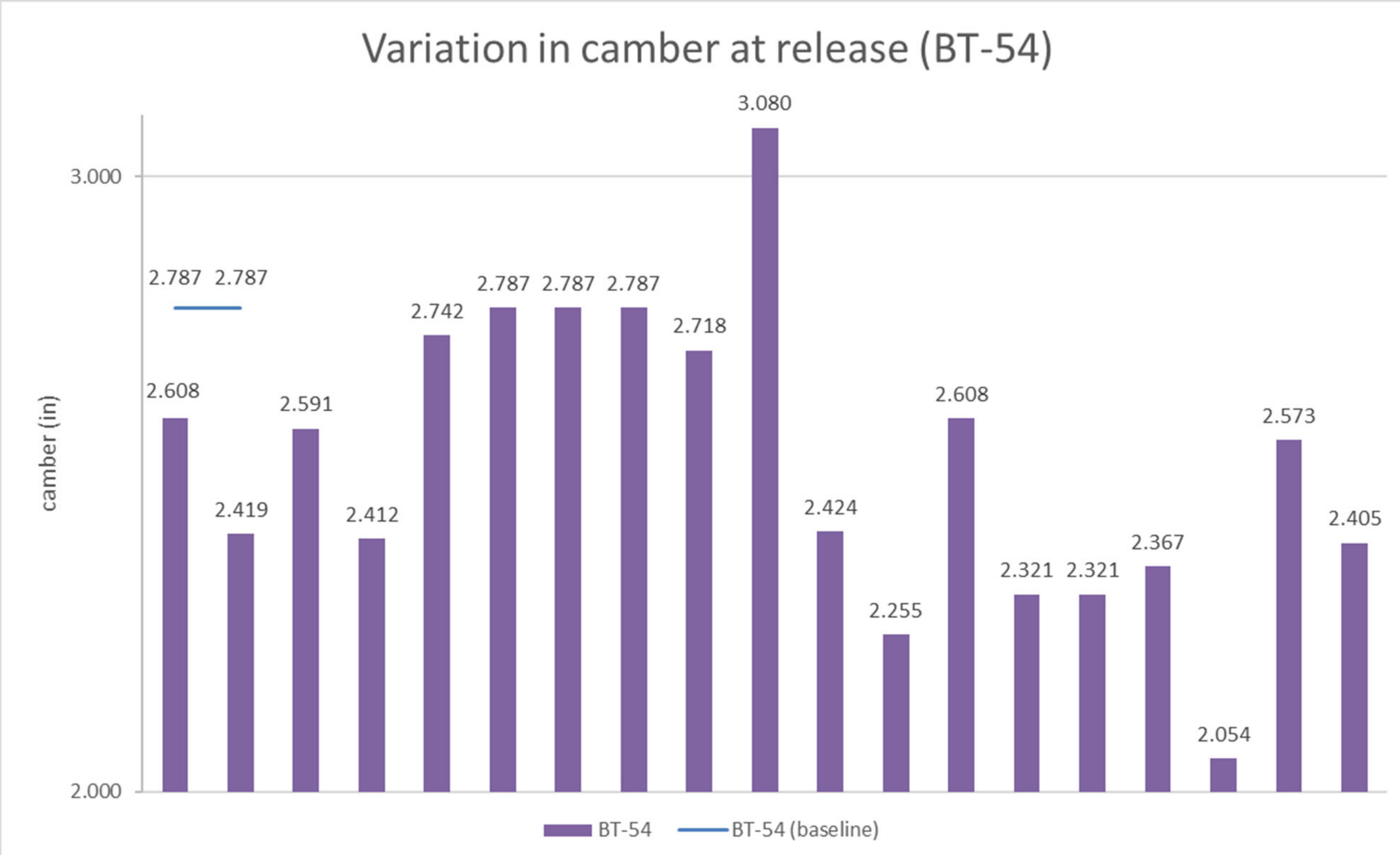
Research Data Sets for Camber Calculations



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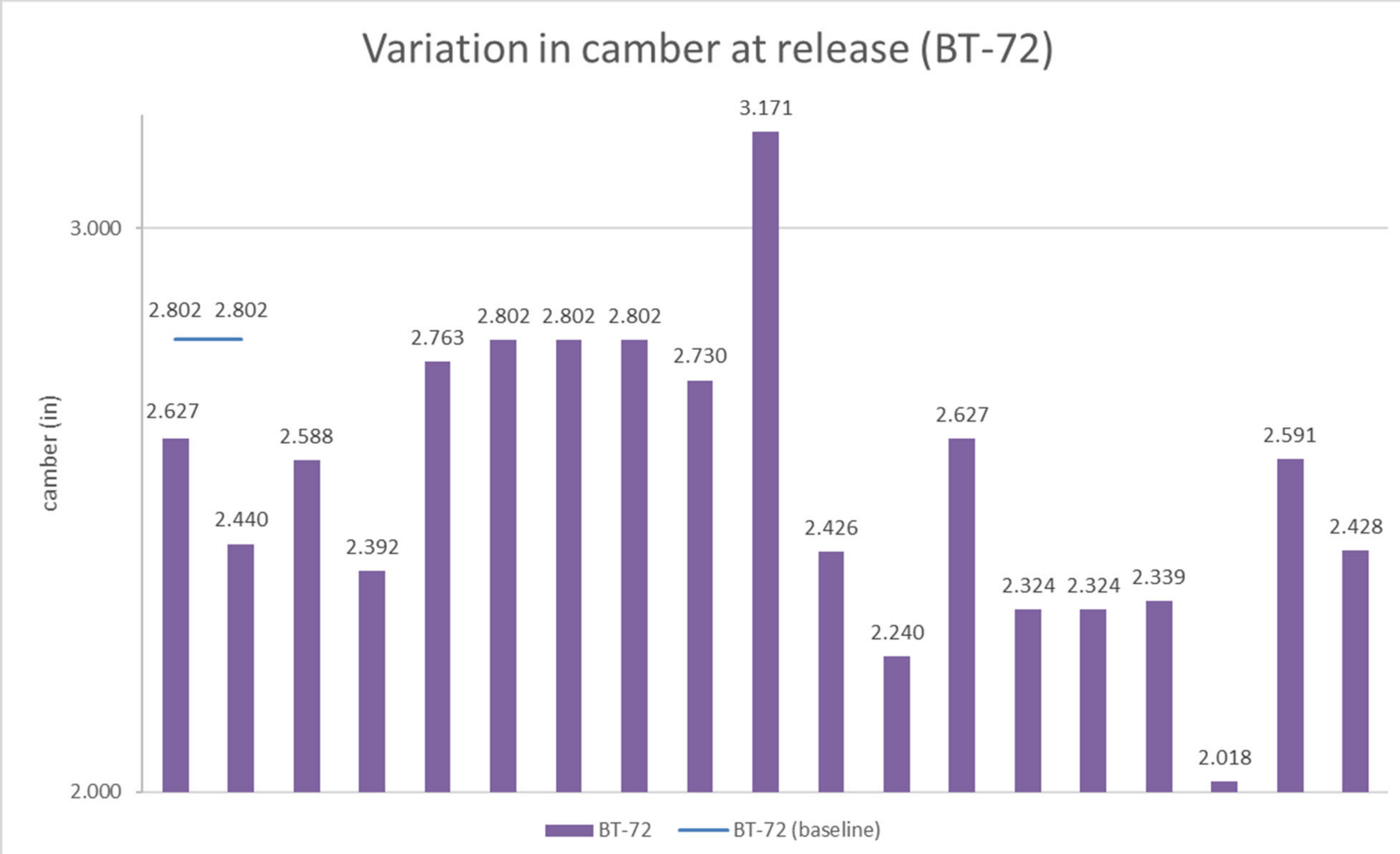

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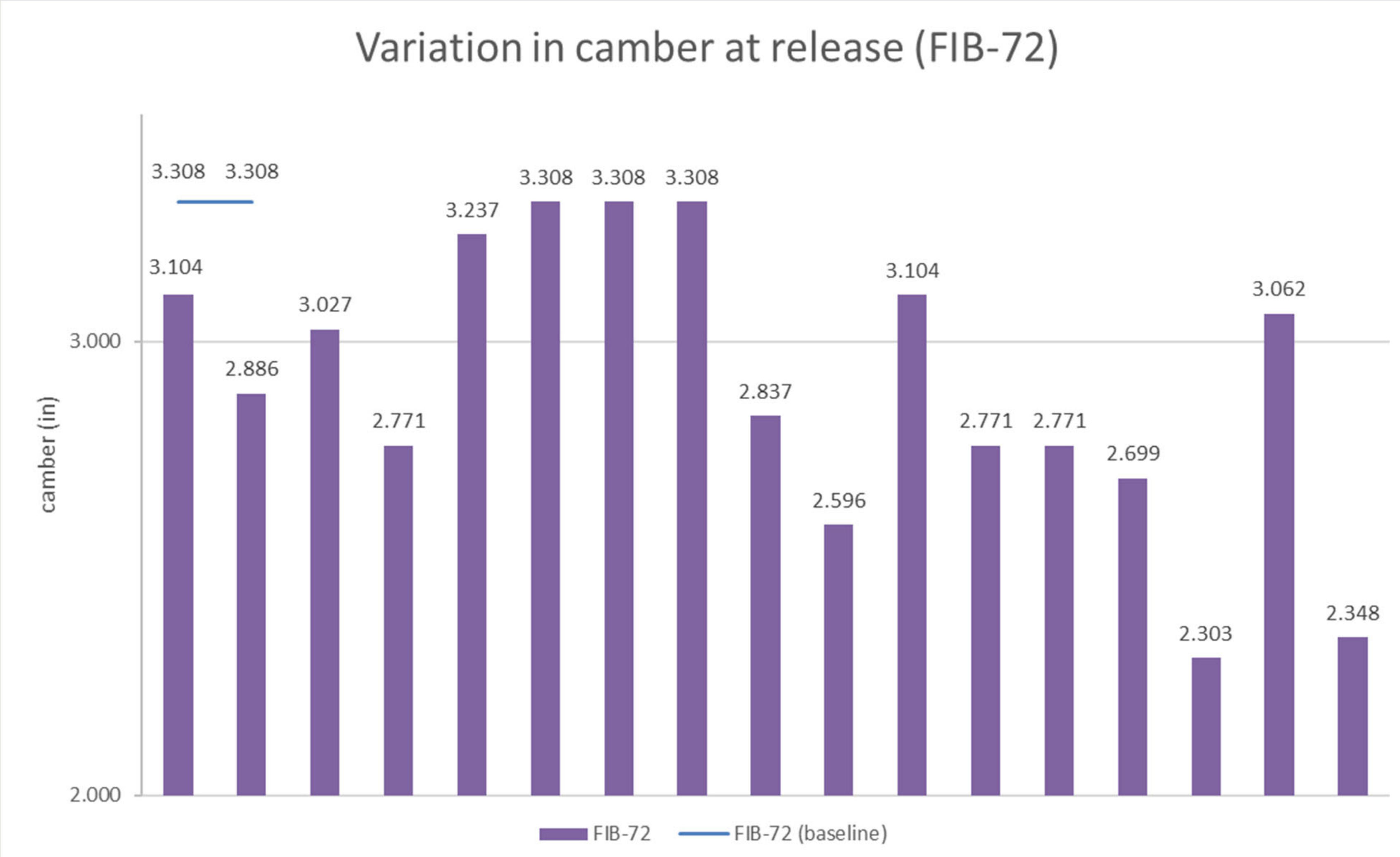
Research Data Sets for Camber Calculations



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Research Data Sets for Camber Calculations

The next sets of graphs depict the variation in the camber estimates at erection for the various items previously noted that influence camber

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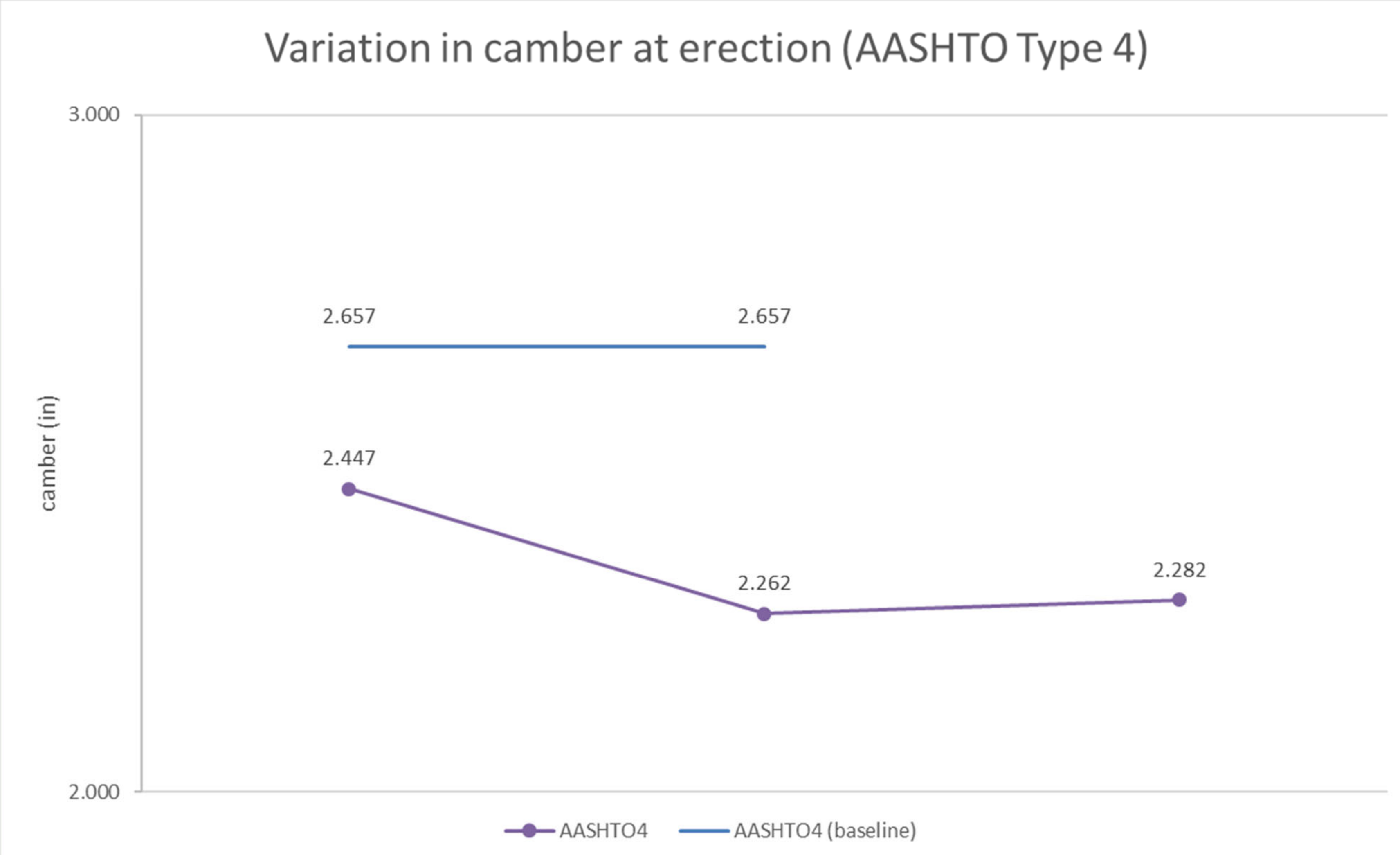
Research Data Sets for Camber Calculations



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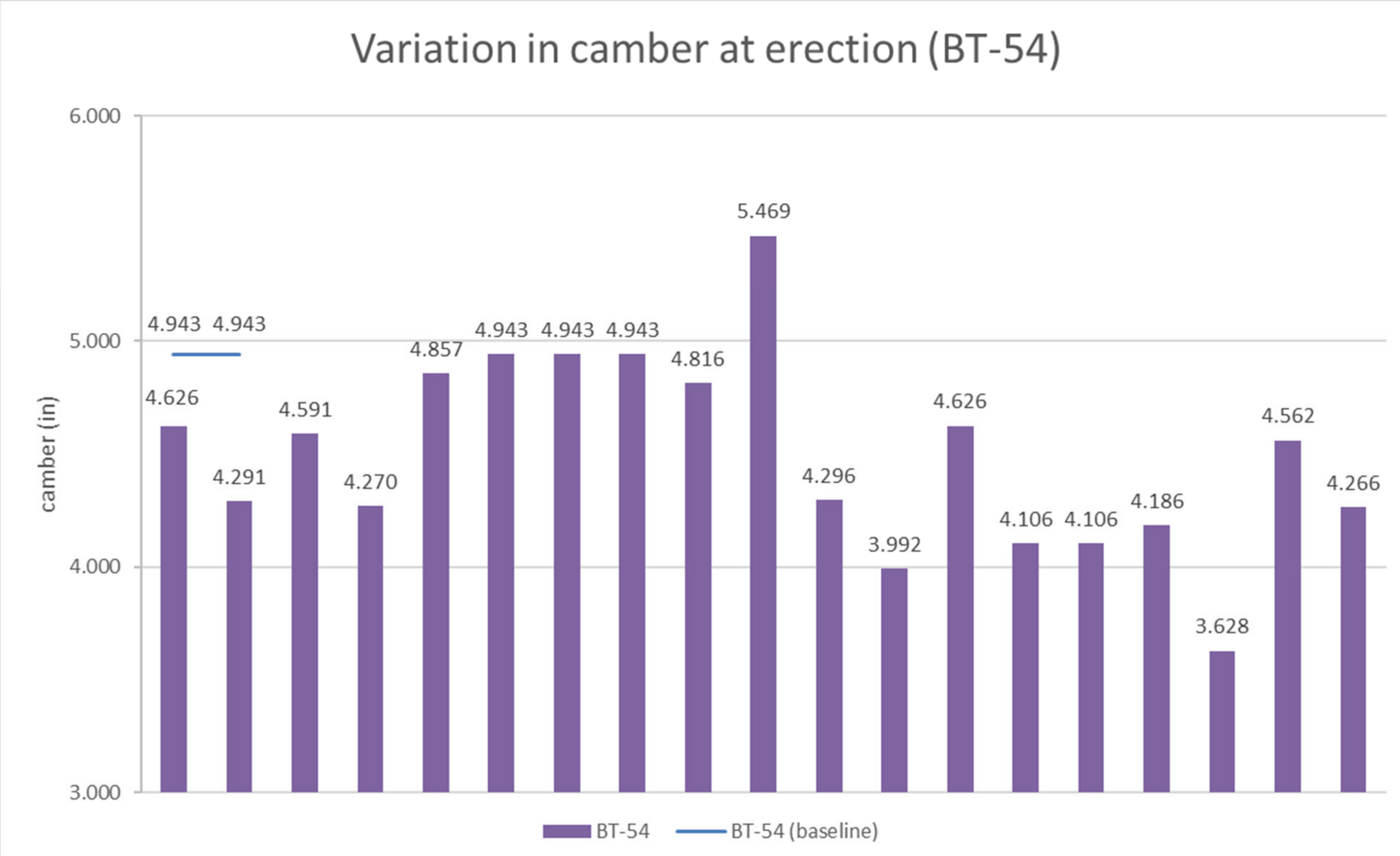
Research Data Sets for Camber Calculations



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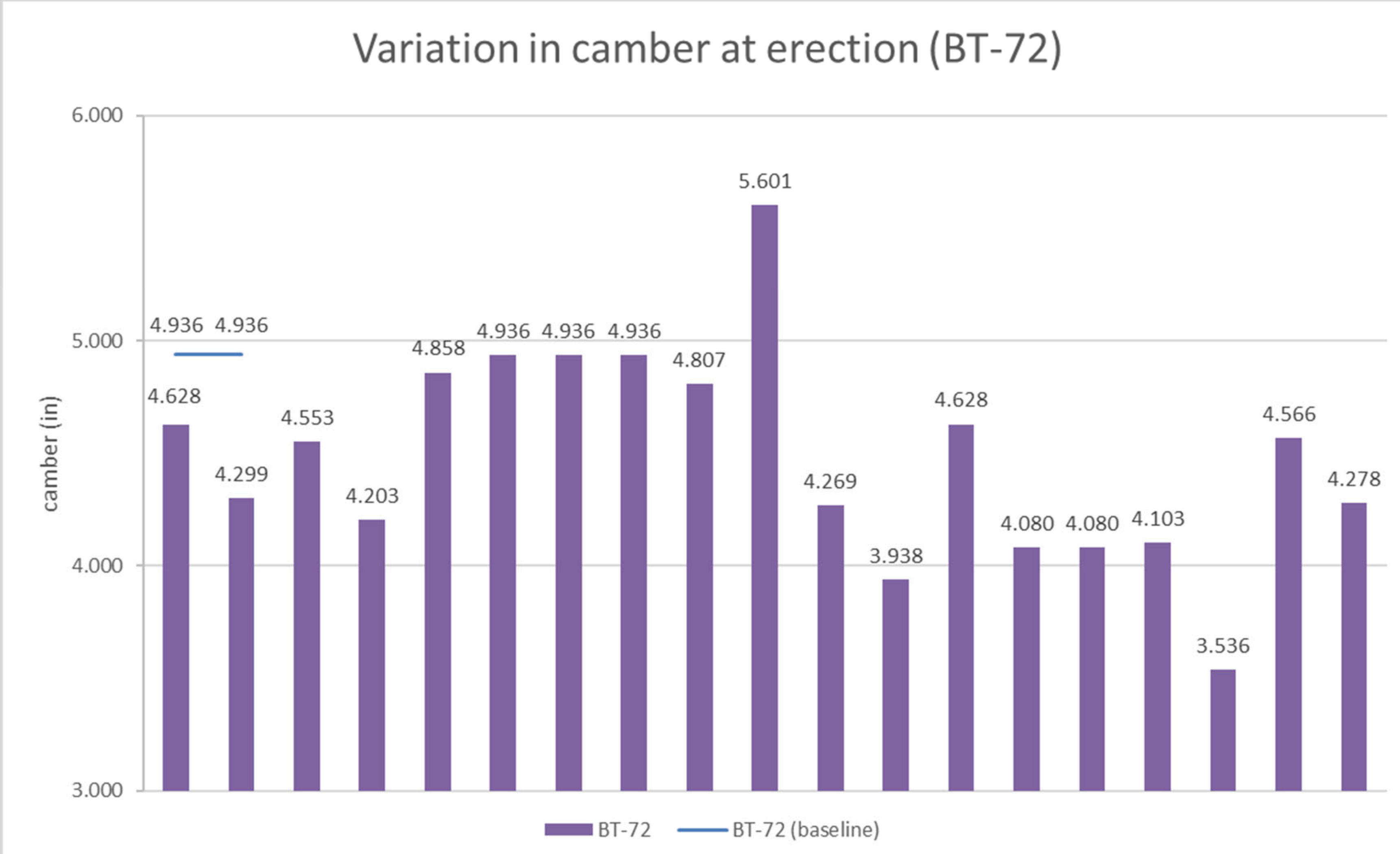
Research Data Sets for Camber Calculations



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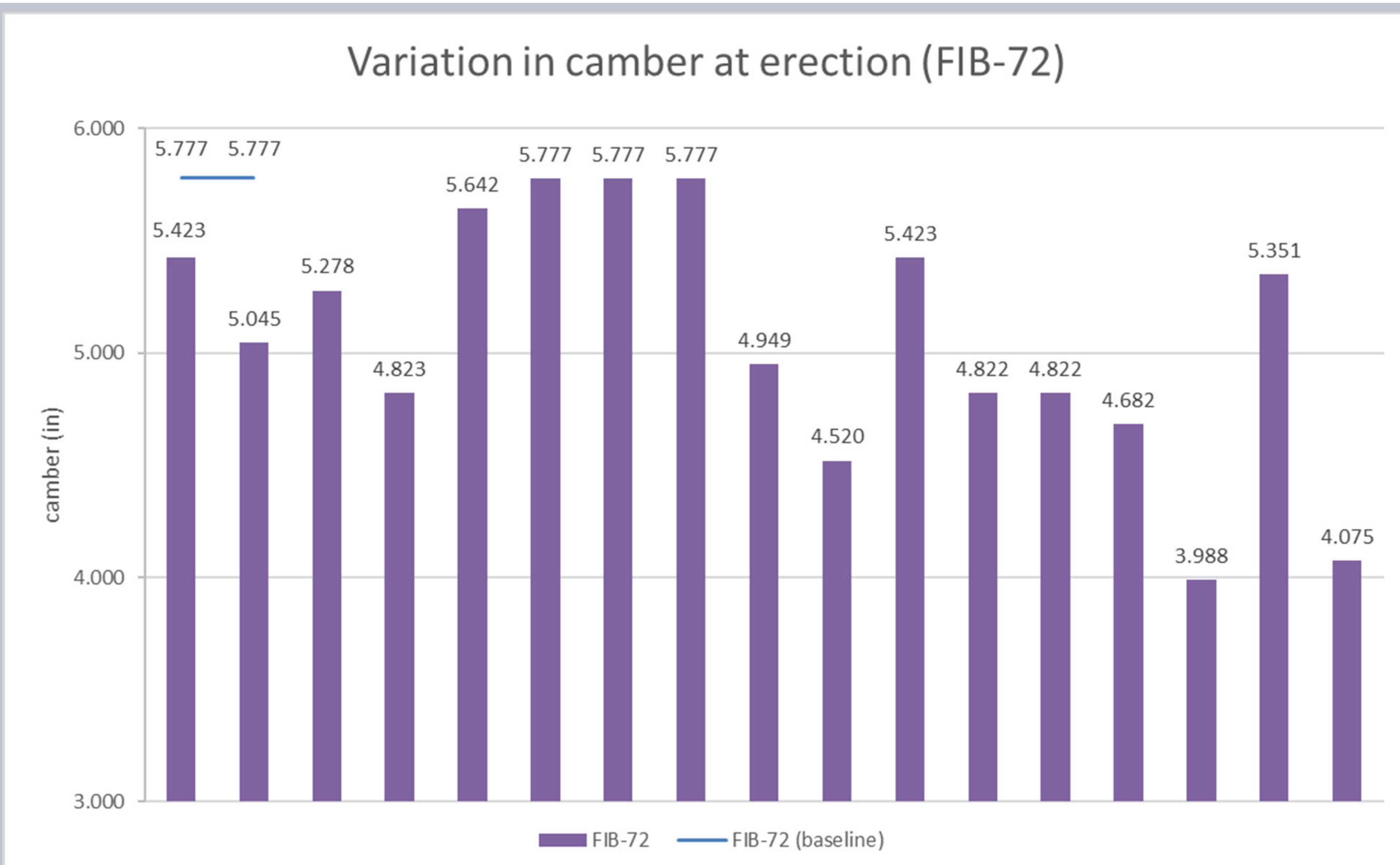
Research Data Sets for Camber Calculations



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Research Data Sets for Camber Calculations

The next sets of graphs depict the variation in the camber estimates at erection (after added dead load deflection) for the various items previously noted that influence camber

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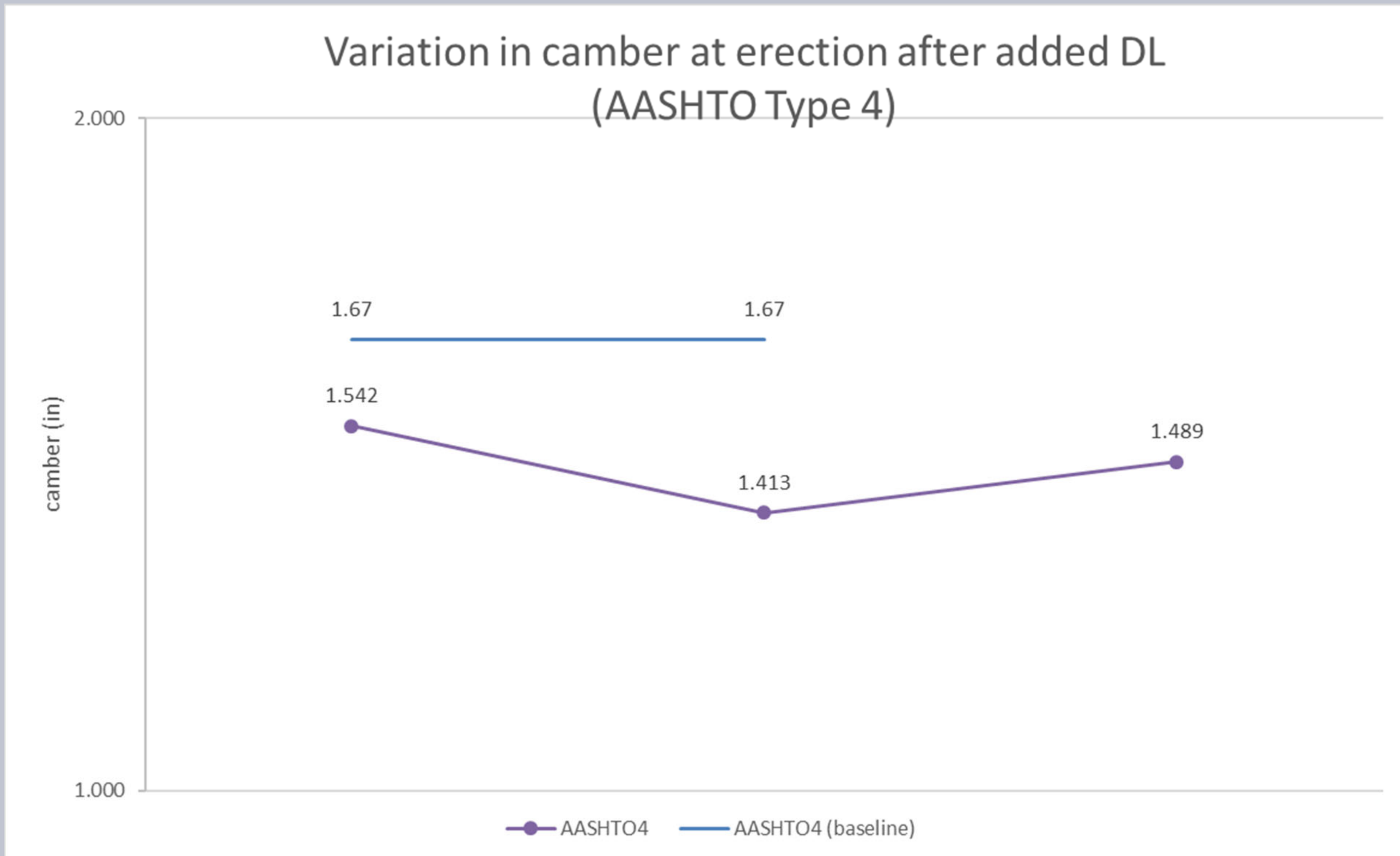
Research Data Sets for Camber Calculations



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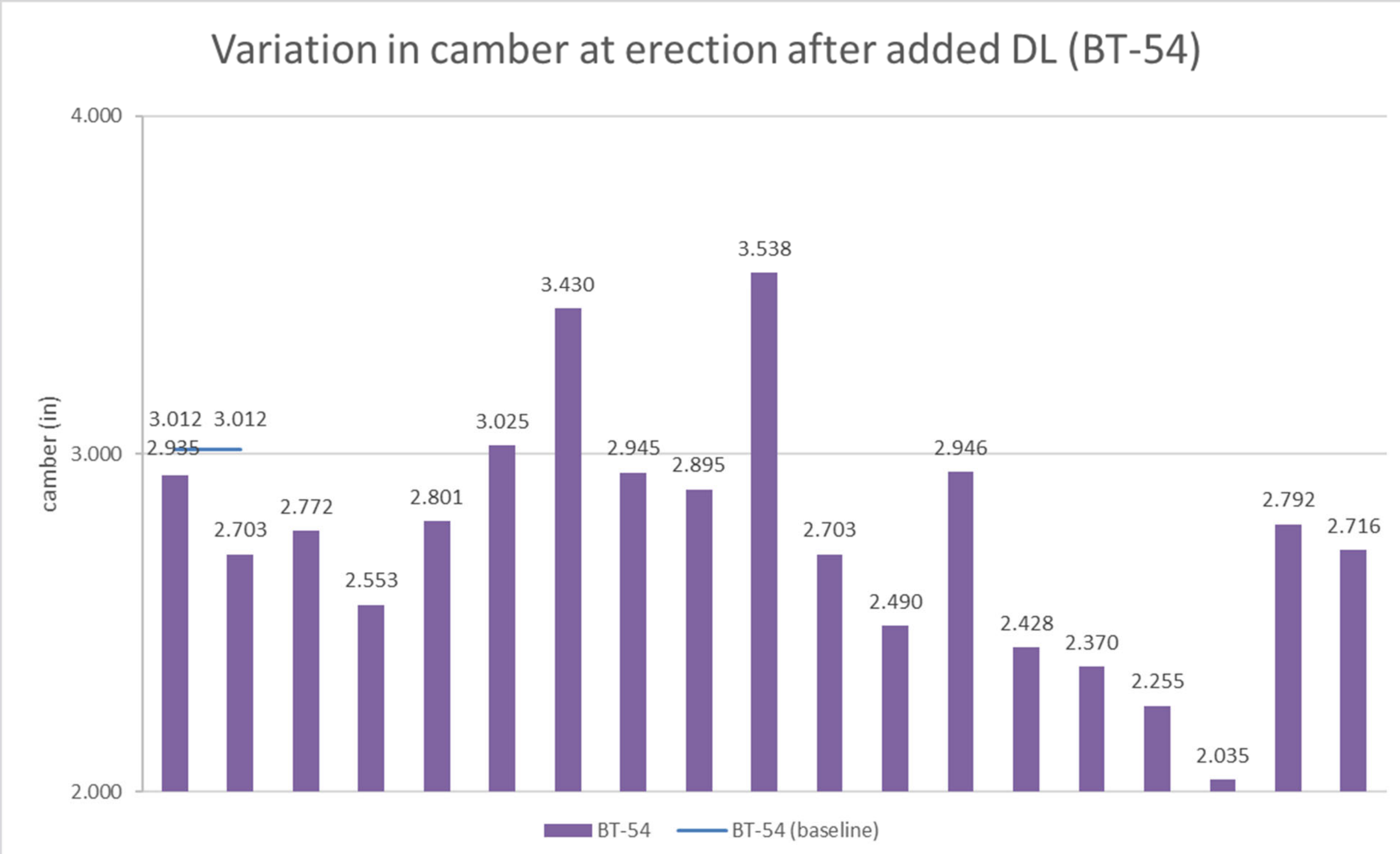
Research Data Sets for Camber Calculations



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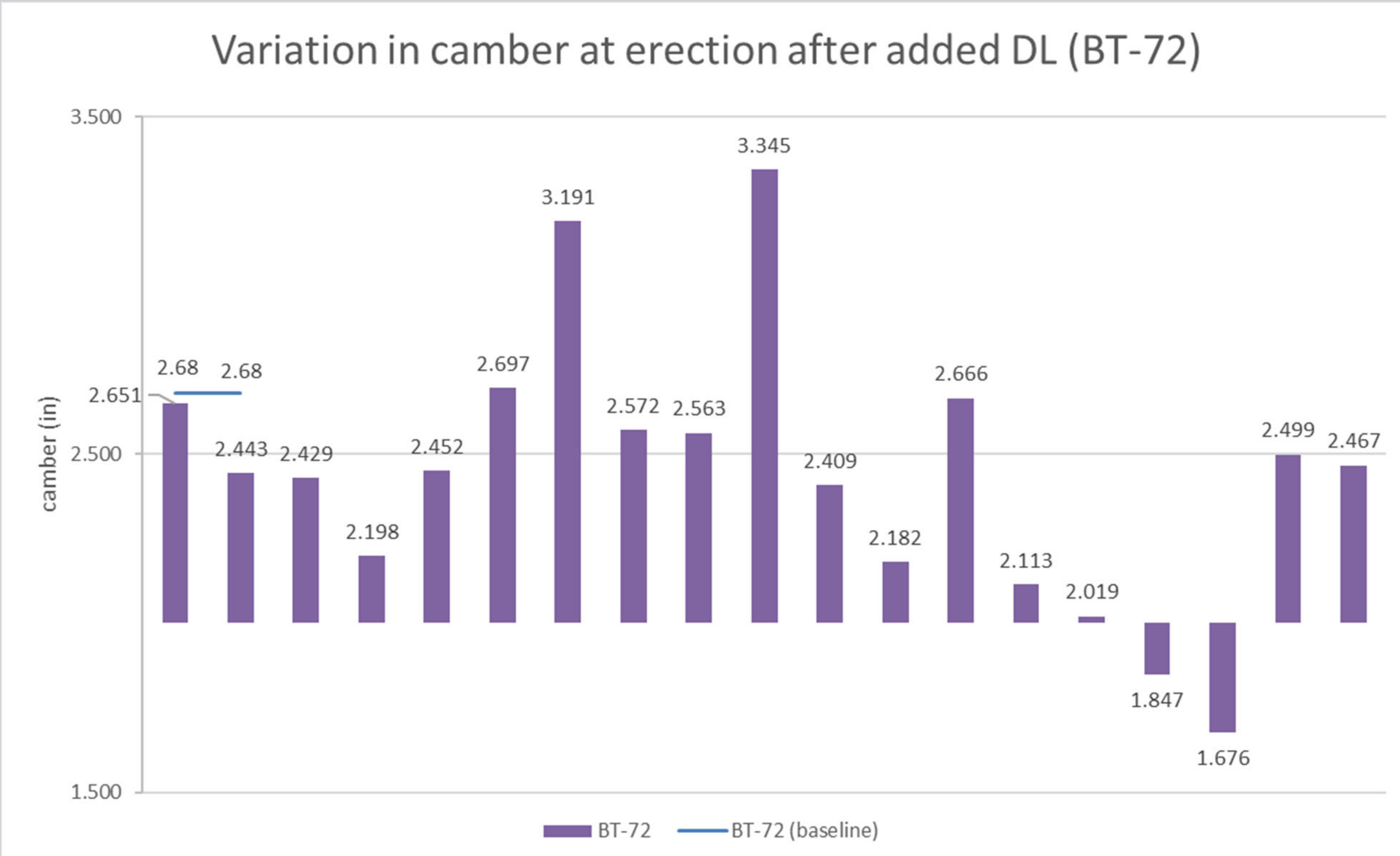
Research Data Sets for Camber Calculations



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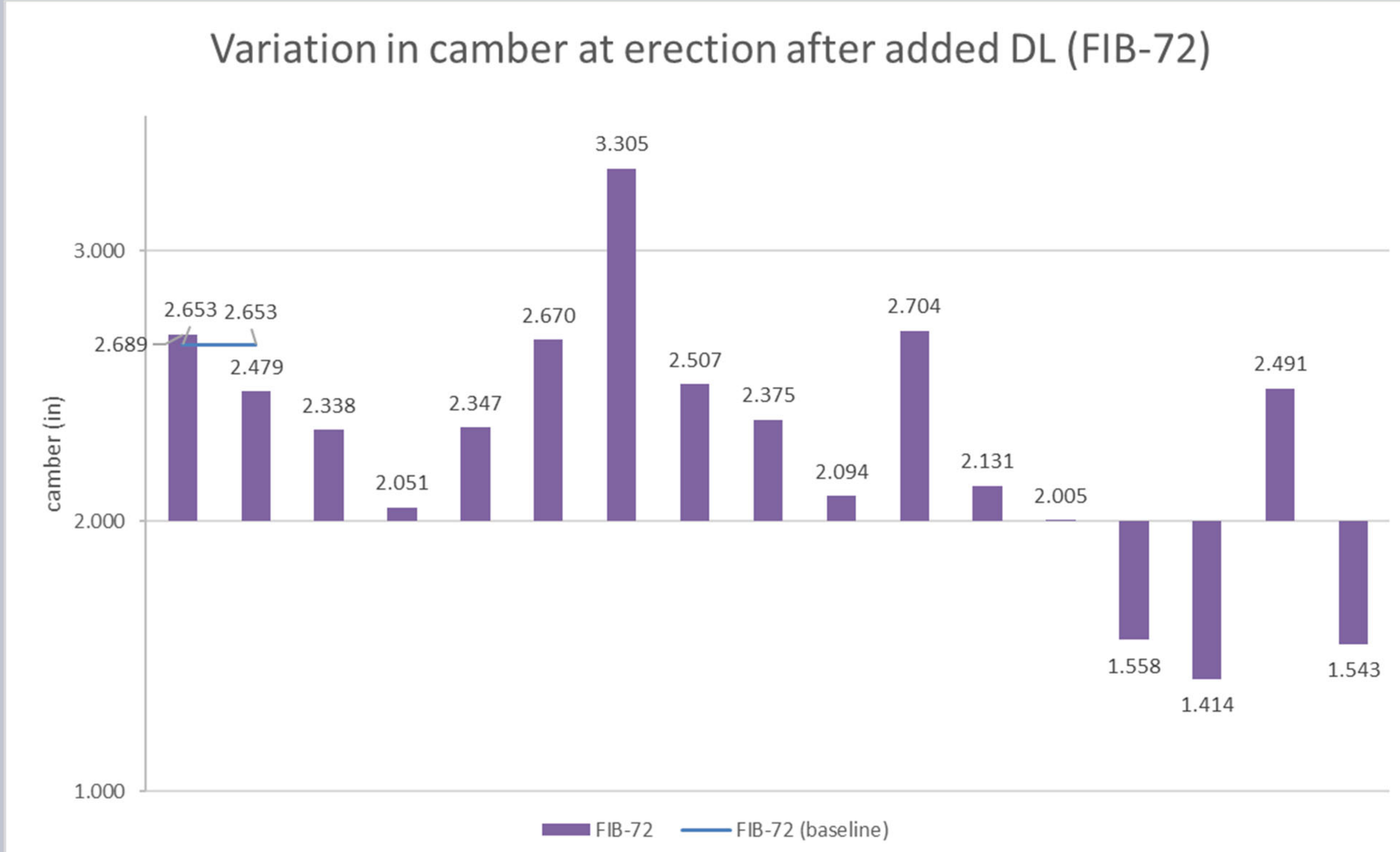

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Research Data Sets for Camber Calculations

Based on the camber data sets information, are there any observations that are apparent?

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Research Data Sets for Camber Calculations

Camber data sets observations:

1. The various items listed that influence camber, do in fact change the values of the camber estimates (both individually and more so in combination with each other)
2. The under-camber girders that MDOT has experienced on several projects is at least validated by the majority of the items (that mostly in combination with each other) show that the estimated camber is less than the baseline (i.e., current design camber estimate)

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Research Data Sets for Camber Calculations

Further questions & insights:

- How much do the items that influence camber change the camber estimates?
  - Chart/list which items increase and/or decrease camber estimates compared to the baseline camber estimate for same span length girder
- What is the range in variability in the camber estimates?
  - Does the data show that the range of variability increases as the span length and/or number of strands increase?
- Can recommendations be made based on the insights and observations or is further research and/or data collection warranted?
  - Based on historic material data and ratio of f'c/f'ci, develop table for design values of f'c and f'ci accounting for upper and lower bounds
  - Consider using adjusted concrete strengths, adjusted modulus of elasticity, and unit weight of 155 pcf when calculating camber estimates based on the historical material data provided by the MS concrete girder manufacturers

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Research Data Sets for Camber Calculations

The next sets of tables depict which items increase or decrease camber estimates compared to the baseline camber estimate for the same span length girder

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Research Data Sets for Camber Calculations

AASHTO Type 4 (90 ft.)						
	camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
baseline run	1.502	% difference	2.657	% difference	1.670	% difference
limestone aggregate	1.383	-7.92%	2.447	-7.90%	1.542	-7.66%
avg f'ci/f'c, unit wt 155, gross section	1.281	-14.71%	2.262	-14.87%	1.413	-15.39%
baseline run, minimum span length 75 ft.	0.801	-46.67%	1.418	-46.63%	0.936	-43.95%
baseline run, maximum span length 105 ft.	2.480	65.11%	4.379	64.81%	2.566	53.65%
avg f'ci/f'c, limestone aggregate	1.290	-14.11%	2.282	-14.11%	1.489	-10.84%

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Research Data Sets for Camber Calculations

	BT-54 (110 ft.)						
#		camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
1	baseline run	2.787	% difference	4.943	% difference	3.012	% difference
2	avg f'ci/f'c	2.608	-6.42%	4.626	-6.41%	2.935	-2.56%
3	high f'ci/f'c	2.419	-13.20%	4.291	-13.19%	2.703	-10.26%
4	unit wt 155	2.591	-7.03%	4.591	-7.12%	2.772	-7.97%
5	unit wt 160	2.412	-13.46%	4.270	-13.62%	2.553	-15.24%
6	gross section	2.742	-1.61%	4.857	-1.74%	2.801	-7.01%
7	fillet included	2.787	0.00%	4.943	0.00%	3.025	0.43%
8	temperature	2.787	0.00%	4.943	0.00%	3.430	13.88%
9	PCI mult 1.0	2.787	0.00%	2.787	-43.62%	0.856	-71.58%
10	increase fillet thickness	2.787	0.00%	4.943	0.00%	2.945	-2.22%
11	top strand	2.718	-2.48%	4.816	-2.57%	2.895	-3.88%
12	straight strands with debonding	3.080	10.51%	5.469	10.64%	3.538	17.46%
13	2, 4	2.424	-13.02%	4.296	-13.09%	2.703	-10.26%
14	2, 5	2.255	-19.09%	3.992	-19.24%	2.490	-17.33%
15	2, 7	2.608	-6.42%	4.626	-6.41%	2.946	-2.19%
16	2, 4, 6, 11	2.321	-16.72%	4.106	-16.93%	2.428	-19.39%
17	2, 4, 6, 10, 11	2.321	-16.72%	4.106	-16.93%	2.370	-21.31%
18	baseline run (including prestress losses)	2.367	-15.07%	4.186	-15.31%	2.255	-25.13%
19	2, 4 (including prestress losses)	2.054	-26.30%	3.628	-26.60%	2.035	-32.44%
22	limestone aggregate	2.573	-7.68%	4.562	-7.71%	2.792	-7.30%
23	avg f'ci/f'c, limestone aggregate	2.405	-13.71%	4.266	-13.70%	2.716	-9.83%

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Research Data Sets for Camber Calculations

	BT-72 (138 ft.)						
#		camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
1	baseline run	2.802	% difference	4.936	% difference	2.680	% difference
2	avg f'ci/f'c	2.627	-6.25%	4.628	-6.24%	2.651	-1.08%
3	high f'ci/f'c	2.440	-12.92%	4.299	-12.91%	2.443	-8.84%
4	unit wt 155	2.588	-7.64%	4.553	-7.76%	2.429	-9.37%
5	unit wt 160	2.392	-14.63%	4.203	-14.85%	2.198	-17.99%
6	gross section	2.763	-1.39%	4.858	-1.58%	2.452	-8.51%
7	fillet included	2.802	0.00%	4.936	0.00%	2.697	0.63%
8	temperature	2.802	0.00%	4.936	0.00%	3.191	19.07%
9	PCI mult 1.0	2.802	0.00%	2.802	-43.23%	0.546	-79.63%
10	increase fillet thickness	2.802	0.00%	4.936	0.00%	2.572	-4.03%
11	top strand	2.730	-2.57%	4.807	-2.61%	2.563	-4.37%
12	straight strands with debonding	3.171	13.17%	5.601	13.47%	3.345	24.81%
13	2, 4	2.426	-13.42%	4.269	-13.51%	2.409	-10.11%
14	2, 5	2.240	-20.06%	3.938	-20.22%	2.182	-18.58%
15	2, 7	2.627	-6.25%	4.628	-6.24%	2.666	-0.52%
16	2, 4, 6, 11	2.324	-17.06%	4.080	-17.34%	2.113	-21.16%
17	2, 4, 6, 10, 11	2.324	-17.06%	4.080	-17.34%	2.019	-24.66%
18	baseline run (including prestress losses)	2.339	-16.52%	4.103	-16.88%	1.847	-31.08%
19	2, 4 (including prestress losses)	2.018	-27.98%	3.536	-28.36%	1.676	-37.46%
22	limestone aggregate	2.591	-7.53%	4.566	-7.50%	2.499	-6.75%
23	avg f'ci/f'c, limestone aggregate	2.428	-13.35%	4.278	-13.33%	2.467	-7.95%

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Research Data Sets for Camber Calculations

	FIB-72 (155 ft.)						
#		camber at release (in.)		camber at erection (in.)		camber at erection (after added DL deflection) (in.)	
1	baseline run	3.308	% difference	5.777	% difference	2.653	% difference
2	avg f'ci/f'c	3.104	-6.17%	5.423	-6.13%	2.689	1.36%
3	high f'ci/f'c	2.886	-12.76%	5.045	-12.67%	2.479	-6.56%
4	unit wt 155	3.027	-8.49%	5.278	-8.64%	2.338	-11.87%
5	unit wt 160	2.771	-16.23%	4.823	-16.51%	2.051	-22.69%
6	gross section	3.237	-2.15%	5.642	-2.34%	2.347	-11.53%
7	fillet included	3.308	0.00%	5.777	0.00%	2.670	0.64%
8	temperature	3.308	0.00%	5.777	0.00%	3.305	24.58%
9	PCI mult 1.0	3.308	0.00%	3.308	-42.74%	0.184	-93.06%
10	increase fillet thickness	3.308	0.00%	5.777	0.00%	2.507	-5.50%
11	2, 4	2.837	-14.24%	4.949	-14.33%	2.375	-10.48%
12	2, 5	2.596	-21.52%	4.520	-21.76%	2.094	-21.07%
13	2, 7	3.104	-6.17%	5.423	-6.13%	2.704	1.92%
14	2, 4, 6	2.771	-16.23%	4.822	-16.53%	2.131	-19.68%
15	2, 4, 6, 10	2.771	-16.23%	4.822	-16.53%	2.005	-24.43%
16	baseline run (including prestress losses)	2.699	-18.41%	4.682	-18.95%	1.558	-41.27%
17	2, 4 (including prestress losses)	2.303	-30.38%	3.988	-30.97%	1.414	-46.70%
21	limestone aggregate	3.062	-7.44%	5.351	-7.37%	2.491	-6.11%
22	avg f'ci/f'c, limestone aggregate	2.348	-29.02%	4.075	-29.46%	1.543	-41.84%

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Research Data Sets for Camber Calculations

The next sets of tables depict the range in variability in the camber estimates

- The **coefficient of variation** represents the ratio of the standard deviation to the mean, and it is a useful statistic for comparing the degree of **variation** from one data series to another, even if the means are drastically different from one another. The higher the ***coefficient of variation***, the greater the level of dispersion around the ***mean***. It is generally expressed as a percentage. ... The lower the value of the ***coefficient of variation***, the more precise the estimate.
- **Standard deviation** is a number used to **tell** how measurements for a group are spread out from the average (mean), or expected value. A low **standard deviation** means that most of the numbers are very close to the average. A high **standard deviation** means that the numbers are spread out.

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Research Data Sets for Camber Calculations

AASHTO Type 4 (90 ft.)							
average release camber	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
1.36	0.32	0.23	1.28	1.50	0.08	0.14	0.22
average erection camber	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
2.41	0.18	0.08	2.26	2.66	0.15	0.25	0.40

Camber values in inches

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Research Data Sets for Camber Calculations

BT-54 (110 ft.)							
average release camber	standard deviation	coefficient of variation	low	high	diff btw avg and low	diff btw avg and high	range
2.55	0.24	0.10	1.03	3.08	1.52	0.53	2.05
average erection camber	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
4.52	0.44	0.10	1.84	5.47	2.69	0.95	3.63

Camber values in inches

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Research Data Sets for Camber Calculations

BT-72 (138 ft.)							
average release	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
2.56	0.26	0.10	0.93	3.17	1.63	0.61	2.24
average erection camber	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
4.51	0.47	0.10	1.65	5.60	2.86	1.09	3.95

Camber values in inches

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Research Data Sets for Camber Calculations

FIB-72 (155 ft.)							
average release camber	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
2.93	0.32	0.11	1.43	3.52	1.50	0.59	2.10
average erection camber	standard deviation	coefficient of variation	low	high	diff (low)	diff (high)	range
5.11	0.57	0.11	2.50	6.14	2.61	1.03	3.63

Camber values in inches

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Research Data Sets for Camber Calculations

Observations related to the range in variability in the camber estimates:

1. The range in variability increases when comparing camber at erection compared to camber at release
2. The range in variability increases as the span length and/or number of strands increases

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Research Data Sets for Camber Calculations

The next sets of slides outlines a few additional observations and recommendations related to the research data sets and historical material data

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Research Data Sets for Camber Calculations

Based on historic material data and ratio of f’ci/f’c, develop a recommended design table for values of f’c and f’ci accounting for upper and lower bounds.

- The ratio of the average actual release concrete strength to average 28-day concrete strength provided by the MS concrete girder manufactures was 0.59, 0.71, and 0.74 with the average of the three ratios being 0.68.
- Assume a 10% and 15% lower and upper bound and round the concrete strength to the nearest 100 psi
- The following recommended design table for f’ci and f’c concrete compressive strengths results in a range of the f'ci/f'c ratio of 0.59 to 0.80

concrete release strength f'ci (psi)	concrete 28-day strength f'c (psi)	15% lower bound	10% lower bound	10% upper bound	15% upper bound
4000	5900	5000	5300	6500	6800
4500	6600	5600	6000	7300	7600
5000	7400	6300	6600	8100	8500
5500	8100	6900	7300	8900	9300
6000	8800	7500	7900	9700	10100
6500	9600	8100	8600	10500	11000
7000	10300	8800	9300	11300	11800
7500	11000	9400	9900	12100	12700
8000	11800	10000	10600	12900	13500
8500	12500	10600	11300	13800	14400
9000	13200	11300	11900	14600	15200
9500	14000	11900	12600	15400	16100
10000	14700	12500	13200	16200	16900

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Research Data Sets for Camber Calculations

The next sets of tables sort the various items that influence camber

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Research Data Sets for Camber Calculations

camber estimate values at release and erection for AASHTO Type 4 (90 ft) girders

description	camber at release	description	camber at erection
baseline run	1.502	baseline run	2.657
limestone aggregate	1.383	limestone aggregate	2.447
avg f'ci/f'c, limestone aggregate	1.290	avg f'ci/f'c, limestone aggregate	2.282
avg f'ci/f'c, unit wt 155, gross section	1.281	avg f'ci/f'c, unit wt 155, gross section	2.262

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Research Data Sets for Camber Calculations

camber estimate values at release and erection for BT-54 (110 ft) girders

description	camber at release	description	camber at erection
straight strands with debonding	3.080	straight strands with debonding	5.469
baseline run	2.787	baseline run	4.943
fillet included	2.787	fillet included	4.943
temperature	2.787	temperature	4.943
PCI mult 1.0	2.787	increase fillet thickness	4.943
increase fillet thickness	2.787	gross section	4.857
gross section	2.742	top strand	4.816
top strand	2.718	avg f'ci/f'c	4.626
avg f'ci/f'c	2.608	avg f'ci/f'c, fillet included	4.626
avg f'ci/f'c, fillet included	2.608	unit wt 155	4.591
unit wt 155	2.591	limestone aggregate	4.562
limestone aggregate	2.573	avg f'ci/f'c, unit wt 155	4.296
avg f'ci/f'c, unit wt 155	2.424	high f'ci/f'c	4.291
high f'ci/f'c	2.419	unit wt 160	4.270
unit wt 160	2.412	avg f'ci/f'c, limestone aggregate	4.266
avg f'ci/f'c, limestone aggregate	2.405	baseline run (including prestress losses)	4.186
baseline run (including prestress losses)	2.367	avg f'ci/f'c, unit wt 155, gross section, top strand	4.106
avg f'ci/f'c, unit wt 155, gross section, top strand	2.321	avg f'ci/f'c, unit wt 155, gross section, increase filklet thickness, top strand	4.106
avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness, top strand	2.321	avg f'ci/f'c, unit wt 160	3.992
avg f'ci/f'c, unit wt 160	2.255	avg f'ci/f'c, unit wt 155 (including prestress losses)	3.628
avg f'ci/f'c, unit wt 155 (including prestress losses)	2.054	PCI mult 1.0	2.787

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Research Data Sets for Camber Calculations

camber estimate values at release and erection for BT-72 (138 ft) girders

description	camber at release	description	camber at erection
straight strands with debonding	3.171	straight strands with debonding	5.601
baseline run	2.802	baseline run	4.936
fillet included	2.802	fillet included	4.936
temperature	2.802	temperature	4.936
PCI mult 1.0	2.802	increase fillet thickness	4.936
increase fillet thickness	2.802	gross section	4.858
gross section	2.763	top strand	4.807
top strand	2.730	avg f'ci/f'c	4.628
avg f'ci/f'c	2.627	avg f'ci/f'c, fillet included	4.628
avg f'ci/f'c, fillet included	2.627	limestone aggregate	4.566
limestone aggregate	2.591	unit wt 155	4.553
unit wt 155	2.588	high f'ci/f'c	4.299
high f'ci/f'c	2.440	avg f'ci/f'c, limestone aggregate	4.278
avg f'ci/f'c, limestone aggregate	2.428	avg f'ci/f'c, unit wt 155	4.269
avg f'ci/f'c, unit wt 155	2.426	unit wt 160	4.203
unit wt 160	2.392	baseline run (including prestress losses)	4.103
baseline run (including prestress losses)	2.339	avg f'ci/f'c, unit wt 155, gross section, top strand	4.080
avg f'ci/f'c, unit wt 155, gross section, top strand	2.324	avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness, top strand	4.080
avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness, top strand	2.324	avg f'ci/f'c, unit wt 160	3.938
avg f'ci/f'c, unit wt 160	2.240	avg f'ci/f'c, unit wt 155 (including prestress losses)	3.536
avg f'ci/f'c, unit wt 155 (including prestress losses)	2.018	PCI mult 1.0	2.802

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Research Data Sets for Camber Calculations

camber estimate values at release and erection for FIB-72 (155 ft) girders

description	camber at release	description	camber at erection
baseline run	3.308	baseline run	5.777
fillet included	3.308	fillet included	5.777
temperature	3.308	temperature	5.777
PCI mult 1.0	3.308	increase fillet thickness	5.777
increase fillet thickness	3.308	gross section	5.642
gross section	3.237	avg f'ci/f'c	5.423
avg f'ci/f'c	3.104	avg f'ci/f'c, fillet included	5.423
avg f'ci/f'c, fillet included	3.104	limestone aggregate	5.351
limestone aggregate	3.062	unit wt 155	5.278
unit wt 155	3.027	high f'ci/f'c	5.045
high f'ci/f'c	2.886	avg f'ci/f'c, unit wt 155	4.949
avg f'ci/f'c, unit wt 155	2.837	unit wt 160	4.823
unit wt 160	2.771	avg f'ci/f'c, unit wt 155, gross section	4.822
avg f'ci/f'c, unit wt 155, gross section	2.771	avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness	4.822
avg f'ci/f'c, unit wt 155, gross section, increase fillet thickness	2.771	baseline run (including prestress losses)	4.682
baseline run (including prestress losses)	2.699	avg f'ci/f'c, unit wt 160	4.520
avg f'ci/f'c, unit wt 160	2.596	avg f'ci/f'c, limestone aggregate	4.075
avg f'ci/f'c, limestone aggregate	2.348	avg f'ci/f'c, unit wt 155 (including prestress losses)	3.988
avg f'ci/f'c, unit wt 155 (including prestress losses)	2.303	PCI mult 1.0	3.308

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Research Data Sets for Camber Calculations

Camber Estimate Recommendations:

1. Based on the historic material data provided by the MS concrete girder manufacturers, it is recommended to use adjusted concrete strengths and adjusted modulus of elasticity values based on the average actual strengths, and a unit weight of 155 pcf when calculated camber estimates.
2. The research data sets used an increase value of 1.27 times the design release concrete compressive strength and an increase value of 1.53 times the design 28-day concrete compressive strength for the various runs designated as average f'ci/f'c.
3. Another approach would be to modify the PCI Multipliers to match the camber estimates using the average actual strengths provided by the MS concrete girder manufacturers and a unit weight of 155 pcf. This was not part of the research but an adjusted multiplier of 1.65 applied to both the deflection and camber components at release provide comparable erection cambers (after added dead load deflections) for the BT-54 (Marshall Co.) and BT-72 (Leake Co.) girders.

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Research Data Sets for Camber Calculations

Camber Estimate Recommendations (cont.):

4. Include camber at release estimate in contract plan drawings.
5. MS Concrete Girder Manufacturers to provide camber measurement prior to shipping girder to project site.

Supplemental Research/Data Collection:

The historic material data provided by the MS Concrete Girder Manufacturers is only an initial sample of data; it is recommended to continue to collect and update MDOT's database with additional historic material data including but not limited to:

1) Measurements of camber at release, 28-days, and prior to shipping
2) Girder shipping weight
3) In addition to release and 28-day concrete cylinder breaks, obtain additional breaks at a later timeframe (e.g., 90, 120, 200, or 365 days) to monitor strength gain
4) Record type of aggregate used for the various mix-designs
5) Obtain modulus of elasticity at release and 28-days

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Research Data Sets for Camber Calculations

One item that became apparent in reviewing the camber estimate data for all the items that influence camber was the ratio of the erection camber to release camber. The following table highlights this information for each of the studied girders.

Girder Type	Average ratio of erection to release camber
AASHTO Type 4 (90 ft.)	1.77
BT-54 (110 ft.)	1.77
BT-72 (138 ft.)	1.76
FIB-72 (155 ft.)	1.74
FIB-72 (155 ft.) time dependent analysis	1.65 1.73 (after forming deck)

One anomaly in comparing the historic material camber data provided by the only MS concrete girder manufacturer who provided 28-day camber information for AASHTO Type 4 girders was an average measured 28-day to average measured camber at release = 3.20

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Effect of Increased stiffness on Live Load Distribution Factor

Evaluate the effect of increased stiffness of the girders and live load distribution factor when using the average actual 28-day concrete compressive strengths based on the historical material data provided by the MS concrete girder manufacturers.

The following table presents results for the BT-54 (Marshall Co.) girder:

Live load distribution factor (shear and moment)	Baseline run	Avg f'c	Avg f'c and with haunch/fillet included in composite section
LLV 1-lane	0.674	0.674	0.674
LLV 2-lanes	0.813	0.813	0.813
LLM 1-lane	0.441	0.446 (1% increase)	0.452 (2% increase)
LLM 2-lanes	0.633	0.641 (1% increase)	0.648 (2% increase)

<u>Note:</u>
Baseline f'c = 7.7 ksi
Average f'c = 11.8 ksi

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Effect of Increased stiffness on Live Load Distribution Factor

Evaluate the effect of increased stiffness of the girders and live load distribution factor when using the average actual 28-day concrete compressive strengths based on the historical material data provided by the MS concrete girder manufacturers.

The following table presents results for the BT-72 (Leake Co.) girder:

Live load distribution factor (shear and moment)	Baseline run	Avg f'c	Avg f'c and with haunch/fillet included in composite section
LLV 1-lane	0.654	0.654	0.654
LLV 2-lanes	0.777	0.777	0.777
LLM 1-lane	0.415	0.420 (1% increase)	0.424 (2% increase)
LLM 2-lanes	0.604	0.611 (1% increase)	0.616 (2% increase)

Note:
Baseline f'c = 7.5 ksi
Average f'c = 11.5 ksi

Compare other State DOT Current Camber Estimating Practices to MDOT

The following publications were reviewed to develop a list of items that are related to either the; design of the precast/prestressed concrete girders, camber estimates, and/or information that is placed on the contract plans/drawings for use by the concrete girder manufacturer to produce shop drawings and production of the girders and also for the contractor.

MS	Ref. MDOT Bridge Design Manual and Bulb-T Design Procedure
FL	Ref. FDOT Structures Design Guidelines, Structures Detailing Manual, and Index 20010 Series Prestressed Florida I-beams (Rev. 01/16)
NE	Ref. NDOR Bridge Division-Bridge Office Policies and Procedures, Section 3.3.1-General Prestressed Girder Policy
TX	Ref. TXDOT Bridge Design Manual-LRFD, Section 4-Pretensioned Concrete I Girders
WA	Ref. WSDOT Bridge Design Manual, Standard Specifications, and Design Memorandums
AL	Ref. ALDOT Structural Design Manual-Prestressed Concrete Girder Design Policy, and Bridge Plan Detailing Manual
LA	Ref. LADOTD Bridge Design and Evaluation Manual

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The following items are related to:

- design of precast/prestressed concrete girders
- camber estimates
- information that is placed on the contract plans/drawings

Item	Design	Camber	Plan Dwgs
strand profile (draped, straight, debonding, top strand)	X	X	X
fillet/haunch thickness	X	X	
roadway vertical curve ordinate	X	X	
camber estimating method (PCI Multiplier, time-dependent)	X	X	
dead load distribution	X	X	
girder section properties & strand templates	X	X	X
material properties (f'ci, f'c, E, unit weight, aggregate type)	X	X	X
prestress loss data (time, humidity, curing method)	X	X	X
temperature gradient	X	X	
prestressed beam detail plan sheet information	X	X	X
camber		X	

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The next sets of slides highlight the various guidelines the selected State DOT's have for the items related to the design, camber estimates, manufacture, and construction of precast/prestressed concrete girders.

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strand profile (draped, straight, debonding, top strand)

AASHTO LRFD	Article 5.11.4.3, the number of partially debonded strands should not exceed 25% of the total, the number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row. Exterior strands in each horizontal row shall be fully bonded.
MS	Bridge Division policy is to drape strands instead of debonding. However, debonding may be used with permission from the Bridge Engineer on straight strands in certain situations to reduce stresses in the beam.
	The drape points on a beam are located one tenth (1/10) of the span length from each side of centerline of beam.
	If draped strands are required, a maximum number of twelve (12) draped strands or a minimum of four (4) draped strands are allowed.
	If the beam design used does not require straight strandsd in the top of the beam, #5 bars (or 1/2" strands stressed to 2,000 lbs) must be used in the top of the beam to aid in positioning shear steel.
	MDOT's Bridge Design Manual (Bulb-T Design Procedure, section 1.f) states "Use Debonding if necessary (Debond to nearest tenth points,………..”
	MDOT's current practice is to use 2 ft. debond increments.
NE	Debonding should be the last option and specified per AASHTO LRFD BDS 5.11.4.3.
	For prestressed I-girders with straight strands only, additional U-shaped bars shall be added to girder ends to reduce the stresses due to lifting and handling.
	Exterior and adjacent strands shall not be debonded.
	Maximum debonded length of strand shall be limited to Span/10.
	Maximum number of strands for the NU bottom flange is 58 strands.
	Four additional top strands shall be tensioned to 2 kips/strand and shall not be accounted for in the design.
	Strand hold-down points shall normally be located at 0.4 and 0.6 points of the prestressed girder (however, quarter and third pints are acceptable).
	Consideration should be given to using only straight parallel strands on short prestressed girders < 50 ft. due to the high hold down force required.
TX	Add and drape strands in the order shown on STD DWG IGND.
	Straight strand designs with and without debonding are permitted provided stress and other limits noted below are satisfied.
	Debonded strands must conform to Article 5.9.4.3.3 except as noted below:
	The maximum debonding length is the lesser of (a) one-half the span length minus the maximum development length, (b) 0.2 times the beam length, or (c) 15 ft.
	Not more than 75% of the debonded strands, or 10 strands, whichever is greater, shall have the debonding terminated at any section, where section is defined as an increment (e.g., 3 feet, 6 feet, 9 feet).
	Use hold-down points shown on STD DWG IGD.
	Keep the end position of depressed strands as low as possible so that the position of the strands does not control the release strength.
	Release strength can be controlled by end conditions when the depressed strands have been raised to their highest possible position.
FL	Full length shielding (debonding) of prestressing strands is prohibited.
	Whenever possible, separate debonded strands in all directions by at least one fully bonded strand and debond strands outside of the horizontal limits of the web. The percentage of debonded strands may exceed the recommended 25% limit in LRFD (5.11.4.3), provided that all strands within the horizontal limits of the web are fully bonded. In no case shall the percentage of debonded strands exeed 30%.
	Recent testing of FIB's under FDOT Project BDK75 977-05 indicates that number of longitudinal reinforcement (tension tie) is provided and the fully bonded strands are grouped close to the web. The 30% debonding limitation is a conservative interim limit until further research is completed under NCHRP Project 12-91.
	When the total initial tensioning force of the fully bonded strands required by design exceeds the values shown below (in table), shield additional strands at the end of the beam when possible.
WA	Strand Tensioning: the slope of the strands is limited to a maximum of 6:1 for 0.5" diameter strands and 8:1 for 0.6" diameter strands.
AL	Debonding of prestressed concrete girders shall be as given in AASHTO LRFD Section 5.11.4.3.
Comments:	
	Update MDOT Bridge Design Manual to reflect current practice of using 2 ft. debond increments.

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fillet/haunch thickness

MS	No guidelines on the minimum haunch/fillet thickness.
	Use 2-inch haunch/fillet as added dead load. Do not include the haunch/fillet thickness in the composite section properties.
	Includes an allowable stress design check using a maximum average fillet/haunch thickness.
NE	A one inch minimum haunch at the CL of the girder between the bottom of the bridge deck and top of girder at midspan is required in design.
	The 1 in. haunch is a construction tolerance that also facilitates future deck removal and must be used to calculate girder seat elevations only, and not used for calculation of composite section properties.
TX	2 in. minimum haunch at centerline of bearing.
	Use 0.5 in. minimum at the edge of the girder at mid-span to accommodate the bedding strips for prestressed concrete panels.
	Regardless of calculated value, the absolute minimum haunch at centerline bearing is 2 inc. (increase in 0.25 in. increments).
	If the height of the girder haunch concrete is greater than 3.5 in., the haunch concrete is reinforced with Bars U (for full depth cast-in-place decks) or Bars UP (when Prestressed Concrete Panels, PCPs, are used).
AL	A minimum one inch haunch shall be provided at girder mid-span, calculated at the critical edge of the girder flange. Minimum buildup at girder ends shall take into consideration vertical curve, superelevation transition, or other complex roadway geometry. The build-up should be investigated for each girder line and adjusted as necessary.
LA	Haunch Thickness: 2" at center of support for spans <= 90 ft, and 0.5" at midspan
	Haunch Thickness: 3" at center of support for spans 90 to 120 ft, and 0.5" at midspan
	Haunch Thickness: 4" at center of support for spans >= 120 ft, and 0.5" at midspan
	Average haunch weight is considered in the analysis. Haunch thickness is ignored in the calculation of section properties.
Comments:	
	MDOT's maximum haunch/fillet check does not take into consideration the actual f'ci or the actual f'c, include actual concrete strengths if known.
	MDOT's current guidelines do not consider the increase in the composite section properties (i.e., composite moment of intertia) as a result of increase in the composite section depth/height due to an "under-camber" girder at erection.
	Specify a minimum haunch thickness in MDOT's Bridge Design Manual.
	Consider actual haunch/fillet thickness over the beam length (i.e., due to the upward camber of the girders, the shape of the haunch/fillet thickness varies over the length of the girder in a parabolic profile).
	Therefore, the added thickness increases both the dead load and composite section properties near the ends of the girder, which can effect deflection estimates.
	To simplify the calculation of the haunch/fillet dead load and section properties, an average thickness is typically used/assumed for the haunch/fillet.

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roadway vertical curve ordinate	
NE	Camber and any correction for grade vertical curvature must be considered when determining girder seat elevations and concrete quantities.
	Bridge plans should indicate typical vertical dimensions from the top of the girder flange to grade at supports.
WA	It is desirable to have points of horizontal and vertical curvature and superelevation transitions off the bridge structure as this greatly simplifies
	the geometric requirements on the slab haunch. However, as new bridges are squeezed into the existing infrastructure it is becoming
	more common to have geometric transitions on the bridge structure.
Comments:	
	Fillet/haunch thickness at the ends of the prestressed concrete girders and beam seat elevations should account for the roadway vertical curve ordinate.

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camber estimating method (PCI Multiplier, time-dependent)	
PCI BDM	8.7.1 Multiplier Method: Perhaps the most used method for predicting time-dependent camber of precast, prestressed members is the set of multipliers given in Table 8.7.1-1 (Martin, 1977).
	Elastic deflections caused by the effects of prestressing, beam self-weight, and other dead loads are calculated using conventional elastic analysis techniques. These are
	multiplied by the appropriate factors selected from Table 8.7.1-1 to determine the deflections that occur as a result of time-dependent behavior.
	Suggested multipliers in estimating long-term cambers and deflectioins for typical members include (at erection):
	1.85 to member weight at transfer of prestress and 1.80 to prestress at time of transfer of prestress
MS	PCI Multiplier method with adjusted multipliers
FL	For precast, pretensioned, normal weight concrete members designed as simply supported beams, use LRFD (5.9.5.3), Approximate Estimate
	of Time-Dependent Losses. For all other members use LRFD (5.9.5.4) with a 180-day differential between girder concrete casting and placement of the deck concrete.
	Commentary: The FDOT cannot practically control, nor require the Contractor to control, the construction sequence and materials for simple span precast, prestressed
	beams. To benefit from the use of refined time-dependent analysis, literally every prestressed beam design would have to be re-analyzed using the proper
	construction times, temperature, humidity, material properties, etc. of both the beam and the yet-to-be-cast composite slab.
WA	Erection: This loading typically occurs around 120 days for a normal construction schedule.

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dead load distribution	
MS	Distribute the railing equally over all beams.
TX	Distribute the weight of one railing to no more than three girders, applied to the composite cross section.
FL	Distribute barrier and railing permanent loads per LRFD 4.6.2.2.
AL	Barrier rail load distribution: The barrier rail dead load shall be considered equally distributed across all girders.
	However, the dead load for girder design shall not be less than 25% of a single barrier rail weight.
Comments:	
	Analytical studies on distribution of the railings, which can effect deflections at erection.

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girder section properties & strand templates

MS	Bridge Division utilizes two types of prestressed concrete girders for the design of bridges: AASHTO and Bulb-T shapes
	Use transformed section properties.
	Do not include the haunch/fillet thickness in the composite section properties.
	For all beam types, the lowest row of strands shall be 2 1/2" above the bottom of the beam. The highest strand location is 3" from the top of the beam.
NE	Use either NU or IT girders for precast concrete girder design, even when widening existing structures
	Designers should check deflections when widening with a different shape.
TX	Use section properties given on the prestressed concrete I-girders STD DWGS.
	Composite section properties may be calculated assuming the girder and slab to have the same modulus of elasticity (for girders with f'c < 8.5 ksi).
	Do not include haunch concrete placed on top of the girder when determining section properties. Section properties based on final girder and slab
	modulus of elasticity may also be used; however, this design assumption must be noted on the plans.
	Beam Types TX28, TX34, TX40, TX46, TX54, TX62, TX70 including recommended span lengths for LRFD.
FL	The Florida I Beams and the AASHTO Type II Beam are the Department's standard prestressed concrete I-shapped beams and will be used in the design of all new
	bridges and bridge widenings with I-shaped beams as applicable.
	The Florida U Beams and the Department's standard prestressed concrete U-shapped beams and will be used in the design of all new
	bridges and bridge widenings with U-shaped beams as applicable.
WA	Ref. Design Memorandum on "Transformed Section Properties" dated October 28, 2011
	Transformed section properties shall not be used for design of prestressed girders. Use of gross section properties remains WSDOT's standard methodology for design of
	prestressed girders including prestress losses, camber and the flexural capacity.
	In special cases transformed sections properties may be used for the design of prestressed girders with the approval of the State Bridge Design Engineer. In these cases
	the live load reduction factor at service III limit state load combination shall be as follows: 0.8 when gross section properties are used and 1.0 when transformed section properties are used.
	For pre-tensioned concrete girders, the number of permanent prestressing strands (straight and harped) shall be limited to 100 total 0.6" diameter strands.
AL	The following standard shape AASHTO-PCI type girders shall be used: Type I, Type II, Type III, BT-54, BT-63, and BT-72, as well as solid and voided slab beams.
	Modifications of these girders may be used under special circumstances (such as clearance problems or freeboard limitiations) when approved by the State Bridge Engineer.
	The transformed area of bonded reinforcement shall not be included in the calculations of section properties for prestressed concrete girders.
LA	Louisiana Girder (LG) types, LG-25, LG-36, LG-45, LG-54, LG-63, LG-72, and LG-78, shall be the standard precast prestressed concrete (PPC) girders used for new
	construction and bridge widening. Quad Beam, AASHTO Type II, III, IV, BT-72 and BT-78 are allowed for bridge rehabilitation projects with the approval of the
	Bridge Design Engineer Administrator.
	Average haunch weight is considered in the analysis. Haunch thickness is ignored in the calculation of section properties.
	The girder spacing shall not exceed 12.0 feet center-to-center for I-shaped girders.
Comments:	
	MDOT's Bridge Design Manual currently does not include the FL Bulb-T beam section properties; recommend including the FL Bulb-T beams.
	Include strand templates in design standards.

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material properties (f'ci, f'c, E, unit weight, aggregate type)

PCI BDM	2.4.7.4 Unit Weight: In the design of reinforced or prestressed concrete structures, unit weight for design is generally taken as 0.005 kcf greater than density of plain concrete.
	However, for members with large quantities of prestressing strand, a higher amount may be more appropriate.
	11.2.1.4 Compressive Strength at Transfer: Higher concrete compressive strength at transfer allows a beam to contain more strands and increases the capacility of the beam to resist
	design loads. However, the availability of high compressive strength concrete at transfer varies throughout the country. Strength at transfer should not be higher than required for the span
	being designed because strengths in excess of 5.5 to 6.5 ksi may increase the required duraction of the production cycle at the manufacturing plant. This would increase the cost
	of the beams. Early compressive strength is influenced by local materials and sometimes by production facilities and regional practices. Producers should be consulted about
	available concrete strengths before beginning design.
MS	The 28-day compressive strength for beam concrete shall be 5,000 psi. Strengths of 5,500 psi and 6,000 psi can be used as required by design.
NE	Girder compressive strengths at final f'c (8, 10, 12, and 15 ksi). 8, 10, and 12 ksi include the use of 0.6 inch prestressing strands.
	12 and 15 ksi include the use of 0.7 inch prestressing strands.
	Girder compressive strengths at release = 0.75f'c (6, 7.5, 9, and 11.25 ksi)
TX	Use class H concrete with a minimum f'ci = 4.0 ksi and f'c = 5.0 ksi and a maximum f'ci = 6.0 ksi and f'c = 8.5 ksi
	Use an effective strand stress after release of 0.75 fpu - ES losses
WA	Ref. Design Memorandum on "Unit Weight of Concrete" dated June 1, 2010
	This memorandum defines the unit weight of concretes for dead load and modulus of elasticity calculations.
	For normal weight concrete (precast pretensioned or post-tensioned spliced girders); use 155 pcf for the modulus of elasticity (plain concrete) and 165 pcf dead load (with reinforcement).
	Background: The unit weight of precast concrete girders is generally taken as 10 pcf greater than the unit weight of plain concrete due to the weight of reinforcement and strands.
	Prestressed Concrete Girders: Nominal 28-day concrete strength (f'c) for prestressed concrete girders is 7.0 ksi. Where higher strengths would eliminate a line of girders, a maximum
	of 10.0 ksi can be specified.
	The minimum concrete comprssive strength at release (f'ci) for each prestressed concrete girder shall be shown in the plans. For high strength concrete, the compressive strength
	at release shall be limited to 7.5 ksi. Release strengths of up to 8.5 ksi can be achieved with extended curing for special circumstances.
	Modulus of Elasticity: The modulus of elasticity shall be determined as specified in AASHTO LRFD Section 5.4.2.4. For calculation of the modulus of elasticity, the unit weight of plain
	concrete (wc) shall be taken as 0.155 kcf for prestressed concrete girders and 0.150 kcf for normal-weight concrete. The correction factor (K1) shall normally be taken as 1.0.
	Shrinkage and Creep: Shrinkage and creep shall be calculated in accordance with AASHTO LRFD Section 5.4.2.3. The relative humidy, H, may be taken as 75 percent for standard
	conditions. The maturity of concrete, t, may be taken as 2,000 days for standard conditions. The volume-to-surface ratio, V/S, is given in Table 5.6.1-1 for standard WSDOT
	prestressed concrete girders.
	In determining the maturity of concrete at initial loading, ti, one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.
	Ref. Technical Report WA-RD 669.1
	The camber was found to depend on the elastic modulus of the concrete, its creep coefficient, and the use of the prestress losses in the calculation of the creep camber.
	To achieve the best match with the measured cambers, the AASHTO recommended values for the elastic modulus and the creep coefficient had to be multiplied by
	adjustment factors and the prestress losses had to be taken into account when computing the creep component of camber.
	Recommendations for practice: For deflection calculations, increase the specified concrete strengths by 10 percent at release and 25 percent at 28 days.
	Use 1.15 times the AASHTO LRFD 2006 equation for predicting the concrete elastic modulus (Ec) for a given concrete strength. An alternative would be to adopt the
	methods recommended by NCHRP or CEB-FIP.
AL	Prestressed Concrete: The following values of f'c shall be used for prestressed concrete structures: girders 5.0 to 8.0 ksi
	(higher strengths will require prior approval by the State Bridge Engineer)
LA	Girder Concrete: f'ci = 6.5 ksi, f'c = 8.5 ksi, density for dead load = 155 pcf, density for modulus of elasticity = 148.5 pcf
	Girder Concrete: f'ci = 7.5 ksi, f'c = 10.0 ksi, density for dead load = 155 pcf, density for modulus of elasticity = 150 pcf
	Prestress Losses: Use gross section and include elastic gains.

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prestress loss data (time, humidity, curing method)	
AL	For calculating losses, use the AASHTO LRFD Approximate Method, neglecting gains. The following values shall be used for calculating losses:
	Time at release: 0.75 days, Age of deck placement: 60 days, final age: 27500 days, relative humidity: 75%
LA	Prestress Losses: Use gross section and include elastic gains.

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temperature gradient	
FL	Include the effects of Temperature Gradient in the design of continuous concrete superstructures only.
	The vertical temperature gradient shall be taken as shown in LRFD Fig. 3.12.3-2.

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prestressed beam detail plan sheet information

MS Camber limit (at release only).
Prestressed concrete girder design information is shown on the contract plans.
Prestress requirements table is shown on the contract plans as follows:

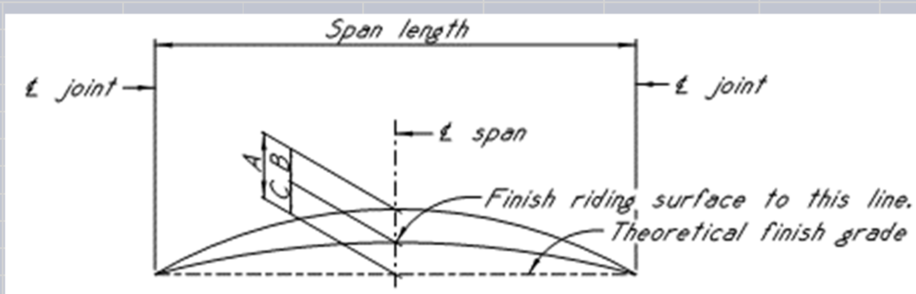
LR indicates low-relaxation strands

PRESTRESS REQUIREMENTS

For deflection diagram, see Misc. Span Details per Sheet No. 22

Strand type	Minimum breaking strength	Initial tension	Required number and location of strands						Centroid for total number of strands (in.)		Distance from ℄ span to hold-down point	Camber limits	Deflection diagram			Minimum concrete strength at time of release (psi)
			Total number strands	Straight strands		Draped strands			At ℄ span	At beam end			A	B	C	
				Number strands	Centroid (in.)	Number strands	Centroid (in.)									
	lbs/strand	lbs/strand					℄ span	Beam end								
0.6" ϕ270 K-LR	45,000	△1 43,940	36	26	4.19	10	6.50	65.00	4.83	21.08	14'-0"	0 to $5\frac{3}{4}$"	$5\frac{1}{16}$"	$2\frac{5}{16}$"	$2\frac{3}{4}$"	6000

A deflection diagram is shown on the contract plans as follows:



DEFLECTION DIAGRAM

A = total recommended allowance for deflection.
B = estimated deflection due to dead load of slab & rail.
C = A-B = net initial camber in riding surface, which includes an allowance or creep.

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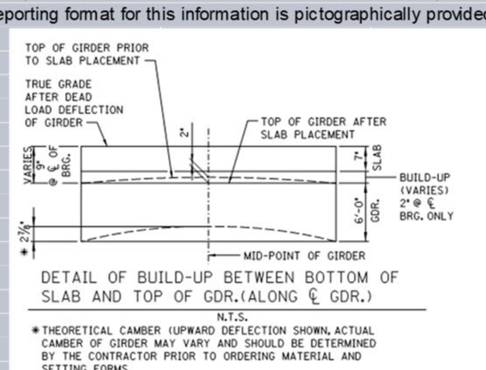
TX	Show predicted slab deflections on the plans although field experience indicates actual deflections are generally less than predicted.
	Include dead load deflection diagram and table of section depths.
FL	Report elastic and time dependent shortening effects (DIM R) at mid-height of the beam at 120 days. The average of the calculated values for the top and bottom
	of the beam may be used.
WS	The "D" dimension is the computed girder deflection at midspan (positive upward) immediately prior to deck slab placement.
	Standard Specifications Section 6-02.3(25)K defines two levels of girder camber at the time the deck concrete is placed, denoted D at 40 Days and D at 120 Days.
	They shall be shown in the plans to provide the contractor with lower and upper bounds of camber that can be anticipated in the field.
	D at 120 Days is the upper bound of expected camber range at a girder age of 120 days after the release of prestress and is primarity intended to mitigate
	interference between the top of the cambered girder and the placement of concrete deck reinforcement. It is also used to calculate the "A" dimension at the girder ends.
	The age of 120 days was chosen because data has shwon that additional camber growth after this age is negligible.
	D at 40 Days is the lower bound of expected camber range at a girder age of 40 days (30 days after the earliest allowable girder shipping age of 10 days).
	To match the profile grade, girders with too little camber require an increased volume of haunch concrete along the girder length. For girders with large flange widths,
	such as the WF series, this can add up to significant quantities of additional concrete for a large deck placement. Thus, the lower bound of camber allows the
	contractor to assess the risk of increased conrete quantities and mitigates claims for additional material.
	Girder stirrups shall all extend at least 5 in. from the top of the girder, but typically no more than the deck thickness minus 2.5 in.
	Hat bars shall be the same size as the girder stirrups. Prebent stirrups may be used with "hat bar" stirrup extensions. Details shall conform to Figure 5.6.2-7.
	Hat bars may be omitted at locations where girder stirrups project at least 3 in. above the bottom of the transverse bar in the bottom mat of the bridge deck.
	Computation of "A" Dimension: The distance from the top of the bridge deck to the top of the girder at centerline bearing at centerline of girder is represented by
	the "A" Dimension. It is calculated in accordance with the guidance of Appendix 5-B1. This ensures that adequate allowance will be made for excess camber,
	transverse deck slopes, vertical and horizontal curvatures. Where temporary prestress strands at top of girder are used to control the girder stresses due to
	shipping and handling, the "A" dimension must be adjusted accordingly.
	Stirrup Length and Precast Deck Leveling Bolt Considerations: For bridges on crown vertical curves, the haunch depth can become excessive to the point where
	the girder and diaphragm stirrups are too short to bend into the proper position. Similary the length of leveling bolts in precast deck panels may need adjustment.
	Stirrup lengths are described as a function of "A" on the standard girder sheets. For example, the G1 and G2 bars of a WF74G girder are 6'-5" + "A" in length.
	For this reason, the stirrups are always long enough at the ends of the girders. Problems occur when the haunch depth increases along the length of the girder to
	accommodate crown vertical curves and superelevation transitions. If the haunch depth along the girder exceeds "A" by more than 2", an adjustment must be made.
AL	Build-up over Top of Prestressed Concrete Girders: The following shall be shown on the prestress camber diagram for the ends of the prestressed concrete girder:
	girder depth, haunch thickness, deck thickness, total deck plus haunch thickness, theoritical camber, dead load deflection
	The reporting format for this information is pictographically provided in the Bridge Plan Detailing Manual.



	A minimum one inch haunch shall be provided at girder mid-span, calculated at the critical edge of the girder flange. Minimum buildup at girder ends
	shall take into consideration vertical curve, superelevation transition, or other complex roadway geometry. The build-up should be investigated for each girder
	line and adjusted as necessary.
	For prestressed concrete members, specific concrete strengths used for design and specified for fabrication shall be stated on the contract drawings.

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LA

Strand pattern details showing strand layouts, number and spacing of strands, concrete cover and edge clearances, and layout of all mild reinforcing steel shall be shown in contract plans.

All girder related design data shall be shown in a girder data table. Refer to LG girder design aids in Part III, Chapter 1 for a girder data table template.

The Camber Data Table shall be included in contract plan.

CAMBER DATA TABLE

SPAN NO.	GIRDER DESIGNATION	DESIGN DATA				FIELD MEASURED DATA *									* DATE OF GIRDER CASTING	* DATE OF RISER POUR
		C1 (IN.)	C2 (IN.)	C3 (IN.)	D5 (IN.)	MC1 (IN.)	MC1a (IN.)	MC2 (IN.)	fb1 (KSI)	fb1a (KSI)	fb2 (KSI)	Eb1 (KSI)	Eb1a (KSI)	Eb2 (KSI)		
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Comments:

Refer to PCI's Recommendations for Camber Tolerances (at release only).

None of the other State DOT's calculate a camber limit.

Consider including camber at release estimate.

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Compare other State DOT Current Camber Estimating Practices to MDOT

camber	
PCI BDM	Refer to section 3.4.2.6 Camber for additional information.
PCI BDM	3.4.6 Tolerances: Good design and detailing practices for precast components and connections always consider allowable tolerances for fabrication, erection, and interfacing
	field construction. PCI Manual 116 lists industry standard tolerances for typical precast concrete bridge members. Details allowing generous tolerances usually result in economies
	during construction, while extremely stringent tolerances can be very expensive and is some cases, may not be achievable. Designers should consult local producers when considering
	tolerances that are tighter than the industry standards.
MS	Calculates camber limit (at release only).
NE	Final deflection due to effective prestress plus dead loads shall be an upward camber.
	No downward camber at 30 days is permitted.
	Camber shall not be considered in the vertical clearance determination under a bridge.
	For the purposes of determining the vertical clearances, the bottom of the girder shall be considered a straight line between the bearings.
	All girder bridge plans shall have deflections calculated at the span tenth points and labeled.
TX	Compute deflections due to slab weight and composite dead loads assuming the girder and slab to have the same modulus of elasticity.
	Assume Ec = 5,000 ksi for girders with f'c < 8.5 ksi.
	Use the deflection due to slab weight only times 0.8 for calculating haunch depth.
	A calculated positive (upward) camber is required after application of all permanent (dead) loads.
FL	Stress and camber calculations for the design of simple span, pretensioned components must be based upon the use of transformed section properties.
	Unless otherwise required as a design parameter, beam camber for computing the build-up shown on the plans must be based on 120-day old beam concrete.
	On the build-up detail, show the age of beam concrete used for camber calculations as well as the value of camber due to prestressing minus the dead load deflection of the beam.
	Consider the effects of horizontal curvature with bridge deck cross slope when determining the minimum buildup over the tip of the inside flange.
	Commentary: In the past, the FDOT has experienced significant deck construction problems associated with excessive prestressed, pretensioned beam camber.
	The use of straight strand beam designs, higher strength materials permitting longer spans, stage construction, long storage periods, improperly placed dunnage, and
	construction delays are some of the factors that have contributed to camber growth. Actual camber at the time of casting the deck
	equal 2 to 3 times the intitial camber at release is no uncommon.
	Design pretensioned beams so that the theoretical design camber at the end of construction is positive (upward) after all non-composite and composite dead loads are applied.
	Camber variability of prestressed components is affected by a number of items such as; aggregates, curing conditions, strand patterns, casting/detensioning temperatures,
	design strength versus actual strength of concrete, weekday versus weekend and holiday casting cycles, support conditions during storage, hauling and handling, and
	component age of time of loading.
	Commentary: Camber variability is common. Requiring steam curing or creep testing of the actual concrete mixes used may improve camber predictions; however, fairly
	large variations in camber may still exist due to other factors as those listed.
WA	Accurate predictions of deflections are difficult to determine, since modulus of elasticity of concrete, Ec, varies with stress and age of concrete. Also, the effects of creep
	on deflections are difficult to estimate. For practical purposes, an accuracy of 10 to 20 percent is often sufficient.
AL	Calculation of camber due to prestress prior to pouring the bridge deck shall be based on a 60-day interval between release of the strand and erection of the girder.
LA	Camber and Deflection: Use PCI Multiplier method. The PCI multiplier method is adopted to calculate the estimated camber at erection
	provided that initial camber due to prestress and deflection due to girder self-weight at transfer are calculated separately. The use of PCI multipliers has shown to
	give reasonable estimates for camber at the time of erection.
	For prestressed girder projects in which the contractor elects to fabricate all the girders at the same time but girder placement will extend months after casting
	(such as for phased construction or very large projects), the contractor must be responsible for camber growth.
	The Camber Data Table shown below shall be included in contract plan.
	The camber design data (C1, C2, C3 and D5) are provided by the EOR. The field measured data (MC1, MC1a, MC2, fb1, fb1a, fb2, Eb1, Eb1a and Eb2) and the dates
	of girder casting and riser pour shall be recorded by the contractor in the Camber Data Table. The Camber Data Table shall be submitted to the EOR for review
	at least 14 days prior to riser pour. When field measured MC1 or MC2 differ more than 1/2" (+ or -) from the estimated "C1" or "C2", the contractor shall notify
	the EOR immediately to investigate corrective measures, such as modify risers and/or roadway profile, etc.
Comments:	
	Refer to PCI's Recommendations for Camber Tolerances (at release only).
	None of the other State DOT's calculate a camber limit.

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Camber
PCI Bridge Design Manual

The next sets of slides highlights the camber information contained in PCI's Bridge Design Manual, Section 3.4.2.6.

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Camber
PCI Bridge Design Manual

3.4.2.6 Camber

Camber is defined as the net upward deflection of an eccentrically prestressed member due to the combined member dead load moment and eccentricity of the prestress force. Camber can increase or decrease with time, depending on the level of prestress and sustained loads. A typical camber versus time graph is shown in **Figure 3.4.2.6-1**. Camber can be predicted with relative accuracy at the time of initial prestress, but the prediction of long-term camber should be considered an approximation.

Measuring and recording actual initial camber, and comparing results to the theoretically computed value, is valuable in quantifying the consistency of production, assumed material properties as compared to actual, and quality control. Small variations in initial camber indicate good consistency in tensioning and concreting procedures, while large camber variations may represent poor consistency. Camber that is significantly lower than expected can indicate inadequate tensioning, improper quantity or placement of strands, or loss of bond between concrete and strand (excessive strand slip). Low camber can also result from concrete transfer strength that is higher than anticipated, such as in members that remain in the form over a weekend prior to initial prestress. Camber significantly higher than expected can result from low concrete strength, excessive force in the strands, or improper quantity or placement of strands.

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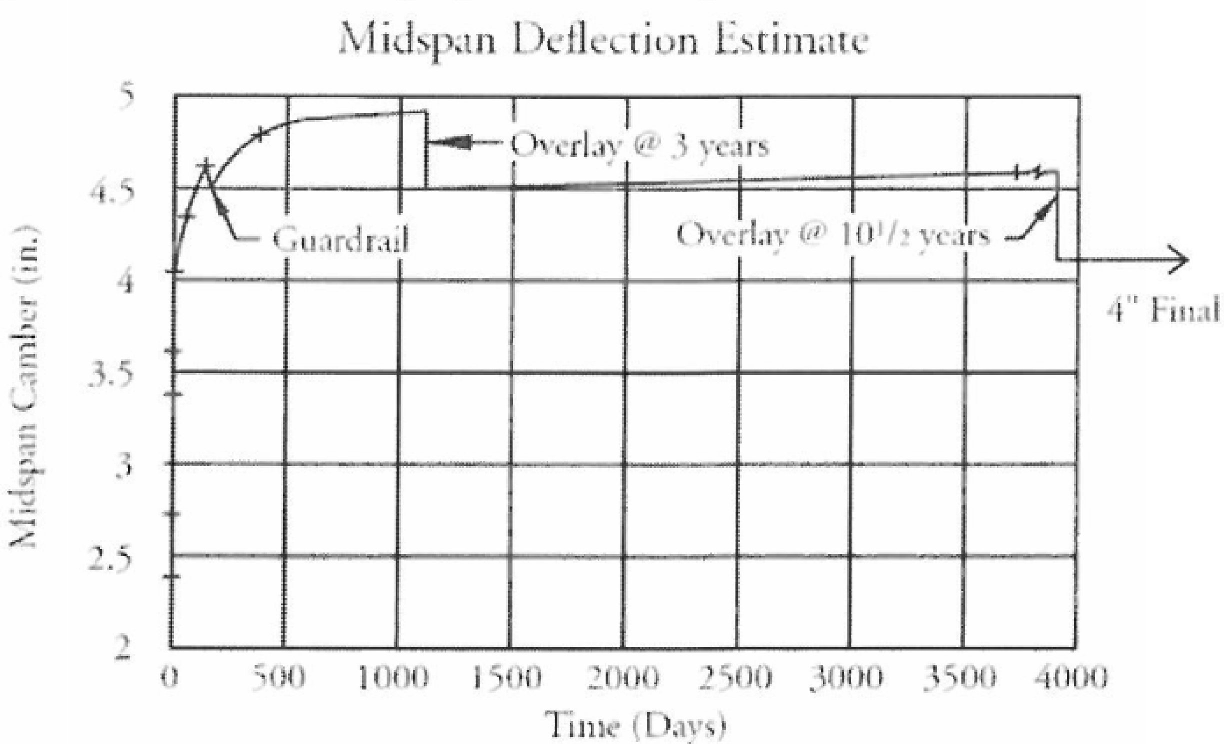
Predicting camber variability should be a mean (average) value, preferably with an indication of the range of variability but it is highly influenced by the modulus of elasticity. The variations in camber become more significant as the use of high-strength concrete, longer spans, and more heavily prestressed concrete beams continues to increase. The variability from the calculated value can be assumed to be ± 50%. See Tadros, et al., 2011.

Figure 3.4.2.6-1
Typical Time-Camber Graph (Deck Bulb-Tee)



Camber
PCI Bridge Design Manual

3.4.2.6.1 Measuring Camber

The PCI Manual 116 requires measurement of camber to be taken on all members produced from the first cast on a new or unusual bed layout, and on no less than 25% of all other members produced each day. This measurement is to be taken as soon as possible after initial prestress, but not to exceed 72 hours after transfer of the prestressing force. The elapsed time to measurement of camber after transfer should remain consistent for a plant.

Several methods are used to measure initial camber. The simplest is to measure the upward deflection at midspan immediately after transfer, but before the member is lifted from the form, using the form soffit as the point of reference. Some products, such as stemmed members, are not easily accessible for this measurement. Once a product is stripped and moved to the yard, camber can be measured with a stringline, laser level, or a surveying level and rod. Camber measurements should be taken to a well defined point on the member, such as the top corner of a bottom flange, and not to an inconsistent surface, such as an intentionally roughened top flange.

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Camber
PCI Bridge Design Manual

3.4.2.6.2 Thermal Influences on Camber

Camber measurements should not be taken when the member is influenced by temporary differences in surface temperature. On a sunny day, the top of the top flange can be significantly warmer than the rest of the member, leading to a temporary increase in camber. Camber readings under these conditions will be misleading.

3.4.2.6.3 Mitigation of Camber Growth

Practical methods for mitigating camber growth are limited. As discussed in **Section 3.3.8.1**, eccentrically pretensioned flexural members should be stored on dunnage located as close to the ends as possible (or final support locations for members with cantilevers). Moving the dunnage away from the ends toward midspan reduces the dead load deflection, and can lead to increased permanent upward deflection. Adding a load to a member in storage to reduce long-term creep and camber is generally not feasible. Control is best accomplished by scheduling production closer to erection or, if not possible, by allowing for increased camber in the design and detailing of the structure. In an unusual situation where camber is not adequate, it can be increased by moving the dunnage in from the ends during storage.

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Beam Camber (Tolerances)

The next sets of slides discuss tolerances for beam camber contained in PCI's Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (Fourth Edition) MNL-116-99 and the most recent recommendations made by PCI's Committee on Bridges-Camber FAST Team who evaluated the current PCI tolerances and made recommendations for PCI tolerances with respect to predicted camber at time of prestress transfer.

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Beam Camber (Tolerances)

- PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (Fourth Edition) MNL-116-99
- Appendix B-10 I-Beam (Girder) or Bulb-Tee Girder

g = Camber Variation from Design Camber

- +/- 1/8 in. per 10 ft.
- maximum of +/- 1/2 in. up to 80 ft.
- maximum of +/- 1 in. for length greater than 80 ft.

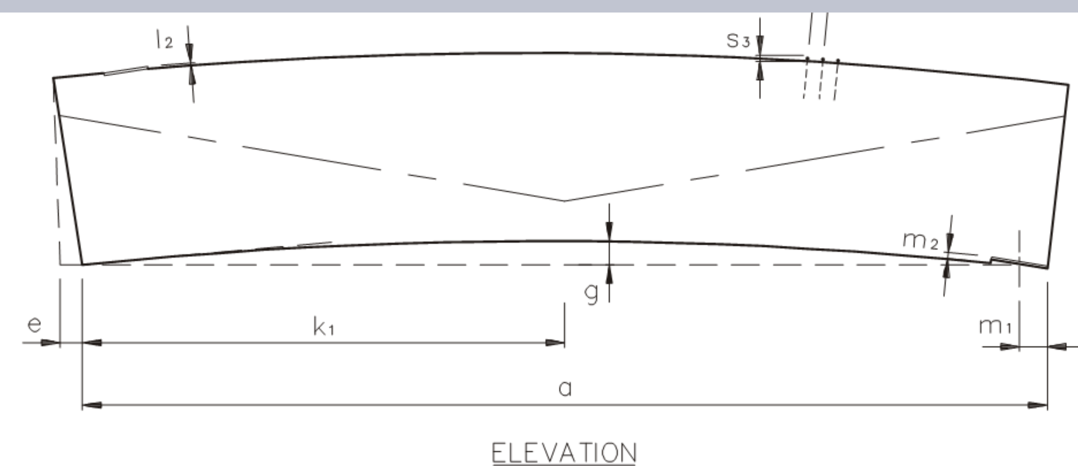

Figure B-10 I-Beam (Girder) or Bulb-Tee Girder

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Beam Camber (Tolerances)

- In 2012 PCI's Committee on Bridges-Camber FAST Team evaluated the current PCI tolerances and made recommendations for PCI tolerances with respect to predicted camber at time of prestress transfer

- Camber is a function of girder cross-section, prestressing force, strand location, concrete properties, girder age, and environmental factors.
- Each of these attributes have variability both within a plant and among plants.
- This variability is independent of the camber prediction method used.

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Beam Camber (Tolerances)

- Camber – The deflection that occurs in prestressed concrete members due to the net bending resulting from the eccentricity of the prestress force
- Prediction of camber is based on empirical formulas
- Accuracy of these estimated values decreases with time
- Measurement of camber from comparison of predicted design values should be completed within 72 hrs of transfer of prestress

- Temperature variation across a member section can have a significant impact on the measured camber.
- Camber should be evaluated under conditions that minimize the effect of temperature variation due to solar radiation, such as early in the morning.

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Beam Camber (Tolerances)

<u>FAST Team Recommendations:</u>

1. Revise ‘g” dimension in Appendix B of PCI MNL-116

g = Camber Variation from Design Camber Within 72 Hours of Release

- + 1/8 in. per ten feet, up to a maximum of 1.50 in.
- - 1/8 inch per ten feet with no lower bound

2. Add a footnote in Appendix B of PCI MNL-116

Out of tolerance camber should not be a sole cause for rejection.

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Remaining Tasks

Interim Report

- Develop Interim Report for submission and review by MDOT

- Organize research tasks into the Interim Report
- Present the research information performed
- Coordinate with TAC on the outcomes, observations, and recommendations
- Address additional research and/or data collection needs to supplement research results
- Coordinate with MDOT/TAC on logistics of research project technology transfer information

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Final Report and last TAC meeting

- ❑ Address review comments to Interim Report
- ❑ Coordinate last TAC meeting
- ❑ Submit Final Report

- After MDOT/TAC's review and comment on the Interim Report, address review comments and prepare the Final Report for submission
- Coordinate last TAC meeting
- Address final review comments to Final Report

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A-3

February 11, 2019

MDOT
Best Practices for Estimating Camber of Bulb T and Florida Girders
State Study No. 288

Technical Advisory Committee (TAC) meeting

February 11, 2019

Jackson, MS

David Tomley, P.E. (Assistant Project Manager & Senior Structural Engineer)

Thompson Engineering

Mobile, AL

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Technical Advisory Committee (TAC) meeting agenda

1. Provide a recap (outcome/insights) to research tasks
   - Literature Review
   - Historic Material Data
   - Other State DOT and MDOT Current Practices
   - Camber Data Sets
   - Conclusions, Recommendations, Implementation Plan
2. Report
   - Review Interim Report comments and additional revisions
   - Additional publications to supplement the research
     - Louisiana Transportation Research Center (LTRC), Technology Transfer Program Final Reports 310, 382, and 395, "Use of High Performance, High Strength Concrete (HPC) Bulb-Tee Girders Saves Millions on I-10 Twin Span Bridge in New Orleans District, Applied Research Pays Dividends: $14,688,267 Saved on One Project"
     - ASPIRE Winter 2019, "AASHTO LRFD Bridge Design Specifications: Stability of Pretensioned Concrete Girders by Dr. Oguzhan Bayrak, University of Texas at Austin
       - "Effects of temporary strands on camber calculations and prestress losses are also acknowledge."

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Research Tasks recap

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Literature Review

26 documents were reviewed to gain insight into the various aspects associated with the research topic and draw from the previous knowledge-base of information related to estimating camber

- Refer to copies of the literature review documents and summary of items related to the research

- Provided insight to the various research/publications and additional topics related to estimating beam camber
- Provided list of common aspects/topics to the research topic which were carried further part of the research for additional investigation

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Historic Material Data

- 3 Mississippi Concrete Girder Manufacturers were collaborated with to provide historic material data consisting of concrete compressive strengths and beam camber
- Historic material data provided a basis for further research

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Other State DOT and MDOT Current Practices

- Other State DOTs (FL, NE, TX, WA, AL, and LA) and MDOT's current practices related to the research topic were reviewed
- Guidelines were categorized into several areas and comparisons made
  - design of precast/prestressed concrete girders
  - camber estimates
  - information that is placed on the contract plans/drawings

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Camber Data Sets

- Based on the literature review, historic material data, and comparison of other State DOT and MDOT current practices for estimating camber, a list of items anticipated to influence camber was made
- Various data sets were developed to investigate camber estimates incorporating the items thought to influence camber

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Conclusions

The following conclusions were drawn from the research tasks:

1. The documents researched emphasized how variable the items are that influence camber and thus camber calculations should be considered an estimate due to the variability in the items that influence beam camber.
2. Historic material data provided by the three Mississippi Concrete Girder Manufacturers provided actual data specific to concrete compressive strengths and beam camber.
3. The various items that were included in the camber data sets did change the values of the camber estimates both individually and more so in combination with each other.
4. The under-camber girders that MDOT has experienced on several projects was validated by the majority of the items included in the camber data sets; which indicated the estimated camber using the research items that influence camber to be less than the current method to estimate camber (i.e., baseline estimated camber).

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Conclusions

5. The Camber data sets provided insight to the range in variability in the camber estimates.
6. The Camber data sets provided insight to the variation and magnitude of camber estimates for various concrete girder types and girder length ranges selected in comparison to the current (i.e., baseline) estimates for camber.
    - Actual average values for the concrete compressive strengths at release and 28-days provided by the Mississippi Concrete Girder Manufacturers were included in the camber data sets and assisted with developing recommendations for estimating camber.
7. Other State DOT current practices provided a comparison with Florida, Nebraska, Texas, Washington, Alabama, and Louisiana current practices related to beam camber. For the most part, the selected other State DOT's have similar practices as MDOT's with a few minor differences as noted in Appendix C.

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Conclusions

8. The effect on the live load moment distribution factor when using the average historic concrete compressive strength was minimal with a 1-2% increase. The increase in the concrete compressive strength when using the average historic concrete compressive strength did not change the live load shear distribution factor.
9. Based on the research findings and recommendations, the following benefits are realized:
   - improved material property versus strength expectation
   - minimize the difference between the estimated and actual beam cambers which will; reduce construction delays and reduce added construction costs to MDOT projects, reduce design and/or functional modifications to MDOT projects, and improve the ride

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Recommendations

1. Design Table for f'ci/f'c
2. Minimum Haunch/Fillet Thickness
3. Estimated Camber at Release
4. Camber Limits
5. Temperature Gradient
6. Prestress Loss Data
7. Florida Bulb-T Beam Section Properties
8. Transformed Section Properties
9. Roadway Vertical Curve Ordinate
10. Debonding Increments
11. Draped vs. Straight Strands
12. Camber Measurements
13. Girder Shipping Weight
14. Additional Concrete Cylinder Breaks
15. Aggregate Types
16. Actual/Measured Modulus of Elasticity
17. Increased Concrete Strengths
18. Estimating Camber

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Implementation Plan

1. Disseminate the research findings within MDOT including the Bridge Design Division, Construction Division, Materials Division, and District staff. Update MDOT research library to include final report.
2. Share research findings with contractors, concrete girder manufacturers, consultants, and industry and discuss whether MDOT will change their current methods for estimating camber.
3. Update MDOT's Bridge Design Manual with reference to the research for best practices for estimating camber of Bulb T and Florida girders.
4. Coordinate with Mississippi Concrete Girder Manufacturers and Contractors any new information required to be provided and/or collected in reference to the research.
5. Continue to collect historical material and beam camber data and update MDOT's knowledge-base/database accordingly.
6. Develop Tech Brief that summarizes the research.
7. Pursue publication.

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Report

- Review Interim Report comments and additional revisions
- Additional publications to supplement the research
  - Louisiana Transportation Research Center (LTRC), Technology Transfer Program Final Reports 310, 382, and 395, “Use of High Performance, High Strength Concrete (HPC) Bulb-Tee Girders Saves Millions on I-10 Twin Span Bridge in New Orleans District, Applied Research Pays Dividends: $14,688,267 Saved on One Project”
  - ASPIRE Winter 2019, “AASHTO LRFD Bridge Design Specifications: Stability of Pretensioned Concrete Girders by Dr. Oguzhan Bayrak, University of Texas at Austin
    - “Effects of temporary strands on camber calculations and prestress losses are also acknowledge.”

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APPENDIX B

Literature Review

B-1

Literature Review Document

Aspects that can effect prestress losses and/or beam camber and/or related topics

1. Curing

   "Study of Prestress Losses Conducted by Lehigh University"
   Ti Huang
   PCI Journal/September-October 1982
   p.56

   The effect of elevated temperature during the curing period on prestress losses in pretensioned strands was also studied. There was concern that as temperature decreases after curing, the steel stress loss due to thermal expansion may not be fully recovered. During the fabrication of the experimental bridge beams for the field study, measurements were made on the strand tension until transfer and on concrete compressive strains at transfer. The results indicated virtually full recovery of the thermal loss of strand stress. To further verify this finding, small specimens were fabricated and tested to decompression immediately after transfer. These tests also indicated nearly complete recovery of the thermal loss of strand stress.

2. Concrete Compressive Strength-gain and high strength concrete
   "High Strength Prestressed Concrete Bridge Girder Performance"
   Charles W. Dolan, Ph.D., P.E., Craig A. Ballinger, P.E., Robert W. LaFraugh, P.E.
   PCI Journal/May-June 1993
   pgs 88-89

Compliance with the release strength and economic requirements for daily production resulted in an actual 28-day concrete strength well above the specified design strength. The actual attained strength in many girders is in the range of high strength concrete. This higher strength allows greater design flexibility and has been used by several states and provinces to increase the span length or to reduce the number of girders in a bridge.

p.89

High strength concrete is not clearly defined and is dependent upon the production capacities and practices in various parts of North America. However, high strength concrete may be defined as any concrete with a specified 28-day strength over 8000 psi (55 MPa). Bridge girder design and construction often consider any concrete in excess of 6000 psi (41 MPa) as the transition point to high strength concrete. The definition of high strength concrete in the precast, prestressed concrete industry is also influenced by the need for a high early strength at transfer of the strand prestress to the girder concrete.

The maximum attainable strength of precast concrete bridge girders varies with the quality of available aggregates and with plant production techniques. This paper examines the development of high strength concrete girders from the perspective of the industry capability to manufacture high quality and high strength members.

Twelve to 16 hours are needed for curing the concrete prior to transfer of the prestressing force to the girder.

The amount of time available for curing prior to the application of the prestressing force has a marked influence on the type of cement and the strength of the concrete used in the girder. Extended initial curing time reduces the energy requirements for accelerated curing and can lower production costs.

The higher the initial prestressing force is specified, the greater are the demands on the initial concrete strength. The initial concrete strength may be "high" if it is high in terms of the final specified concrete strength, e.g., 70 percent of the final strength, or if it is high in absolute value, i.e., $f'_{ci} > 5000$ psi (34 MPa) at 18 to 20 hours.

Production operations using a one-day work cycle will require a concrete that gains strength very rapidly in the first few hours after casting. The strength gain is obtained by using more cement, Type III cement, lower water-cement ratios, accelerating the concrete cure by using admixtures, and by heating the concrete after the initial set. Multiple-day production cycles can use Type I cement and ambient curing temperature more effectively and may not have to resort to accelerated curing.

p.90

Table d1 summarizes both the release strength and the design strength for several states, and provides a projected 28-day strength. The projected 28-day strength is based on the assumption that the one-day release strength is 50 percent of the 28-day strength. The one-day

value of 50 percent is high for normally cured concrete, but is reasonable for accelerated curing used in the precast concrete industry.
Actual available design strength must allow for the properties and strength variation within the actual concrete mix.
Several important issues are raised in Table 1. First, approximately 4000 psi (28 MPa) of additional strength is projected to be available for design use. Second, if this strength is available, what is the best method to utilize it in design? Fig. 1 shows a typical strength gain curve for a concrete girder produced in Washington State. The specified 28-day strength is 7000 psi (48 MPa) and the actual strength is 9190 psi (63 MPa).
Allowing for a 1.34 x standard deviation reduction in strength to satisfy the specified performance guidelines, the concrete would meet an 8500 psi (59 MPa) design strength, 1500 psi (10 MPa) above the specified strength. This concrete mix did not use a high range water reducing (HRWR) admixture, but did use a low water-cement ratio and a high cement factor.
p.91-92
The strength gain that is available for a given mix is a function of the mix design and the accelerated curing.
Thus, historically, two types of concrete emerged:

1. Concrete with high quality aggregate and high cement factors continued to have substantial strength gain at 28 days.
2. Concrete with lower cement factors and HRWRs had much less strength gain between 18 hours and 28 days.
   This behavior led to an investigation of the strength gain beyond 28 days and the performance of newer high strength concrete girders.

Since concrete continues to gain strength with time, additional strength gain beyond the 28-day standard may be available.

In considering strength gain in excess of 28 days, it is noted that the initial uses of high strength concrete in building design were specified at 56 and 90 days. Since many precast girder bridges do not need their full strength until well past 56 days, a 56-day concrete strength is a pragmatic possibility.

The post-28-day strength gain is a function of the mix design. Mixes that gain their strength from a high cement factor, e.g., Type I cement and/or which use pozzolanic fly ash, are more favorable candidates for supplemental strength gain.

Strength gain of moist-cured cylinders is widely documented for laboratory conditions. Field conditions offer highly variable curing environments and it is logical to question if the laboratory strength gain is available in actual structures. Tests at CTL on a 25-year-old prestressed concrete girder removed from the Illinois Tollway, provided data indicating that a 5000 psi (34 MPa) increase in compressive strength occurred over the originally specified 5000 psi (34 MPa) design strength. Tests of two 30-year-old bridge girders in Belgium indicated a 77 percent increase in compressive strength beyond the original specified strength. The actual strength at the time of testing was 13,800 psi (95 MPa).

p.95

Under normal construction sequencing, deferring the application of full live load for 56 days appears quite reasonable. If the extra strength gain can be specified and utilized during the bridge design, then the increased strength due to aging is particularly valuable.

p.96-97 Strategies for Using High Strength Concrete

2. Examine the historical strength gain to determine if a higher final design strength is available. If a significant strength gain is present, high strength concrete may be "free".

3. Increase the cure time before the prestress transfer to two days in order to obtain a higher initial strength compared to the design strength requirements.

4. If cementitious fly ash or Type I cement is used in the mix, consider using a 56-day strength to allow the cementitious material to react more completely.

7. When evaluating the condition of older prestressed concrete bridges, consider time-dependent strength increases. The increased strength may allow the bridge to continue to function at its full capacity or for a higher load capacity than the original design.

p.97 Conclusions

Many prestressed concrete girders display an actual concrete strength in excess of the specified strength. The additional strength is available to the designer and has economic benefits in the form of longer spans and greater girder spacing.

The analyses in this paper and past performance of prestressed concrete girders suggest that high strength concrete and bulb-tee girders are very cost-effective bridge members.

3. Prestress Losses

"Prestress Losses In Pretensioned High-Strength Concrete Bridge Girders"
Maher K. Tadros, Nabil Al-Omaishi, Stephen J. Seguirant, James G. Gallt
NCHRP Report 496
Transportation Research Board, 2003

p.1

It was concluded that local material properties significantly impacted the prediction of modulus of elasticity, shrinkage, and creep. The proposed formulas produce national averages: factors are given to adjust these averages for the four states covered in the project.

p.3

If one underestimates prestress losses, there is a risk of cracking the girder bottom fibers under full service loads. On the other hand, if prestress losses are overestimated, a higher prestress force must be provided, which will result in larger amounts of camber and shortening than is necessary. It is, therefore, important to have a reasonably accurate estimate of prestress losses.

p.5

The prestress losses prediction formulas are used by current AASHTO-LRFD and AASHTO Standard Specifications for considering the effects of variation in material properties, especially concrete strength.

p.6

In early-age concrete, the strength of the cement paste is the primary contributor to the strength while the stiffness of the coarse aggregates is the primary contributor to the modulus of elasticity.

Accurately estimating the value of Ec allows for accurate prediction of the initial camber and initial elastic prestress loss and helps improve the accuracy of the prediction of creep loss.

The modulus of elasticity increases approximately with the square root of the concrete compressive strength; empirical equations have been developed to estimate the modulus of elasticity based on the compressive strength of the concrete.

p.13

The AASHTO-LRFD provisions need to be updated (1) to consider high-strength concrete in Sections 5.4.2.3 and 5.4.2.4, (2) to improve the prestress loss calculation methods of Section 5.9.5 for high-strength concrete, and (3) to link the material property formulas of Sections 5.4.2.3 and 5.4.2.4 with prestress loss prediction formulas of Section 5.9.5 into one integrated approach.

p.50

Whether gross or transformed section properties are used, the calculated concrete stresses are essentially the same if the proper components of the prestress loss are used. Either long-term losses due to creep, shrinkage, and relaxation in conjunction with transformed section properties, or total losses (including elastic losses and gains) in conjunction with gross section properties should be used.

p.56-57 Conclusions

(a) The prestress losses prediction formulas used by current AASHTO Specifications do not account for the variability in material properties.

(b) The modulus of elasticity of concrete has been shown to have a high degree of variability, attributed to such factors as properties and the proportion of the coarse aggregates used, moisture content and temperature of the constituents at time of mixing, methods of mixing and curing, method of testing, size and shape of specimens tested, and difference between compaction of concrete in the precast member and that in a test cylinder. A formula has been proposed for estimating modulus of elasticity that assumes a concrete unit weight relationship to concrete strength. The proposed formula has been shown to give more accurate estimates than those obtained by the current AASHTO-LRFD and ACI-363 formulas.

(c) The research has determined that concrete compressive strength, V/S ratio, curing methods, and time elapsed after the end of curing influence shrinkage. A proposed shrinkage formula produced results that averaged 105% of the measured values, compared to 174% when using the AASHTO-LRFD method and 155% when using the ACI-209 method.

(d) The creep coefficient is influenced by the same factors that influence the shrinkage coefficient in addition to the age of the concrete at the time of loading and time elapsed after loading. A proposed creep formula produced results that averaged 98% of the experimental values, compared to 161% for AASHTO-LRFD and 179% for those estimated using ACI-209.

(e) Predictions of modulus of elasticity, shrinkage, and creep are influenced by local materials and practices. Therefore, data for local materials and mixture proportions should be used when available.

(k) Test results reported in the literature showed that the total prestress losses averaged 38.5 ksi; the initial elastic loss was 19.0% of the jacking stress of 202.5 ksi.

(l) The AASHTO-LRFD Refined method tends to over-estimate creep effects because it does not consider the reduction in the creep coefficient associated with the increase in concrete strength.

(m) The AASHTO-LRFD Lump-Sum method results showed a better agreement with test results that the Refined method, because it accounts for the variability of the loss with concrete strength.

(n) The proposed approximate method produces better estimates of long-term prestress losses than those obtained by the AASHTO-LRFD Lump-Sum method because the Lump-Sum method does not account for the level of prestressing or ambient relative humidity.

Further research is also needed to investigate initial and long-term girder camber. Data from field installations could be used to calibrate the analytical results obtained on the basis of the theory developed in this project.

4. Girder camber variability

"Precast, prestressed girder camber variability"

Maher K. Tadros, Faten Fawzy, and Kromel E. Hanna

PCI Journal/Winter 2011

p.135

Precast concrete girder camber can vary significantly between the time of prestress release and the time of erection.

The variations in camber become more significant as the use of high-strength concrete, longer spans, and more heavily prestressed concrete girders continues to increase.

This paper addresses several issues related to prediction, design, and construction to accommodate variability in prestressed concrete girder camber.

Camber at prestress release is not affected by creep and shrinkage estimates, but it is highly influenced by the modulus of elasticity. Also, accurate estimates of elastic shortening losses at prestress release would allow for more accurate prediction of camber at release.

With the increased use of high-strength concrete, most of the creep and shrinkage takes place in the first few months of the concrete age.

p.136-137

The 2005 interim revision to the American Association of State Highway and Transportation Officials' AASHTO LRFD Bridge Design Specifications introduced extensive revisions to the formulas for prestress losses, as well as those for modulus of elasticity, creep, and shrinkage. These revisions extended the application of these formulas to concrete strengths from 5000 psi to 15,000 psi (34,000 kPa to 100,000 kPa).

This paper also discusses camber variability. It recommends user-friendly detailing and construction methods to acknowledge camber variability and minimize conflicts between designers, producers, and contractors.

Instantaneous camber, which occurs at the time of release of the prestressing force from the bed to the concrete member, is well defined. The prestressed concrete member cambers upward because the upward bending due to initial prestress is generally larger than the downward deflection due to member self-weight. The camber at that time is a result of the combination of these two effects. Due to the assumed linear elastic behavior of the system, the conventional theory of elasticity and method of superposition are valid. Thus, deflection due to self-weight is calculated separately from camber due to initial prestress, though the two quantities cannot be physically separated.

For camber analysis at prestress release, common practice historically uses the following assumptions:

- The span length is assumed to equal the overall member length. The reasoning behind this assumption is that when prestress is released, the member cambers and bottom of the girder separates from the bed except at the extreme ends. Some design guides use the span length between bearings on the bridge. This is done for convenience and is illustrated in this paper.
- The modulus of elasticity is the concrete modulus at time of prestress release. This quantity is most often predicted from the density of the member and the specified concrete strength at prestress release.

- The prestressing force is assumed to be the force in the concrete after allowance for elastic shortening losses. As the prestress transfers to the concrete, the member shortens due to two equal and opposite forces: tension in the prestressing strands and compression in the concrete. At the time of release, the prestressing-strand tension is smaller than the tension before release due to the member deformation.
- The properties ec, ee, and I are the gross cross-section properties. Theoretically, they should be the net section properties because the calculation of elastic loss presumes separation of the steel and concrete. However, the two sets of properties are close.
  p.137
  A more rigorous approach would be to use the prestressing force just before release and apply it to the transformed section properties when calculating the initial camber. With this approach it is not necessary to calculate the elastic shortening loss. Because the elastic loss varies from one section to another along the span, this helps avoid the error of assuming constancy. This approach was introduced in 2005 in the AASHTO LRFD specifications. Proposed equations in this paper follow this design approach.
  Equation (2) does not take into account the loss of prestressing force due to strand shielding. The proposed formula includes this effect.
  As discussed earlier, the span length at this stage is usually assumed to be the full member length. This may be true during the short duration when the prestess is released and before the member is removed from the bed. However, when the member is stored in the precasting yard, it is usually placed on hard wood blocking. This condition remains until the member is shipped for erection on the bridge. It is important to model the storage support condition due to the increasing use of long-span girders over 150 ft (45 m) long. Optimal placement of wood blocking is at a distance of about 7% to 10% of the member length.
  There is a need to standardize storage conditions in order to allow for more accurate camber prediction. At a minimum, the designer should recognize that support location during girder storage is a factor in estimating camber at release and at erection.
  p.139-140
  Most designers ignore the overhangs in estimating the initial deflection due to self-weight. This is reasonable for conventional beam lengths with supports near the beam ends. However, long girders, approaching 200 ft (60 m) in length, have been produced in recent years. These long girders should be supported at a distance about 7% to 10% of the length. This helps improve stability, camber, and sweep during storage. Ignoring the overhangs for these conditions may underestimate the elastic loss effect and overestimate camber. Equation (15) yields more-accurate results than equations developed for a simple span.
  The prestressing force just before release along with section properties of the transformed section should be used in the aforementioned analysis. This is the method promoted by the AASHTO LRFD specifications, section 5.9.5. A common alternate method is to use gross section properties along with prestressing force just after release, which is equal to the initial prestressing force less the elastic shortening loss. This proposed method is theoretically equivalent to the assumption that the elastic loss is constant for the entire length.

p.142

With CONSPAN, the modulus of elasticity was 90% of that predicted by AASHTO LRFD specifications, to reflect the Florida Department of Transportation design guide for soft Florida limerock aggregates.

Studies by Tadros et al. (NCHRP project 18-07 report 496) and Al-Omaishi et al.[4,5] demonstrated that the values of Eci can vary by +/-22% relative to the mean value for levels of confidence between the 10th and 90th percentiles.

p.143

Impact of coarse aggregates on Eci

The impact of coarse aggregates on Eci is a well-documented but often ignored factor. The NCHRP project 18-07 report 496 gives recommendations for this effect for the states of Nebraska, Washington, Texas, and New Hampshire. In Florida, the use of soft native limerock is frequent enough to have a standard recommendation use a 0.9 factor. If this factor is used, the camber changes from 3.01 in. (76 mm) to 3.34 in. (85 mm). Because of these factors, it is recommended that records of the modulus of elasticity be kept for concrete mixture proportions used in precast concrete bridge girder production.

Actual concrete strength versus specified strength

Designers specify a minimum concrete strength at prestress release. For Example 1, this strength is 6 ksi (40 MPa). However, it is not an uncommon practice or a code violation release the prestress at higher concrete strengths of 7 ksi or 8 ksi (50 MPa to 55 MPa). Occasionally, precast concrete fabricators leave girders scheduled for release on Saturday in the bed until the following Monday morning, two days later than the due date. Actual initial concrete strength can change dramatically in a short period, depending on the mixture, and the designer may not know the actual time for camber prediction. Its variability significantly affects the value of Eci.

Differential temperature at prestress release

Concrete temperature is elevated in the first hours after concrete placement. Temperature rise is caused by the heat generated through the cement hydration process and also due to externally applied heat for curing. The temperature is higher with increased cement content.

Cooling of the girder concrete is not uniform as it balances with the ambient temperature. The top flange and web cool more quickly than the bottom flange. The temperature gradient through the girder depth can create a deflection that is generally not considered in estimating the initial camber. This deflection component eventually diminishes, but creep and shrinkage effects begin to take place.

p.144

Storage span length versus final span length

Often the girders are placed on hardwood supports in storage at a significant distance away from the ends. They are kept in storage in this manner until they are moved for shipping to the jobsite. This is recommended for long girders to enhance their stability.

Friction at girder ends due to prestress release

The friction effect is highly variable. There is no guidance in the literature to quantify it. It is more related to quality-control issues than to true camber variability. To reduce the difference between theoretical and actual initial camber, recommendations are to lift the girder off the bed and reset it on the bed before measuring camber.

Initial prestress and girder weight

A designer may specify strands to be tensioned to a specific stress, but the actual stress will be higher or lower. Strand locations can alter specified stresses on the design drawings, which can lead to a difference in initial prestressing force by 5%. The density of concrete is usually specified between 0.14 kip/ft^3 to 0.15 kip/ft^3 (22 kN/m^3 to 24 kN/m^3), but it can be lower or higher based on aggregate type and the use of mild reinforcement with the strands. This can result in a 5% change in the density of concrete. The corresponding net camber when assuming initial prestress is 5% lower and member weight is 5% higher is calculated:

9.462 up – 6.132 down = 3.33 in. (85 mm).

The resulting 13% increase validates the effect that initial prestress and girder weight may have on the actual initial net camber.

p.145

Background and methods of long-term camber prediction

The PCI Bridge Design Manual has a detailed discussion in sections 8.7 and 8.13 of both the approximate and detailed methods of time-dependent analysis for prestress loss, camber, and deflection. The discussion includes the simple constant multiplier method originally proposed by Martin in 1977. This method is still predominant in current commercial software because of its extreme simplicity and the belief that it is difficult to accurately predict time-dependent effects. Some believe that if modulus of elasticity, creep, and shrinkage can only be predicted within +/- 20% at best, one should not worry about fine turning the camber equations. However, as explained in the following discussion, one should never worry about the random variables that cannot be controlled. Errors from controllable and/or know variable can and should be minimized as much as possible.

In 1985, Tadros et al.[10] published a paper on the topic of multipliers in terms of creep variability. The contents of the paper were extensively covered as the improved multiplier method in section 8.7.2 of the PCI Bridge Design Manual. This method was further advanced in the study for NCHRP project 18-07 report 496, which was adopted in the 2005 interim AASHTO LRFD specifications and covered previously in this paper. The variable multiplier method allows for adjustment due to high-strength concrete and high levels of prestress as currently used in bridge practice. High-strength concrete can cause significant reduction in the creep coefficient. However, high-strength concrete also allows for use of high prestress levels. Thus, it is observed in current practice that initial camber can be significantly greater than cambers from a decade ago. However, camber growth as a percentage of initial camber is somewhat slower. High-strength concrete tends to undergo most of its creep in the first several months, as opposed to the more slowly developing creep in lower-strength concrete. For these reasons, it is important to accurately model modulus of elasticity, creep, and, to a lesser degree, shrinkage in order to obtain reasonable camber averages.

p.147

Variability of long-term camber

There are a number of causes for variability of camber growth between the time of prestress release and the time of erection. Time of erection is defined here as the time at which the deck placement operation is completed.

Rose et al. [12] completed a study in 2007 that included analysis as well as field measurements. It concluded with a recommendation to endorse the new AASHTO LRFD specifications prestress-loss provisions to directly account for loss effects on camber and to apply modification factors to the modulus of elasticity and creep multipliers in AASHTO LRFD specifications. The study reflects conditions in the state of Washington related to the environment and to local materials. The recommended factors are 1.15 for modulus of elasticity and 1.4 for creep. The report makes an important observation of the impact of support conditions. The camber measured 41% to 46% smaller with temporary oak blocking that with the permanent elastomeric bearing pads.

Accuracy of long-term multipliers

Martin's multipliers are simply 1.85 and 1.80 for girder weight and initial prestress. They were derived from the assumption that the creep coefficient is a constant 2.00 with additional constants to account for prestress loss, change in elasticity modulus, and partial development at an intermediate time. Proposed multipliers separate these effects and allow for actual conditions to be incorporated. Even with this refinement, concrete properties are random variables and cannot be deterministically accounted for in calculations. The NCHRP project 18-07 did not come to a specific recommendation for upper- and lower-bound values, as was done for the modulus of elasticity, because of a lack of data for high-strength concrete creep at the time of the study. It is reasonable to assume these bounds to be +/- 25%.

Girder support condition while in storage

Girder support condition while in storage affects the initial camber, as previously discussed. This, it affects the long-term camber as well.

p.147-148

Time elapsed before girder installation

Most designers do not and cannot enforce a specific girder age at the time of deck placement. At best, some state highway agencies require that the girder be at least 28 days old when the deck concrete is placed. Few agencies require a minimum age of 90 days. None of the specifications, to the authors' knowledge, require an upper limit on girder age at time of deck placement. It is possible that the girders will be six months old before they are erected. While this variability is outside of the control of the designer, construction documents should be prepared to minimize conflicts and delays during construction.

p.149-150

Variability of deflection due to superimposed dead loads

Whether the forms are permanent stay-in-place metal forms or temporary wood forms, their weight should be included in predicting net camber immediately after deck placement. Sources of variability of deflection include magnitude of superimposed dead load, estimate of modulus of elasticity Ec, bearing resistance, and support condition.

Magnitude of superimposed dead load

The diaphragms, forms, and deck weight can be assumed to be relatively accurately. The haunch buildup can be a significant load that is a function of the camber itself. Density of concrete containing normal weight aggregates is assumed to be 0.15 kip/ft^3 (24 kN/m^3). This is reasonable and consistent with AASHTO LRFD specifications for concrete strength in the 4000 psi to 5000 psi (28,000 kPa to 34,000 kPa) range.

Estimate of modulus of elasticity Ec

The modulus of elasticity is subjected to the same +/- 22% random variability discussed earlier.
p.151-152

Detailing and construction considerations

Assuming the best possible preconstruction data, assumptions, and camber prediction theories, there is still a likelihood of significant variation in the camber prediction. Camber at the time of deck placement is an important measurement during construction, yet it may vary by as much as +/- 50%. This range could be even larger if a girder is stored in the yard for several months. For example, if theory predicts a 3 in. (80 mm) camber immediately after deck placement, it could end up ranging from 1.5 in. to 4.5 in. (40 mm to 110 mm). If the camber is larger than predicted, the girder could have a negative haunch at midspan or even interfere with the bottom mat of deck reinforcement. If camber is lower than predicted, it would increase the quantity of concrete in the haunch, which is often an item of contractual disagreement. Increasing the quantity also increases the load on the girder.

Another important factor is the possibility of infringement on vertical clearances below the bridge. Furthermore, camber that is too small may be a cause for aesthetic concern, especially if the long-term camber ends up being negative, or a downward deflection. Girder sag, while acceptable for structural capacity, may not be as accepted for serviceability by the general public.

The following guidelines are recommended in design to alleviate some of the camber variability concerns:

- Design for a minimum haunch of 2.5 in. (60 mm) over the girder. This would allow for an actual camber that is 3.5 in. (90 mm) higher than estimated without interfering with deck reinforcement. For a 4-ft-wide (1.2 m) girder top flange and a 2 % deck cross slope, the available distance is actually 3 in. (80 mm), not 3.5 in. (90 mm).
- Detail shear reinforcement in girder to accommodate camber variability. Some designers use a different hook height above the top flange and in the outer quarter lengths of the girder.
- The height of girder seats should be finalized only near the time of girder installation. At that time, the actual girder camber can be measured and the seat elevation determined. For example, if the estimated camber is 3 in. (80 mm) and the actual camber is 1.5 in. (40 mm), the seat elevations can be raised 1.5 in. (40 mm) using cementitious grout, steel plates, or other means.
- The contractor pay item for concrete quantities in the haunches could be structured in a way that it is not adjustable during construction. The contractor would have to assume the variability and account for it in the initial bid. This is a small item in the overall cost of the bridge and arguments during construction could be avoided if both parties acknowledge that the engineer estimate of haunch thickness is highly variable.

p.152

Conclusion

It is not possible to have the actual camber at prestress release or at deck placement match the calculated estimates. Random variability beyond the control of the engineer does not allow for such precision.

- The 2005 interim AASHTO LRFD specifications (and later editions) include prestress loss, modulus of elasticity, creep, and shrinkage prediction formulas that can be effectively used to improve camber prediction.
- Local material properties, girder storage, and construction practices should be considered in design, as much as is practical, rather than defaulting to embedded conditions in commercial design software. This recommendation may not be easy to implement while owners do not have specifications that govern storage and erection conditions. Currently, there are unique practices of specific producers and contractors that cannot be regulated by the designer unless the project is design-build, not the conventional design-bid-build.
- In design, allow for variability of camber by 50%. Future research may offer refinements of this figure.
- Allowance in design should include flexibility in adjusting the horizontal shear reinforcement and the girder-seat elevations.

5. Commercial Software

PSBeam V4 Prestresed Concrete Bridge Girder Design software by Eriksson technologies The following items are noted in the features brochure that are related to the MDOT research study.

Section Properties

- Option to transform strands

Include Rebar in Design

- Top tension rebar at release
- Bottom tension tie (shear)

Debonding

- End debonding
- All effects accounted for in analysis
- Debond straight and draped patterns

All Critical Design Checks

- Camber and deflections

6. Commercial Software
   Comparison of Precast Prestressed Bridge Girder Design Software
   The following items are noted (as related to the MDOT research study) in the feature comparison between PGSuper and two other popular precast prestress girder design programs, ConSpan and PSBeam.
   - Girder camber
   - Girder deflections
   - Bursting in anchorage zones
   - Straight patterns
   - Harped patterns
   - Debonded patterns
   - Longitudinal reinforcement for shear
   - Slab haunch offset ("A" Dimension)
   - Release and final concrete strength
   - Prestress losses

7. Early age behavior, SCC, high strength concrete, instrumentation & monitoring, integrated sensing systems

   "Evaluating the early-age behavior of full-scale prestressed concrete beams using distributed and discrete fibre optic sensors"

   Liam J. Butler, Niamh Gibbons, Ping He, Campbell Middleton, Mohammed Z.E.B. Elshafie

   Construction and Building Materials 126 (2016) 894-912

   p.894

   An analysis of the curing strains within the beams revealed the significant effect that ambient temperature, curing duration, and formwork restraint has on the development of prestress losses prior to detensioning.

   p.895

   Introduction and Background

   Permanently integrating FOS into one or more of these critical elements before they are placed in service allows the entire load history of these elements to be captured over time. Therefore, engineers and researchers can assess design assumptions against measurements of real behavior and asset manager can more confidently address future questions about an assets' existing and/or remaining capacity and its potential for reuse I other structures. Research into the time-dependent behavior of precast prestressed concrete beams has been ongoing for several decades. Prestressed concrete offers many advantages over reinforced concrete in terms of controlling cracking and minimizing long-term deflections In addition, by using high strength self-consolidating concrete (SCC) mixtures, the overall constructability and quality of the finished product can be greatly improved. The use of high strength concrete in bridge beams offers economic advantages over traditional mixes due to increased stiffness and reduced deflections, permitting longer spans and smaller section sizes. However, accurate prediction of the prestress losses in high-strength concrete, in particular at an early age, is required for design. An underestimation of the prestress losses may lead to cracking uner service conditions and an associated reduction in section efficiency which can lead to long term durability issues. In contrast, overestimating prestress losses in a beam can lead to excessive camber and unnecessary additional elastic shortening. Therefore, it is important to provide accurate prestress loss predictions to ensure that the remaining prestressing force is adequate to control deflections of prestressed members under permanent load. Guidance in current European (EN 1992-1-1:2004) and American (AASHTO-LRFD) standards pertaining specifically to high strength concrete is limited and has been identified as an area requiring further investigation. Calculating reasonable estimates of early age prestress losses in prestressed beams that use high strength SCC before they are made composite with the concrete bridge deck has also not been studied extensively. There have been several experimental studies that have investigated quantifying early age time-dependent behavior in prestressed concrete beams. Examining the prestress losses that occurred during the first year, it was found that 90% of the 1 year prestress loss took place within the first 4 months and that the prestress losses were highly influenced by the concrete stiffness properties.

   It was found that the measured prestress losses were 4-24% lower that the losses predicted using Eurocode 2.

Khayat and Mitchell [7] investigated the structural performance of four full-scale AASHTO Type II precast pretensioned beams constructed using SCC. Using embedded vibrating wire strain gauges, it was identified that the SCC mixtures developed higher autogenous shrinkage in the first 28 days and higher drying shrinkage and creep strains in the first 300 days compared to the control beams. The authors concluded that due to the greater drying shrinkage values, SCC beams may experience higher prestress losses and smaller camber values.

p. 896

Fibre optic sensor technology

In general, FOS offer several advantages over conventional sensors including being relatively small and lightweight, resistant to corrosion and electromagnetic interference, and readily embeddable into structural materials. When used in combination, these systems can provide a highly detailed strain profile of a structural element.

p. 898

Monitoring programme

Table 1 also describes the time-dependent effects that were captured as part of the recorded data at each of the monitoring stages.

p. 899

Fibre optic monitoring results and discussion

The following section presents the results recorded from the monitoring of the prestressed beams during their first six months following casting. Results are presented chronologically beginning with a summary of the tested concrete and prestressing steel properties, the beam manufacturing process which includes the casting, curing and detensioning process of the beams, and then presents the overall strain changes including readings taken at approximately 3 months (storage outdoors in casting yard) and at approximately 6 months (after beams were transported to site and lifted onto the bridge abutments). By recording data at each of these stages, this study presents a detailed evaluation of the internal strain evolution within the beams prior to being placed in service.

Self-compacting concrete was used to eliminate the need for mechanical consolidation and to attain a higher quality finished concrete surface.

p. 903

Beam storage, transport and installation

Following the detensioning process, the beams were transported to an outdoor temporary storage area within the prestressing yard prior to being transported to the bridge site. The beams were cast in January 2015 and were not transported to site until July 2015 and therefore, continued to undergo strain changes due to concrete shrinkage, creep and steel relaxation.

p. 904

Similar to the TY7 beams these results indicate that the majority of the early age creep and shrinkage experienced by the TYE7 beams occurred within the first 3 months after casting.

Early-age prestress loss estimation

Prestress loss predictions

Total prestress losses are divided into two components: instantaneous losses and time-dependent losses. Instantaneous losses for prestressed concrete include the relaxation of steel prior to detensioning, losses associated with shrinkage of concrete, and the losses due to elastic shortening of the concretet at the time of prestress transfer. The time-dependent losses include long-term losses due to relaxation of steel, shrinkage and creep. Formulae provided in Eurocode 2 have been used as the basis for calculating the predicted prestress losses. Losses due to steel relaxation, elastic shortening of concrete, shrinkage, and creep were all considered.

Tendon relaxation

Relaxation of the tendons, due to the loss of tension over time for a fixed length and temperature, is a property of the steel and a function of the ratio between the initial prestressing force and the tensile strength. Relaxation progresses more rapidly than either concrete shrinkage or creep.

p. 905

Elastic shortening of concrete

After the concrete has reached adequate compressive strength, usually between 80 and 90% of fck, the prestressing strands are detensioned at the jacking head and the prestress is transferred as a compressive force into the concrete. Both the concrete and the bonded tendons shorten due to the applied compressive force, thereby reducing the prestressing tension in the tendons. In cases where the tendon arrangements is such that the centroid of the prestressing steel and the centroid of the concrete section are not concurrent, an eccentricity, e, exists which causes an additional moment equal to the product of the total prestressing force and the eccentricity This moment creates bending stresses in the section and generates curvature in the beam resulting in an upward (negative) deflection or camber.

p. 906

Combined calculation of time-dependent losses

Time-dependent losses are often calculated in a combined form to account for the interaction between creep, relaxation and shrinkage as presented in Eq. (9).

p. 910-911

Conclusions

This study evaluated the early-age behavior of four full-scale prestressed concrete bridge beams utilizing the combined technologies of distributed (BOTDR) and discrete (FBG) fibre optic sensors. The entire curing and detensioning process of the beams were captured in great detail along their length. Additional monitoring data captured the strain evolution of the beams from just after they were detensioned up until they were lifted onto the bridge abutments.

The fibre optic cables themselves could last as long as the structure itself with only the optical connectors and analysers requiring long term attention.

It was hypothesized that the restraint created by the beam formwork led to locked-in thermal strains that caused net tension in the top of the beam and net compression in the bottom of the beam. Overall, this effect counteracted the early-age concrete shrinkage strains and created insignificant prestress losses within the TYE7 beams prior to the transfer of prestress.

The TY7 beams experienced slightly higher average prestress losses due to elastic shoreing as compared to the TYE7 beams.

In both the TY7 and TYE7 beams, the majority of the time-dependent losses (creep, shrinkage and relaxation) occurred within the first three months after the beams were cast. Strains along the lengths of the beams were uniform along both the tops and bottoms of all beams during each stage of measurement.

Measured prestress loss values for the TY7 and TYE7 beams approximately 6 months after casting were 79% and 72% of the ultimate prestress losses predicted by Eurocode 2, respectively.

The theoretically calculated cambers (at time of detensioning) were within 9% of the estimated cambers 9based on fibre optic sensor data) for the TY7 beams. In the case of the TYE7 beams, the theoretically calculated cambers (at time of detensioning) overestimated the estimated cambers (based on fibre optic sensor data) by 32%. By estimating the cambers at various times up until the beam erection, they were found to increase significantly (between 1.2 and 1.7 times the estimated camber at detensioning). This preliminary study demonstrated the possibility of using the installed sensor system in combination with simple beam theory to measure deflections under service load conditions.

The primary aim of this research was to demonstrate that integrated sensing systems can become viable tools for monitoring strain evolution in concrete bridges and can be used to establish comprehensive baselines to inform long term bridge monitoring and asset management programmes.

8. High Strength Concrete, Welded Wire Reinforcement (WWR), benefit/cost analysis, the value of research

"Use of High Performance, High Strength Concrete (HPC) Bulb-Tee Girders Saves Millions on I-10 Twin Span Bridge in New Orleans District"

Louisiana Transportation Research Center (LTRC) Implementation Update: Research in Practice, Technology Transfer Program Final Reports 310, 382, and 395

By using high performance, high strength concrete, wider girder spacing was achieved for the I-10 Twin Span Bridge. The use of 8,500 psi (59 MPa) design compressive strength concrete allowed the use of six lines of girders spaced at 10 ft.-9 in. (3.28 m) rather than seven spaced at 8 ft.-10 in. (2.69 m), which would have been required with a concrete compressive strength of 6,500 psi (45 MPa). Furthermore, the use of 8,500 psi (59 MPa) concrete allowed the BT-78 girder to span a length of 135 ft. (41 m).

In addition, with the lower permeability associated with high performance concrete, the Louisiana Department of Transportation and Development (LADOTD) expects a minimum 75-year service life for the bridge instead of the standard 50-year service life for concrete structures.

History

LADOTD has been gradually introducing high performance, high strength concrete into its bridge construction program. At the same time, LTRC has been sponsoring research work to address design and construction issues related to the utilization of high performance concrete.

In 1988, a bridge project was used as an experiment to determine if a concrete compressive strength of 8,000 psi (55 MPa) could be obtained on a production project. The contractor achieved strengths over 6,000 psi (41 MPa), an improvement over standard strengths, but generally fell short of the 8,000 psi (55 MPa) target.

In 1992, a 130-ft (39.6-m) long, square, prestressed concrete pile with a compressive strength of 10,453 psi (72 MPa) was produced, shipped, and successfully driven without damage as part of the State Route 415 Bridge over the Missouri Pacific Railroad.

In 1993, two bridges on the Inner Lop Expressway near Shreveport were built using AASHTO Type IV girders with a 28-day specified compressive strength of 8,500 psi (59 MPa). A 1994 LTRC report recommended that LADOTD consider the implementation of concrete with compressive strengths up to 10,000 psi (69 MPa) in a bridge and that the bridge should be instrumented to measure long-term behavior [1]. This recommendation was implemented with the design and construction of the Charenton Canal Bridge, which opened to traffic November 1992 [2]. The successful construction of the Charenton Canal Bridge demonstrated that a high performance concrete bridge could be designed and built in Louisiana using locally available materials.

Research Results

Based on the results of the shear tests, it was determined that the existing limitation of 60,000 psi (414 MPa) for the design yield stress of transverse reinforcement cited in both AASHTO specifications is conservative. Higher reinforcement yield strengths can be utilized in the design of prestressed concrete beams. Welded wire deformed reinforcement can be used as an equally effective alternate to deformed bars as shear reinforcement.

Benefit/Cost Analysis

The value of research is sometimes hard to quantify. But in this case, using just one project, researchers found huge savings. The calculations below are based on initial cost savings in bulb-tee girders alone and do not include the reduction in bent locations and increase in structure design life.

9. Temperature variations on camber
"The effect of temperature variations on the camber of precast, prestressed concrete girders"
Hang Nguyen, John Stanton, Marc Eberhard, and David Chapman
PCI Journal/September-October 2015

Abstract
It is important to estimate girder camber accurately because differences between expected and actual camber can lead to construction challenges or girder rejection. Field measurements of daily variations in temperature profile and camber for two precast, prestressed concrete girders provided data with which to calibrate models of the effect of temperature variations on camber. Using measured temperature profiles over the height of the girder, the associated camber history was accurately computed, assuming a coefficient of thermal expansion of 5.5 x 10-6 degree Fahrenheit. Two practical methods were also developed using 164 observations from 24 girders. To implement the simpler method (peak temperature camber method), the designer needs only to know the girder's length and depth and to estimate the maximum change in air temperature during the day, which is available from meteorological stations. The errors in the resulting models had root mean square average camber over time of about 0.1 in. (2.5 mm).

p. 48-49

The prediction of girder camber at a particular time is difficult because it depends on the concrete properties, curing conditions, prestress losses, and temperature variations within the girder, all of which vary with time. One contributor to this difficulty is the effect of variations in the profile of internal temperatures over a day. Such variations induce thermal strains, which, if they vary over the height of the girder, result in camber even in the absence of external loads.

Thermal camber typically has the most important consequences during construction.

The methods proposed herein may be used for estimating thermal camber for a known temperature environment. Knowledge of the thermal camber is expected to be helpful for making appropriate elevation allowances when setting formwork for the cast-in-place deck slab, for estimating the additional concrete required in the haunch over the girder, and, in extreme cases, for ensuring that girders are not rejected for having camber that lies outside the specified range.

p. 49-50

Previous Work

For the purpose of design, the American Association of State Highway and Transportation Officials' AASHTO LRFD Bridge Design Specifications provide four temperature gradients corresponding to four solar radiation zones in the United States. The gradient depends on the location of the bridge girder, the superstructure depth, and the materials in the superstructure.

Barr monitored changes in camber of five simply supported I-girders during a day to evaluate the bridge's response to daily temperature variations. He found that the camber varied by 0.63 in. (16 mm) between 9:00 a.m. and 9:00 p.m. in one day.

Hinkle observed that the cambers of three girders with an average length of 128 ft (39.0 m) varied by up to 0.50 in. (13 mm) between 7:45 a.m. and 1:25 p.m. in the same day.

The camber of the monitored girders varied during each day by 0.70 to 0.95 in. (18 to 24 mm) for girders ranging in length from 119 to 132 ft (36.3 to 40.2 m).
p. 50
Experimental program
All of the measurements discussed in this paper were taken approximately two months after the prestressing force had been applied to the girder concrete, by which time the effects of creep, shrinkage, and relaxation were expected to be constant over the course of any one day.
p. 60
Conclusion

- The temperature profile in a precast concrete girder can be highly nonlinear during the afternoon, with a large temperature gradient within the top flange.
- Both the temperature history camber model and the peak temperature camber model had root mean square average camber errors over time of about 0.1 in. (2.5 mm) when applied to a range of girders, all with I-shaped cross sections. Such small errors would not significantly affect the installation of girders onsite.
- The accuracy of the camber prediction depended on the details of the girder's exposure to the sun. It is possible that the differences in accuracy arise from differences in temperature, humidity, or solar radiation conditions.

10. Predicting camber

“Improving the Accuracy of Camber Predictions for Precast Pretensioned Concrete Beams”
Tech transfer summary
Iowa State University, Institute for Transportation
Bridge Engineering Center

Problem Statement

Construction schedule delays and additional costs are common problems when the actual camber of precast pretensioned concrete beams (PPCBs) are different from those expected during bridge design.

Background and Research Overview

The camber of PPCB is relatively complex because it is sensitive to variations in several parameters, including the mix design, tolerances on prestressing forces and moisture control, bed configuration, curing process and handling, storage environment, and support location during storage. In addition, the aggregate types, cement, and admixtures used on the concrete mix play a significant role in the mix’s creep and shrinkage behavior, which in turn significantly affect the long-term camber.

The method that the Iowa Department of Transportation (DOT) was using to formulate the camber for PPCBs frequently overpredicted the long-term camber of some of the most often used long PPCBs in Iowa bridges, while it underestimated the long-term camber of shorter PPCBs.

Therefore, a systematic study was undertaken to identify the key parameters affecting camber, needed improvements to construction practices, and potential refinements to the predictive analytical models. The discrepancy between the predicted and actual camber is reduced by addressing the concerns associated with each of these areas.

Objectives

- Quantify the engineering and time-dependent properties of concrete to reduce the uncertainties associated with the variability of material properties in the camber prediction of PPCBs
- Propose suitable refinements to the measurement approach to accurately capture the instantaneous camber and recommend appropriate modifications to the PPCB fabrication process to decrease variations in the camber of identical PPCBs
- Improve the method for predicting the instantaneous camber and verify the accuracy of the method using data collected from PPCBs produced at three local precast plants
- Address the long-term camber variability resulting from thermal effects and the locations of temporary supports
- In conjunction with the measured field data and the accurate analyses of PPCBs, develop a new set of multipliers to predict the camber accurately when the PPCBs are erected in the field

Material Characterization

Three normal concrete (NC) and four high-performance concrete (HPC) mix designs, which were representative mixes from three precast plants, were investigated for their engineering and time-dependent properties.

- The AASHTO LRFD creep and shrinkage models were found to give the best estimates when compared to the measurements taken from the four HPC and three NC mixes over one year.
- Shrinkage strains taken from sealed specimens corresponded well with the strains obtained from a segment of full-scale PPCB, suggesting that strains taken from sealed rather than unsealed specimens would produce more realistic creep and shrinkage strains for PPCBs.

Instantaneous Camber Measurements

Using data collected for 105 PPCBs from three precast plants, the causes of error associated with the instantaneous camber measurement were investigated. The following conclusions were drawn:

- Values obtained from field measurements showed that the camber on average is affected by 0.030 in. +/- 0.062 in. due to bed deflection, 0.392 in. +/- 0.294 due to friction between the beam and steel bed, 0.099 in. +/- 0.142 in. due to the inconsistent top flange surfaces along the beam length, and 0.113 in. +/- 0.119 in. due to inconsistencies in the top flange surfaces resulting from local effects.
- Data obtained from the PPCBs at the transfer of the prestress by tape measure, rotary laser level, and string potentiometers showed good agreement when adjusting for possible camber measurement errors. Despite good agreement between the tape measure and rotary level, tape measure data are easily affected by human error and are not recommended.
- The reverse friction is small in magnitude and can be ignored. The contribution of vertical displacement due to the friction can be obtained by lifting/setting the PPCBs on the precast bed and then taking the camber measurement.

Instantaneous Camber Predictions

Challenges faced in predicting the instantaneous camber during design are related to the designer’s ability to accurately estimate the material properties and model the applied forces exerted on the PPCB after accounting for the effect of the prestress losses. Therefore, using the moment area method, different parameters affecting the instantaneous camber prediction, including the modulus of elasticity, prestress force, prestress losses, transfer length, influence of sacrificial strands, and section properties, were investigated.

The instantaneous camber was consequently predicted using the AASHTO modulus of elasticity based on the measured compressive strength with due consideration given to the applied prestress force, prestress losses, the AASHTO transfer length, sacrificial prestressing strands, and transformed moment of inertia.

Comparing these results with the measured camber of 50 PPCBs, for which measurement errors were eliminated, produced an agreement of 98.2 +/-14.9%, which is a significant improvement in the predictive accuracy of the instantaneous camber.

Long-Term Camber Measurements

The resulting overhang length was found to vary from less than 20 in. (0.015 L, where L is the overall length of the beam) to as high as 87 in. (0.05 L), with a mean overhang length of L/30. As a consequence, the researchers found that long-term camber is affected by the overhang length.

Variations from expected trends in the long-term camber data, including unusually high camber at early ages and a reduction or no significant increase in camber, were frequently observed. Because the camber measurements were performed at different times during the day, the thermal effects created by the vertical temperature gradients down the beam depth were suspected to be contributing to the scatter in the data.

An investigation of the thermal effects of 22 PPCBs confirmed the effects of the temperature gradients on the long-term camber. The researchers found that temporary camber growth of as much as 0.75 in. is possible on a warm summer day, which explains the unusual trends in long-term camber.

Long-Term Camber Predictions

- Based on a sensitivity analysis, the scatter in the long-term camber data was adequately captured by using a linear temperature gradient down the beam depth with a temperature difference of 15 degrees Fahrenheit between the top and bottom flanges.
- By incorporating the 15 degrees Fahrenheit temperature difference in the long-term camber predictions, the corresponding errors were reduced to -1.2% +/- 10.7% and -14.7% +/- 22.5% for the large- and small- camber PPCBs, respectively.

Key Findings

- When AASHTO LRFD is used to estimate the modulus of elasticity, the release strength should be taken 40% and 10% higher than the specified concrete strength for PPCBs when the design value is in the 4500-5500 psi and 6000-8500 psi range, respectively.
- The sources of errors caused by the current instantaneous camber measurement techniques were identified and subsequently eliminated by the proposed measurement technique.
- By isolating the measurement errors from the errors caused by the prediction methods, the accuracy of the instantaneous camber prediction was improved using a combination of appropriate material properties and design procedures.
- Using the FEM of the PPCBs developed with consideration given to measured creep and shrinkage, thermal effects, and changes in the prestress and support locations significantly improved the accuracy of the long-term camber predictions.
- The multipliers, which include adjustments for the overhang and thermal effects, improved the long-term camber predictions compared to those from the method used by the Iowa DOT.

Recommendations for Precasters and Contractors

To eliminate the differences in the camber due to the measurement technique, a simplified procedure was formulated that can be used by both precasters and contractors to accurately measure the camber.

Observations and independent camber measurements at three separate precast plants led to the following recommendations for minimizing the difference between the expected and measured camber of PPCBs:

- The camber is highly sensitive to the prestress force; therefore, monitor and apply the designed prestress force as accurately as possible.
- Aim for reaching and not exceeding the design strength of concrete at the transfer of prestress.
- Ensure consistency of the concrete mixes and base materials (e.g., aggregates) regardless of the time and day of casting.
- Ensure consistent curing conditions and ensure that PPCB curing conditions match those of the sample cylinders used for obtaining the release strength.
- When there is change in material or the curing process, time-dependent properties including creep and shrinkage behavior of concrete should be appropriately revised.
- Minimize the error in the instantaneous camber of identical PPCBs cast on different beds or at different times or days.
- Use the proposed camber measurement procedure to measure instantaneous camber.
- When PPCBs are stored in precast plants, use an overhang length of zero or L/30 (where L is the PPCB length).

Implementation Benefits

Implementing the study recommendations is expected to significantly improve the accuracy of the camber measurements and predictions.

As a result, the difficulties during bridge construction by the inaccurate camber values when the PPCBs are erected on-site will be alleviated, thereby reducing construction schedule delays, improving bridge serviceability, and decreasing costs.

11. Prestress losses, Camber Growth, Self-Consolidating Concrete (SCC)

"Prestress losses and camber growth in wing-shaped structural members"

Marco Breccolotti and Annibale L. Materazzi

PCI Journal/January-Febrary 2015

p.98

- An experimental program on pretensioned wing-shaped members has investigated the evolution of prestressing stresses and camber during the different stages of concrete hardening.
- The result allowed the careful evaluation of prestress losses in the first three weeks during concrete placement and curing, emphasizing the influence of temperature history on prestress losses and calibrating parameters for the prediction of camber growth.
- Calculations based on Eurocode 2 equations compared favorably with the data, indicating that these equations can be applied to prestressed wing-shaped members made of self-consolidating concrete as investigated in the present work.

The exploitation of the mechanical properties of the materials, the prestress force necessary to ensure an appropriate load-carrying capacity and the requirement for in-service camber for aesthetic and functional reasons necessitate monitoring of prestress and camber. Inaccurate prediction of the camber can lead to unsatisfactory service conditions.

p. 99

According to their findings, current analytical techniques, such as the time-step method, can correctly predict the camber provided that the material properties are accurately known. The found that high curing temperatures during fabrication can reduce the calculated prestress by up to 7%, reduce the initial camber by up to 40%, and increase the in-service bottom tensile stress by up to 27%.

p. 100

Attention was also placed on the use of self-consolidating concrete (SCC). In fact, while most of the properties of hardened SCC are comparable with those of conventional concrete, the modulus of elasticity at release Eci, which plays an important role in the evaluation of prestress losses, has been found to be less than that of conventional concrete mixtures with comparable compressive strength of concrete at release fci.

p. 112

Calibration of the corrective terms for camber prediction

The numerical procedure to evaluate the total camber at release and at time t was adjusted to take into account the effective material properties and the effective prestress losses that depend on the ambient temperature, on the strand tensioning procedure, and so on, but also on the uncertainties due to variation in relative humidity, temperature distribution, effective surface area in contact with the environment, and manufacturing tolerances (mainly referring to the actual geometry of the sections).

p. 113

No corrective term has been used for the deflection due to self-weight because the theoretical estimate can be considered reliable.

Figure 15 plots the ratios between the experimental camber and the camber predicted by Eq. (7) and Eq. (8). Good agreement has been found for the camber at 14 days, with a ratio between the experimental and the theoretical values between 0.95 and 1.10. Slightly worse results have been found for the camber at release.

p. 113-114

Conclusion

- The temperature history has a non-negligible influence on the development of prestress losses for steel relaxation and concrete creep.

Appropriate corrective actions are proposed as well to reduce as much as possible the prestress loss during concrete casting, curing, and hardening, and suitable values for the design parameters have been calibrated to correctly predict prestress losses and camber growth.

12. Camber, prestress losses, High Performance Concrete (HPC)
"Camber and Prestress Losses in Alabama HPC Bridge Girders"
J. Michael Stallings, Ph.D., P.E., Robert W. Barnes, Ph.D., Sam Eskildsen
PCI Journal/September-October 2003
p.90-91
Overestimation of camber and prestress losses for high performance precast concrete bridge girders may discourage the efficient use of longer spans.
Current analytical techniques can result in accurate predictions of camber and prestress losses for HPC girders if the material properties used in the analysis are representative of the actual concrete used in girder production.
The combination of longer span lengths and higher prestressing forces may lead to large calculated cambers and prestress losses. Overestimating camber and prestress losses during the design stage may unfairly discourage the use of high strength concrete and long spans – ultimately nullifying the potential increase in efficiency.
Camber is a function of time-dependent concrete creep and loss of prestress force. Prestress loss is also a function of creep, as well as concrete shrinkage and steel relaxation. Research in North America has shown that HPC tends to exhibit less creep and shrinkage than conventional concrete[1-3]. This improved performance results in reduced time-dependent effects on camber and prestress losses. Methods for estimating camber and prestress losses that were developed for conventional concrete mixes may not provide accurate results for HPC bridge girders.
The work presented here constitutes an investigation of the accuracy of existing methods for calculating camber at the time of erection for standard AASHTO girders fabricated with HPC. Applications of these methods to predict the time-dependent behavior of actual HPC bridges are scarce in the literature.
p.92
Although the design 28-day compressive strength for the girder concrete was approximately 6500 psi (45 MPa), actual strengths averaged approximately 9300 psi (64 MPa). These researchers recommended that prestress loss prediction techniques be updated to reflect the properties of the rapidly changing materials used in pretensioned concrete construction.
Prestress losses predicted by incorporating measured material properties into the PCI general time-step approach[14] were 5 to 10 percent larger than measured in the instrumented girders. Camber measured at an age of 60 days for the girder containing a limestone coarse aggregate was well predicted by the PCI multiplier method.[15] However, this method over-estimated the 60-day camber by approximately 2 in. (50 mm), or 40 percent, for the girder containing a glacial gravel coarse aggregate.
Predicted values of initial camber were significantly higher than those measured. However, the difference between experimental and analytical results remained approximately constant with time, indicating that the time-dependent growth of camber predicted by the time-step analysis closely matched that exhibited by the instrumented beams. Thus, the researchers suggested that the disagreement between measured and predicted values might be due to an underestimation of the modulus of elasticity of the concrete in the beams.

p.93

The coarse aggregate was No. 67 crushed limestone for the first twelve castings.[19]

Specified minimum release and 28-day compressive strengths were 8000 and 10,000 psi (55 and 69 MPa), respectively.

p.94

The HPC mix used in this study appears to undergo roughly half as much creep as predicted by the ACI 209 method for standard conditions.

p.95

Modeling of Prestress Losses

Prestress losses in pretensioned girders result from five primary factors: seating at the strand anchorages, elastic shortening, strand relaxation, concrete shrinkage, and concrete creep.

Relaxation is a reduction in strand stress without a corresponding change in strain.

p.98

Material and Geometric Properties

The ACI 209R-92 correction factors for these girders are listed in Table 5. An average relative humidity of 70 percent was taken from Reference 23 for the central Alabama area.

p.99

TEST RESULTS

Camber

Use of the standard parameters results in overestimation of camber. The average age for the measured cambers is 200 days; the average camber is 4.44 in. (113 mm), and the standard deviation is 0.27 in. (7 mm).

The 200-day camber calculated using the HPC parameters is 4.18 in. (106 mm), which is 6 percent less than the average measured camber. The calculated 200-day camber resulting from use of the standard parameters is 5.49 in. (139 mm), which is 24 percent greater than the average measured camber.

Erection cambers are generally considered to correspond to ages of 30 to 60 days. Final cambers correspond to long-term cambers years into the future.

p.100

These results indicate that both the incremental time-steps method and the approximate time-steps method can produce good estimate of girder cambers when material parameters are accurately known. Data from this study suggest that accurately modeling the creep characteristics of the concrete is a necessary step towards accurately estimating camber.

Strains

As it did for camber, use of the standard parameters significantly overestimates the strains.

p.101

Prestress Losses

For example, a loss in prestress force causes a loss in camber and an increase in tensile strain at the level of the prestress.

p.102

CONCLUSIONS

1. Accurate predictions of camber are possible using the incremental time-steps method and the approximate time-steps method with material parameters that are

representative of the actual concrete used in girder production. Cambers calculated by both methods using the HPC material parameters agreed well with the measured cambers.

2. New analytical techniques for camber prediction are not required for HPC. However, accurate predictions require the use of accurate material parameters. Cambers calculated using standard material parameters consistently exceeded measured values. This error resulted primarily from overestimation of the shrinkage and creep characteristics of the HPC.
3. Measured cambers were significantly less than the camber at erection estimated using the PCI multipliers method.

5. Time-dependent losses calculated using the incremental time-steps method with HPC material parameters showed good agreement with measured time-dependent losses.

6. AASHTO bridge design specifications may overestimate prestress losses due to creep and shrinkage in HPC girders.

p.102-103

RECOMMENDATIONS

1. If accurate estimate of camber or prestress losses in HPC girders are required, tests of representative concrete should be performed to assess the relevant material properties.

2. Currently available analytical techniques for computing camber and prestress losses are adequate provided that the correct time-dependent material properties are appropriately integrated into the selected procedure.

13. prestress losses
"Prestress loss calculations: Another perspective"
David B. Garber, Jose M. Gallardo, Dean J. Deschenes, and Oguzhan Bayrak
PCI Journal/May-June 2016
p.68
The accuracy, precision, and conservatism of prestress loss estimation must be carefully balanced to ensure a safe, serviceable, durable, and economically viable girder. When prestress loss is underestimated, the designer assumes a greater stress in the strands than is actually present. This underestimation can lead to service-load cracking and long-term durability concerns due to corrosion. Overestimation of prestress loss may lead to uneconomical designs and large cambers, which are both a result of an excessive number of strands being required.
p.69
Attempting to improve the accuracy of prestress loss estimation and applicability to modern materials and structural shapes, the National Cooperative Highway Research Program (NCHRP) project 18-07 was funded in 2000.[1] The end product of this research was NCHRP Report 496,[1] which provided new approximate and refined methods to estimate prestress losses. The NCHRP Report 496 methods were then incorporated into the American Association of State Highway and Transportation Officials' AASHTO LRFD Bridge Design Specifications, 3rd Edition-2005 Interim Revisions with minimal modifications.
The prestress loss procedure found in the PCI Bridge Design Manual will also be used as a point of comparison in this paper to show differing design philosophies.
Experimental procedure
The research conducted for this project was accomplished through full-scale experimental testing,[7] the assembly of a comprehensive experimental database,[8] and an analytical study investigating the sensitivity of the refined AASHTO LRFD specifications procedure and the design implications of its use.
Experimental program
In total, 30 full-scale pretensioned, precast concrete beams were fabricated to provide a relevant experimental basis for investigating the parameters influencing prestress loss and in order to assess the existing prestress loss provisions.
The concrete and coarse aggregate types were intentionally varied from series to series to investigate their effect on prestress loss. Series I and III were fabricated in San Antonio, Tex., with conventional concrete and limestone coarse aggregate; series II was fabricated in Elm Mott, Tex., using conventional concrete and river gravel coarse aggregate; and series IV was fabricated near Eagle Lake, Tex, using both conventional and self-consolidating concrete with river gravel coarse aggregate.
The specimens were conditioned at a total of five different storage locations across the state of Texas in order to investigate the effect of different relative humidities: San Antonio (average relative humidity RH of 63%), Austin (62%), Lubbock (51%), Elm Mott (63%), and Eagle Lake (75%).

p.70

The relative humidity of the conditioning sites and the modulus of elasticity of concrete at time of release Eci for each of the different concrete mixtures is also included in the figure. The stiffness of the concrete had the most effect on the development of prestress loss. The beams made with concrete with a greater modulus of elasticity (series II and IV) developed significantly smaller prestress losses than those made with concrete with a lesser modulus of elasticity (series I and III). The relative humidity had only a slight effect on the prestress loss development, with a higher relative humidity generally resulting in slightly smaller prestress losses.

p.72

The average relative humidity of the conditioning locations varied from 45% to 80%, with the majority of the specimens being conditioned in climates with an average relative humidity between 60% and 70%.

In addition, a variety of concrete mixtures with different types of aggregates are captured within the evaluation database. The majority of the specimens were fabricated using conventional concrete, though some specimens were fabricated using self-consolidating concrete. The two main types of coarse aggregate used in common practice (river gravel and limestone) make up the majority of the specimens in the database. Concrete release strengths within the database range from 4.0 to 13 ksi (28 to 90 MPa) and concrete 28-day strengths range from 5 to 15 ksi (34 to 103 MPa), with 89 of the 140 specimens attaining a 28-day compressive strength of more than 10 ksi (69 MPa).

Analytical investigation

A parametric study was undertaken to investigate the influence of various inputs on output loss estimation parameters (sensitivity analysis) and assess the impact of prestress loss estimation on beam design (impact analysis).

Sensitivity analysis

A sensitivity analysis was conducted on the 2012 AASHTO LRFD specifications' loss procedure using an extreme value analysis. In an extreme value analysis, the effect of the maximum and minimum possible values for the input variables on the output parameters is investigated. For this study, the extreme value analysis was used to investigate the effect that various input parameters have on the calculation of the different component of prestress loss.

Two different factorial designs were used in the experimental design for the extreme value analysis, one factor at a time (Table 1) and full factorial (Table 2). The one factor at a time design was used to investigate the effect of each individual input variable on the output variables. For this analysis, only one variable is set to a design extreme while the other input variables are kept at an average value (Table 1).

p.73

The general trends observed in the one factor at a time analysis can be observed in the input parameters' effect on the total prestress loss (Fig. 3). A larger spread in the plot signifies that the input variable (for example, cross-section type) has a significant effect on the output (for example, the total prestress loss). The concrete release strength and the beam length were found to have the largest impact on the total prestress loss

estimate. The cross-section size, relative humidity, and coarse aggregate correction factor were each found to have a relatively similar impact on the loss estimate. The time of deck placement does not significantly affect the loss estimate.
p.74

Optimizing existing methods

The concrete material models[13-15] typically have been developed and calibrated based on comprehensive creep and shrinkage databases primarily composed of standard concrete cylinder size samples (4 in. [100 mm] diameter and 8 in. [200 mm] length). The researchers found that these models did not adequately represent the observed behavior in full-scale prestressed beam specimens (when directly implemented).
p.76

The simplicity of the various proposals was investigated by comparing the total variables and total number of mathematical operations required to estimate the final prestress loss for each of the procedures,

- The PCI Design Manual has approximately 40 operations and 14 variables.
- The 2004 AASHTO LRFD specifications has approximately 40 operations and 12 variables.
- The 2012 AASHTO LRFD specifications has approximately 600 operations and 70 variables.
- The proposed method has approximately 60 operations and 24 variables.

p.77

Consideration of typical construction details

Two different variables introduced in the 2012 AASHTO LRFD specifications procedure were found to have minimal variation throughout the analytical investigation, the transformed section coefficient Kid and the shape factor ks. Figure 7 shows a small sample of these results. The transformed section coefficient was found to always fall between 0.8 and 0.9 for typical bridge configurations in which the prestress loss would affect design. The shape factor was found to nearly always be 1.0 (other than in a few situations where it was only up to 1.05). With these two observations in mind, the procedure was simplified by setting the transformed section coefficient to 0.9 (a conservative simplification) and setting the shape factor to 1.0.
p.78

As the concrete release strength increases, the conservatism of the 2012 AASHTO LRFD specifications' prestress loss estimate decreases, as demonstrated by the line of best fit. From the investigation of the elastic shortening procedures, there was little variation observed between the performances of each of the methods. For this reason, a gross-section approximation was chosen because it offered both conservatism and simplicity.
p.82

Conclusion and recommendations

In the authors' opinion, additional complexity is warranted if a benefit from it can be derived in terms of accuracy and/or precision. The 2012 AASHTO LRFD specifications do not offer additional precision over the procedure proposed in this paper and is 10 times more computationally intensive. Approximately 600 mathematical operations are required for the AASHTO LRFD specifications' prestress loss procedure compared with the 60 operations required for the proposed procedure.

14. probabilistic comparison of prestress loss methods, variability of prestress losses
"A Probabilistic Comparison of Prestress Loss Methods in Prestressed Concrete Beams"
Christopher G. Gilbertson, P.E and Theresa M. (Tess) Ahlborn, Ph.D., P.E.
PCI Journal/September-October 2004
P.52-53

Camber and long-term deflections are directly linked to the loss of prestressing force, a characteristic of all prestressed concrete members. In this investigation, the effects of the inherent variability of the parameters used to estimate prestress loss was studied for two typical bridge beam cases using several prestress loss predictive methods at final service conditions: A parametric study was conducted to assess the general effect of single parameter variation on the calculation of prestress loss. Monte Carlo simulations were used to assess the distribution of prestress loss considering the variability of all parameters simultaneously. Results of the Monte Carlo simulations were also used to evaluate the impact of loss variability on deflection and cracking moment calculations. Results show that the variability of parameters has a notable effect on prestress loss variation and that this variation significantly influences the estimated deflection and cracking moment of prestressed concrete beams. Indeed, the variation in camber between beams cast on the same prestressing bed can be partially explained through the variability of materials and predictive methods used.

Loss of prestress is a characteristic of all prestressed concrete members wherein the level of prestress force first applied to the member is reduced over time due to short- and long-term conditions. Many methods have been used to estimate the prestress losses in prestressed concrete members. These methods produce discrete values representing the expected losses, and vary considerably in both length and complexity of calculations. The methods consider many factors in their respective calculations but do not consider the effect of variability due to the parameters such as concrete strength, strand stress, strand area, dimensional properties, and environmental conditions. The purpose of this study was to investigate the effect of input variation on the computed losses from six common methods used to estimate prestress losses.

BACKGROUND

An accurate prediction of the prestress loss in a prestressed concrete member is important in the design phase to assess the expected behavior of the member over its life. Calculated losses are used to predict service conditions such as expected concrete stress levels, camber, deflection, and cracking loads. However, an accurate prediction of prestress loss is difficult because of the complex interaction between the various source of losses and the inherent non-homogeneity of concrete members.

The primary objective of the research presented herein was to compare the variability of total prestress losses computed by several methods while accounting for parameter variability through a probabilistic assessment.

p.61

DISCUSSION OF RESULTS

Those parameters having the greatest effect on total losses were initial strand stress, initial concrete strength at release, relative humidity, and strand eccentricity.

p.62-63

CONCLUDING REMARKS

Prestress losses, as well as camber and deflection, are affected by several parameters with inherent variability. This study compares predictive methods for prestress losses and the impact of losses on predicted camber and deflection due to the variability of parameters used for such predictions.

For the two typical bridge systems studied herein, the primary influencing parameters were jacking stress (fpj), compressive strength of concrete at strand release (f'ci), relative humidity (RH), and eccentricity of strand €. The inherent variability of these and all prestress loss parameters, compounded by the complex interactions of prestress loss predictions, helps to justify camber and deflection variations often seen in the field.

Lastly, the authors hope that this study will provide designers with some insight into the relative importance of the various factors that influence the determination of prestress losses.

15. effects of production practices on camber
"Effects of production practices on camber of prestressed concrete bridge girders"
Tyler K. Storm, Sami H. Rizkalla, and Paul Z. Zia
PCI Journal/Winter 2013
p.96-97

- This paper presents the results of research to investigate factors related to prestressed concrete girder production that could affect the camber and to recommend camber prediction methods.
- These factors include higher concrete compressive strengths that specified, curing method, girder type, changes in cross section due to deformation of internal void forms, strand debonding, and transfer length.
- A refined camber prediction method was developed that uses creep coefficients and prestress losses based on the 2010 AASHTO LRFD Bridge Design Specifications. An approximate method based on the PCI camber multipliers was also proposed. Both methods compared well with the measured cambers of 382 prestressed concrete bridge girders, though the former was more accurate for the majority of girders.

Accurate predictions of camber and deflection often pose a challenge for bridge engineers. Excessive discrepancy between the predicted and actual camber can cause problems for deck construction. Many state departments of transportation (DOTs) have previously investigated some aspects of this problem[1-5] and found considerable variations between the predicted and actual cambers. For example, Kelly et al.[1] noted that the camber for eight identical American Association of State Highway and Transportation Officials (AASHTO) Type IV girders that were 127 ft (38.7 m) in length varied from 2 to 6 in. (50 to 150 mm) at the time of prestress transfer. Several other studies[3,6-9] also examined this issue relative to the use of high-strength concrete.

To predict camber accurately is difficult because camber depends on many random variables, some of which are interdependent and change over time. Some of the most important variables are the compressive strength and elastic modulus of concrete, amounts of creep and shrinkage, thermal gradients within the girder, and the time-dependent variations in prestressing force. When predicting camber at the design stage, bridge engineers typically calculate prestress losses and concrete properties based on the specified concrete strength at various ages because they have no knowledge of the actual concrete properties prior to manufacture. Camber is also influenced by the time history of loading and environmental conditions. Complicating matters further is that camber is the net result of two large opposing quantities: upward deflection due to prestress and downward deflection due to dead loads. Because these two quantities are each subject to some inherent variability, one cannot expect to always predict the net camber accurately.

p.97-98

The 2010 AASHTO LRFD Bridge Design Specifications provide both simplified and detailed methods for estimating creep, shrinkage, and prestress losses. They also require camber and deflection to be calculated but do not provide specific procedures. The PCI Design Handbook: Precast and Prestressed Concrete recommends the approximate method developed by Zia et al.[12] for estimating loss of prestress and provides a simplified procedure for camber and deflection calculation using multipliers, a concept originally developed by Martin.[13]

These PCI methods were developed more than 30 years ago, largely based on the properties of lower-strength concrete than what is typically used today and calibrated primarily against the performance of prestressed concrete building members. The specified constant multipliers account for the creep effect due to sustained load and are suitable for conventional building designs under average environmental conditions. For bridges, more detailed analysis methods are needed to account for widely varying environmental conditions and other time-dependent factors. However, many commercially available design software programs and even in-house developed design software programs used by state departments of transportation and others have used the constant multiplier method because of its simplicity. These programs are still being used today by many bridge designers in both the public and private sectors.

In 1985, Tadros et al, [14]developed refined time-dependent multipliers for long-term deflection calculations and a refined method for estimating prestress losses. These refined approaches, unlike the PCI method, would account for the effects of various environmental conditions and the presence of nonprestressed steel reinforcement, which tends to restrain creep and shrinkage of concrete.

The PCI Bridge Design Manual provides excellent commentaries on the complexity of estimating prestress losses and its implications on design. Methods for estimating loss of prestress prescribed by the AASHTO LRFD Bridge Design Specifications are described and illustrated by examples. The PCI Bridge Design Manual also recommends a set of multipliers for computing long-term camber and deflection but cautions that the use of the multipliers only gives "reasonable estimates of cambers at the time of erection" and "the method does not properly account for the significant effects of a large cast-in-place deck." It also warns designers that "prestressing levels should not be increased in order to reduce or eliminate long-term downward deflection that might be predicted if the give multipliers are used."

Based on the camber measurements, it was shown that the PCI Design Handbook multiplier method significantly overestimated the camber at the time of girder erection. Both the approximate time-step method and the incremental time-step method predicted camber reasonably well.

The most important factors in camber prediction are the elastic modulus and creep of the concrete, which vary with its constituents, the production process, and age. For example, Tadros et al.[6] showed that the stiffness of the coarse aggregate used in the concrete, which typically varies with the aggregate source, can introduce significant variations when estimating elastic modulus. They recommended the application of an elastic modulus adjustment factor K1, applied to the 2004 AASHTO LRFD specifications equation to account for aggregate stiffness. Their recommendation was subsequently adopted in the AASHTO LRFD specifications beginning with the 2005-2006 interim revisions. Kelly et al.[1] also noted that the actual concrete compressive strength is often much higher than the specified strength.

Based on their study of eight prestressed concrete girders with specified 28-day strengths of 6500 psi (45 MPa), they found that the average measured 28-day strength was approximately 9300 psi (64 MPa), more than 40% higher than the specified strength. This discrepancy results in a higher elastic modulus than would be predicted using the specified strength, consequently reducing the measured camber compared with the predicted value. Tadros et al,[6] also observed that it is typically assumed by the designer that the time for prestress transfer s one day after girder casting, though it is fairly common in practice to allow girders to cure over the weekend, thus delaying the prestress transfer. The extra curing time allows the elastic modulus of the concrete to become higher than the value predicted for early prestress transfer, resulting in poor predictions of initial camber. Because creep is sensitive to the strength of the concrete at the time that prestress transfer occur, it could also lead to poor predictions of camber at later stages.

The prestressing force may also be affected by the thermal expansion of the prestressing strands, prior to prestress transfer, caused by changes in the concrete temperature during curing. Bruce et al.[7] showed that due to cement hydration during concrete curing, the temperature of the strands can increase, causing a reduction of the prestress force by as much as 11% due to thermal expansion. However, because the concrete likely bonds to the steel within six to eight hours of casting, some portion of the force would likely be regained with cooling. They estimated that the loss of prestress due to this effect was approximately 6%.

Tadros et al.[2] noted that thermal gradients can develop through the depth of the girder due to uneven heating and cooling or due to solar effects. This gradient can temporarily cause additional camber or deflection in the girder, thereby introducing scatter into the camber measurements. Byle et al.[8] estimated that thermal gradient effects could produce deflections of approximately 0.5 in. (13 mm) for the U girders that they studied, which ranged from 115 to 145 ft (35.1 to 45.2 m) in length.

This paper presents the results of study[19] conducted by the authors, including field and laboratory measurements, to examine the various parameters affecting the camber predictions, with particular attention to factors related to girder production. Based on the finding of this study, two camber prediction methods are proposed and compared with the results obtained from the field.

Field and laboratory measurements

A large number of companion cylinders for each girder were also obtained from the producers to determine compressive strength, elastic modulus, and unit weight in the laboratory at different ages. In general, camber was measured for each girder immediately after prestress transfer, at the beginning of storage in the yard, prior to shipment to the bridge site, and after erection.

Factors affecting the prediction of camber

Several factors related to the production of prestressed concrete girders were found to significantly affect the prediction of camber. These include the concrete

properties, deformation of the internal voids of box beams and cored slabs during casting, strand debonding, prestress transfer length, temperature changes in the strands after initial stressing, production schedule, and curing method.
p.99-100

Concrete properties

Predictions of prestress losses and camber depend on the properties of the concrete being used for the girder. Important properties are the compressive strength and the elastic modulus.

Compressive strength

To ensure acceptance, each girder producer generally has several preapproved concrete mixture designs that will produce quality concrete with average compressive strength significantly higher than the minimum strength specified by the DOT. Therefore, the elastic modulus is generally underestimated by using the specified strength. Similarly, the prestress losses, which are also related to the concrete strength, may be overestimated. Therefore, to improve the predictions of camber and prestress losses, it is critical to have a good estimate of the actual compressive strength.

Based on the collected data for the girders included in this study, the average ratio of the measured compressive strength at prestress transfer to the specified strength at transfer was found to be 1.24 with a range of approximately 1.0 to 2.1 (Fig. 3). Based on this result, it is recommended that the concrete strength at prestress transfer to be used for predicting camber fci be calculated using Eq. (1). Fci* = 1.25 fci

A similar analysis of the test results for 78 sets of concrete cylinders showed that the average ratio of the measured 28-day compressive strength to the specified 28-day strength was 1.45 with a range of approximately 1.0 to 2.2 (Fig. 4). Based on this result, it is recommended that the 28-day compressive strength to be used for predicting camber fc should be calculated using Eq. (2). Fc* =1.45fc

For production in other states, it is advisable to validate the two coefficient in Eq. (1) and (2) based on local/regional conditions.

Elastic modulus

To evaluate the accuracy of the AASHTO LRFD specifications equation for estimating the elastic modulus of concrete Ec, the average ratio of the measured elastic modulus to the predicted value for girders produced for North Carolina Dot (NCDOT) bridges was calculated. The average ratio for 153 concrete cylinders tested in the laboratory was found to be 0.85 with a range of approximately 0.62 to 1.15. Based on this analysis, it is recommended that the 2010 AASHTO LRFD specifications equation be used to predict the elastic modulus for the camber predictions with the aggregate adjustment factor K1 taken as 0.85 and the unit weight of concrete wc taken as 150 lb/ft^3
p.101-102

Debonding and transfer length

Partial debonding of prestressing strands near the ends of prestressed girders reduces the prestressing moment in this region and thus reduces the camber. The prestressing moment is also reduced over the transfer length at the ends of a girder. However, both effects are typically ignored in camber calculations by the design engineers. Based on an analysis of the 382 girders in the database, considering the effects of debonding and transfer length reduced the predicted camber by less than 3% for the vast majority of the girders. The effect was more pronounced, however, for girders having partial debonding lengths of approximately 10 ft (3 m) or greater at each end, for which the error could be as high as 13%.[19,20] Based on this analysis, it is considered appropriate to include the effects of debonding and transfer length when calculating camber, particularly for girders with long debonding lengths.

p.102

Temperature of the strands

Prior to prestress transfer, the prestressing force was found to vary according to the temperature in the strands after initial stressing due to thermal expansion of the strands. Significant temperature fluctuations can be caused by exposure to the sun, cement hydration-induced heating during curing, or heat curing. A theoretical analysis as well as direct measurements showed that the prestressing force could be temporarily reduced by more than 7% due to this effect.

Girder production schedule

It is typically assumed when predicting camber that transfer of the prestressing force will occur one day after casting, and the elastic modulus and prestress loss calculations are therefore based on this assumption. However, it is often the case that the girders remain in the forms over the weekend, with prestress transfer occurring after three days. Because creep is highly sensitive to the concrete properties and prestressing force at the time of prestress transfer and because both of these properties are changing rapidly during this time, the delay has the potential to affect the predictions of both the initial and long-term cambers. In addition, the timing for casting the composite deck often varies greatly from project to project. Some girders are kept in storage for several months and in extreme cases up to a year before being shipped for installation, causing increased uncertainty in the predicted camber at the time of erection.

Curing method

Precast, prestressed concrete girders are typically cured either by moist curing or by heat curing using steam pipes. The particular curing method used was found to significantly affect the net camber at the time of prestress transfer, as is discussed later in this paper.

p.104-105

Approximate method

The approximate method is based on the PCI multiplier method.[11,16] This method does not require calculation of the time-dependent losses.

Calculate the camber at 28 days using 1.80 times camber due to prestressing and 1.85 times deflection due to self-weight.

Calculate the camber at one year using 2.45 times camber due to prestressing and 2.70 times deflection due to self-weight.

This multiplier method gives reasonable estimates for cambers at the time of erection, but it does not properly account for the significant effects of a large cast-in-place concrete deck.[13]

Refined method

The 2010 AASHTO LRFD specifications provide a detailed method for estimating the prestress losses at any given time. However, they do not specify a procedure to predict camber. Therefore, this paper introduces a detailed method for predicting camber that uses the time-dependent loss calculations given by the 2010 AASHTO LRFD specifications.

This method can be used to predict camber at any time before placement of the deck or superimposed dead loads. However, because the exact date of girder erection is often not known during design, it is recommended that prestress losses and camber be estimated at transfer, at 28 days, and at one year to obtain a representative range of values.

p.106

Evaluation of the proposed prediction methods

The camber data were grouped by girder type, curing method, and the time at which camber was measured.

Camber at prestress transfer

The calculation of the camber at prestress transfer is identical for both methods. Figure 8 shows that the effect of the curing method on the camber at prestress transfer is significant.

Because the predicted camber value is always equal for moist-cured and heat-cured versions of the same girder, it follows from the graph that the average measured camber at transfer for most girder types was significantly less for the head-cured versions that for the moist-cured versions. This discrepancy may be caused by at least two factors that are potentially significant in head-cured girders: the presence of a thermal gradient within the concrete at transfer due to uneven cooling and the reduction in the prestressing force due to the thermal expansion of the strands. For modified bulb tees, the curing method did not seem to affect the camber at transfer as significantly as it did the other member types. This could be due to a potentially less substantial thermal gradient effect in the modified bulb tee because its unique shape and greater depth result in a different thermal profile during cooling.

p.107-108

Camber at 24 days and later

The data for camber measurements taken at 24 days after casting or later provide the best means to evaluate the prediction methods. The focus for this research is to improve the prediction of camber at the time of girder erection, which typically occurs at least four weeks after casting.

The analysis indicates that both the approximate method and the refined method provide reasonably accurate camber predictions, though the refined method is more accurate for most of the girder types and curing methods (Fig. 9). When the refined method is used, the average error is less than 10% for most of the data groups. When the approximate method is used, the error is between approximately 10% and 20% for most of the data groups.

Conclusion

- The camber predictions should account for the typically higher concrete strength at prestress transfer and at 28 days compared with the specified values.
- The coefficients in Eq. (1) and (2), developed from this study based on concrete materials normally provided for NCDOT, should be validated or determined for concrete supplied from other localities and regions. However, these coefficients are simply the averages of a widely varying value and therefore should not be viewed as anything more than an estimate.
- The camber predictions should consider the reduced curvature at the ends of the girder due to debonding and transfer length, especially for girders with long debonding lengths.
- The camber of girders at the time of prestress transfer can be significantly affected by the curing method used. Heat-cured girders-especially box beams and cored slabs-tend to have significantly less camber at transfer than moist-cured girders, though there was not a significant difference in the camber at later stages between girders cured using either method.
- Due to production variables, the measured camber can vary significantly among girders that are identical in their design even if the girders are cast at the same time on the same casting bed, in part because multiple batches of concrete are typically used for a single casting.
- The refined method provides the most accurate camber predictions for most girder types and curing methods. The approximate method generally overestimates camber at erection slightly, but it is suitable for preliminary estimates and rough by-hand calculations.

16. extending span capabilities WSDOT

"New Deep WSDOT Standard Sections Extend Spans of Prestressed Concrete Girders"

Stephen J. Seguirant, P.E.

PCI Journal/July-August 1998

p.93

The primary goal of the study was to increase the span range capability of standard prestressed concrete girders, and to improve economy by increasing the allowable girder spacing over previous designs.

p.96

For safety reasons, the slope of the harped strands should not exceed 8 horizontal to 1 vertical.

p.98-99

Pretensioned Simple Span Capabilities

The span capability envelope for the W21MG girder section is shown in Fig. 11. This envelope assumes that the maximum pretensioning capability of precasting plants in the Northwest is sixty-four each 0.6 in. (15.24 mm) diameter strands, and that 200 kips (889.6 kN) is the maximum weight that can be handled and shipped. This translates into a maximum single-piece girder length of approximately 185 ft (56.39 m).

Prestress Losses

One of the most significant variables influencing the span capability of prestressed concrete girders is the method used to calculate long-term prestress losses.

Pessiki et al.[12] have recently evaluated the effective prestress force in two 28-year-old prestressed concrete bridge beams. Their conclusion was that the average actual loss experienced by these beams was approximately 60 percent of the losses predicted using current calculation methods. Similar results have been reported over the past decade.

All predicted values overestimate the actual losses, with the ACI-ASCE Committee 423 Method providing the closest estimate. For the purpose of this paper, subsequent sections will use either the AASHTO LRFD Approximate or Refined Methods, whichever results in the lesser value.

Allowable Tension

Another significant variable influencing span capability is the amount of tension allowed in the precompressed tensile zone at the service limit state.

Currently, WSDOT allows no tension in the precompressed tensile zone under service loads.

p.101-102

Concrete Strengths

Overnight strengths of up to 7.2 ksi (49.64 MPa) have been consistently attained at Concrete Technology Corporation (CTC) with concrete mixes containing silica fume. Higher release strengths can be achieved on an every-other-day basis with added cost.

As mentioned previously, the design concrete strength used to develop the span capability envelopes was 10.0 ksi (68.95 MPa). This value is the maximum that PNW/PCI members feel they can consistently achieve at this time.

The applied criteria of zero tension and simple spans results in the concrete strength at release governing the design.

Additionally, industry statistics have shown a strong correlation between concrete release strength, curing time and temperature, and the design concrete strength. For a given mix design, the more aggressively the concrete is cured to achieve a high release strength, the lower the long-term strength will be.

p.105-106

Weight Limitations

Girders shipped in some states have weighed in excess of 200 kips (889.6 kN). The net weight limitation with trucking equipment currently available in Washington State is approximately 167 to 180 kips (742.9 to 800.7 kN), if a reasonable delivery rate (number of pieces per day) is to be maintained.

Product weights of up to 200 kips (889.6 kN) can be hauled with currently available equipment at a limited rate.

Length Limitations

Length limitations are generally governed by turning radii on the route to the jobsite. Potential problems can be circumvented by moving the support points closer together (away from the ends of the girder), or by selecting alternate routes. A rule of thumb of 130 ft (39.62 m) between supports is commonly used.

p.108

CONCLUDING REMARKS

The development of new standard deep girder sections gives WSDOT the ability to extend spans and remove piers from environmentally sensitive areas, all with the superstructure material they prefer to specify.

17. camber tolerances
Committee on Bridges-Camber FAST Team
PCI 2012 Committee Days
Revision #0, dated 3/27/12

Mission Statement

To evaluate the current PCI tolerances for camber of bridge girders published in PCI MNL-116 and make recommendations for PCI tolerances with respect to predicted camber at time of prestress transfer.

The PCI Manual for Quality Control for Plants and Production of Structural Precast Concrete Products (PCI MNL-116-99) defines the tolerance for camber variation from design camber as +/- 1/8" per 10 feet of girder length with a maximum of +/- ½" for girders up to 80 feet long and +/- 1" maximum for girders over 80 feet long. Today, girder lengths commonly exceed 100 feet with some exceeding 200 feet. Therefore, the current tolerances with their maxima are overly restrictive. Furthermore, some agencies are arbitrarily applying these camber variation tolerances to girders at ages other than 72 hours within release. The PCI Committee on Bridges formed a FAST Team to evaluate the current PCI tolerances for camber of bridge girders published in PCI MNL-116 and make recommendations for PCI tolerances with respect to predicted camber at time of prestress transfer.

To gain a better understanding of actual camber variability, measured and predicted camber values at release were collected for a variety of girders from various regions around the United States. The data represents I-girders ranging in depth from 35 to 100 inches. Girder lengths ranged mainly from 70 to 170 feet. Locations represented in the data pool include CA, WA, SD, KS, IN, KY, VA, and VT.

Evaluating or recommending a particular method of estimating camber was not within the scope of the FAST Team.

The following language has been taken from Appendix B, page B.3 of the Manual for Quality Control for Plants and Production of Structural Precast Concrete Products, Fourth Edition, also known as MNL-116-99.

"Camber – The deflection that occurs in prestressed concrete members due to the net bending resulting from the eccentricity of the prestress force. For members with span-to-depth ratio at or exceeding 25, the camber tolerance given herein may not apply. If the application requires control of camber to the listed tolerance in beams with high span-to-depth ratio, special production measures may be required.

Prediction of camber in a prestressed member is based on empirical formulas. The accuracy of these estimated values decreases with time. Measurement of camber for comparison of predicted design values should be completed within 72 hours of transfer of prestress.

Temperature variation across a member section can have a significant impact on the measured camber. Camber should be evaluated under conditions that minimize the effect of temperature variation due to solar radiation, such as early in the morning."

Camber FAST Team Recommendation #1:

Revise the 'g' dimension on Page B.25 in Appendix B of PCI MNL-116 to:

g = camber variation from design camber within 72 hours of release…………

+1/8 inch per ten feet, up to a maximum of 1-1/2 inches
-1/8 inch per ten feet with no lower bound

Camber FAST Team Recommendation #2:

Add a footnote on Page B.25 in Appendix B of PCI MNL-116:
Out of tolerance camber should not be a sole cause for rejection.

18. bond behavior of debonded strands

"Predicting the Bond Behavior of Prestressed Concrete Beams Containing Debonded Strands"

Bruce W. Russell, Ph.D., P.E., Ned H. Burns, Ph.D., P.E., and Leslie G. ZumBrunnen

PCI Journal/September-October 1994

p.60

This research also shows that the currently required multiplier of 2.0 for the development length of debonded strands can be significantly reduced for some cases.

In the construction of pretensioned concrete beams, prestressing strands are concentrated in the bottom of the cross section to provide maximum efficiency to resist flexural loads. Because of the concentrated prestressing forced, the allowable tensile and/or compressive stresses can be exceeded in the end regions of a simply supported beam.

p.61

The debonding, or blanketing, of strands is an alternative to draping strands in an effort to control the maximum tensile and compressive stresses in pretensioned concrete highway girders. Debonding, by definition, is the intentional breaking of bond between prestresseing strand and concrete. This can be done by applying grease to the strands in the regions requiring debonding; however, the most common practice is to wrap specially made split plastic tubing around the strand to prevent bond of the strand to the concrete. Debonding strands can simplify girder construction; draping of strands is more difficult and more dangerous. Debonding of strands like-wise exhibits economic advantages when compared to draping of strands.

CURRENT AASHTO AND ACI CODE REQUIREMENTS

Current code provisions of the American Concrete Institute (ACI) and the American Association of State Highway and Transportation Officials (AASHTO) governing the use of debonded strands are nearly identical.

This provision requires that debonded strands be bonded for a length equal to twice the required development length for fully bonded strands.

Even though the AASHTO Specifications allow debonded strands, many state DOTs do not specify their use because they fear that debonding strands significantly weakens the pretensioned beam. The states of Texas and Oklahoma do not currently allow debonded strands as an alternative to draping strands for I-shaped girders.

p.62

Debonding Strands: Lower Effective Prestress Force

By debonding strands, the effective prestress force is reduced in the end regions of the beams, when compared with beams that contain fully bonded strands.

p.63

Staggered Debonding vs. Concurrent Debonding

However, the behavior of concurrently debonded specimens was quite different from specimens with staggered debonding. Staggering the debonding has the effect of gradually increasing the effective prestress force through the debonded regions, thus improving the beam's resistance to cracking.

p.66

First, the embedment length necessary to prevent bond failures is dependent on the length of debonding. Second, longer debonded lengths require greater embedment lengths to ensure strand anchorage. Therefore, it is incumbent upon the designer to maintain the debonded lengths of strand at their shortest possible distance.

p.74

Staggered Debonding vs. Concurrent Debonding

These tests clearly demonstrate that beams with staggered debonding can outperform beams with concurrent debonding.

p.75-76

CONCLUSIONS

6. The behavior of beams made with debonded strands is predictable and reliable. Therefore, the use of debonded strands should be considered safe, provided the transfer zone of debonded strands is not allowed to extend into regions where cracking will occur at ultimate limit states.

7. In general, staggered debonding should be employed; concurrent debonding may lead to premature anchorage failures. Concurrent debonding results in a lower cracking moment in the debond/transfer zone when compared with staggered cutoff points. Consequantly, concurrent debonding can lead to bond failures where staggered debonding will not.

RECOMMENDATIONS

1. Debonded strands may be employed as an alternative to draped strands; however, the debond/transfer zone should not extend into regions of flexural cracking. In simply supported beams, strand debonding should be terminated within 15 percent of the span length, measured from the end of the beam.

2. Debond termination points should be staggered to increase the beam’s resistance to cracking in the debond/transfer zone.

3. Code provisions should be restructured to reflect the relationship between cracking and anchorage failures, and to more accurately reflect the behavior of beams made with debonded strands.

19. strand development length, strand transfer length
“A Review of Strand Development Length for Pretensioned Concrete Members”
C. Dale Buckner, Ph.D., P.E.
PCI Journal/March-April 1995
p.84
The specific objectives of the study were: (1) conduct a review of literature related to strand transfer and development length research; (2) analyze data from recent studies and rationalize discrepancies among conclusions drawn from these studies; and (3) recommend equations for strand transfer and development lengths consistent with the current state-of-knowledge.
p.86
In October 1988, the FHWA issued a memorandum that imposed the following restrictions on seven-wire strands in bridge applications:[9]
1. The use of 0.6 in. (15.2 mm) diameter strand in a pretensioned application shall not be allowed.
2. Minimum strand spacing (center-to-center) will be four times the nominal strand diameter.
3. Development length for all strand sizes up to and including 9/16 in. (14.3 mm) shall be determined as 1.6 times AASHTO Eq. (9-32).
4. Where strand is debonded (blanketed) at the end of a member in the precompressed tensile zone, the development length shall be determined as 2.0 times AASHTO Eq. (9-32), as currently required by AASHTO Article 9.27.3.
p.97
SUMMARY AND RECOMMENDATIONS
The objectives of the present study were to conduct a review of literature relating to transfer and development of seven-wire pretensioning strand, to rationalize discrepancies among conclusions drawn from various studies, and to recommend equations for strand transfer and development lengths. At present, there are several research projects in progress related to strand development length. Thus, recommendations made in this paper will need re-evaluation as additional data become available.

20. Anchorage zone reinforcing
"Design of Anchorage-Zone Reinforcement in Prestressed Concrete Beams"
Peter Gergely and Mete A. Sozen
PCI Journal/April 1967
p.63
The object of this paper is to introduce a simple method for the design of transverse reinforcement to restrain anchorage-zone cracks and to present a series of test results to confirm the validity of the method.
p.69-72
DESIGN RECOMMENDATIONS
The steel stress is controlled by a limiting crack width through an approximate force-slip relationship. The pivotal assumption is that there is a longitudinal crack in the anchorage zone. The prime role of the reinforcement is to confine the crack.
It is advisable not to have stirrup spacing larger than h/5 over a distance of h from the end of the beam.

21. horizontal cracking in the ends
"Control of Horizontal Cracking in the Ends of Pretensioned Prestressed Concrete Girders"
W. T. Marshall and Alan H. Mattock
p.56-58
SYNOPSIS
This paper describes a limited investigation of the stresses which occur in the ends of pretensioned prestressed concrete girders at the time of transfer of prestress, and which can result in the formation of horizontal cracks in the ends of such girders. Also reported is a study of the stresses set up in vertical stirrup reinforcement near the ends of pretensioned prestressed girders when horizontal end cracking does occur. On the basis of experimental data obtained in this study, an equation is proposed for the design of vertical stirrup reinforcement necessary to restrict the size of any horizontal end cracks which may occur in a pretensioned prestressed concrete girder.
When an adequate amount of vertical stirrup reinforcement is provided in the end regions of pretensioned prestressed girders, the development of the horizontal cracks is restricted.
p.72
The total cross-sectional area of stirrups necessary, At, will then be given by Eq. (4) as:
$A_t = S/(f_s/2) = 0.021(T/f_s)(h/l_t)$ Eq. (5)
The amount of stirrup reinforcement calculated using Eq. (5) should be distributed uniformly over a length equal to one fifth of the girder depth, measured from the end face of the girder. For most efficient crack control the first stirrup should be placed as close to the end face of the girder as possible, since surface crack width is to some extent controlled by concrete cover[6]. It is suggested that for design purposes lt may be assumed to be 50 times the strand diameter.

22. Estimating camber and deflection
"A Rational Method for Estimating Camber and Deflection of Precast Prestressed Members"
Leslie D. Martin
p.100
The determination of long-time cambers and deflections in precast prestressed members is somewhat more complex because of the:
1. Effect of prestress and the loss of prestress over time;
2. Strength gain of the concrete after release of prestress; and the
3. Camber or deflection is important not only at the "initial" and "final" stages, but also at erection, which occurs at some intermediate stage, usually from 30 to 60 days after casting.
Much research has been done on the effects of creep and shrinkage on prestressed concrete members, not only regarding camber/deflection behavior, but also on the related issue of prestress losses.
Some relatively precise and complex equations have been developed for predicting these long-time behaviors. However, the data on which these equations are based usually has a scatter of at least 15 to 10 percent,[2,3] using laboratory controlled specimens.
p.101-102
Synopsis
The author presents a step-by-step rational procedure for determining long-time multipliers for camber and deflection of precast prestressed concrete members.
Table 2 illustrates the sensitivity of the equations to various variables and shows that except for extremely long members that variations are within the tolerances prescribed by the PCI Manual for Quality Control.
Section 4.1 of the PCI Design Handbook, first edition, illustrates that use of "multipliers" for determining the long-time cambers and deflections.
The Handbook suggest a range of 1.5 to 3.0 for these multipliers, but does not provide a guide to the designer for determining the values.
It should be noted that because of the inherent variables that affect camber and deflection, such as concrete mix, storage method, time of release of prestress, time of erection and placement of superimposed loads, relative humidity, etc., and the data scatter under the most closely controlled tests, calculated long-time values should never be considered any better than estimates.
Determination of Multipliers
Since the release strength of precast, prestressed members is usually about 70 percent of the 28-day strength, Eci is about 85 percent of the final.
p.103
Erection Camber
The camber at the time of erection is also important. This occurs usually at 30 to 60 days following casting. Research has shown[3] that creep and shrinkage, the primary factors in long-term behavior, will have reached about 40 to 60 percent of ultimate in that time.

Therefore, it is reasonable to assume that one-half of the long-time camber, deflection, and losses will have occurred by then. The multiplier for the erection phase would then be:

p.104-105

Sensitivity of Cambers and Deflections to the Variables

In order to determine the sensitivity of the foregoing equations to the variables encountered, cambers of five different precast prestressed members were computed using the stated assumptions, and then were recalculated by changing the variables one at a time.

The changes in the variables and the results of the camber calculations are shown in Table 2. Note that only with Member No. 5, which is an extremely long span for a precast product, does the difference exceed ¾ in., which is the maximum tolerance allowed (from the calculated value) by the PCI Manual for Quality Control.

23. Estimating prestress losses
"Recommendations for Estimating Prestress Losses"
PCI Committee on Prestress Losses
PCI Journal/July-August 1975
p.46

COMMITTEE STATEMENT

This recommended practice is intended to give the design engineer a comprehensive summary of research data applicable to estimating loss of prestress. It presents a general method whereby losses are calculated as a function of time.

This report contains information and procedures for estimating prestress losses in building applications. The general method is applicable to bridges, although there are some differences between it and the AASHTO Standard Specifications for Highway Bridges with respect to individual loss components.

A precise determination of stress losses in prestressed concrete members is a complicated problem because the rate of loss due to one factor, such as relaxation of tendons, is continually being altered by changes in stress due to other factors, such as creep of concrete. Rate of creep in its turn is altered by change in tendon stress. It is extremely difficult to separate the net amount of loss due to each factor under different conditions of stress, environment, loading, and other uncertain factors.

In addition to the foregoing uncertainties due to interaction of shrinkage, creep, and relaxation, physical conditions such as variations in actual properties of concrete made to the same specified strength, can vary the total loss. As a result, the computed values for prestress loss are not necessarily exact, but the procedures here presented will provide more accurate results than by previous methods which gave no consideration to the actual stress levels in concrete and tendons.

An error in computing losses can affect service conditions such as camber, deflection, and cracking. It has no effect on the ultimate strength of a flexural member unless the tendons are unbonded or the final stress after losses is less than 0.5fpu.

24. camber predictions
"Improving Predictions for Camber in Precast, Prestressed Concrete Bridge Girders"
Michael A. Rosa, John F. Stanton, and Marc O. Eberhard
Washington State Transportation Center (TRAC)
Research Report Agreement T2695, Task 68 Camber Prediction
March 2007
p.xi-xii

EXECUTIVE SUMMARY

This research was conducted to develop improved methods of predicting camber in prestressed concrete girders.

The results showed that the response was sensitive to the predicted prestress losses and that the 2006 AASHTO values for prestress loss provided much better estimates than did the 2004 provisions. In addition, the camber was found to depend on the elastic modulus of the concrete, its creep coefficient, and the use of the prestress losses in the calculation of creep camber. Predicted cambers were compared to the measured cambers to calculate a predicted error. To achieve the best match with the measured cambers, the AASHTO-recommended values for the elastic modulus and the creep coefficient had to be multiplied by adjustment factors. The adjustment factor for the elastic modulus was found by minimizing the predicted error on the camber immediately after release, resulting in a factor of 1.15. The adjustment factor for the creep coefficient was found by minimizing the predicted error on the second camber measurement, resulting in an adjustment factor of 1.4. The prestress losses had to be taken into account when computing the creep component of camber.

The cambers measured after placement of the girders in their final locations were compared with predicted values to evaluate the influence of the support conditions. The supports provided partial restraint to longitudinal movement of the girder's bottom flange and clearly affected the camber. However, significant scatter in the recorded data made trends difficult to see and reinforced the need for further measurement and analysis of the issue. As an indication of the importance of the end conditions, it may be noted that, in the evaluation of the changes in camber due to release of the temporary strands and placement of the deck, the girders that were seated on oak blocks at both ends were 41 percent to 46 percent stiffer than those seated on elastomeric bearings.

p. 1-2

INTRODUCTION

The Washington State Department of Transportation (WSDOT) has been using precast prestressed concrete girders in bridge applications for many years. These girders have allowed for longer spans, provided economical design, and accelerated construction times by allowing precast fabricators to deliver ready-made products at the contractor's convenience. The girders are often built with an upward camber after initial stressing. However, because of material and environmental properties, this initial camber at release can change over time.

WSDOT typically uses standard designs for prestressed concrete girder cross sections. These sections were designed in collaboration with precast fabricators to provide economical fabrication in conjunction with increased span capabilities. WSDOT will typically design a bridge by using design programs, such as PGSuper (PGSuper 2006), and provide the contractor with detailed plans and specifications to fabricate the girders. The fabricator will then build the girder according to the details provided and deliver the girder to the contractor on site.

1.2 Camber Prediction Challenges to WSDOT

Most of the bridges built in Washington State today are constructed with precast, prestressed concrete girders. However, the uncertainty of the predicted camber in precast, prestressed girders can lead to problems during construction. Excessive camber leads to interference between the top of the girder and the deck reinforcement. Insufficient camber leads to an increase in the concrete required to meet the bottom of the deck slab, resulting in additional weight to the superstructure. Both are undesirable conditions that can lead to construction delays, as well as increased material and labor costs.

p.22-25

2.8 WSDOT Practice for Camber Prediction

The design length of the girder is defined as the length from the centerline of the final bearing at both ends. All deflection calculations are based on the span length using a simply supported model. The procedures of Concrete Technology and Central Pre-Mix Prestress for supporting the girders at release and storage were observed by the researches to be different than those assumed by WSDOT. At release, the girder is typically supported by lifting loops that vary in distance from the ends. When placed in the storage yard before shipping, the girder is supported on bunks anywhere from 2 to 3 feet in from the ends. Because creep deflection is proportional to the elastic deflection, small variations in elastic deflection caused by different support conditions carry over to the creep deflection.

p.38-39

4.3 Data Collection Procedure

Camber measurements were taken only in the morning to minimize the effects of temperature differentials over the height of the girder caused by the sun. Measurements were taken between 6:00 AM and 10:00 AM.

p.40-41

4.4 Observed Behavior

Temperature variations might also have contributed to the camber variations. In particular, heating the tops of the girders creates a temperature gradient over the height. To minimize this effect, camber measurements were taken only in the morning, before the ambient air temperature changed.

p.43

MATERIALS TESTING

5.1 Purpose

Both moist-cured and accelerated-cured cylinders were tested to compare the difference in curing methods and the influences on the material properties.

p.45-46

5.3 Concrete Compressive Strength

The AASHTO LRFD method suggests that one day of moist-cure is equivalent to seven days of accelereated-cure.

p.48-49

5.4 Elastic Modulus

The equations recommended by the AASHTO LRFD and ACI underestimated nearly all the measured data. This was consistent with the predicted camber data collected at release, as discussed in Chapter 4. On average, the measured elastic modulus was 8 percent higher than the value predicted by the AASHTO LRFD method.

p.75

6.5.1 Effect of Actual Concrete Strength

According to Table 6.6, the measured cambers (on average) were smaller than the calculated ones when the design concrete strength was used. This was true for exterior and interior girders, for one-end and two-end continuous spans, and for all three loading conditions (strand release, deck placement and deck creep). The accuracy increased and the calculated cambers decreased significantly when the actual (rather than the design) concrete strength was used. This trend was expected because the actual stiffness of the concrete was greater than the value assumed in design. This resulted in smaller values of estimated deflection. The use of the actual concrete strength thus improves the prediction accuracy.

p.87-88

CHAPTER 8

EVALUATION OF FABRICATOR DATA

8.2 Data Collection Procedure

Most of the information obtained from the fabricator was related to the girder cross-section, length, and prestressing strand size and quantity. Those properties were taken from the fabrication drawing plans. Quality control records were also reviewed. These records documented the fabrication history, including the time and date of casting, time and date of the release of prestress, concrete cylinder strengths at release and 28 days after casting, and all camber measurements.

8.3 Description of Dataset

8.3.1 Data Collection from Concrete Technology Corporation

The data set from Concrete Technology Corporation contained 103 girders from four projects and included the eight girders monitored for a detailed time history of camber and materials testing (see chapters 4 and 5).

p.91

8.3.4 Age at Release

The age of the concrete at release influences the girder stiffness because the elastic modulus changes over time and with maturity.

Therefore, while most girders will be released between 12 and 24 hours of casting, some girders will cure for between 48 and 96 hours prior to release.

p.92-93

8.3.5 Seasonal Variations

Temperature during casting and curing is known to influence material properties and may affect prestress loss. The two girder fabricators are located in areas that exhibit very different annual temperature and humidity cycles. Central Pre-Mix uses outdoor casting beds. It covers the girders with thermal blankets and, if necessary, raises the concrete's temperature by releasing steam adjacent to the form, under the blanket. In the winter months, the temperature in Eastern Washington frequently drops below freezing, so special measures are needed to achieve proper concrete curing. In the summer months, the climate is typified by high temperatures and low humidity, both of which might be expected to promote shrinkage and creep.

Concrete Technology in Tacoma has a covered casting bed and heats the girders with electric elements inside insulated forms, even though the climate in Western Washington is much milder. Girders made at both plants are susceptible to some thermal effects during casting and curing, but the seasonal differences suggest that the effects might be larger in girders cast by Central Pre-Mix Prestress.

To illustrate the potential for seasonal effects on the data set, Figure 8.4 shows the number of girders that were cast in each month of the year.

p.95

8.4.2 Influence of Compressive Strength on Release Camber

The compressive strength at release can influence the release camber by changing the stiffness of the girder. Figure 8.7 illustrates this trend for the Black Lake Bridge project. As the compressive strength of the concrete increased at the time of release, the initial camber decreased.

p.96

8.4.3 Long-Term Camber

The long-term trends were more difficult to distinguish. The quantities that fabricators measure after the release measurements are 28-day compressive strengths and an additional camber measurement. This additional camber measurement was not taken at a consistent time after casting because WSDOT requires only that one additional measurement be taken prior to shipping. Shipping dates vary, and often change, in accordance with the contractor's schedule. This leads to a significant scatter in the age of the girder at the second camber measurement. The effects of creep and shrinkage will also vary with time, material properties, environmental conditions, and even girder support conditions in the yard, so large scatter must be expected in this second camber measurement.

p.98-99

CALIBRATION OF CAMBER MODEL

9.1 Introduction

This chapter discusses calibration of the camber model with field data. Those data consisted of reading from girders that were fabricated by the two major manufacturers in the state Of Washington and placed in six different bridges.

9.2 Evaluation of Current Procedure

Thus a negative error indicates that the predicted upward camber is larger than the measured, and that the real girder is flatter than the calculations suggest. This was the outcome in most cases when the standard WSDOT method of calculation was used. The WSDOT method over-predicted the camber in almost all girders that were over 100 feet long.

p.102-103

9.3 Effect of Using Measured Concrete Compressive Strength

Consequently, the girder tends to be stiffer and deflect less than would be the case if the actual strength had been based on the long-term requirements.

Table 9.4 shows the average ratio of the measured concrete compressive strength to the design concrete compressive strength for all the girders and the two fabricators. As expected, the average ratio was higher at 28 days than at release.

Figure 9.3 shows how concrete strength affects the predicted camber for a W74G girder. A 10 percent increase in concrete strength results in decreases in the predicted camber of approximately 0.10 in. at release and 0.25 in. after 200 days.

p.106-107

9.4 Prestress Loss Deflection Adjustment

By using the actual concrete compressive strength and including the effects of prestress loss on deflection, the error in the prediction was reduced as shown in Table 9.3.

p.108

9.5 Calibration of the Camber Prediction Model

Creep deflection is affected by the elastic modulus, but the converse is not true.

p.115

9.5.3 Calibration of Prestress Losses Due to Creep

After the initial optimization had been completed in this project with the AASHTO LRFD 2004 equation, AASHTO adopted the recommendation in NCHRP Report 496, which generally predicts lower losses. WSDOT has since adopted the AASHTO 2006 “Refined Methods of Time-Dependant Losses”.

p.123-128

10.2 Conclustions

a) Concrete strength. On average, the measured concrete compressive strength exceeded the specified strength by 10 percent at release and 25 percent at 28 days. The excess strength at release was particularly large when the girder was cured for more than one day, as often happens over weekends.

4) a) Girder support locations

The girder is supported at different locations at different times, such as at release, during storage and on the bridge piers. These support locations affect the camber, but are typically not accounted for during design.

b) Restrain by support

Placement of a girder on a fixed support creates some restraint to the longitudinal shortening of the bottom flange. Lifting and reseating a girder in the storage yard released that restraint and caused an increase in camber that averaged 0.15 in.

d) Environmental conditions

Ambient air temperature, relative humidity, and temperature gradient in the girder could affect measured cambers but were not included in this research.

1) Effect of support conditions

Some girder ends were supported on oak blocks and then built in to partial-height diaphragms, while others were seated on elastomeric bearings. Girders supported on oak blocks at both ends deflected less than those that were supported on an elastomeric bearing at one end.

The lower deflections were attributed to fact that the bottom flange of the girder was partially restrained against longitudinal movement. The restrained girders behaved as if they were 41 percent to 46 percent stiffer than those seated on elastomeric bearings.

p.128

10.3 Recommendations

10.3.1 Recommendations for Practice

The following recommendations are made for practice:

1) Concrete strength

For deflection calculations, increase the specified concrete strengths by 10 percent at release and 25 percent at 28 days.

2) Elastic modulus

Use 1.15 times the AASHTO LRFD 2006 equation for predicting the concrete elastic modulus (Ec) for a given concrete strength. An alternative would be to adopt the methods recommended by NCHRP or CEB-FIP.

25. camber, prestress losses, deflections
"Predicting Camber, Deflection, and Prestress Losses in Prestressed Concrete Members"
Dr. Sami Rizkalla, Dr. Paul Zia, and Tyler Storm
North Carolina State University
Final Report Research Project # 2010-05
FHWA/NC/2010-05
July 2011

Abstract

Accurate predictions of camber and prestress losses for prestressed concrete bridge girders are essential to minimizing the frequency and cost of construction problems. The time-dependent nature of prestress losses, variable concrete properties, and problems related to production variables make it difficult to predict camber accurately. The recent problems experienced by NCDOT during construction are mainly related to inaccurate prediction of camber. In this report, several factors related to girder production are shown to have a significant impact on the prediction of camber.

p.vi-viii

SUMMARY

In recent years, NCDOT has experienced increasing construction problems related to discrepancies between the predicted and measured camber for prestressed concrete bridge girders, as well as problems with differential camber between identical girders. In addition, current prestress loss predictions used by NCDOT are based on the 2004 AASHTO LRFD Bridge Design Specifications, which has been superseded by the 2010 edition.

This report examines the accuracy of the current NCDOT method for predicting the prestress losses and camber for prestressed concrete girders as compared to field measurements. Other methods available in the literature are also reviewed, including the PCI method and the AASHTO 2010 method.

The report presents the findings from the testing of a large number of concrete cylinders that was conducted to evaluate the properties of the concrete. It also presents the findings of several site visits to precasting plants that were conducted by the research team to identify factors related to girder production that could potentially affect the accuracy of the camber predictions. Specific findings related to the concrete properties and other production factors include the following:

1) The concrete compressive strength at transfer was found to be on average 25% higher than the specified design value.

2) The concrete compressive strength at 28 days was found to be on average 45% higher than the specified design value.

3) The elastic modulus of the concrete was found to be on average 15% less than the value predicted by the AASHTO specifications using a unit weight of 150 pcf for the concrete.

4) Concrete properties can potentially vary from girder to girder within the same casting bed due to the use of multiple batches of concrete along the bed as well as delays in concrete batching that occasionally occur during a casting.

6) The prestressing force was found to be significantly affected by the temperature fluctuations of the prestressing strands during fabrication.
7) Temporary thermal gradients in the girder could cause significant scatter of the measured camber data.
8) The debonding and transfer length of the prestressing strands were found to be significant sources of error in the camber predictions for girders with debonded lengths greater than ten feet and should therefore be considered in the prediction of camber.
This report provides specific recommendations to account for several of these factors to enhance the prediction of camber.
The research introduces two methods for the prediction of camber for prestressed concrete bridge girders, including an "approximate" method based on multipliers and a "refined" method based on the detailed losses calculations given in the 2010 AASHTO specifications. The current NCDOT method was also modified to account for the factors related to girder production. The current NCDOT method, the modified NCDOT method, and the two proposed methods were compared with measured cambers of 382 prestressed concrete girders in the field, some of which were taken by the research team and others that were collected with the help of NCDOT inspectors and Resident Engineers. The girder types that were considered in the study include AASHTO Type III and Type IV girders, box beams, cored slabs, and modified bulb-tees. The findings from the comparison of the prediction methods are summarized briefly as follows:
1) The current NCDOT method was found to overestimate the camber of prestressed girders by an average of 52%. The modified NCDOT method overestimated the camber by an average of 39%. The proposed approximate method overestimated the camber by an average of 16%. The proposed refined method underestimated the camber by an average of 6%.
2) The accuracy of the predictions of camber at prestress transfer was found to vary between different girder types and curing methods. Steam cured box beams and cored slabs exhibit lower cambers at the time of prestress transfer than the moist cured members. However, the accuracy of the predictions at later stages is less significantly affected by the curing method and girder type.
Based on the findings of this research, the two proposed methods are recommended to provide the most accurate prediction of camber. The proposed approximate method is more convenient for simple hand calculation, while the proposed refined method is suited for more accurate computer calculations.
A spreadsheet program to predict prestress losses and camber using each of the methods considered in this research is provided. A spreadsheet that calculates the modified section properties for box beams and cored slabs due to void deformation is also provided.
p.8
2.7 Camber Experiences of Other States
A brief questionnaire was sent to bridge design engineers at the Nebraska Division of Roads (NDOR), the Texas Department of Transportation (TXDOT), and the Florida Department of Transportation (FDOT). These questionnaires were designed to explore the methods used by these states to predict prestress losses and camber. They also

requested information about any problems experienced with the prediction of camber. This section provides a summary of the responses. The full text of the responses are provided in Appendix C.

2.7.1 Nebraska Division of Roads (NDOR)

NDOR uses both the “approximate method” and the “refined method” of the 2004 AASHTO LRFD Bridge Design Specifications to estimate prestress losses. In their experience, both give approximately the same prediction of camber at the time of the erection of the bridge, which is assumed to be 30 days after casting.

NDOR has observed that the camber predictions are often higher than the measured values, particularly for very long spans (over 150 feet) when the specified concrete strength exceeds 10 ksi.

p.9

2.7.2 Texas Department of Transportation (TXDOT)

TXDOT typically uses the “refined method” of the 2004 AASHTO LRFD Bridge Design Specifications to estimate prestress losses. To predict camber, they use a single set of assumed creep values for all girders.

In contrast with NDOR, TXDOT has observed that their camber predictions are often significantly lower than measured values for long span girders. In rare cases, the girders had to be re-cast. In addition, TXDOT has observed that girders cast at the same time will often have different cambers on the bridge if the project phasing requires that some girders remain in storage in the casting yard longer than others. They observed that these differential camber problems are most problematic with box beam girders since these are placed immediately adjacent to each other on the bridge.

2.7.3 Florida Department of Transportation (FDOT)

FDOT uses the AASHTO equation (Equation 2-1) to calculate the elastic modulus of concrete. They use the specified concrete strength and a concrete unit weight of 145 pcf. For concrete made with coarse aggregate native to Florida, which is typically limestone, the elastic modulus is factored by 0.9.

FDOT estimates prestress losses using the refined calculations specified by the 2004 AASHTO LRFD Bridge Design Specifications. Camber is calculated using either the PCI multiplier method or the approximate time-step method. FDOT engineers have not experienced persistent problems with camber prediction using either method, although construction difficulties related to camber occasionally occur.

Problems related to camber prediction are prevented to some extent by FDOT’s practice of requiring that the contractor measure the camber of the girders before setting the seat elevations on the bridge bents.

p.14

3.2.2 PCI Method

The method recommended by the Precast and Prestressed Concrete Institute for estimating prestress losses is similar to the current NCDOT method in that it only estimates the ultimate time-dependent losses rather than time-specific values. However, the prediction equations themselves are different.

p.25

3.3.2 PCI Method

The PCI method also uses multipliers to predict camber at prestress transfer, at bridge erection, and at an arbitrary “final” time in the distant future, which represents the ultimate deflection.

3.3.2.1 Camber at Transfer

The calculation of the camber at prestress transfer is identical to the NCDOT method (Equation 3-35).

3.3.2.2 Camber at Time of Bridge Erection

In estimating the camber at the time of bridge erection, the PCI method is similar to the NCDOT method except that the multipliers are reduced:

3.3.2.3 Camber at Final Time

The net camber at an arbitrary “final” time in the distant futre is estimated using additional multipliers for the initial deflections. The deflection due to superimposed loads applied at bridge erection, if such loads are present, is also adjusted by a multiplier.

If the superimposed load applied at bridge erection is a composite topping, then its contribution to deflection is multiplied by 2.30 instead of 3.00 in the above equation.

p.26-27

4 FIELD MEASURMENTS AND SITE VISITS

4.1 Introduction

A significant part of the effort of this study was the development of an extensive database of field measurements that could be used to evaluate the various prediction models. The field data included camber, concrete properties, and production details. The development of the database is discussed in Section 4.2.

4.2 Field Measurments

To develop the extensive database of field measurements, it was necessary to enlist the help of NCDOT inspectors and Resident Engineers. For quality assurance purposed, the inspectors are required to be present during the casting of every prestressed bridge girder produced for NCDOT. Since there is an NCDOT inspector stationed at each precasting yard, they were well-positioned to take camber measurements before the girders were shipped. The resident engineers, on the other hand, are present at the erection of the bridge and were thus able to take camber measurements once the girders were in place.

4.2.1 Camber Data Sheets

In order to collect the camber measurements and the related girder data, a data sheet was developed on which the inspectors would record the measurements and data at the precasting yards. Once the girders were sent to the construction site, the data sheets were sent by the inspectors to the resident engineers so that the camber measurement of the in-place girder could also be recorded.

The measurements and data included on the data sheets consisted of the following items:

- Position of each girder along the casting bed
- Curing method used

- Ultimate compressive strength of the concrete at transfer
- Ultimate compressive strength of the concrete at a later age
- Locations of the supports in storage
- Exposure conditions and geologic orientation during storage
- Measurements of the camber at transfer, at the beginning of storage, at the end of storage, and in place on the bridge
- Weather and temperature at the time of each camber measurement

p.37

5 EVALUATION OF PRODUCTION FACTORS

5.1 Introduction

Throughout this research, several production variables were identified as factors that could affect the prediction of camber. In this chapter, these production factors are evaluated to consider their effects. Recommendations to account for these factors to improve the predictions are provided.

p.41-42

5.4 Concrete Properties

Predictions of prestress losses and camber are highly depended on the properties of the concrete used for the girder. The elastic modulus of the concrete is used for predicting deflections and prestress loss due to elastic shortening. The compressive strength of the concrete is used to predict losses and elastic modulus. The unit weight of concrete is also needed to determine the self-weight load, and it influences the elastic modulus equations provided by AASHTO, PCI, and ACI.

Due to the importance of the concrete properties in the predictions, the research team used physical tests and collected data to evaluate these properties. The following sections describe the tests, the collected data, and the analyses of these properties.

5.4.2 Unit Weight of Concrete

The 88 cylinders that were tested for unit weight were tested in 29 sets of three or four cylinders. As shown in Figure 5-2, the average unit weight of the cylinder sets was 148 pcf. The data ranged from approximately 140 to 154 pcf.

Currently, NCDOT engineers assume a unit weight of 150 pcf for the elastic modulus and deflection predictions. Due to the weight of the reinforcing steel in the girder, it is reasonable to assume that the actual unit weight of the section will be slightly higher than the measured value of 148 pcf. Therefore, the value of 150 pcf currently used by NCDOT engineers for design is considered appropriate.

p.60

5.7 Curing Method

The two primary curing methods used for precast, pretensioned girders are moist curing and heat (or steam) curing. Moist curing typically consists of using a hose to drip water on the top surface of the girder, while heat curing involves the use of steam lines to heat the girder, consequently accelerating the cement hydration process.

As discussed in Section 5.3, the temperature of the strands can affect the prestressing force through thermal expansion and relaxation of the strands. When heat curing is used, the temperature of the girders and strands rises very quickly after casting.

Therefore, girders that are heat cured could experience a greater reduction in the prestressing force than moist cured girders due to the effect of the strand temperature. In addition to affecting the prestressing force, heat curing can also generate a thermal gradient within the girder since the girder is typically heated from below while the top of the girder is more exposed to cooling. This could reduce the camber of the girder at the time of prestress transfer due to differential thermal expansion, as discussed in Section 5.6.

The effect of the curing method on camber is evidenced by the analysis of the collected camber data, which is discussed in detail in Section 7.3. The data suggest that the camber at the time of prestress transfer is significantly reduced for heat cured members as compared to moist cured members. The data also suggest that the effect seems to depend on the girder type. For box beams, the camber at transfer for heat cured members is roughly 50% lower than for moist cured members; for cored slabs, the difference is approximately 75%; and for Type IV girders, the difference is approximately 20%. For modified bulb-tees, there is not a significant difference in the camber at transfer for head cured versus moist cured girders.

Although the effect of curing method on the camber at the time of transfer is significant, analysis of the camber measurements at later ages suggest that the effect may be only temporary, since the difference between the camber measurements for heat cured and moist cured girders is significantly reduced at ages greater than 24 days.

Based on these observations, it is concluded that the effect of curing method on camber prediction accuracy can be significant at the time of transfer, although it varies for different girder types. In addition, the fact that the effect of the curing method is most significant for shallow girders such as cored slabs and box beams leads to the conclusion that thermal gradients present at the time of transfer may be a significant cause of camber discrepancy at this stage for head cured members. This conclusion is drawn from the fact that thermal gradients with the same temperature differential tend to cause a greater deflection in shallow members that in deep members, as discussed in Section 5.6.

This analysis leads to the further conclusion that the measured camber at the time of transfer is not necessarily a reliable indicator of the eventual long-term camber since the camber at this early stage may be temporarily reduced due to effects related to the curing method.

Adjustments to the camber predictions due to this factor are not practical since the curing method is generally not known at the design stage. However, for camber analysis, girder data should be grouped according to the curing method used.

p.62

5.9 Project Scheduling

Factors related to project scheduling can also affect the camber predictions. For example, predictions of the camber at the time of prestress transfer as well as predictions of losses are based on the assumption that prestress transfer occurs one day after casting, which is typically the case. However, it is also common for girders cast on a Friday to have the prestressing transferred on the following Monday, three days after casting. During this extra time, the strength and elastic modulus of the concrete increases significantly. As a result, these properties can be significantly greater than the specified values used in the camber and prestress loss predictions. In addition, the increased strength could result in significantly less creep than predicted. Therefore, two girders cast for the same bridge could have very different cambers if the prestressing force is transferred to the girders at different ages.

Another factor related to project scheduling that could affect the camber predictions is the amount of time between prestress transfer and erection of the bridge. There is often wide variation in this respect from project to project. For example, it is not uncommon for cored slabs to be erected within 15 days of casting, while other girder types may be in storage for six months or more before being erected. Due to the effects of creep on the measured camber, this could result in significant discrepancies between predicted and measured camber since the time of bridge erection is typically assumed to be 28 days in the predictions.

Due to the inherent scheduling uncertainties involved with bridge projects, adjustments to account for the project scheduling at the design stage are not practical. However, camber behavior will be more consistent among the girders in a particular bridge or span if they experience the same amounts of time between casting, prestress transfer, and bridge erection.

p.63-64

5.10 Summary of the Proposed Adjustments

Many of the factors discussed in this chapter introduce errors in the camber predictions that are additive. Specifically, the effects of neglecting debonding, transfer length, concrete over-strength, and void deformation in the camber predictions all tend to overestimate the actual camber. When considered together, the impact of these effects can be significant. Therefore, it is recommended that the adjustments for all four of these factors be included in the predictions.

The following recommended adjustments should be used when predicting camber and when predicting prestress losses for the camber calculations. The adjustments are summarized as follows:

<u>Concrete Strength</u>

The specified concrete strength at transfer, f’ci, should be adjusted to determine the best estimate of the actual concrete strength at transfer, f*ci, as follows:

f*ci = 1.25f’ci

The specified 28-day concrete strength, f’c, should be adjusted to determine the best estimate of the actual 28-day strength, f*c, as follows:

f*c = 1.45f’c

These adjusted strengths should replace the specified strengths in all of the calculations, including concrete elastic modulus, creep coefficients, shrinkage strains, losses, and deflections.

<u>Concrete Elastic Modulus</u>

The AASHTO equation to estimate the elastic modulus of concrete at transfer should be adjusted by a factor of 0.85 to account for local production factors, as follows:

Eci = (0.85)33$wc^{1.5}$sqrt(f*ci)

The 28-day elastic modulus equation should be similarly adjusted:

$E_c = (0.85)33w_c^{1.5}\text{sqrt}(f^*c)$

p.64

Debonding and Transfer Length

The reduction in camber due to debonding and transfer length, discussed in section 5.2, is currently neglected by NCDOT engineers. To account for this effect, the elastic deflection due to prestressing only should have the following general form:

p.74

7 EVALUATION OF PREDICTION METHOD

7.1 Introduction

To evaluate the accuracy of the methods for predicting camber, the predicted values from each method were compared to the field measurements. The following four prediction methods were compared:

1) current NCDOT method
2) modified NCDOT method
3) proposed approximate method
4) proposed refined method

The detailed calculations for these methods are given in Chapters 3 and 6. All of the methods except the current NCDOT method include adjustments to account for the effects of the production factors and material properties as discussed in Chapter 5.

7.2 Method of Comparison

A series of spreadsheets was developed to predict the camber of each girder in the database using each prediction method. Since no camber measurements were taken after placement of superimposed dead loads, the predictions were limited to consider only the camber up to the time of girder erection.

p.78

7.3.2 Camber at Transfer

The calculation of the camber at prestress transfer is nearly identical for all methods. The primary difference is that the recommended adjustments to account for the effects of the production factors described in Chapter 5 are included for the modified NCDOT method, the proposed approximate method, and the proposed refined method, while they are not included for the current NCDOT method.

p.80-81

7.3.3 Camber at 24 Days and Later

The data for camber measurements taken at 24 days or more after casting provide a more reliable comparison for the performance of the prediction methods since the bilinear approximations of the prediction curves more closely represent the realistic camber growth during this time. This analysis is of primary importance since the focus for this research is to improve the prediction of the camber at the time of girder erection, which typically occurs several weeks after casting.

The analysis indicates that both the proposed approximate method and the proposed refined method provide significant improvement to the camber predictions compared to the current NCDOT method and the modified NCDOT method, as shown by the data in Table 7-1 and Figure 7-5.

p.86-90

8 CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusions

1) Many factors related to the design of girders as well as the production process can have significant impacts on the prediction of camber and prestress losses. Significant factors which should be considered at the design stage include the following:

a) Concrete properties

The accurate estimation of the concrete properties is essential to obtaining reliable predictions of camber and prestress losses. NCDOT currently uses the specified concrete strength in the predictions, which is often significantly lower than the actual concrete strength of the girder. A survey of cylinder test data as well as tests conducted by the research team showed that the actual concrete strength at transfer was on average 25% higher than the specified strength for transfer, while the concrete strength at 28 days was 45% higher than the specified 28-day strength. The cylinder tests also revealed that the elastic modulus for locally produced girders was on average 85% of the value predicted using the AASHTO equation. In addition, concrete properties can vary from girder to girder on the same casting bed since multiple batches of concrete are used for a single casting.

c) Debonding and transfer length

Neglecting the reduced curvature at the ends of the girder due to debonding and transfer length can result in overestimation of camber. For most girders, the effect on camber is less than 3%. However, for girders with especially long debonded lengths (10 feet or greater), the camber may be overestimated by as much as 13% if this effect is not considered. The effect is also more significant for short span girders since the affected end region comprises a larger proportion of the overall span.

2) Factors that are less significant or for which little can be practically done at the design stage to improve the predictions include the following:

a) Temperature of the concrete

Temporary thermal gradients through the depth of the girder caused by heat curing or solar effects can result in temporary changes in camber. This effect is most severe for box beams and cored slabs.

b) Curing method

The camber of girders at the time of prestress transfer can be significantly affected by the curing method used. Heat cured girders other than modified bulb-tees tend to have significantly less camber at transfer than moist cured girders, although there does not seem to be significant difference in the camber at later stages, suggesting that the discrepancy could be due to temporary thermal gradients.

c) Temperature of the strands

The prestressing force can undergo significant fluctuations between the time of tensioning and the time of prestress transfer due to temperature changes in the strands caused by the ambient temperature, solar effects, and concrete curing temperatures. This can affect the camber of the girder.

d) Project scheduling

Girders that experience different amounts of time between casting and prestress transfer or between transfer and girder erection can have different cambers.

3) Due to the variations in production, some of which are unpredictable, the measured camber was observed to vary significantly among girders that were otherwise identical in design.

4) The measured camber at the time of transfer should not be used as a reliable indicator of the camber at later stages due to the variability caused by rapidly changing concrete properties and by thermal gradient effects.

5) The current NCDOT method was shown to significantly overestimate the camber for most girder types. Camber was overestimated by an average of 52% among all of the girders studied.

6) The modified NCDOT method provided improved camber predictions compared to the current NCDOT method, but is still overestimated the camber by an average of 39% among all of the girders studied.

7) The proposed approximate method overestimated camber by an average of 16% among all of the girders studied, which was significantly better than the NCDOT methods.

8) The proposed refined method provided the best estimates of camber for most girder types. It underestimated camber by an average of only 6% among all of the girders studied, although it underestimated the camber by an average of approximately 25% for head cured modified bulb-tee girders and moist cured Type IV girders.

8.2 Recommendations for Practice

Based on the findings of this research, the following design and production practices are recommended:

1) For deflection calculations, the specified concrete strength at transfer should be increased by 25%. The specified 28-day concrete strength should be increased by 45%. These changes account for the average relationship between specified and actual concrete strength.

2) The concrete unit weight should be assumed to be 150 pcf.

3) To estimate elastic modulus, the equation provided in the 2010 AASHTO specifications should be multiplied by 0.85 and should be calculated using the adjusted concrete strength and the recommended unit weight of 150 pcf.

6) The effect of debonding and transfer length should be accounted for in the camber predictions. The calculations for the proposed methods provided in Chapter 6 include this adjustment.

7) The proposed refined method provided in Section 6.5 should be used to predict camber. The proposed approximate method provided in Section 6.4 may be used to predict camber when a simple rough estimate is desired. The elastic shortening loss used in the approximate method may be calculated according to the AASHTO 2010 method instead of the PCI method, since this will not significantly affect the camber predictions and will provide consistency with the losses calculations that are performed for the proposed refined method.

8) Whenever practical, camber should be measured before dawn before the sun induces thermal gradients within the girders.

9) Girders should be stored with the supports as close as possible to their design bearing locations to minimize camber variability.

p.139-145

Appendix C Camber Experiences of Other States

Texas Department of Transportation

3. What procedure or approach do you use to minimize the impact of the camber problem?

Smart contractors will lower the bearing seat elevations on their own if they suspect there could be a camber issue. Most contractors are aware that some fabricators have a history of producing beams with high cambers while others have a history of low cambers. The problem occurs mainly with widenings or phased construction where profile grade adjustments are not usually possible. For new construction, it is usually easy to adjust the profile grade to account for higher cambers.

6. What code(s) and design software(s) do you currently use to calculate prestress losses, camber, and deflection?

Camber

The maximum camber calculations are derived from the hyperbolic function method developed by Sinno [6]. Sinno formulated hyperbolic functions for unit shrinkage and unit creep from field data of full-sized, Texas Type B, prestressed concrete bridge beams.

The prestressing steel for the beams consisted of seven wire, 7/16-in. diameter, 250 ksi stress relived strands. The beams were fabricated of both normal-weight and lightweight concrete and stored for a 300-day period. At the end of the storage period, the beams were installed in the 40 ft. and 56 ft. spans of a bridge on IH-610 over South Park Boulevard in Houston, Texas. The camber calculations are, therefore, strongly correlated with the particular structure they were calibrated to.

The calculation method developed by Sinno was fully implemented in the TxDOT "Camber Prediction Program" – PSTRS11 (a.k.a. Prestressed Beam Stresses and Camber). PSTRS11 was written by Sinno and employs empirically based unit hyperbolic creep and shrinkage functions and a step-wise time-increment numerical procedure. This program was never used for predicting camber in bridge design production, so the source code was not incorporated into PSTRS10 or PSTRS14. Instead, a very simplified single step method of calculating the camber at mid-span, using a single set of assumed creep and shrinkage values, is included in PSTRS14. The justification for this simplification is the presumption that camber calculation is inherently inaccurate so there is no need to improve the prediction of what cannot be reliably predicted. The design engineer may not agree with this logic and may choose to employ other means to determine beam camber. But in practice PSTRS14's calculated camber is assumed to be good enough by most bridge design engineers in spite of some field data to the contrary.

PSTRS14's camber calculation method may not provide an adequately accurate prediction of beam camber for design purposes when applied to other beam types (i.e., volume/surface ratios) than I-beams, other material constituents, other storage periods, other final location and framing plan, as well as other factors not considered in Sinno's method. Some of these factors and other factors described by Kelly, Bradberry, and Breen [7] significantly influence camber at erection. Under no circumstances should the calculated camber be considered as having the same degree of certainty as, for example, the concrete strength required. The value printed for erection camber should be considered a rough estimate only and is not applicable to all possible design options, beam types, aggregate types, etc.

The user should verify the calculated camber versus the dead load deflection due to placement of all superimposed dead load results in a beam with a positive net camber. If the camber needs to be increased, the user may do so by adding additional strands using the analysis option. This would not, however, guarantee that the profile of the beam will always have a net positive camber throughout the service life of the bridge. Camber calculations performed by the program are thus insensitive to the beam type and many other factors that affect the actual camber of prestressed concrete beams. Furthermore, final concrete strength alone is used to determine beam stiffness so the method is not affected by the initial concrete stiffness associated with f'ci.

Deflections are simple span deflections and may be based on the moduli derived from final concrete strengths of beam and composite areas such as shear keys. However, TxDOT policy is to set all concrete moduli to 5000 ksi for both stress and deflection calculations and to indicate on the plans that the deflections shown are calculated assuming an E'c of 5000 ksi.

Nebraska Division of Roads

3. What procedure or approach do you use to minimize the impact of the camber problem?

We have shown PCI tolerances (at release) on the plans so our inspectors can enforce it. We have required the fabricator to check the camber before shipping. We are in process of identifying the "K" factors due to our local ingredients so we can use it in our calculation for losses.

6. What code(s) and design software(s) do you currently use to calculate prestress losses, camber, and deflection?

We use CONSPAN software for design using LRFD. We use approximate losses method and sometimes refined losses. (Both give about the same camber prediction at 30 days)

7. How do you determine the modulus of elasticity of concrete at different ages? If you calculate camber at prestress transfer, do you use the specified f'ci for the calculation? How do you determine shrinkage and creep in your calculations?

We provide two camber numbers on the plans: camber at release calculated based on the fci and camber at 30 days (assumed girder erection) based on the 28 days strength (design strength). In general we use AASHTO LRFD approximate method to determine creep and shrinkage.

Florida Department of Transportation

3. What procedure or approach do you use to minimize the impact of the camber problem?

The contractor is required to monitor the camber in storage. We are considering additional loads during design to allow for more tolerance when the camber is overestimated.

26. “Estimating Camber, Deflection, and Prestress Losses in Precast, Prestressed Bridge Girders, TRC 1606” presentation provided at the 2018 ARDOT Transportation Research Committee Spring Meeting

Ahmed Al-Mohammedi
Micah Hale
Department of Civil Engineering, University of Arkansas

- The current design methods results in differences between the **design** and the **actual** camber.
- The main goal of this project is to improve the accuracy of estimating camber and long-term deflection in precast prestressed concrete girders
  - (1) Current Methods for Camber Prediction:
    - 1.80 × initial camber – 1.85 × deflection = camber at erection
    - 2.45 × initial camber – 2.70 × deflection – (others) = final camber
  - (2) Underestimation of concrete properties.
  - (3) Inaccurate prediction of strands stress.
- All camber/deflection calculations are estimates
  - Affected by uncertainties relating to material properties
  - Time dependent
  - Temperature
  - Humidity
  - Load application
- Several visits were made to two plants which are: **(1)** Coreslab Structures in Tulsa, OK **(2)** JJ Ferguson Prestress/Precast in Greenwood, MS.
  - The objectives from these visits are to:
    - Evaluate concrete properties.
    - Measure strand stress
    - Monitor camber in the field.
    - Evaluate the current design methods.

Evaluating Concrete Properties

Concrete was sampled during the casting of each girder. More than 30 cylinders and 6 prisms were cast from each mix. The specimens were brought to the lab the following day and tested for:
1- Compressive strength. **2-** Elastic modulus. **3-** Unit weight. **4-** Creep. **5-** Shrinkage.

Monitoring camber

- The initial camber was measured immediately after release while the girders were still on the prestressing bed and again immediately after moving the girder to storage yard.
- Camber was measured multiple times until the girders were shipped to the job site.
- Once the girders arrived to the bridge site, camber was measured before and after casting the deck.
- **Current Methods for Camber Prediction:**

1.80 × initial camber – 1.85 × deflection **=** camber at erection
2.45 × initial camber – 2.70 × deflection – (others) = final camber

- The ACI 363 equation and the AASHTO LRFD equation underestimated the modulus of elasticity of concrete by 15% to 20%.
- Modification factors are proposed in the final report to overcome the underestimation.

Recommended Changes to Camber Equations

The recommended method utilizes a single multiplier of 1.4 times the elastic camber (initial camber at release) calculated using gross section properties as shown in the equation below. This multiplier was validated using the camber measurements conducted at girders erection.

(1) **Current Methods for Camber Prediction:**

1.80 × initial camber – 1.85 × deflection = camber at erection

2.45 × initial camber – 2.70 × deflection – (others) = final camber

Prestress Losses

- The 2014 AASHTO LRFD Detailed Method overestimated the prestress losses by **45%**.
  - The high compressive strength at release also affects modulus of elasticity.
  - The measured elastic shortening losses was very close to the predicted when using transformed section properties.
- Results
  - The 2014 AASHTO LRFD detailed method overestimated the prestress losses at the time of deck placement by **154% and 121%** for Type II and III, respectively.
  - The high compressive strength at release decreased the prestress losses.
  - Elastic shortening losses can be accurately estimated by applying the prestressing force directly to the transformed section properties.
- Recommended Changes to Modulus of Elasticity Equations
  - The MOE of concrete is a significant parameter in determining the initial camber and deflection of prestressed concrete girders.
  - MOE of concrete at release is necessary for estimating both downward and upward defection components of initial camber.
  - MOE at service is more important for quantifying the downward component which directly affects the long-term deflection.
- Coarse Aggregate Stiffness Coefficient (*K1*)
  - The AASHTO LRFD (2014) gives a better estimate for the modulus of elasticity when the concrete compressive strength is higher than 6500 psi. Therefore, it was more realistic to derive two K_1 coefficients with a range of applicability below and above 6500 psi. K_1 coefficients are summarized in the table below.

Range of Applicability	Crushed limestone (Sulphur Springs, AR)	River Gravel (Greenwood, MS)	Crushed limestone (Springdale, AR)
$f'_c < 6.5$ ksi	1.15	1.20	1.1
$f'_c > 6.5$ ksi	1.05	1.10	1.0

  - Conclusions
    - Use measured compressive strengths
      - Instead of specified

    - Measured release strength were 26 to 80% higher than design strength
- Use a multiplier of 1.4 vs 2.45 for final camber
- Measure Modulus of Elasticity
    - Use recommended K1 values
    - Measured E's at release were 20% to 50% greater than design

B-2

Literature Review Items Related to Research Topic Table

Topics Related to Research Study Table

	TOPICS																															
literature doc	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	BB	CC	DD	EE	FF
1																															X	
2	X																															
3		X																														
4		X	X	X	X																											
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25			X																					X		X	X	X	X	X		
26		X	X			X																				X				X		

Note: Literature docs shown in blue contain 3 or more topics related to the research

Note: Topics shown in yellow show up in 3 or more of the literature documents reviewed

topics	
A	strength gain > 28 days
B	material properties
C	camber prediction methods
D	camber variability
E	section properties
F	instrumentation & monitoring
G	high strength concrete using local materials (LADOTD)
H	temerature effects on camber
I	design procedures
J	measured camber
K	prestress losses
L	experimental program
M	AASHTO specifications
N	Sensitivity Study (TXDOT)
O	probabalistic comparison/effect of variability on prestress losses and camber & deflections
P	test data
Q	transportation weight limits
R	factors that influence span capabilities (prestress losses, allowable tension, local producer member capabilities f'c)
S	camber tolerances
T	debonded strands
U	anchor zone reinforcing
V	QC records (WSDOT)
W	humidity
X	historical material data
Y	support conditions
Z	modification factors for camber estimates
AA	camber experiences by other State DOT's
BB	when to measure initial camber
CC	scheduling pours
DD	recommendations for practice
EE	curing
FF	strand development and transfer lengths

APPENDIX C

Other State DOT Guidelines and Practices

C-1

Strand profile (draped, straight, debonding, top strand)

strand profile (draped, straight, debonding, top strand)

AASHTO LRFD	Article 5.11.4.3, the number of partially debonded strands should not exceed 25% of the total, the number of debonded strands in any horizontal row shall not exceed 40% of the strands in that row. Exterior strands in each horizontal row shall be fully bonded.
MS	Bridge Division policy is to drape strands instead of debonding. However, debonding may be used with permission from the Bridge Engineer on straight strands in certain situations to reduce stresses in the beam.
	The drape points on a beam are located one tenth (1/10) of the span length from each side of centerline of beam.
	If draped strands are required, a maximum number of twelve (12) draped strands or a minimum of four (4) draped strands are allowed.
	If the beam design used does not require straight strandsd in the top of the beam, #5 bars (or 1/2" strands stressed to 2,000 lbs) must be used in the top of the beam to aid in positioning shear steel.
	MDOT’s Bridge Design Manual (Bulb-T Design Procedure, section 1.f) states “Use Debonding if necessary (Debond to nearest tenth points,……….”
	MDOT’s current practice is to use 2 ft. debond increments.
NE	Debonding should be the last option and specified per AASHTO LRFD BDS 5.11.4.3.
	For prestressed I-girders with straight strands only, additional U-shaped bars shall be added to girder ends to reduce the stresses due to lifting and handling.
	Exterior and adjacent strands shall not be debonded.
	Maximum debonded length of strand shall be limited to Span/10.
	Maximum number of strands for the NU bottom flange is 58 strands.
	Four additional top strands shall be tensioned to 2 kips/strand and shall not be accounted for in the design.
	Strand hold-down points shall normally be located at 0.4 and 0.6 points of the prestressed girder (however, quarter and third pints are acceptable).
	Consideration should be given to using only straight parallel strands on short prestressed girders < 50 ft. due to the high hold down force required.
TX	Add and drape strands in the order shown on STD DWG IGND.
	Straight strand designs with and without debonding are permitted provided stress and other limits noted below are satisfied.
	Debonded strands must conform to Article 5.9.4.3.3 except as noted below:
	The maximum debonding length is the lesser of (a) one-half the span length minus the maximum development length, (b) 0.2 times the beam length, or (c) 15 ft.
	Not more than 75% of the debonded strands, or 10 strands, whichever is greater, shall have the debonding terminated at any section, where section is defined as an increment (e.g., 3 feet, 6 feet, 9 feet).
	Use hold-down points shown on STD DWG IGD.
	Keep the end position of depressed strands as low as possible so that the position of the strands does not control the release strength.
	Release strength can be controlled by end conditions when the depressed strands have been raised to their highest possible position.
FL	Full length shielding (debonding) of prestressing strands is prohibited.
	Whenever possible, separate debonded strands in all directions by at least one fully bonded strand and debond strands outside of the horizontal limits of the web. The percentage of debonded strands may exceed the recommended 25% limit in LRFD (5.11.4.3), provided that all strands within the horizontal limits of the web are fully bonded. In no case shall the percentage of debonded strands exeed 30%.
	Recent testing of FIB's under FDOT Project BDK75 977-05 indicates that number of longitudinal reinforcement (tension tie) is provided and the fully bonded strands are grouped close to the web. The 30% debonding limitation is a conservative interim limit until further research is completed under NCHRP Project 12-91.
	When the total initial tensioning force of the fully bonded strands required by design exceeds the values shown below (in table), shield additional strands at the end of the beam when possible.
WA	Strand Tensioning: the slope of the strands is limited to a maximum of 6:1 for 0.5" diameter strands and 8:1 for 0.6" diameter strands.
AL	Debonding of prestressed concrete girders shall be as given in AASHTO LRFD Section 5.11.4.3.
Comments:	
	Update MDOT Bridge Design Manual to reflect current practice of using 2 ft. debond increments.

C-2

Fillet/haunch thickness

fillet/haunch thickness

MS	No guidelines on the minimum haunch/fillet thickness. Use 2-inch haunch/fillet as added dead load. Do not include the haunch/fillet thickness in the composite section properties. Includes an allowable stress design check using a maximum average fillet/haunch thickness.
NE	A one inch minimum haunch at the CL of the girder between the bottom of the bridge deck and top of girder at midspan is required in design. The 1 in. haunch is a construction tolerance that also facilitates future deck removal and must be used to calculate girder seat elevations only, and not used for calculation of composite section properties.
TX	2 in. minimum haunch at centerline of bearing. Use 0.5 in. minimum at the edge of the girder at mid-span to accommodate the bedding strips for prestressed concrete panels. Regardless of calculated value, the absolute minimum haunch at centerline bearing is 2 inc. (increase in 0.25 in. increments). If the height of the girder haunch concrete is greater than 3.5 in., the haunch concrete is reinforced with Bars U (for full depth cast-in-place decks) or Bars UP (when Prestressed Concrete Panels, PCPs, are used).
AL	A minimum one inch haunch shall be provided at girder mid-span, calculated at the critical edge of the girder flange. Minimum buildup at girder ends shall take into consideration vertical curve, superelevation transition, or other complex roadway geometry. The build-up should be investigated for each girder line and adjusted as necessary.
LA	Haunch Thickness: 2" at center of support for spans <= 90 ft, and 0.5" at midspan Haunch Thickness: 3" at center of support for spans 90 to 120 ft, and 0.5" at midspan Haunch Thickness: 4" at center of support for spans >= 120 ft, and 0.5" at midspan Average haunch weight is considered in the analysis. Haunch thickness is ignored in the calculation of section properties.
Comments:	
	MDOT's maximum haunch/fillet check does not take into consideration the actual f'ci or the actual f'c, include actual concrete strengths if known. MDOT's current guidelines do not consider the increase in the composite section properties (i.e., composite moment of intertia) as a result of increase in the composite section depth/height due to an "under-camber" girder at erection. Specify a minimum haunch thickness in MDOT's Bridge Design Manual. Consider actual haunch/fillet thickness over the beam length (i.e., due to the upward camber of the girders, the shape of the haunch/fillet thickness varies over the length of the girder in a parabolic profile). Therefore, the added thickness increases both the dead load and composite section properties near the ends of the girder, which can effect deflection estimates. To simplify the calculation of the haunch/fillet dead load and section properties, an average thickness is typically used/assumed for the haunch/fillet.

C-3

Roadway vertical curve ordinate

roadway vertical curve ordinate

NE	Camber and any correction for grade vertical curvature must be considered when determining girder seat elevations and concrete quantities. Bridge plans should indicate typical vertical dimensions from the top of the girder flange to grade at supports.
WA	It is desirable to have points of horizontal and vertical curvature and superelevation transitions off the bridge structure as this greatly simplifies the geometric requirements on the slab haunch. However, as new bridges are squeezed into the existing infrastructure it is becoming more common to have geometric transitions on the bridge structure.
Comments:	
	Fillet/haunch thickness at the ends of the prestressed concrete girders and beam seat elevations should account for the roadway vertical curve ordinate.

C-4

Camber Estimating Method (PCI Multiplier, time-dependent)

camber estimating method (PCI Multiplier, time-dependent)

PCI BDM	8.7.1 Multiplier Method: Perhaps the most used method for predicting time-dependent camber of precast, prestressed members is the set of multipliers given in Table 8.7.1-1 (Martin, 1977). Elastic deflections caused by the effects of prestressing, beam self-weight, and other dead loads are calculated using conventional elastic analysis techniques. These are multiplied by the appropriate factors selected from Table 8.7.1-1 to determine the deflections that occur as a result of time-dependent behavior. Suggested multipliers in estimating long-term cambers and deflectioins for typical members include (at erection): 1.85 to member weight at transfer of prestress and 1.80 to prestress at time of transfer of prestress
MS	PCI Multiplier method with adjusted multipliers
FL	For precast, pretensioned, normal weight concrete members designed as simply supported beams, use LRFD (5.9.5.3), Approximate Estimate of Time-Dependent Losses. For all other members use LRFD (5.9.5.4) with a 180-day differential between girder concrete casting and placement of the deck concrete. Commentary: The FDOT cannot practically control, nor require the Contractor to control, the construction sequence and materials for simple span precast, prestressed beams. To benefit from the use of refined time-dependent analysis, literally every prestressed beam design would have to be re-analyzed using the proper construction times, temperature, humidity, material properties, etc. of both the beam and the yet-to-be-cast composite slab.
WA	Erection: This loading typically occurs around 120 days for a normal construction schedule.

C-5

Dead Load Distribution

dead load distribution

MS	Distribute the railing equally over all beams.
TX	Distribute the weight of one railing to no more than three girders, applied to the composite cross section.
FL	Distribute barrier and railing permanent loads per LRFD 4.6.2.2.
AL	Barrier rail load distribution: The barrier rail dead load shall be considered equally distributed across all girders. However, the dead load for girder design shall not be less than 25% of a single barrier rail weight.
Comments:	
	Analytical studies on distribution of the railings, which can effect deflections at erection.

C-6

Girder section properties & strand templates

girder section properties & strand templates

MS	Bridge Division utilizes two types of prestressed concrete girders for the design of bridges: AASHTO and Bulb-T shapes
	Use transformed section properties.
	Do not include the haunch/fillet thickness in the composite section properties.
	For all beam types, the lowest row of strands shall be 2 1/2" above the bottom of the beam. The highest strand location is 3" from the top of the beam.
NE	Use either NU or IT girders for precast concrete girder design, even when widening existing structures
	Designers should check deflections when widening with a different shape.
TX	Use section properties given on the prestressed concrete I-girders STD DWGS.
	Composite section properties may be calculated assuming the girder and slab to have the same modulus of elasticity (for girders with f'c < 8.5 ksi).
	Do not include haunch concrete placed on top of the girder when determining section properties. Section properties based on final girder and slab modulus of elasticity may also be used; however, this design assumption must be noted on the plans.
	Beam Types TX28, TX34, TX40, TX46, TX54, TX62, TX70 including recommended span lengths for LRFD.
FL	The Florida I Beams and the AASHTO Type II Beam are the Department's standard prestressed concrete I-shapped beams and will be used in the design of all new bridges and bridge widenings with I-shaped beams as applicable.
	The Florida U Beams and the Department's standard prestressed concrete U-shapped beams and will be used in the design of all new bridges and bridge widenings with U-shaped beams as applicable.
WA	Ref. Design Memorandum on "Transformed Section Properties" dated October 28, 2011
	Transformed section properties shall not be used for design of prestressed girders. Use of gross section properties remains WSDOT's standard methodology for design of prestressed girders including prestress losses, camber and the flexural capacity.
	In special cases transformed sections properties may be used for the design of prestressed girders with the approval of the State Bridge Design Engineer. In these cases the live load reduction factor at service III limit state load combination shall be as follows: 0.8 when gross section properties are used and 1.0 when transformed section properties are used.
	For pre-tensioned concrete girders, the number of permanent prestressing strands (straight and harped) shall be limited to 100 total 0.6" diameter strands.
AL	The following standard shape AASHTO-PCI type girders shall be used: Type I, Type II, Type III, BT-54, BT-63, and BT-72, as well as solid and voided slab beams.
	Modifications of these girders may be used under special circumstances (such as clearance problems or freeboard limitiations) when approved by the State Bridge Engineer.
	The transformed area of bonded reinforcement shall not be included in the calculations of section properties for prestressed concrete girders.
LA	Louisiana Girder (LG) types, LG-25, LG-36, LG-45, LG-54, LG-63, LG-72, and LG-78, shall be the standard precast prestressed concrete (PPC) girders used for new construction and bridge widening. Quad Beam, AASHTO Type II, III, IV, BT-72 and BT-78 are allowed for bridge rehabilitation projects with the approval of the Bridge Design Engineer Administrator.
	Average haunch weight is considered in the analysis. Haunch thickness is ignored in the calculation of section properties.
	The girder spacing shall not exceed 12.0 feet center-to-center for I-shaped girders.
Comments:	
	MDOT’s Bridge Design Manual currently does not include the FL Bulb-T beam section properties; recommend including the FL Bulb-T beams.
	Include strand templates in design standards.

C-7

Material Properties (f'ci, f'c, E, unit weight, aggregate type)

material properties (f'ci, f'c, E, unit weight, aggregate type)

PCI BDM	2.4.7.4 Unit Weight: In the design of reinforced or prestressed concrete structures, unit weight for design is generally taken as 0.005 kcf greater than density of plain concrete. However, for members with large quantities of prestressing strand, a higher amount may be more appropriate.
	11.2.1.4 Compressive Strength at Transfer: Higher concrete compressive strength at transfer allows a beam to contain more strands and increases the capacility of the beam to resist design loads. However, the availability of high compressive strength concrete at transfer varies throughout the country. Strength at transfer should not be higher than required for the span being designed because strengths in excess of 5.5 to 6.5 ksi may increase the required duraction of the production cycle at the manufacturing plant. This would increase the cost of the beams. Early compressive strength is influenced by local materials and sometimes by production facilities and regional practices. Producers should be consulted about available concrete strengths before beginning design.
MS	The 28-day compressive strength for beam concrete shall be 5,000 psi. Strengths of 5,500 psi and 6,000 psi can be used as required by design.
NE	Girder compressive strengths at final f'c (8, 10, 12, and 15 ksi). 8, 10, and 12 ksi include the use of 0.6 inch prestressing strands. 12 and 15 ksi include the use of 0.7 inch prestressing strands.
	Girder compressive strengths at release = 0.75f'c (6, 7.5, 9, and 11.25 ksi)
TX	Use class H concrete with a minimum f'ci = 4.0 ksi and f'c = 5.0 ksi and a maximum f'ci = 6.0 ksi and f'c = 8.5 ksi
	Use an effective strand stress after release of 0.75 fpu - ES losses
WA	Ref. Design Memorandum on "Unit Weight of Concrete" dated June 1, 2010
	This memorandum defines the unit weight of concretes for dead load and modulus of elasticity calculations.
	For normal weight concrete (precast pretensioned or post-tensioned spliced girders); use 155 pcf for the modulus of elasticity (plain concrete) and 165 pcf dead load (with reinforcement).
	Background: The unit weight of precast concrete girders is generally taken as 10 pcf greater than the unit weight of plain concrete due to the weight of reinforcement and strands.
	Prestressed Concrete Girders: Nominal 28-day concrete strength (f'c) for prestressed concrete girders is 7.0 ksi. Where higher strengths would eliminate a line of girders, a maximum of 10.0 ksi can be specified.
	The minimum concrete comprssive strength at release (f'ci) for each prestressed concrete girder shall be shown in the plans. For high strength concrete, the compressive strength at release shall be limited to 7.5 ksi. Release strengths of up to 8.5 ksi can be achieved with extended curing for special circumstances.
	Modulus of Elasticity: The modulus of elasticity shall be determined as specified in AASHTO LRFD Section 5.4.2.4. For calculation of the modulus of elasticity, the unit weight of plain concrete (wc) shall be taken as 0.155 kcf for prestressed concrete girders and 0.150 kcf for normal-weight concrete. The correction factor (K1) shall normally be taken as 1.0.
	Shrinkage and Creep: Shrinkage and creep shall be calculated in accordance with AASHTO LRFD Section 5.4.2.3. The relative humidy, H, may be taken as 75 percent for standard conditions. The maturity of concrete, t, may be taken as 2,000 days for standard conditions. The volume-to-surface ratio, V/S, is given in Table 5.6.1-1 for standard WSDOT prestressed concrete girders.
	In determining the maturity of concrete at initial loading, ti, one day of accelerated curing by steam or radiant heat may be taken as equal to seven days of normal curing.
	Ref. Technical Report WA-RD 669.1
	The camber was found to depend on the elastic modulus of the concrete, its creep coefficient, and the use of the prestress losses in the calculation of the creep camber.
	To achieve the best match with the measured cambers, the AASHTO recommended values for the elastic modulus and the creep coefficient had to be multiplied by adjustment factors and the prestress losses had to be taken into account when computing the creep component of camber.
	Recommendations for practice: For deflection calculations, increase the specified concrete strengths by 10 percent at release and 25 percent at 28 days.
	Use 1.15 times the AASHTO LRFD 2006 equation for predicting the concrete elastic modulus (Ec) for a given concrete strength. An alternative would be to adopt the methods recommended by NCHRP or CEB-FIP.
AL	Prestressed Concrete: The following values of f'c shall be used for prestressed concrete structures: girders 5.0 to 8.0 ksi (higher strengths will require prior approval by the State Bridge Engineer)
LA	Girder Concrete: f'ci = 6.5 ksi, f'c = 8.5 ksi, density for dead load = 155 pcf, density for modulus of elasticity = 148.5 pcf
	Girder Concrete: f'ci = 7.5 ksi, f'c = 10.0 ksi, density for dead load = 155 pcf, density for modulus of elasticity = 150 pcf
	Prestress Losses: Use gross section and include elastic gains.

C-8

Prestress Loss Data (time, humidity, curing method)

prestress loss data (time, humidity, curing method)

AL	For calculating losses, use the AASHTO LRFD Approximate Method, neglecting gains. The following values shall be used for calculating losses: Time at release: 0.75 days, Age of deck placement: 60 days, final age: 27500 days, relative humidity: 75%
LA	Prestress Losses: Use gross section and include elastic gains.

C-9

Temperature Gradient

temperature gradient	
FL	Include the effects of Temperature Gradient in the design of continuous concrete superstructures only. The vertical temperature gradient shall be taken as shown in LRFD Fig. 3.12.3-2.

C-10

Prestressed Beam Detail Plan Sheet Information

prestressed beam detail plan sheet information

MS Camber limit (at release only).
Prestressed concrete girder design information is shown on the contract plans.
Prestress requirements table is shown on the contract plans as follows:

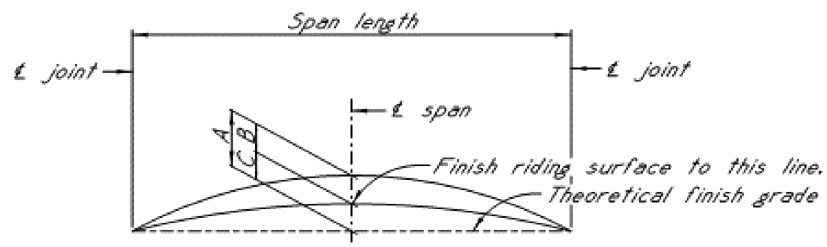
LR indicates low-relaxation strands

PRESTRESS REQUIREMENTS

For deflection diagram, see Misc. Span Details per Sheet No. 22

Strand type	Minimum breaking strength (lbs/strand)	Initial tension (lbs/strand)	Required number and location of strands: Total number strands	Straight strands: Number strands	Straight strands: Centroid (in.)	Draped strands: Number strands	Draped strands: Centroid (in.) ℄ span	Draped strands: Centroid (in.) Beam end	Centroid for total number of strands (in.): At ℄ span	Centroid for total number of strands (in.): At beam end	Distance from ℄ span to hold-down point	Camber limits	Deflection diagram: A	Deflection diagram: B	Deflection diagram: C	Minimum concrete strength at time of release (psi)
0.6"ϕ270 K-LR	45,000	△1 43,940	36	26	4.19	10	6.50	65.00	4.83	21.08	14'-0"	0 to 5¾"	5 1/16"	2 5/16"	2¾"	6000

A deflection diagram is shown on the contract plans as follows:



DEFLECTION DIAGRAM

A = total recommended allowance for deflection.
B = estimated deflection due to dead load of slab & rail.
C = A-B = net initial camber in riding surface, which includes an allowance or creep.

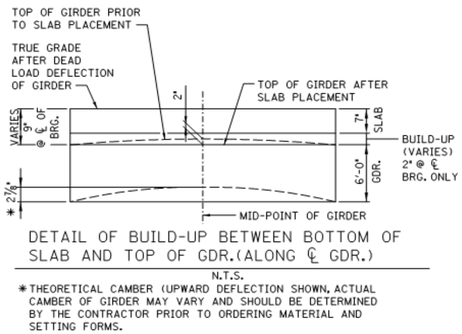
TX Show predicted slab deflections on the plans although field experience indicates actual deflections are generally less than predicted.
Include dead load deflection diagram and table of section depths.

FL Report elastic and time dependent shortening effects (DIM R) at mid-height of the beam at 120 days. The average of the calculated values for the top and bottom of the beam may be used.

WS The "D" dimension is the computed girder deflection at midspan (positive upward) immediately prior to deck slab placement.
Standard Specifications Section 6-02.3(25)K defines two levels of girder camber at the time the deck concrete is placed, denoted D at 40 Days and D at 120 Days. They shall be shown in the plans to provide the contractor with lower and upper bounds of camber that can be anticipated in the field.
D at 120 Days is the upper bound of expected camber range at a girder age of 120 days after the release of prestress and is primarity intended to mitigate interference between the top of the cambered girder and the placement of concrete deck reinforcement. It is also used to calculate the "A" dimension at the girder ends.
The age of 120 days was chosen because data has shwon that additional camber growth after this age is negligible.
D at 40 Days is the lower bound of expected camber range at a girder age of 40 days (30 days after the earliest allowable girder shipping age of 10 days).
To match the profile grade, girders with too little camber require an increased volume of haunch concrete along the girder length. For girders with large flange widths, such as the WF series, this can add up to significant quantities of additional concrete for a large deck placement. Thus, the lower bound of camber allows the contractor to assess the risk of increased conrete quantities and mitigates claims for additional material.
Girder stirrups shall all extend at least 5 in. from the top of the girder, but typically no more than the deck thickness minus 2.5 in.
Hat bars shall be the same size as the girder stirrups. Prebent stirrups may be used with "hat bar" stirrup extensions. Details shall conform to Figure 5.6.2-7.
Hat bars may be omitted at locations where girder stirrups project at least 3 in. above the bottom of the transverse bar in the bottom mat of the bridge deck.
Computation of "A" Dimension: The distance from the top of the bridge deck to the top of the girder at centerline bearing at centerline of girder is represented by the "A" Dimension. It is calculated in accordance with the guidance of Appendix 5-B1. This ensures that adequate allowance will be made for excess camber, transverse deck slopes, vertical and horizontal curvatures. Where temporary prestress strands at top of girder are used to control the girder stresses due to shipping and handling, the "A" dimension must be adjusted accordingly.
Stirrup Length and Precast Deck Leveling Bolt Considerations: For bridges on crown vertical curves, the haunch depth can become excessive to the point where the girder and diaphragm stirrups are too short to bend into the proper position. Similary the length of leveling bolts in precast deck panels may need adjustment. Stirrup lengths are described as a function of "A" on the standard girder sheets. For example, the G1 and G2 bars of a WF74G girder are 6'-5" + "A" in length. For this reason, the stirrups are always long enough at the ends of the girders. Problems occur when the haunch depth increases along the length of the girder to accommodate crown vertical curves and superelevation transitions. If the haunch depth along the girder exceeds "A" by more than 2", an adjustment must be made.

AL Build-up over Top of Prestressed Concrete Girders: The following shall be shown on the prestress camber diagram for the ends of the prestressed concrete girder:

girder depth, haunch thickness, deck thickness, total deck plus haunch thickness, theoritical camber, dead load deflection
The reporting format for this information is pictographically provided in the Bridge Plan Detailing Manual.



A minimum one inch haunch shall be provided at girder mid-span, calculated at the critical edge of the girder flange. Minimum buildup at girder ends shall take into consideration vertical curve, superelevation transition, or other complex roadway geometry. The build-up should be investigated for each girder line and adjusted as necessary.

For prestressed concrete members, specific concrete strengths used for design and specified for fabrication shall be stated on the contract drawings.

LA

Strand pattern details showing strand layouts, number and spacing of strands, concrete cover and edge clearances, and layout of all mild reinforcing steel shall be shown in contract plans.

All girder related design data shall be shown in a girder data table. Refer to LG girder design aids in Part III, Chapter 1 for a girder data table template.

The Camber Data Table shall be included in contract plan.

CAMBER DATA TABLE																
		DESIGN DATA				FIELD MEASURED DATA *										
SPAN NO.	GIRDER DESIGNATION	C1 (IN.)	C2 (IN.)	C3 (IN.)	D5 (IN.)	MC1 (IN.)	MC1a (IN.)	MC2 (IN.)	fb1 (KSI)	fb1a (KSI)	fb2 (KSI)	Eb1 (KSI)	Eb1a (KSI)	Eb2 (KSI)	* DATE OF GIRDER CASTING	* DATE OF RISER POUR
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

Comments:

Refer to PCI's Recommendations for Camber Tolerances (at release only).
None of the other State DOT's calculate a camber limit.
Consider including camber at release estimate.

C-11

Camber

camber

PCI BDM	Refer to section 3.4.2.6 Camber for additional information.
PCI BDM	3.4.6 Tolerances: Good design and detailing practices for precast components and connections always consider allowable tolerances for fabrication, erection, and interfacing field construction. PCI Manual 116 lists industry standard tolerances for typical precast concrete bridge members. Details allowing generous tolerances usually result in economies during construction, while extremely stringent tolerances can be very expensive and is some cases, may not be achievable. Designers should consult local producers when considering tolerances that are tighter than the industry standards.
MS	Calculates camber limit (at release only).
NE	Final deflection due to effective prestress plus dead loads shall be an upward camber. No downward camber at 30 days is permitted. Camber shall not be considered in the vertical clearance determination under a bridge. For the purposes of determining the vertical clearances, the bottom of the girder shall be considered a straight line between the bearings. All girder bridge plans shall have deflections calculated at the span tenth points and labeled.
TX	Compute deflections due to slab weight and composite dead loads assuming the girder and slab to have the same modulus of elasticity. Assume Ec = 5,000 ksi for girders with f'c < 8.5 ksi. Use the deflection due to slab weight only times 0.8 for calculating haunch depth. A calculated positive (upward) camber is required after application of all permanent (dead) loads.
FL	Stress and camber calculations for the design of simple span, pretensioned components must be based upon the use of transformed section properties. Unless otherwise required as a design parameter, beam camber for computing the build-up shown on the plans must be based on 120-day old beam concrete. On the build-up detail, show the age of beam concrete used for camber calculations as well as the value of camber due to prestressing minus the dead load deflection of the beam. Consider the effects of horizontal curvature with bridge deck cross slope when determining the minimum buildup over the tip of the inside flange. Commentary: In the past, the FDOT has experienced significant deck construction problems associated with excessive prestressed, pretensioned beam camber. The use of straight strand beam designs, higher strength materials permitting longer spans, stage construction, long storage periods, improperly placed dunnage, and construction delays are some of the factors that have contributed to camber growth. Actual camber at the time of casting the deck equal 2 to 3 times the intitial camber at release is no uncommon. Design pretensioned beams so that the theoretical design camber at the end of construction is positive (upward) after all non-composite and composite dead loads are applied. Camber variability of prestressed components is affected by a number of items such as; aggregates, curing conditions, strand patterns, casting/detensioning temperatures, design strength versus actual strength of concrete, weekday versus weekend and holiday casting cycles, support conditions during storage, hauling and handling, and component age of time of loading. Commentary: Camber variability is common. Requiring steam curing or creep testing of the actual concrete mixes used may improve camber predictions; however, fairly large variations in camber may still exist due to other factors as those listed.
WA	Accurate predictions of deflections are difficult to determine, since modulus of elasticity of concrete, Ec, varies with stress and age of concrete. Also, the effects of creep on deflections are difficult to estimate. For practical purposes, an accuracy of 10 to 20 percent is often sufficient.
AL	Calculation of camber due to prestress prior to pouring the bridge deck shall be based on a 60-day interval between release of the strand and erection of the girder.
LA	Camber and Deflection: Use PCI Multiplier method. The PCI multiplier method is adopted to calculate the estimated camber at erection provided that initial camber due to prestress and deflection due to girder self-weight at transfer are calculated separately. The use of PCI multipliers has shown to give reasonable estimates for camber at the time of erection. For prestressed girder projects in which the contractor elects to fabricate all the girders at the same time but girder placement will extend months after casting (such as for phased construction or very large projects), the contractor must be responsible for camber growth. The Camber Data Table shown below shall be included in contract plan. The camber design data (C1, C2, C3 and D5) are provided by the EOR. The field measured data (MC1, MC1a, MC2, fb1, fb1a, fb2, Eb1, Eb1a and Eb2) and the dates of girder casting and riser pour shall be recorded by the contractor in the Camber Data Table. The Camber Data Table shall be submitted to the EOR for review at least 14 days prior to riser pour. When field measured MC1 or MC2 differ more than 1/2" (+ or -) from the estimated "C1" or "C2", the contractor shall notify the EOR immediately to investigate corrective measures, such as modify risers and/or roadway profile, etc.
Comments:	
	Refer to PCI's Recommendations for Camber Tolerances (at release only). None of the other State DOT's calculate a camber limit.

APPENDIX D

Camber Data Sets

D-1

Camber Data Sets Outline

MDOT

Camber Best Practices

Camber Data Sets outline

10-26-2018

The following provides an outline to information that will be researched part of the camber data sets used to evaluate the effects on camber estimated based on; literature review, historic material information provided by the MS Concrete Girder Manufacturers, MDOT projects that have experienced "under-camber" on girders at erection, and MDOT's current practices for estimating camber.

AASHTO LRFD Code provisions related to estimating camber will be included along with considering other State DOT practices for estimating camber.

<u>Known information:</u>

1. Actual concrete compressive strengths both at release and at 28-days are greater than design concrete compressive strengths.

2. Several MDOT projects have experienced under-camber on girders at erection. Data provided by MDOT will provide insight to the amount of differences between the design camber compared to actual/measured camber at erection. These differences will try to be simulated part of the camber data sets.

<u>Plan:</u>

Evaluate the differences between camber estimates using actual concrete compressive strengths based on the historic material information provided by the MS concrete girder manufacturers compared to design concrete compressive strengths.

Various sets of data will be developed through example camber estimate calculations that capture the sensitivity of the difference in camber estimates between using actual concrete compressive strengths compared to using design concrete compressive strengths.

For a particular girder type, evaluate the minimum and maximum span capabilities to capture the range of camber variations. Vary both girder lengths and girder spacings. Use AASHTO Type 4 girders and FL Bulb-T girders. Use the bridge data provided by MDOT's Bridge Division for the Leake County and Marshall County projects.

<u>Items to evaluate that influence estimating camber:</u>

- Release concrete compressive strength (f'ci)
- 28-day concrete compressive strength (f'c)
- Release modulus of elasticity (Eci)
- 28-day modulus of elasticity (Ec)
- Unit weight (wc)
    - Vary the self weight of the girders by using different units weights (e.g., 150 pcf, 155 pcf, and 160 pcf) to evaluate the effect of unit weight on estimated camber.
    - Run a sample calculation on the girder self weight (for an AASHTO Type 4 and FL Bulb-T) by accounting for the additional weight of the strand to see if there is an appreciable difference in the girder self weight.
        - Recommend MS concrete girder manufactures collect all girder self weights to be able to have a historical database of unit weights produced. Girder weights are required for shipping/hauling permits also and the girder weights may already be documented by the MS concrete girder manufactures.
    - Recommend further research be performed to obtain unit weights and modulus of elasticity information from the MS concrete girder manufactures.
        - Also note the type of aggregate used in the mix-design to further understand whether the AASHTO LRFD formula to compute the modulus of elasticity for concrete can be adjusted based on the type of aggregate and historic information. Since historical information does not exist related to the types of aggregates used to manufacture the girders, the research will not utilize the adjustment factor in the AASHTO LRFD Bridge Design Specifications but the effect of adjusting the aggregate factor (K1) will be included.
        - The modulus of elasticity formula in the AASHTO LRFD Bridge Design Specifications will be used for both the design values of f'ci and f'c and the average actual values of f'ci and f'c to evaluate the effects on estimating camber.
- Transformed section properties
    - Compare prestress losses with and without transformed section properties to evaluate whether prestress losses differ; if prestress losses differ between using gross section properties compared to transformed section properties then isolate the moment of inertia effects when using transformed section properties by manually entering the percentage of prestress losses (%) to be the same for gross section properties and transformed section properties.

Items to evaluate that influence estimating camber (continued):

- Prestress losses
    - It is suggested to specify what the various parameters are to be used part of the prestress loss estimates (refer to ALDOT's guidelines)
        - Time at release = 0.75 days
        - Age of deck placement = 60 days
        - Final age = 27500 days
        - Relative humidity = 75%
    - PCI's Bridge Design Manual (design examples) assume the following construction schedule; 1-day at transfer, 90-days at erection/deck placement, and 20,000-days at final stage. Section 9.0.1 Service Life discusses the assumed age for the various design examples as related to long-term (i.e., final) prestress losses
    - Age of girder at erection/deck placement
    - Time-dependent analysis
- Haunch/fillet thickness
    - Compare deflections at erection with and without the haunch/fillet thickness in the composite section properties
- PCI multipliers used in the PCI Multiplier Method for estimating camber
- Temperature gradients
- Straight vs. draped strand patterns
- Where required to satisfy allowable stresses using straight strand patterns, top strand with and/or without reduced pull
- Increased stiffness of girders using actual 28-day concrete compressive strengths
    - Evaluate the effect of increased stiffness of the girders when using the actual 28-day concrete compressive strengths on the live load distribution factor
- Roadway Vertical curve ordinate
    - Although this item is not directly related to estimating camber, it could have an effect on the haunch thickness at the ends of girders and is related to calculating beam seat elevations, therefore procedures are recommended to include the roadway vertical curve ordinate in the calculation of the haunch/fillet thickness at the ends of the girders.
    - The research will not consider the roadway vertical curve ordinate.

Historic Material Information Provided by MS Concrete Girder Manufacturers:

	Producer 1	Producer 2	Producer 3
design f'ci	5600 psi	4480 psi	6800 psi
Average actual f'ci	5622 psi	7503 psi	7725 psi
Ratio of average actual f'ci/ design f'ci	1.004	1.67	1.14
Design f'c	6500 psi	5625 psi	8500 psi
Average actual f'c	9501 psi	10644 psi	10441 psi
Ratio of average actual f'c/design f'c	1.46	1.89	1.23
Ratio of Average actual f'ci/average actual f'c	0.59	0.71	0.74

Observations with respect to f'ci and f'c:

1. Use the low, high, and average ratios to vary f'ci and f'c to evaluate the effects on camber estimates.
   a. Values for f'ci include; low = 1.004, average = 1.27, high = 1.66
   b. Values for f'c include; low = 1.23, average = 1.53, high = 1.89
2. Producer 3 provided 2-day break data (not shown in the above table) that can be considered in utilizing higher design release strengths. For example, the average 2-day actual concrete compressive strength was 8984 psi which is a ratio increase from the 6800 psi design f'ci of 1.32.
3. The relationship between f'ci/f'c can be used to understand the strength gain during design. The above values based on historic material information can assist in establishing guidelines. For example the lowest ratio was 0.59, the highest was 0.74, and the average ratio was 0.68. Compare these values with other State DOT guidelines on design values for f'ci and f'c.

Historic Camber Data Provided by MS Concrete Girder Manufacturers:

	Producer 1	Producer 2	Producer 3
Estimated camber (at release)	Data not provided	0.94 in.	Data not provided
Average measured camber (at release)	no camber data provided	0.74 in.	Provided data on 15 projects with difference beam types
Average measured camber (28-days)	no camber data provided	2.37 in.	
Ratio of Average measured camber (28-days)/average measured camber (at release)	no camber data provided	3.2	

Observations with respect to camber:

1. For Producer 2; use both the average actual f'ci and design f'ci to estimate camber at release for similar beam type and span length and compare the estimated camber differences at release to see if there is a correlation between the estimated camber and measured camber at release.
    a. Compare 28-day camber data to estimated camber data sets
2. For Producer 3; the camber data shows variation in the measured camber; therefore calculate the range of variation (i.e., low and high values from the average measured camber).
    a. Look for consistencies between various beam types on the spread/magnitude the variations in the measured cambers.
    b. Vary f'ci and Eci using average actual f'ci values and compare effects with measured camber to see if there is a correlation to the relationship between 1) the design f'ci and estimated camber and 2) the actual f'ci and measured camber.
        i. Compare results with estimated camber using PCI multiplier method (at release only). MDOT is currently adjusting the PCI multipliers when estimating camber; comment on this approach.

MDOT Guidelines for Estimating Camber:

MDOT's Bridge Division has beam design details published in the Bridge Design Manual (Version 6.1) and has a Bulb-T Design Procedure (revised 3/24/2017). The following information is drawn from MDOT's current procedures and guidelines for estimating camber and will be used as a basis to compare against other State DOT procedures and guidelines for estimating camber:

- Maximum haunch/fillet check
  - MDOT's maximum haunch/fillet check does not take into consideration the actual f'ci or the actual f'c
  - MDOT's current guidelines do not consider the increase in the composite section properties (i.e., composite moment of intertia) as a result of increase in the composite section depth/height due to an "under-camber" girder at erection.
- Minimum haunch/fillet thickness at edge of the girder top flanges.
  - MDOT currently does not provide guidelines on the minimum haunch/fillet thickness whereas other State DOT's do.
- Zero tension
- Transformed section properties
- Current procedures to set the PCI Multipliers to all 1.0 values
- MDOT's Bridge Design Manual currently does not include the FL Bulb-T beam section properties; it is recommended to include the FL Bulb-T beams
- Allows the use of either #5 reinforcing bars or 0.5-inch strands with 2 kip pull in the top of the beam when required to satisfy release stresses
- Use 2-inch haunch/fillet as added dead load. Do not include the haunch/fillet thickness in the composite section properties.
  - Part of the camber data sets, compare deflections at erection with and without the haunch/fillet thickness in the composite section properties
- Distribute railings equally over all beams
  - Not included part of the research scope of work, recommend further analytical studies on distribution of the railings, which can effect deflections at erection. Refer to ALDOT's distribution guidelines.
- Consider actual haunch/fillet thickness over the beam length (i.e., due to the upward camber of the girders, the shape of the haunch/fillet thickness varies over the length of the girder in a parabolic profile). Therefore, the added thickness increases both the dead load and composite section properties near the ends of the girder, which can effect deflection estimates. To simplify the calculation of the haunch/fillet dead load and section properties, an average thickness is typically used/assumed for the haunch/fillet.
- MDOT's Bridge Design Manual (Bulb-T Design Procedure, section 1.f) states "Use Debonding if necessary (Debond to nearest tenth points,………."
  - *Note: MDOT's current practice is to use 2 ft. debond increments; therefore suggest updating Bridge Design Manual to reflect this*

  Comment on other State DOT practices to use 2 ft. debond increments.

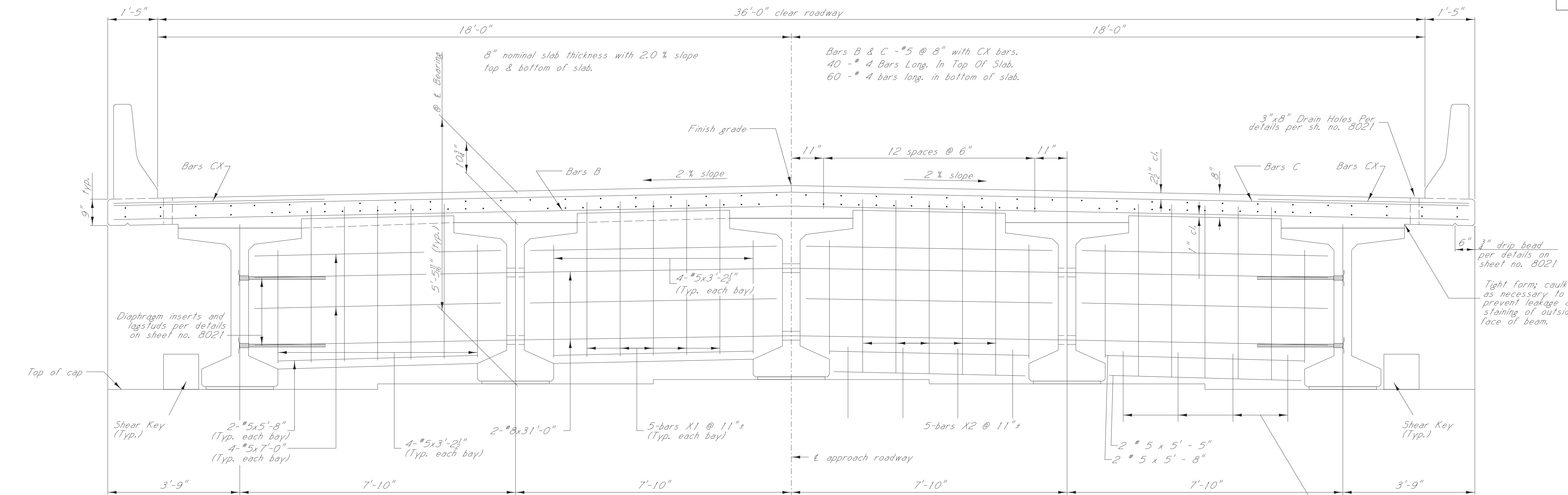
- Camber limit (at release only). Comment on the calculation and use of the camber limit compared to other State DOT practices and PCI's Recommendations for Camber Tolerances (at release only).
- Compare camber/deflection values placed on MDOT girder detail sheets vs. other State DOT values placed on the girder detail sheets.

D-2

Sample Plans for MDOT Project in Marshall County

2nd O.REV.

STATE	PROJECT NO.
MISS.	BR-0060-03(021)



PART SECTION NEAR END OF SPAN

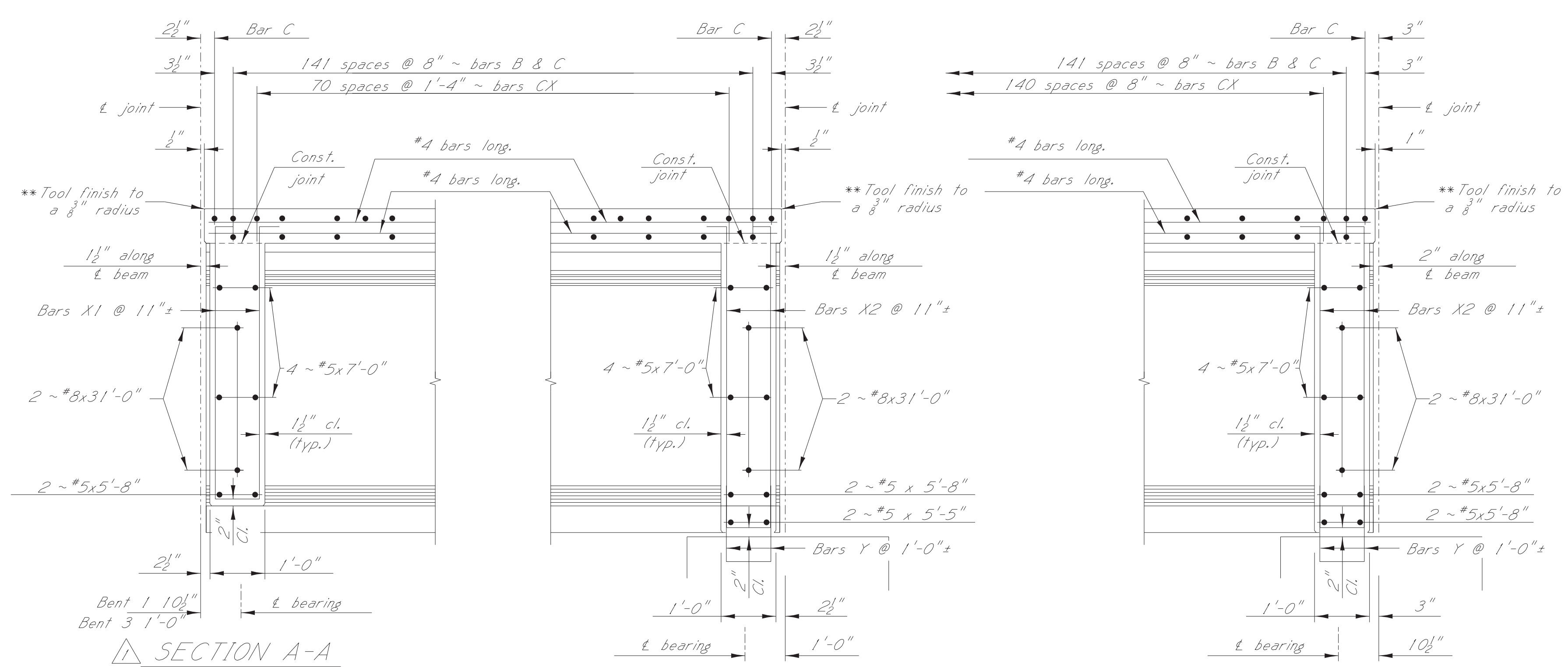
Showing end diaphragm details

PART SECTION NEAR FIXED END DIAPHRAGM

Showing end diaphragm details and slab reinforcement

Bars Y (Typ. each bay) details this sheet. 1'-0" embeddment (typ.) For placement see sheet no. 8009 for span 2 and sheet no. 8011 for span 3.

**NOTE: ⅛" seat required at joint. See sealing details on sheet no. 8021



SECTION A-A

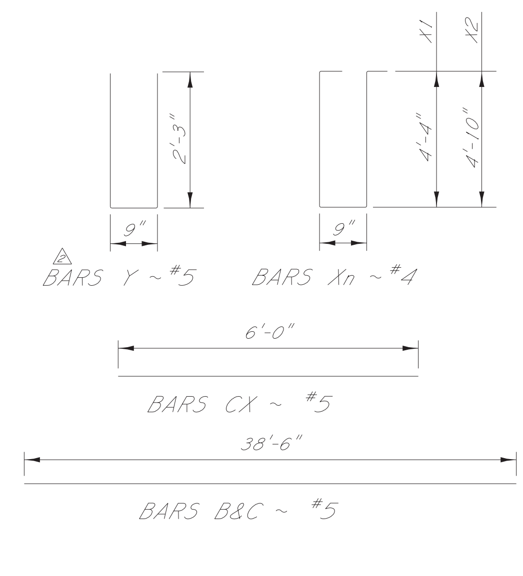
Showing end section detail for Span No. 1 and 3

SECTION B-B, SPAN NO. 1

Showing end section detail at Span No. 1, Bent No. 2

SECTION B-B, SPAN NO. 3

Showing end section detail at Span No. 3, Bent No. 4 Detailing 2" Joint



BAR BENDING DETAILS

Dimensions are out to out

NOTE:
Contractor should be aware of possible tilting of exterior beams during construction of the superstructure and should take precautionary steps to prevent such tilting of beams.

NOTE:
Insure that holes in beam webs are completely filled with diaphragm concrete.

NOTE:
The volume of concrete in the fillets between the bottom of the nominal slab and the top of the beam has been estimated by using one half (1/2) of the fillet height, at the bearing, multiplied by the top flange width and the full length of the beam. This volume shall be used for final pay quantity.
*Any additional concrete required in the fillet resulting from an unexpected camber in the beam will not be directly paid for and shall be considered an absorbed item.

DESIGN DATA:
Specifications........................A.A.S.H.T.O. LFRD 2014 with 2016 interims
Loading................................HL-93
Slab stresses.....................f_s=24,000 p.s.i. ; f_c=1,600 p.s.i. ; n=8
Prestressed beam details...See sheets no. 8022,8024.

DATE	REVISION	BY
2-5-2018	Revised Details	TNT
2-9-2018	Added Detail and Note	TNT

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

BRIDGE AT STA. 1077+37.88

SR-4 OVER CUFFAWA CREEK
95' SPAN DETAILS
SPAN NO. 1 AND 3

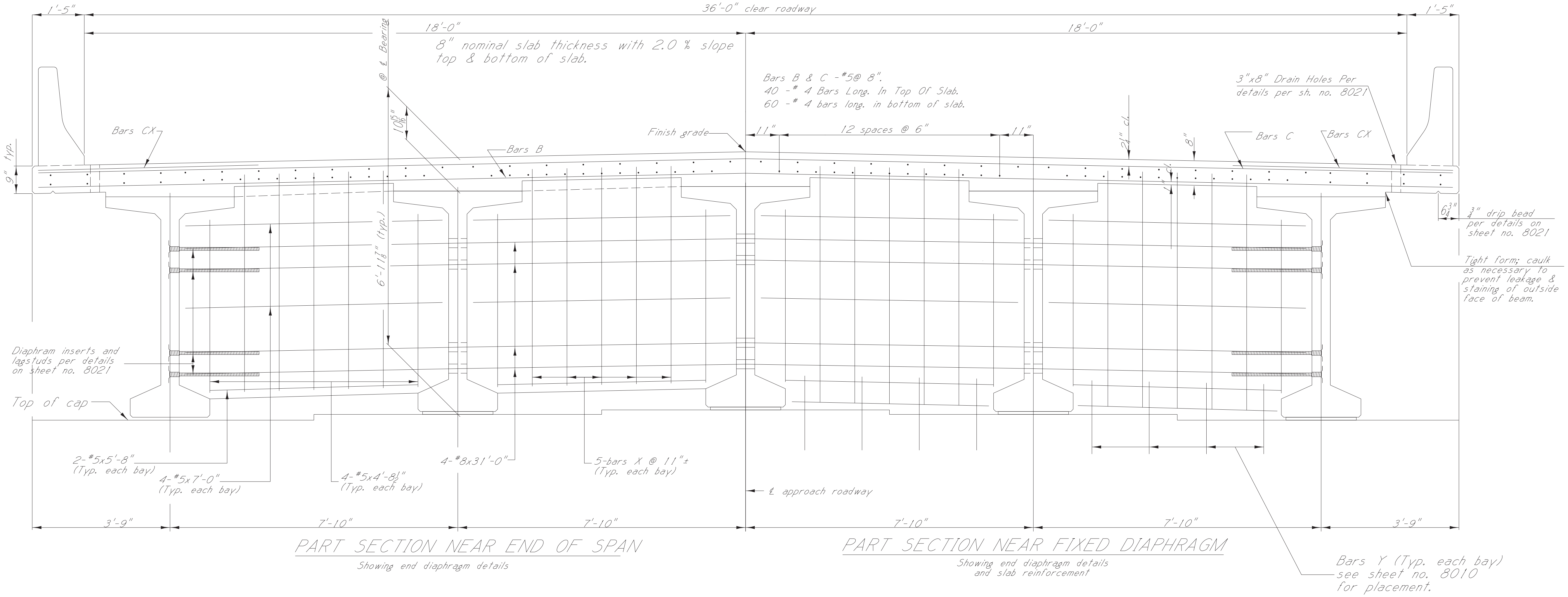
FMS: 102207 / 302000
COUNTY: Marshall
PROJECT NUMBER: BR-0060-03(021)

DESIGNER Thomas Terry CHECKER Spencer Yates
DETAILER Thomas Terry ISSUE DATE 2017-07-12
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

WORKING NUMBER 13 OF 30
SHEET NUMBER 8015

PLAN
BRIDGE DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

STATE	PROJECT NO.
MISS.	BR-0060-03(021)

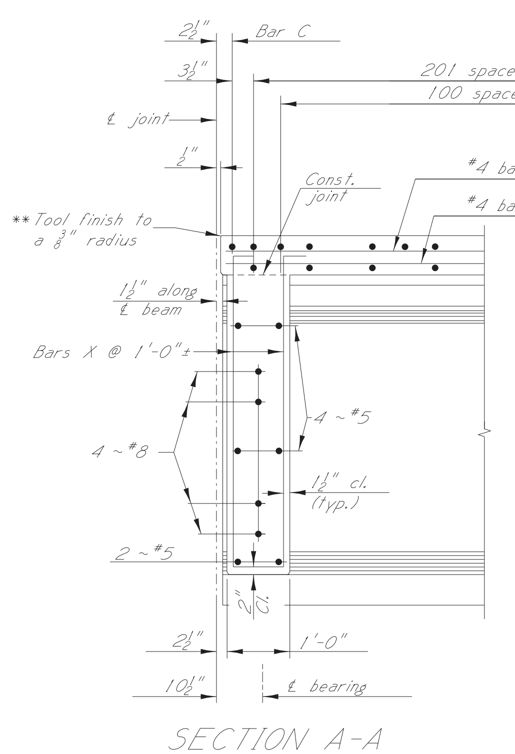


PART SECTION NEAR END OF SPAN

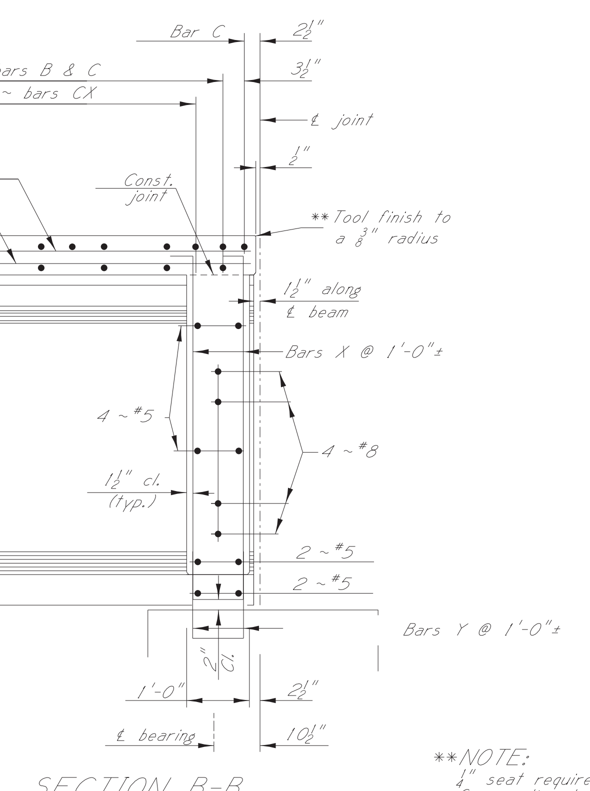
Showing end diaphragm details

PART SECTION NEAR FIXED DIAPHRAGM

Showing end diaphragm details and slab reinforcement

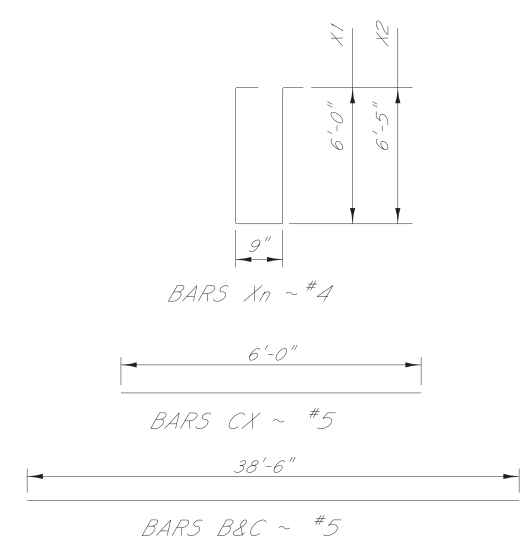


SECTION A-A



SECTION B-B

**NOTE:
$\frac{1}{4}$" seat required at joint. See sealing details on sheet no. 8021



BAR BENDING DETAILS

Dimensions are out to out

NOTE:
Contractor should be aware of possible tilting of exterior beams during construction of the superstructure and should take precautionary steps to prevent such tilting of beams.

NOTE:
Insure that holes in beam webs are completely filled with diaphragm concrete.

NOTE:
The volume of concrete in the fillets between the bottom of the nominal slab and the top of the beam has been estimated by using one half (1/2) of the fillet height, at the bearing, multiplied by the top flange width and the full length of the beam. This volume shall be used for final pay quantity.
*Any additional concrete required in the fillet resulting from an unexpected camber in the beam will not be directly paid for and shall be considered an absorbed item.

DESIGN DATA:
Specifications......................A.A.S.H.T.O. LFRD 2014 with 2016 interims
Loading...............................HL-93
Slab stresses.......................f_s=24,000 p.s.i. ; f_c=1,600 p.s.i. ; n=8
Prestressed beam details...See sheets no. XXXX

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

BRIDGE AT STA. 1077+37.88

SR-4 OVER CUFFAWA CREEK
135' SPAN DETAILS
SPAN 2

BY | REVISION | DATE

FMS: **102207 / 302000**

COUNTY: **Marshall**

PROJECT NUMBER: **BR-0060-03(021)**

DESIGNER Thomas Terry — CHECKER Spencer Yates
DETAILER Thomas Terry — ISSUE DATE 2017-07-12

DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

WORKING NUMBER 15 OF 30

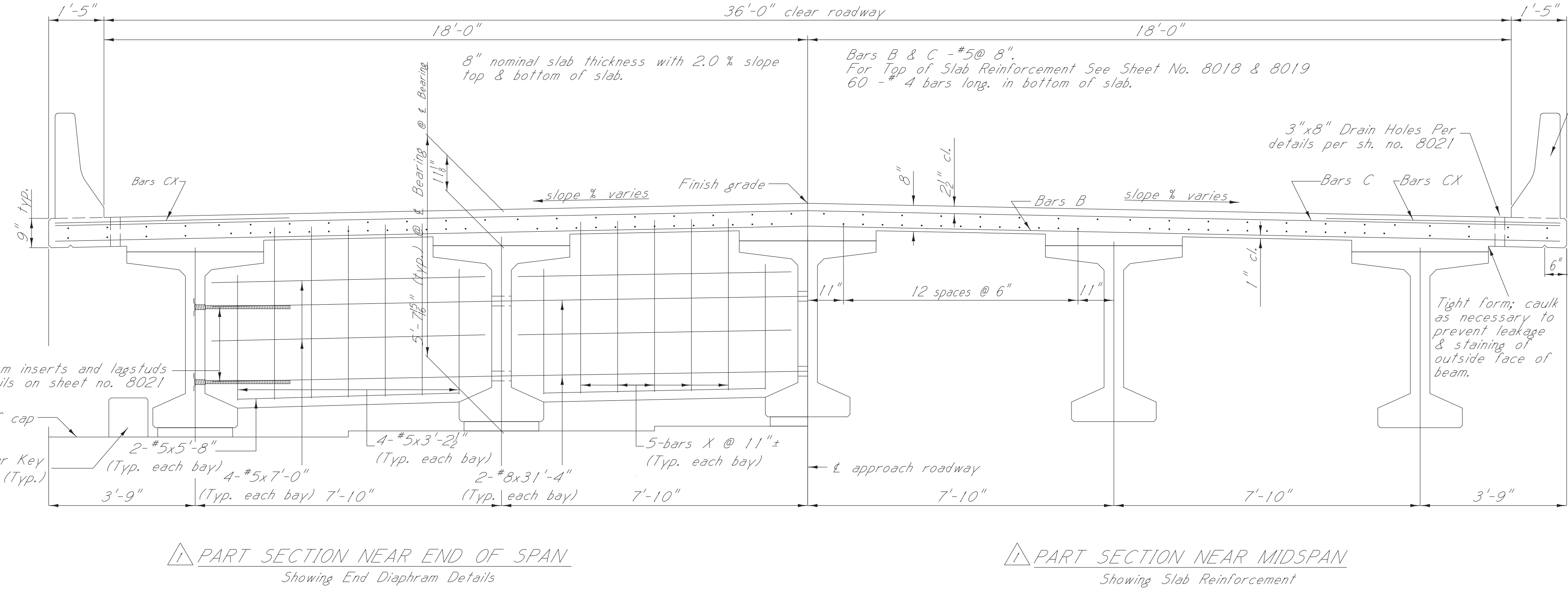
SHEET NUMBER 8017

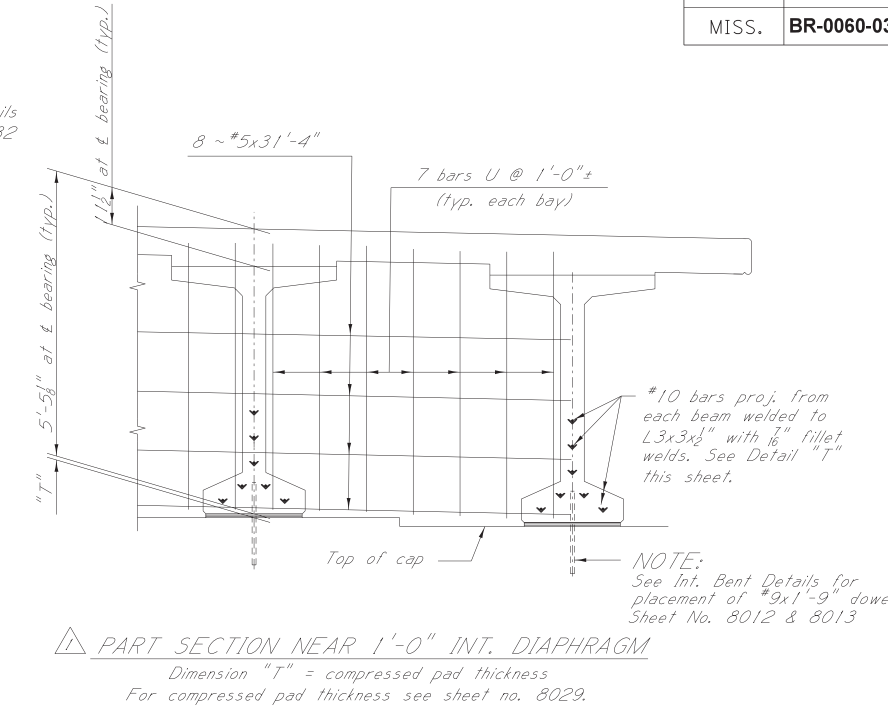
PLAN
BRIDGE DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

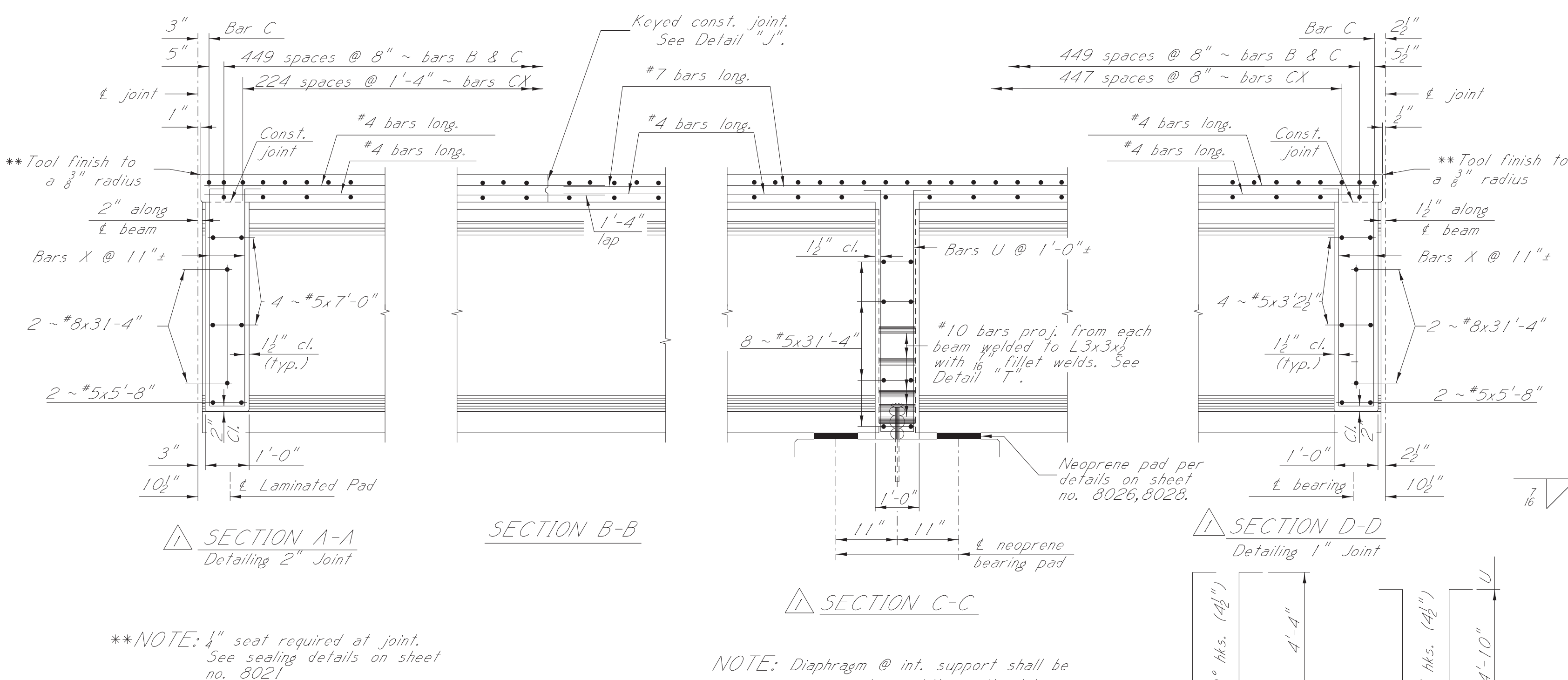
$DATE$ $TIME$ $FILENAME$

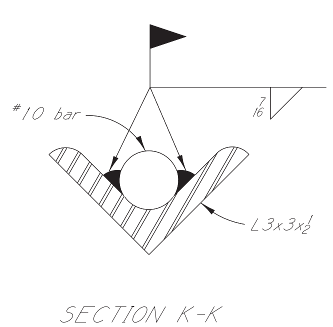
2nd O.REV.

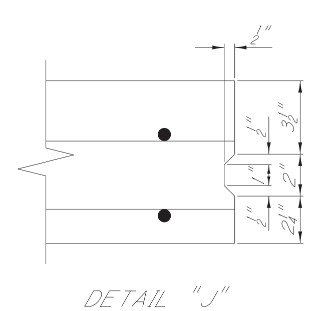
STATE	PROJECT NO.
MISS.	BR-0060-03(021)

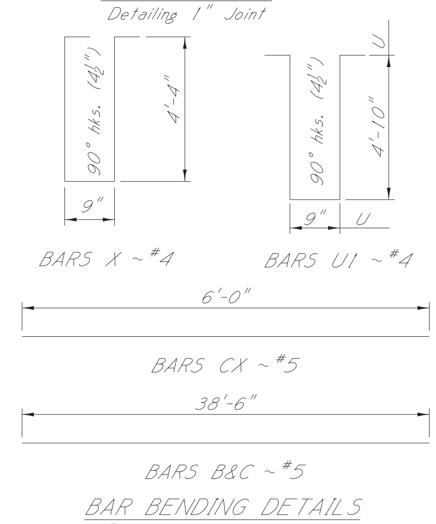


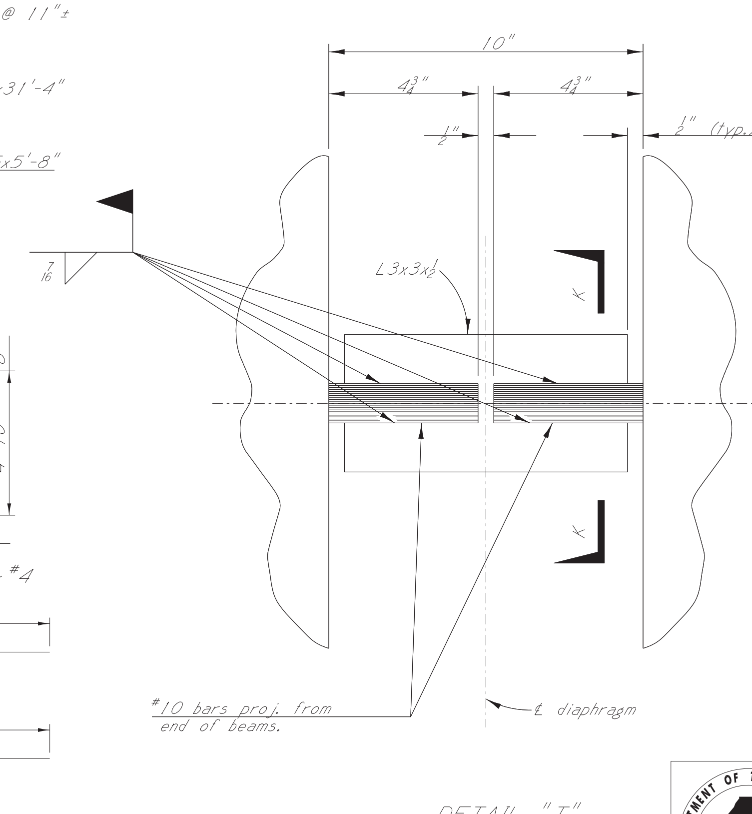












NOTE:
Contractor should be aware of possible tilting of exterior beams during construction of the superstructure and should take precautionary steps to prevent such tilting of beams.

NOTE:
Insure that holes in beam webs are completely filled with diaphragm concrete.

NOTE:
The volume of concrete in the fillets between the bottom of the nominal slab and the top of the beam has been estimated by using one half (1/2) of the fillet height, at the bearing, multiplied by the top flange width and the full length of the beam. This volume shall be used for final pay quantity.
*Any additional concrete required in the fillet resulting from an unexpected camber in the beam will not be directly paid for and shall be considered an absorbed item.

DESIGN DATA:
Specifications......................A.A.S.H.T.O. LRFD 2014 with 2016 interims
Loading..............................HL-93
Slab stresses....................f_s=24,000 p.s.i. ; f_c=1,600 p.s.i. ; n=8
Prestressed beam details...See sheets no. 8025,8026,8027

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 1077+37.88
95-110-95 ft. SPAN DETAILS
SPANS 4 THRU 6

FMS: 102207 / 302000
COUNTY: Marshall
PROJECT NUMBER: BR-0060-03(021)

WORKING NUMBER 18 OF 30
SHEET NUMBER 8020

DATE	REVISION	BY
2-5-2018	Revised Details	TNT

DESIGNER Thomas Terry — CHECKER Spencer Yates
DETAILER Thomas Terry — ISSUE DATE 2017-07-12
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

MISSISSIPPI DEPARTMENT OF TRANSPORTATION BRIDGE DIVISION PLAN

STATE	PROJECT NO.
MISS.	BR-0060-03(021)

GENERAL NOTES:

Mississippi Standard Specifications for Road and Bridge Construction, 2017.

No change of plans will be permitted except by written approval of the Director of Structures, State Bridge Engineer. Minor changes in detail of design or construction procedure may be authorized by the Director of Structures, State Bridge Engineer provided such changes will not be cause for contract price adjustment.

The final surface texture of the bridge deck shall be mechanically transverse grooved in accordance with Sections 501 and 804 of the specifications. See Misc. Span Details for limits of transverse grooving on bridge deck.

Bridge concrete shall be class "AA" or class "BD" as indicated in plans.

Railing expansion joint material shall be bituminous fiber type unless otherwise noted.

No payment will be allowed for excavation incidental to the construction of end bents.

Bar bending details shall be in accordance with "Manual of Standard Practice for Detailing Reinforced Concrete Structures" (ACI 315R-94).

Reinforcement order lists and required placing plans shall be furnished in accordance with Section 805 of the Mississippi Standard Specifications. Partial submittals are not acceptable.

Shop drawings of prestressed beams, including an erection plan, shall be submitted in duplicate to the Director of Structures, State Bridge Engineer for approval prior to the manufacture of beams.

The Fabricator shall provide camber data at release and immediately prior to shipping.

The Contractor shall provide camber data after erection. The Contractor should be aware that the deflection diagram may be modified based on the provided camber data. Therefore, deck grades should be set only after notification from the Director of Structures, State Bridge Engineer.

Concrete surfaces shall receive a Class 2 rubbed or spray finish in accordance with the specifications.

Reinforcing steel shall be ASTM A615, Grade 60, unless otherwise noted.

Work for which no pay item is provided in the proposal will not be paid for directly and compensation therefor will be included in the prices and payments for bid items.

NOTE:

The girder deflection diagrams shown in these plans were prepared and intended for design and estimation purposes only. Actual bridge girder deflections may differ from the deflection diagrams shown in these plans.

It is the Contractor's responsibility to construct the bridge to meet the requirements of the plans and specifications including, but not limited to, the requirements for bridge deck smoothness.

Prior to formwork construction, the Contractor shall submit three (3) copies of a proposed bridge superstructure construction plan to the Director of Structures, State Bridge Engineer for review, through the Project Engineer. This submittal shall include all calculations, assumptions and parameters used by the Contractor to determine bridge girder deflections and form grade elevations. This submittal shall also include an erection and construction procedure that addresses the construction means and methodologies used by the Contractor and shall consider effects including, but not limited to, construction phasing, pouring schedules, applied permanent and construction loading, and shall include calculations and details of temporary girder bracing systems used to ensure girder stability and to counter the effects of girder tilt.

After girder erection and prior to deck construction, the Contractor shall submit deck thickness verification calculations for each girder. These calculations shall include a comparison of the erected girder top flange profiles versus the plan deck grade elevations over each girder plus the anticipated girder deflection due to applied permanent dead load and creep.

Three (3) copies of the deck thickness verification calculations and any proposed remediation measures to correct for thin deck areas shall be submitted to the Director of Structures, State Bridge Engineer for review, through the Project Engineer.

The bridge superstructure construction plan and the deck thickness verification calculations shall be prepared and stamped by a Mississippi Registered Professional Engineer.

STEEL PIPE PILE NOTES:

PDA test piles shall be driven with an approved impact hammer as an indicator test pile or production pile at the location shown in the PDA TEST PILE SCHEDULE and will be paid for as test piles only.

The first PDA test pile driven shall be an indicator PDA test pile as shown on the Foundation Plan. The indicator PDA test pile shall be driven continuously using an approved impact hammer. The full length of the indicator PDA test pile shall be monitored using PDA.

The PDA monitored Indicator Test Piles will be out-of-position piles driven with mandatory restrikes and PDA results analyzed prior to driving any PDA Test Piles. Based on the results of the PDA Indicator Test Piles, the plan lengths of the PDA Test Piles may change. Therefore, recommend ordering PDA Test Piles after analysis of the PDA Indicator Test Pile is complete.

Remaining test piles all be driven as a continuos operation, to the tip elevation shown in the PDA TEST PILE SCHEDULE, unless otherwise directed by the Director of Structures, State Bridge Engineer.

Permanent piles shall be driven to an elevation no higher than the elevation shown in the REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE.

The Director of Structures, State Bridge Engineer may authorize test piles driven outside the structural limits.

When feasible, bearing piles shall be driven full length and be spliced, only, as approved by the Director of Structures, State Bridge Engineer.

Welding shall be done by the ELECTRIC ARC process. Welders shall be certified and electrodes shall be approved.

When loading tests are required, the maximum test load shall be one and one half ($1\frac{1}{2}$) times the minimum pile bearing capacity.

PDA test piles shall require a 1 day and 7 day restrike unless otherwise directed by the Engineer.

Pile lengths and driving criteria shall be provided based on the results of the PDA test piles.

The required ultimate pile bearing shown in the REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE includes the LRFD resistance factor for PDA of 0.65.

Pile hammer leads used for all PDA test piles and PDA restrikes shall be large enough to provide a minimum of 3" clearance on each side of the pile in order to properly place and protect PDA gages.

Steel pipe piles shall be driven with a maximum rated energy no less than 70,000 ft-lbs, to the tip elevations specified unless the Contractor's drivability analysis utilizing the Contractor's selected alternative hammer is approved by the Director's of Structures, State Bridge Engineer.

All Steel Pipe Piles shall be ASTM A252, Grade 3 (Fy = 45,000 psi.).

Steel Pipe Piles are intended to be open ended.

Welding shall comply with ANSI/AWS D1.5 Bridge Welding Code and be performed by a certified welder.

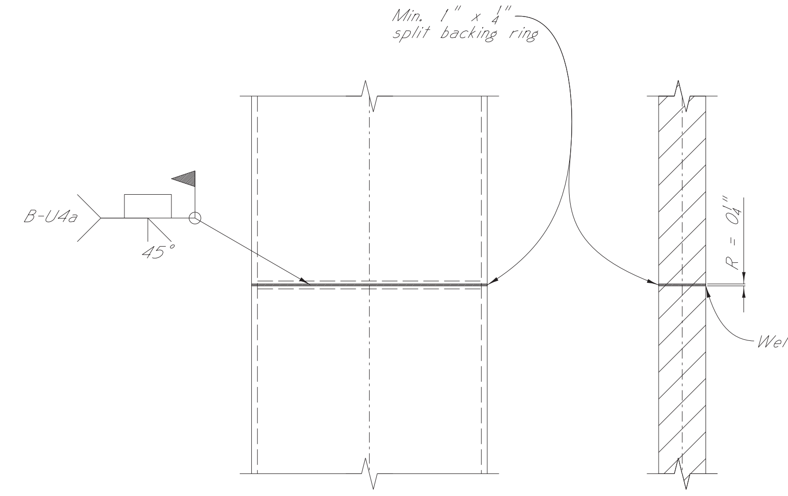
The tip elevation of piling, for hydraulic structures, may be determined by scour line but under no circumstances shall be greater than the minimum tip elevation shown in the REQUIRED ULTIMATE PILE BEARING CAPACITY.

Pipe piles shall receive a protective coating beginning at the bottom of the cap and extending to the 100 yr. scour elevation as shown on the Layout Sheet. The coating shall be one of the following, applied according to the manufacturer's specifications in two coats of 16mil minimum dry film thickness:

a) Bitumastic 300-M Coal Tar Epoxy
   manufactured by Carboline Company in St. Louis, MO
   www.carboline.com
b) Carotech Coal Tar Epoxy
   manufactured by INSL-X Company in Montvale, NJ
   www.carotechcoatings.com
c) Series 46-143 TNEMEC-Tar
   manufactured by TNEMEC Co Inc in Kansas City, MO
   www.tnemec.com

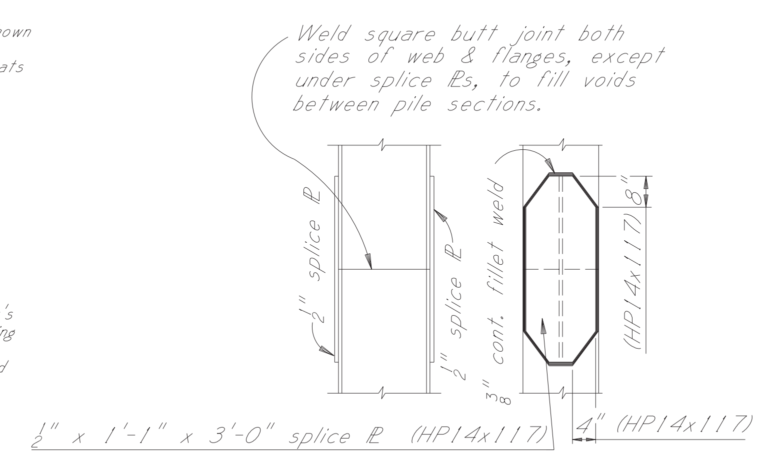
Any areas of coating above the ground line that become damaged during shipping or driving shall be repaired per the manufacturer's specifications. Any areas of coating affected by pipe pile splicing shall be repaired per the manufacture's specification. Protective coating, including surface preparation and application, will be paid for as Steel Pipe Piling, (not a separate pay item).

REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE							
Bent No.	Pile Type	Pile Size	Required Ultimate Bearing (Tons)	Min. Tip Elevation	Est. Length (ft.)	Controlling Limit State	LRFD Resistance Factor
1	Steel	HP14x117	122	403.2	70	Strength I	0.65
2	Steel pipe pile	30"	361	358.7	90	Strength I	0.65
3	Steel pipe pile	30"	361	358.7	90	Strength I	0.65
4	Steel pipe pile	30"	305	364.9	90	Strength I	0.65
5	Steel pipe pile	30"	345	371.0	90	Strength I	0.65
6	Steel pipe pile	30"	345	371.0	90	Strength I	0.65
7	Steel	HP14x117	128	404.0	70	Strength I	0.65



PIPE PILE SPLICING DETAIL

30" Diam. steel pipe piles



PILE SPLICE DETAIL

HP14x117 steel piles

PILE HAMMER REQUIREMENTS*			
Pile Type	Pile Size	Min Energy (ft.-lb.)	Max Energy (ft.-lb.)
Steel pipe pile	All	70,000	N/A

*NOTE: Based on preliminary drivability analysis

PDA TEST PILE SCHEDULE		
Bent No.	Min. Length (ft.)	Tip Elevation
1	80	349.6
2*	110	318.5
5	110	322.2
7	80	354.8

*Indicator pile to be driven within 3-5 pile diameters. Uncoated for the entire length.

PILE NOTES:

Test piles shall be driven as permanent piles at the location shown in the PDA TEST PILE SCHEDULE and will be paid for as test piles only.

The Director of Structures, State Bridge Engineer may authorize test piles driven outside the structural limits.

Test piles shall be driven as a continuous operation, to the bearing capacity and the tip elevations shown in the PDA TEST PILE SCHEDULE, unless otherwise directed by the Director of Structures, State Bridge Engineer.

Permanent piles shall be driven to an elevation no higher than the elevation shown in the REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE.

The tip elevation of piling, for hydraulic structures, may be determined by the scour line.

When feasible, bearing piles shall be driven full length and be spliced, only, as approved by the Director of Structures, State Bridge Engineer.

Welding shall be done by the ELECTRIC ARC process. Welders shall be certified and electrodes shall be approved.

When loading tests are required, the maximum test load shall be one and one half ($1\frac{1}{2}$) times the minimum pile bearing capacity.

PDA test piles shall require a 1 day and 7 day restrike unless otherwise directed by the Engineer.

Pile lengths and driving criteria shall be provided based on the results of the PDA test piles.

The required ultimate pile bearing shown in the REQUIRED ULTIMATE PILE BEARING AND TIP ELEVATION SCHEDULE includes the LRFD resistance factor for PDA of 0.65.

Pile hammer leads used for all PDA test piles and PDA restrikes shall be large enough to provide a minimum of 3" of clearance on each side of the pile in order to properly place and protect PDA gages.

Steel HP piles shall be driven with a maximum rated energy no less than 70,000 ft-lbs to the tip elevations specified unless the Contractor's drivability analysis utilizing the Contractor's selected alternative hammer is approved by the Director's of Structures, State Bridge Engineer.

NOTE: In lieu of splice plates, prefabricated splicers may be used. Prefabricated splicers shall be submitted for approval by the Director of Structures, State Bridge Engineer.

ESTIMATED QUANTITIES															
Item / Location	Trans. Grooving	Conventional Static Pile Load Test	HP14x117 Steel Piling	PDA Test HP Steel Pile	PDA Test Steel Pipe Pile	Pile Restrike	30" Diam. Steel Pipe Piling	Bridge Concrete Class "AA"	Bridge Concrete Class "BD"	95 Ft. Prest. Conc. Beam BT-54	110 Ft. Prest. Conc. Beam BT-54	135 Ft. Prest. Conc. Beam BT-72	Reinforcement	Concrete Railing, 32"	Loose Riprap (300#)	Geotextile Under Riprap
	S.Y.	Each	L.F.	Each	Each	Each	L.F.	C.Y.	C.Y.	L.F.	L.F.	L.F.	LbS.	L.F.	Ton	S.Y.
Spans	2222.22								707.82	1,891.67	545.83	673.75	155,042	1,250		
End bents			1960	2		1		70.70					12,804	4.33	992.0	1005.0
Int. bents		1			2	1	2,070	92.94					10,308			
Totals	2222.22	1	1960	2	2	2	2,070	163.63	707.82	1,891.67	545.83	673.75	178,154	1,254.33	992.0	1005.0

PLAN
BRIDGE DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

00:00AMPM DGNFILENAME

BY | REVISION | DATE

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

BRIDGE AT STA. 1077+37.88

SR-4 OVER CUFFAWA CREEK

ESTIMATED QUANTITIES & GENERAL NOTES

FMS: **102207 / 302000**

COUNTY: **Marshall**

PROJECT NUMBER: **BR-0060-03(021)**

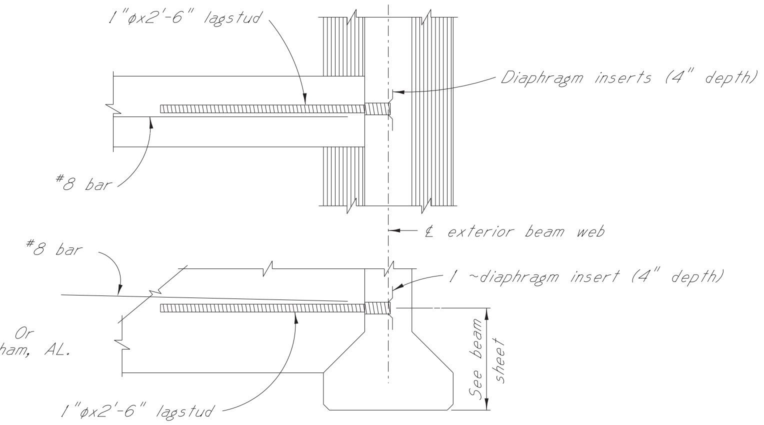
WORKING NUMBER 1 OF 30

DESIGNER Thomas Terry CHECKER Spencer Yates
DETAILER Thomas Terry ISSUE DATE 2017-07-12

DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

SHEET NUMBER 8003

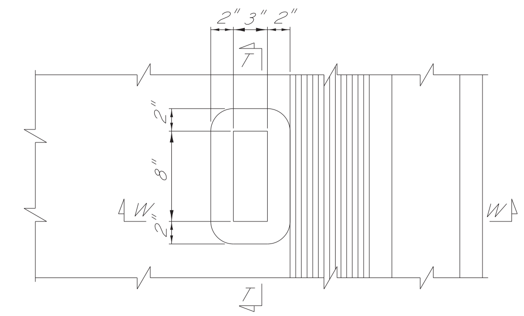
STATE	PROJECT NO.
MISS.	BR-0060-03(021)



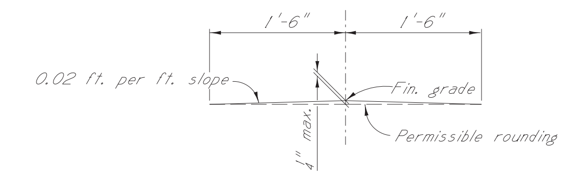
PART SECTION

NOTE: Continuous threaded lagstuds and diaphragm inserts shall be as manufactured by the Richmond Screw Anchor Co., Inc., Atlanta, GA; By Meadow Steel Products Co., Inc., Birmingham, AL Or Dayton Superior Co., Inc., Birmingham, AL.

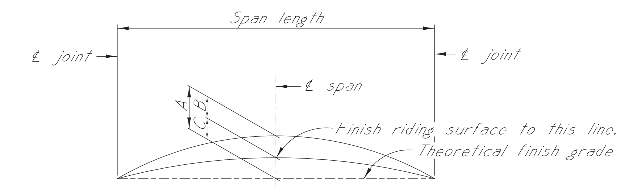
DIAPHRAGM INSERT AND LAGSTUD DETAILS



DETAIL "D"



CROWN DETAILS



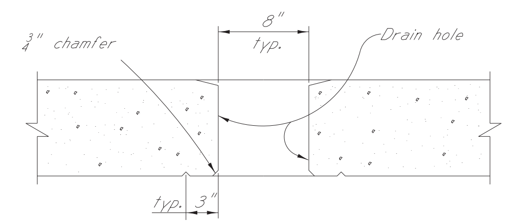
DEFLECTION DIAGRAM

A = total recommended allowance for deflection.
B = estimated deflection due to dead load of slab & rail.
C = A-B = net initial camber in riding surface, which includes an allowance or creep.

NOTE:
For values of A, B & C, see Beam Detail sheets.

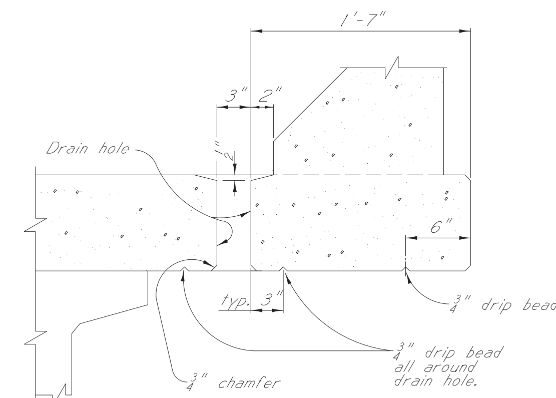
NOTE:
The Girder Deflection Diagrams shown in these plans were prepared and intended for design and estimation purposes only. Actual bridge girder deflections may differ from the deflection diagrams shown in these plans. It is the Contractor's responsibility to construct the bridge to meet the requirements of the plans and specifications including, but not limited to, the requirements for bridge deck smoothness. Prior to formwork construction, the Contractor shall submit three (3) copies of a proposed BRIDGE SUPERSTRUCTURE CONSTRUCTION PLAN to the Director of Structures, State Bridge Engineer for review, through The Project Engineer. This submittal shall include all calculations, assumptions and parameters used by the Contractor to determine bridge girder deflections and form grade elevations. This submittal shall also include an erection and construction procedure that addresses the construction means and methodologies used by the Contractor and shall consider effects including, but not limited to, construction phasing, pouring schedules, applied permanent and construction loading, and shall include calculations and details of temporary girder bracing systems used to ensure girder stability and to counter the effects of girder tilt. After girder erection and prior to deck construction, the Contractor shall submit deck thickness verification calculations for each girder. These calculations shall include a comparison of the erected girder top flange profiles versus the plan deck grade elevations over each girder plus the anticipated girder deflection due to applied permanent dead load and creep. Three (3) copies of the deck thickness verification calculations and any proposed remediation measures to correct for thin deck areas shall be submitted to the Director of Structures, State Bridge Engineer for review, through the Project Engineer. The BRIDGE SUPERSTRUCTURE CONSTRUCTION PLAN and the deck thickness verification calculations shall be prepared and stamped by a Mississippi Registered Professional Engineer.

GENERAL NOTES:
All concrete in span shall be class "BD".
All concrete in railing shall be class "AA".
Chamfer all edges ¾", unless otherwise noted.
See Layout sheet for finishing of concrete surfaces.
Placing dimensions for reinforcing steel to concrete surfaces are clear distances.
To determine the dimension from finish grade to cap, the assumption is made that the compressed thickness of the neoprene pad is as shown in table, and that the original camber of the beams will be within the limits shown on the Beam Detail sheets. The Director of Structures, State Bridge Engineer shall be notified if the cambers are not within these limits.



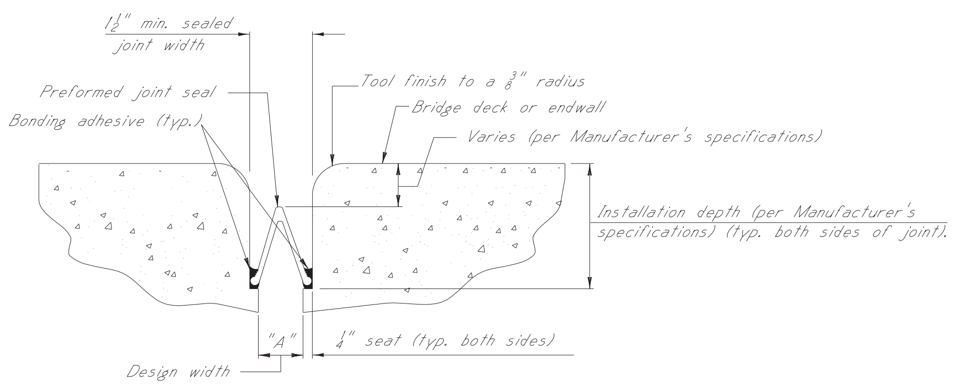
SECTION T-T

NOTE: Drain holes shall be located so that bars B & C will not be cut.

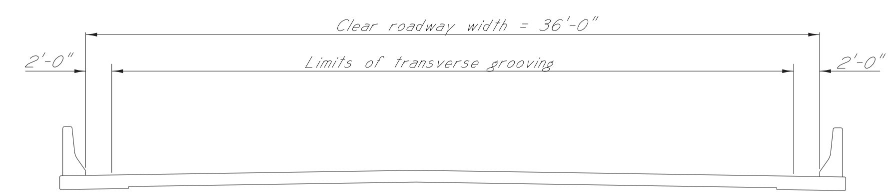


SECTION W-W

DRAIN HOLE DETAILS
Use where shown on the Span Detail sheet.



TYPICAL SECTION PREFORMED JOINT SEAL



LIMITS OF TRANSVERSE GROOVING

NOTES:

1. Joint installation and sealing on newly constructed bridge decks shall not be paid for directly and shall be considered an absorbed item of work.

2. The preformed joint seal shall be one of the following, installed according to the Manufacturer's specifications:

   A. Silicoflex Joint Sealing System, manufactured by R.J. Watson, Inc www.rjwatson.com

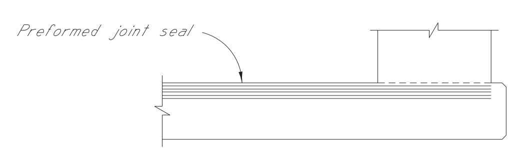
   B. Wabo SPS Joint System manufactured by Watson Bowman Acme Corporation www.wbacorp.com

   C. Silspec SSS Silicone Strip Seal manufactured by SSI Commercial & Highway Construction Materials www.ssicm.com

3. For estimating purposes, The RJ Watson Silicoflex Joint Sealing System was selected. However, should another supplier be chosen, it is the Contractor's responsibility to ensure that the Manufacturer's recommendations are followed for joint preparation, installation depths and widths, adhesive setting times, and any other variances between the specifications provided by the Manufacturers. A Manufacturer representative shall be present at the time joint sealing begins to ensure that the Contractor is properly schooled in installation of the joint material. All open joints shall be sealed at their design widths, dimension "A", as indicated on the end bent and span details.

4. Dimension "A" is defined as the design width of the joint opening, which does not account for the ¼" seat required on both sides of the joint. Preformed Joint Seal, Type I, shall be used for design widths less than 2". Preformed Joint Seal, Type II, shall be used for design widths greater than or equal to 2", with the maximum design width being 2½". In cases where design widths are greater than 2½", another type of expansion material shall be required as directed by the Director of Structures, State Bridge Engineer.

5. Joints in newly constructed bridge decks shall be protected from damage until accepted for maintenance by the State. Damaged joints shall be repaired at no additional cost to the State.



ELEVATION AT END OF SPAN

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 1077+37.88
SR-4 OVER CUFFAWA CREEK
MISCELLANEOUS SPAN DETAILS

FMS: 102207 / 302000
COUNTY: Marshall
PROJECT NUMBER: BR-0060-03(021)

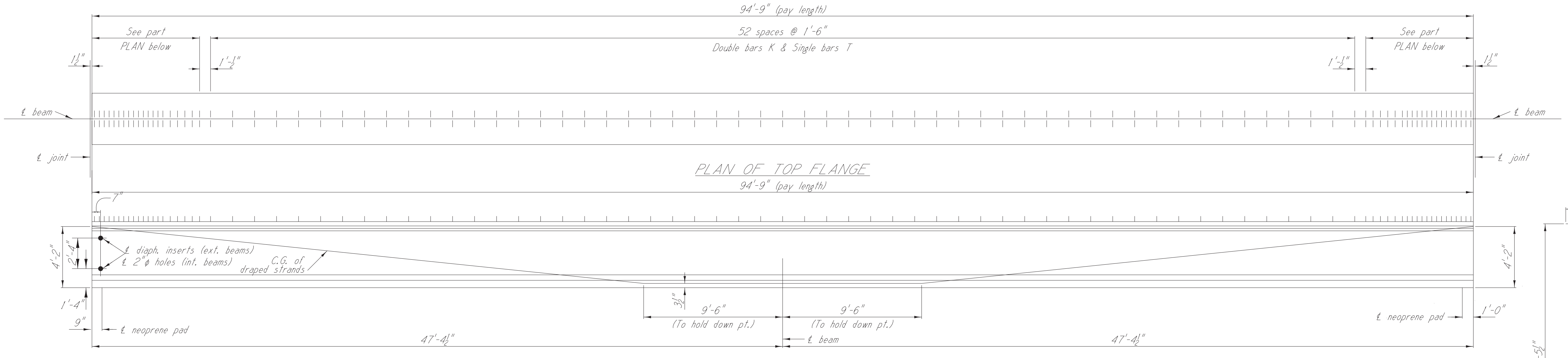
WORKING NUMBER 19 OF 30
SHEET NUMBER 8021

BY | REVISION | DATE

DESIGNER Thomas Terry CHECKER Spencer Yates
DETAILER Thomas Terry ISSUE DATE 2017-07-12
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

PLAN BRIDGE DIVISION MISSISSIPPI DEPARTMENT OF TRANSPORTATION

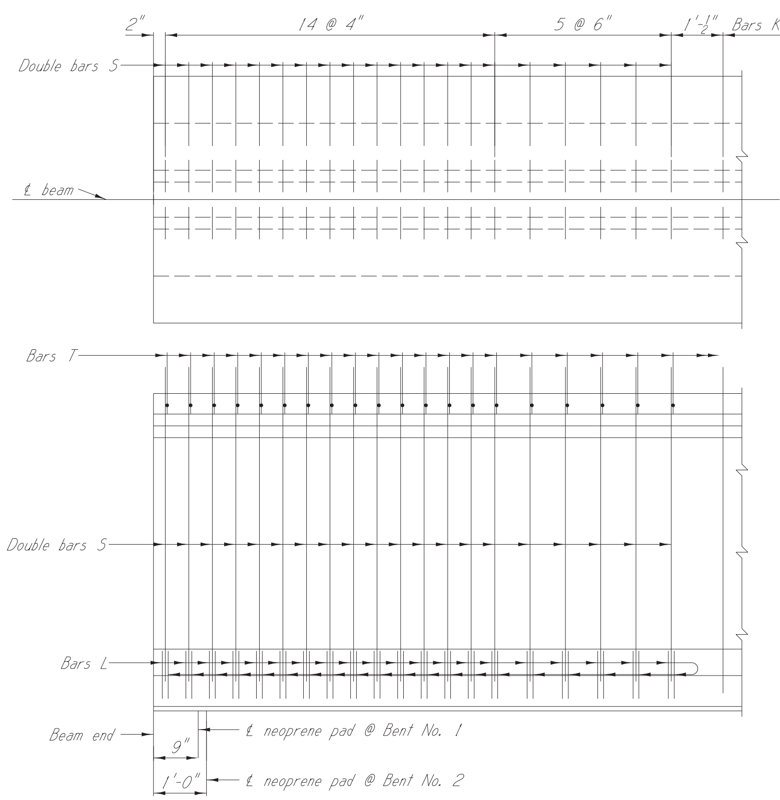
STATE	PROJECT NO.
MISS.	BR-0060-03(021)



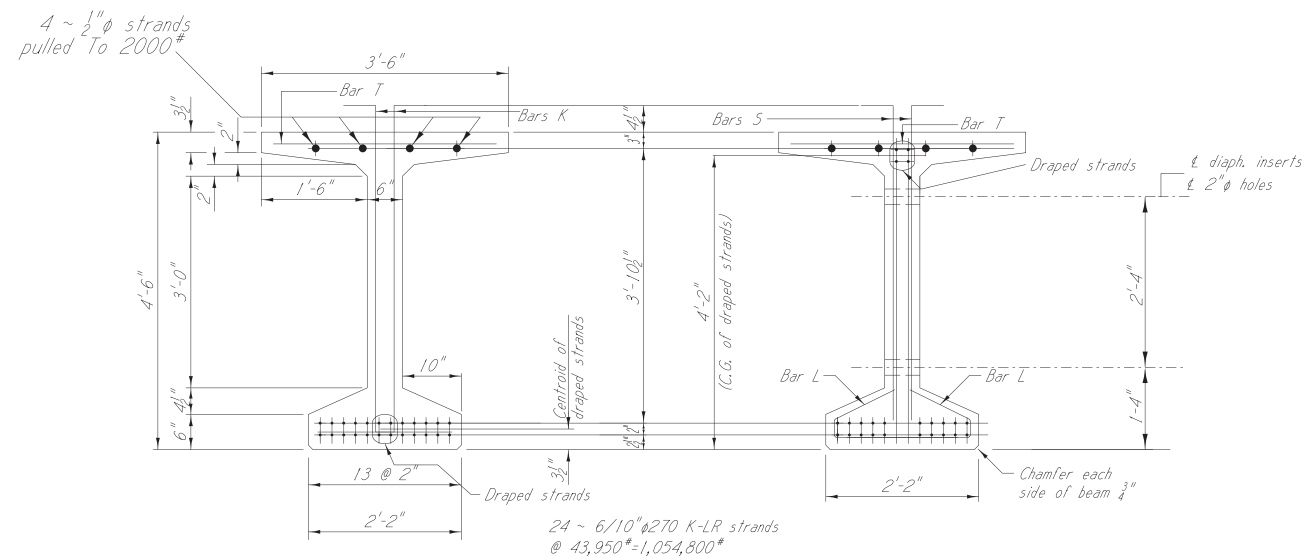
PLAN OF TOP FLANGE

ELEVATION

NOTE:
Cut strands flush and weatherproof with limestone colored "Sonolastic" (Sonneborn Building Products), "GC-9 Synthacalk" (Pecora Corp.), or approved equal, meeting the requirements of Federal Specification No. TT-S-00227E Or TT-S-00230C, applied according to Manufacturer's directions.

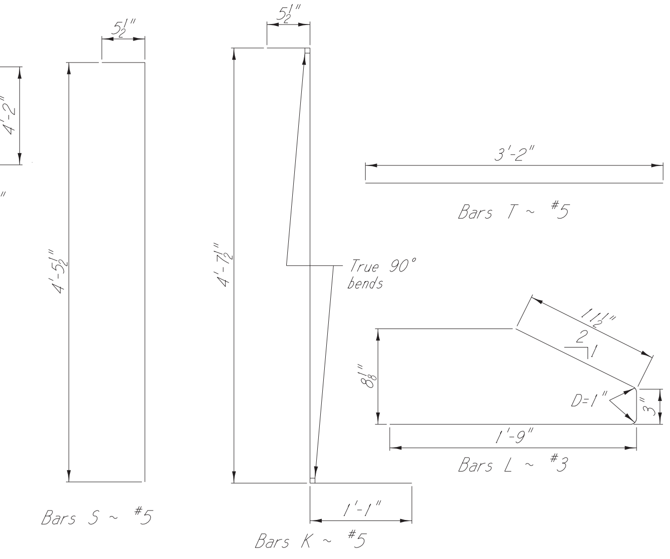


PART ELEVATION
Strands not shown for clarity



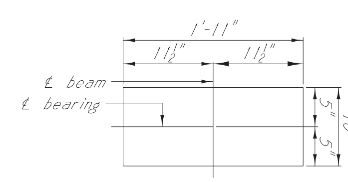
SECTION NEAR ℄ SPAN

END ELEVATION



BAR BENDING DETAILS
Dimensions are out to out

NOTE:
For beam general notes, beam bevels, sloped plates, bearing pad details, other beam details see sheets no. 8028,8029.



PLAN

ELEVATION

NEOPRENE PAD DETAILS
In no case shall neoprene pads be field cut. Bearing area on top of cap shall be cast smooth and true to grade.

GENERAL NOTES:

Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2017.
The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.
Strand pattern detailed is for 6/10"φ270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.
The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.
The Fabricator shall provide camber data at release and immediately prior to shipping.
Concrete shall be class "FX" and:
(a) shall have a 28-day cylinder strength of 6500 p.s.i.
(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.
At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 6500 p.s.i. concrete.
If any cylinder tests below 6500 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

DESIGN DATA

Unit stresses are in accordance with A.A.S.H.T.O., 2014 with 2016 interims.
Stay-In-Place Metal Deck Forms............18 lb/ft (BTW Flanges)

LR indicates low-relaxation strands

PRESTRESS REQUIREMENTS

For deflection diagram, see Miscellaneous Span Details per sheet no. 8021

Strand type	Minimum breaking strength lbs/strand	Initial tension lbs/strand	Required number and location of strands: Total number strands	Straight strands: Number strands	Straight strands: Centroid (in.)	Draped strands: Number strands	Draped strands: Centroid (in.) ℄ span	Draped strands: Centroid (in.) Beam end	Centroid for total number of strands (in.): At ℄ span	Centroid for total number of strands (in.): At beam end	Distance from ℄ span to hold-down point	Camber limits	Deflection diagram A	Deflection diagram B	Deflection diagram C	Minimum concrete strength at time of release (psi)
6/10"φ270 K-LR	58,600	43,950	24	20	3.50	4	3.50	50.00	3.50	11.25	9'-6"	0 to 2 5/16"	1 3/16"	1 3/16"	0"	5400

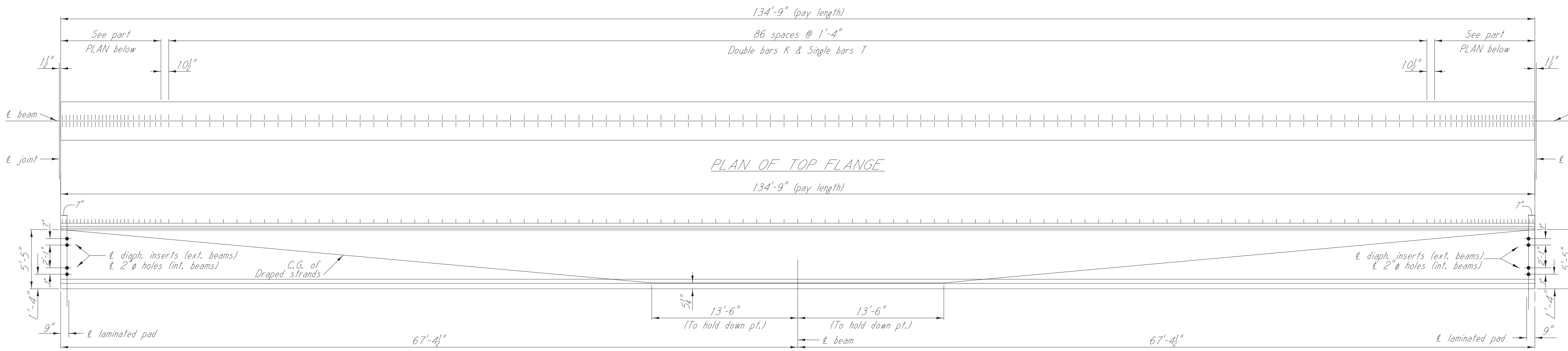
MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 1077+37.88
95 FT BEAM DETAILS
BEAM NO. 95-1 (BT-54)

FMS: 102207 / 302000
COUNTY: Marshall
PROJECT NUMBER: BR-0060-03(021)

WORKING NUMBER 20 OF 30
SHEET NUMBER 8022

DESIGNER Thomas Terry CHECKER Spener Yates
DETAILER Thomas Terry ISSUE DATE 2017-07-12
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

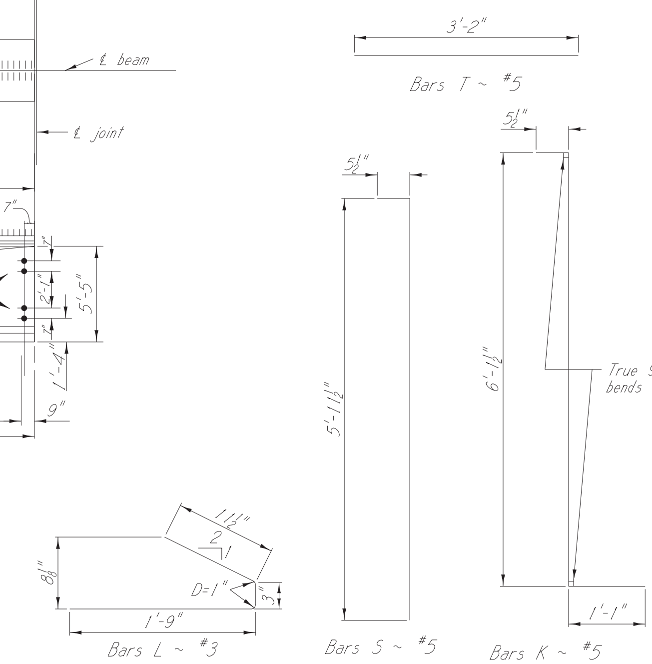
STATE	PROJECT NO.
MISS.	BR-0060-03(021)



PLAN OF TOP FLANGE

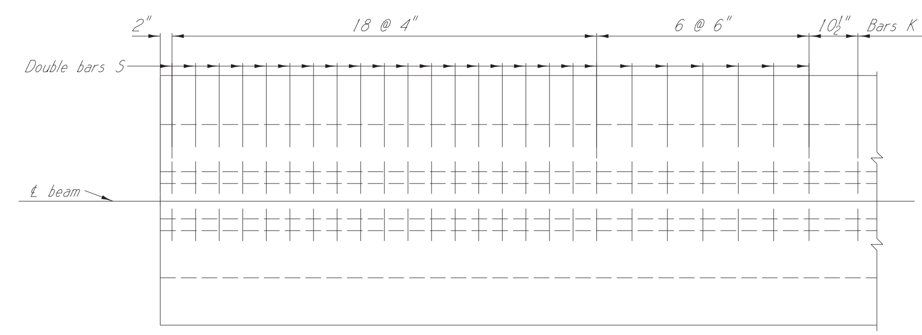
ELEVATION

NOTE: Cut strands flush and weatherproof with limestone colored "Sonolastic" (Sonneborn Building Products), "GC-9 Synthacalk" (Pecora Corp.), or approved equal, meeting the requirements of Federal Specification No. TT-S-00227E Or TT-S-00230C, applied according to Manufacturer's directions.

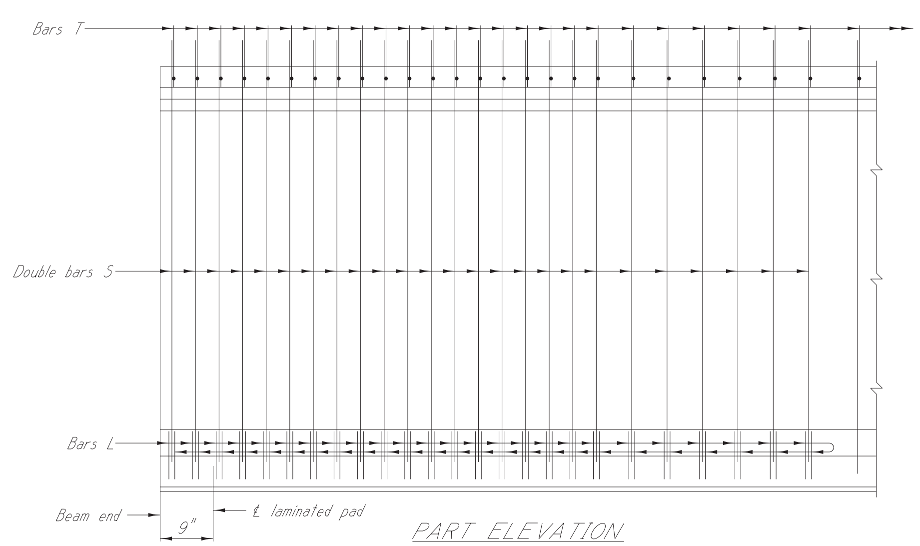


BAR BENDING DETAILS

Dimensions are out to out

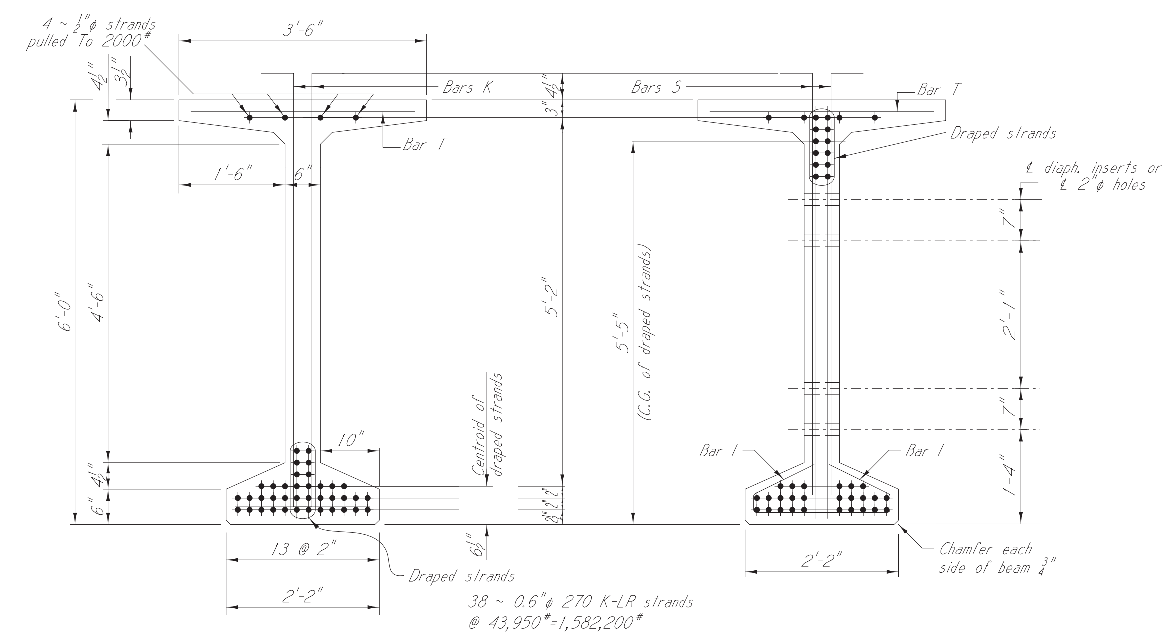


PART PLAN



PART ELEVATION

Strands not shown for clarity



SECTION NEAR ℄ SPAN

END ELEVATION

GENERAL NOTES:

Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2017.

The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.

Strand pattern detailed is for 0.6"ϕ 270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.

The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.

The Fabricator shall provide camber data at release and immediately prior to shipping.

Concrete shall be class "FX" and:

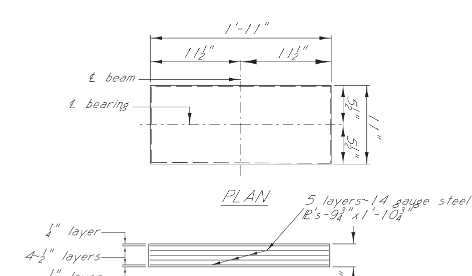
(a) shall have a 28-day cylinder strength of 7800 p.s.i.

(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.

At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 7800 p.s.i. concrete.

If any cylinder tests below 7800 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

NOTE:
For beam general notes, beam bevels, sloped plates, bearing pad details, other beam details see sheets no. 8028,8029.



LAMINATED PAD DETAILS

Testing acceptance procedure shall be in accordance with section 714.10.6 of the Specifications.
Elastomer shall have a hardness of 50 durometer with a minimum shear modulus at 73°F of .085 k.s.i. And A maximum shear modulus at 73°F of .100 k.s.i.
Bearing area on top of cap shall be cast smooth and true to grade.

DESIGN DATA

Unit stresses are in accordance with L.R.F.D. 2014 with 2016 interims.
Stay-in-place Metal Forms . . . . 18lbs/ft (between flanges)

PRESTRESS REQUIREMENTS

LR indicates low-relaxation strands

For deflection diagram, see Miscellaneous Span Details per sheet no 8021.

Strand type	Minimum breaking strength	Initial tension	Required number and location of strands						Centroid for total number of strands (in.)		Distance from ℄ span to hold-down point	Camber limits	Deflection diagram			Minimum concrete strength at time of release (psi)
			Total number strands	Straight strands		Draped strands										
				Number strands	Centroid (in.)	Number strands	Centroid (in.)									
	lbs/strand	lbs/strand					℄ span	Beam end	At ℄ span	At beam end			A	B	C	
6/10"ϕ 270 K-LR	58,600	43,950	38	26	4.19	12	7.5	63.5	5.24	22.92	12'-9"	0 to 3 9/16"	2 5/16"	2 5/16"	0"	6500

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

BRIDGE AT STA. 1077+37.88

135 FT. BEAM DETAILS
BEAM NO. 135-1 (BT-72)

FMS: 102207 / 302000
COUNTY: Marshall
PROJECT NUMBER: BR-0060-03(021)

WORKING NUMBER 21 OF 30

SHEET NUMBER 8023

DESIGNER Thomas Terry CHECKER Spencer Yates
DETAILER Thomas Terry ISSUE DATE 2017-07-12

DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

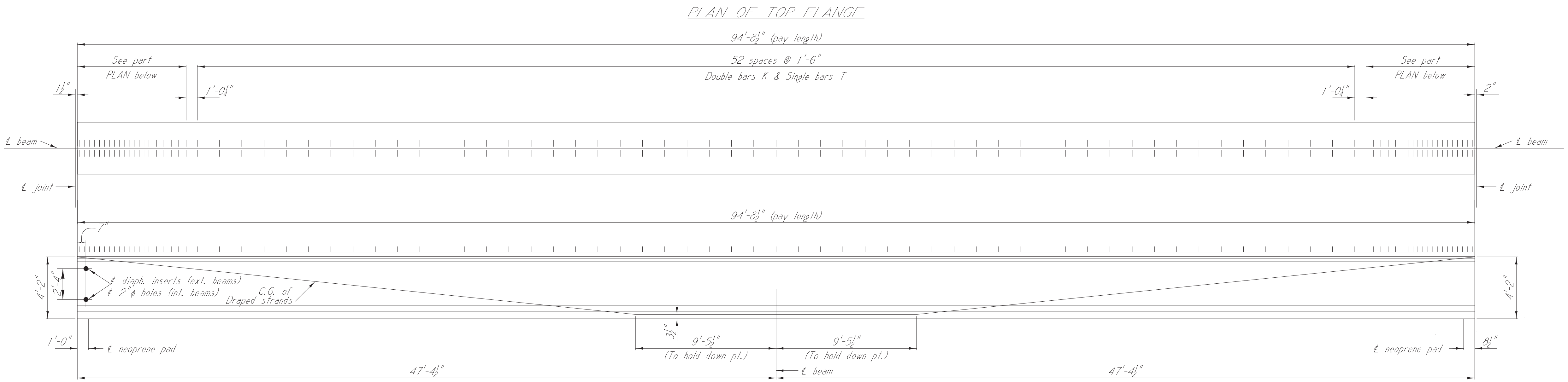
BY | REVISION | DATE

PLAN
BRIDGE DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

00:00AMPM DGNFILENAME

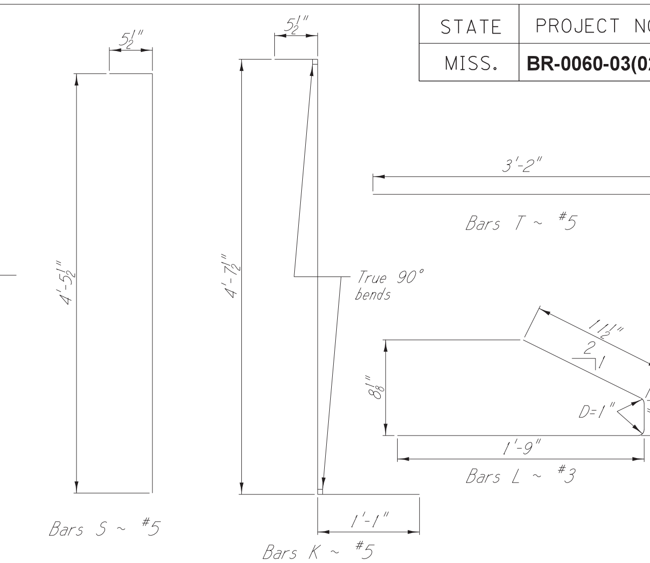
STATE	PROJECT NO.
MISS.	BR-0060-03(021)

PLAN OF TOP FLANGE



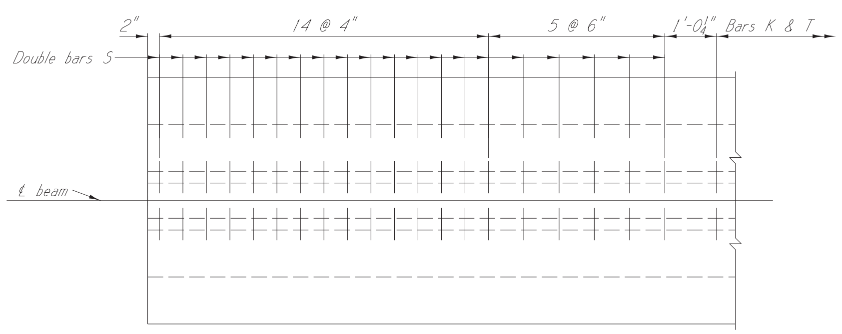
ELEVATION

NOTE: Cut strands flush and weatherproof with limestone colored "Sonolastic" (Sonneborn Building Products), "GC-9 Synthacalk" (Pecora Corp.), or approved equal, meeting the requirements of Federal Specification No. TT-S-00227E Or TT-S-00230C, applied according to Manufacturer's directions.

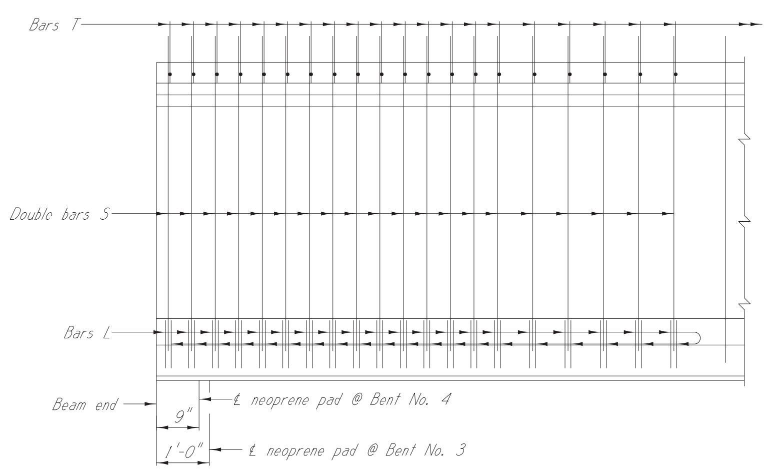


BAR BENDING DETAILS

Dimensions are out to out



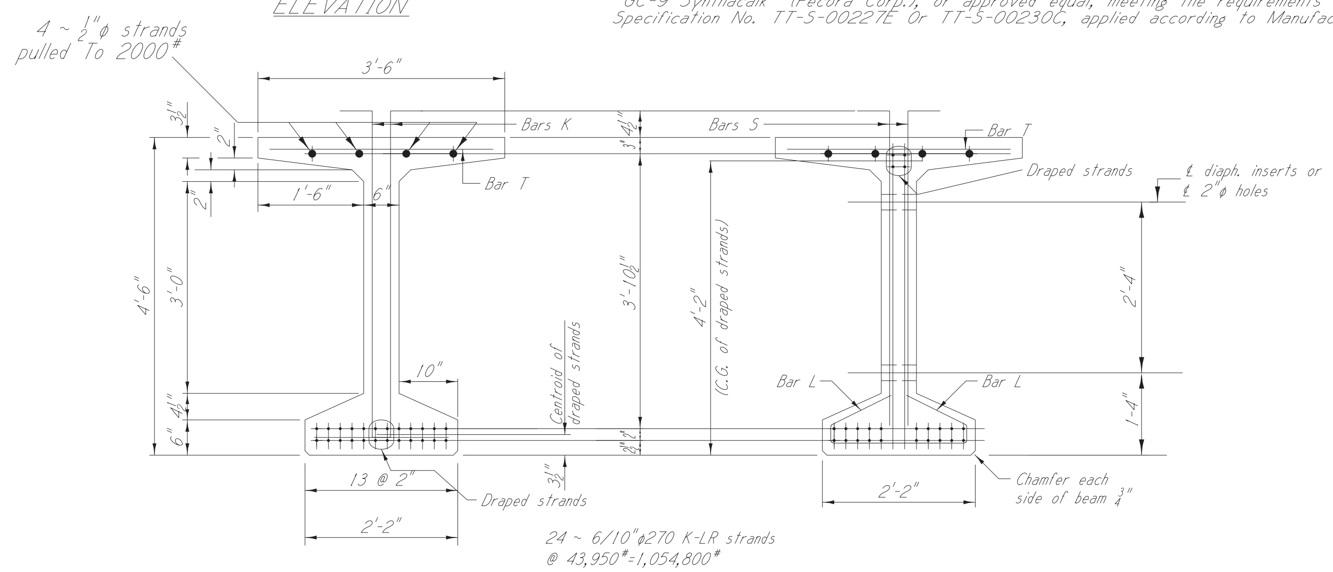
PART PLAN



PART ELEVATION

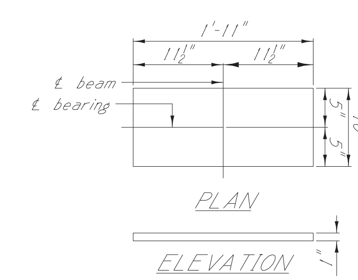
Strands not shown for clarity

NOTE:
For beam general notes, beam bevels, sloped plates, bearing pad details, other beam details see sheets no. 8028,8029.



SECTION NEAR ℄ SPAN

END ELEVATION



PLAN

ELEVATION

NEOPRENE PAD DETAILS

In no case shall neoprene pads be field cut. Bearing area on top of cap shall be cast smooth and true to grade.

GENERAL NOTES:

Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2017.

The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.

Strand pattern detailed is for 6/10"ϕ270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.

The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.

The Fabricator shall provide camber data at release and immediately prior to shipping.

Concrete shall be class "FX" and:
(a) shall have a 28-day cylinder strength of 6500 p.s.i.
(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.

At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 6500 p.s.i. concrete.

If any cylinder tests below 6500 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

DESIGN DATA

Unit stresses are in accordance with A.A.S.H.T.O., 2014 with 2016 interims.
Stay-In-Place Metal Deck Forms............18 lb/ft (BTW Flanges)

PRESTRESS REQUIREMENTS

LR indicates low-relaxation strands

For deflection diagram, see Miscellaneous Span Details per sheet no. 8021

Strand type	Minimum breaking strength	Initial tension	Required number and location of strands						Centroid for total number of strands (in.)		Distance from ℄ span to hold-down point	Camber limits	Deflection diagram			Minimum concrete strength at time of release (psi)
			Total number strands	Straight strands		Draped strands										
				Number strands	Centroid (in.)	Number strands	Centroid (in.)									
	lbs/strand	lbs/strand					℄ span	Beam end	At ℄ span	At beam end			A	B	C	
6/10"ϕ270 K-LR	58,600	43,950	24	20	3.50	4	3.50	50.00	3.50	11.25	9'-5½"	0 to 2 5/16"	1 3/16"	1 3/16"	0"	5400

MISSISSIPPI DEPARTMENT OF TRANSPORTATION

BRIDGE AT STA. 1077+37.88

95 FT BEAM DETAILS
BEAM NO. 95-2 (BT-54)

FMS: **102207 / 302000**
COUNTY: **Marshall**
PROJECT NUMBER: **BR-0060-03(021)**

BY	REVISION	DATE

DESIGNER Thomas Terry
DETAILER Thomas Terry
CHECKER Spencer Yates
ISSUE DATE 07-12-2017

DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

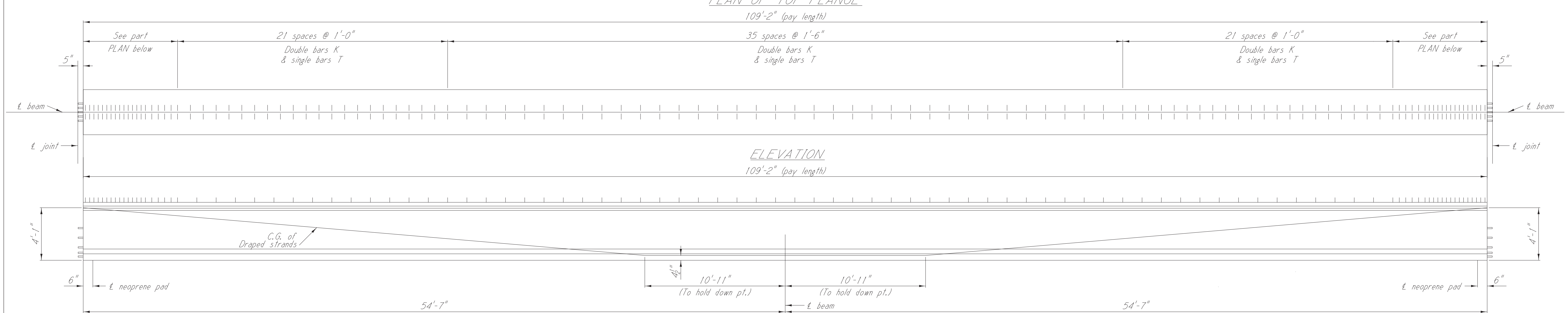
WORKING NUMBER 22 OF 30

SHEET NUMBER 8024

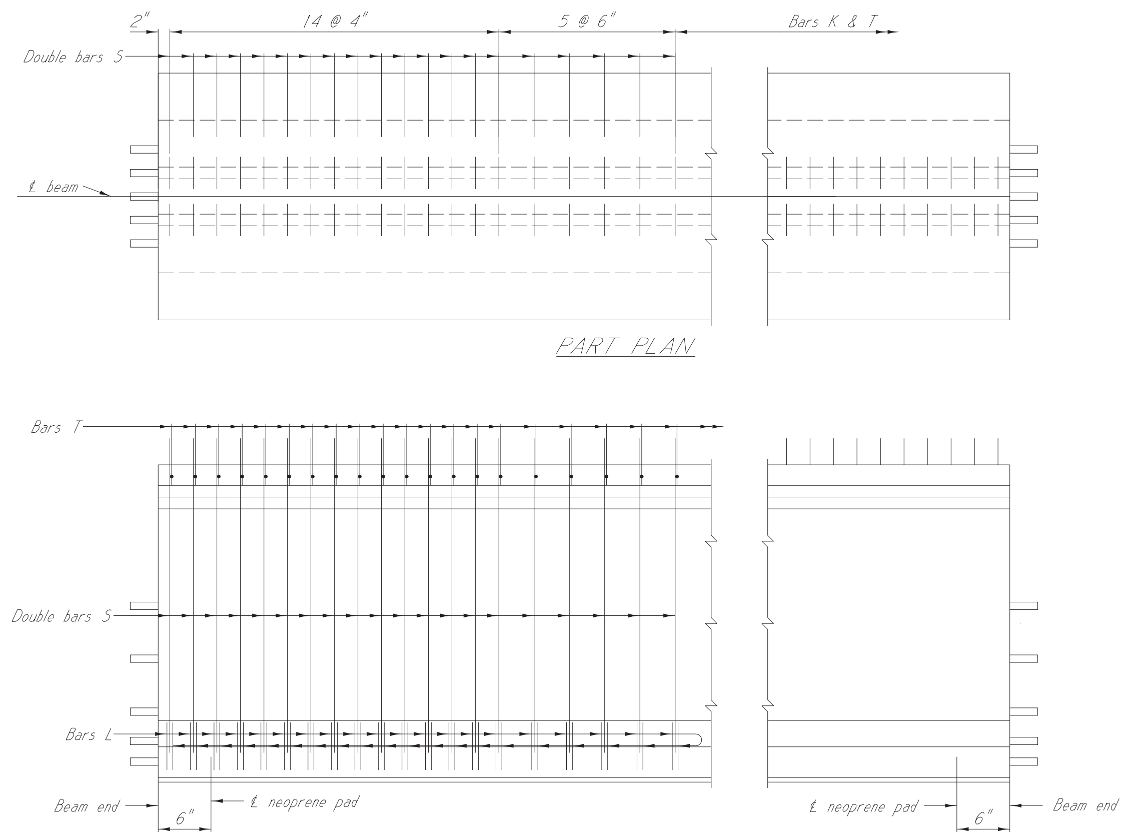
PLAN
BRIDGE DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

00:00 AMPM DGNFILENAME

PLAN OF TOP FLANGE



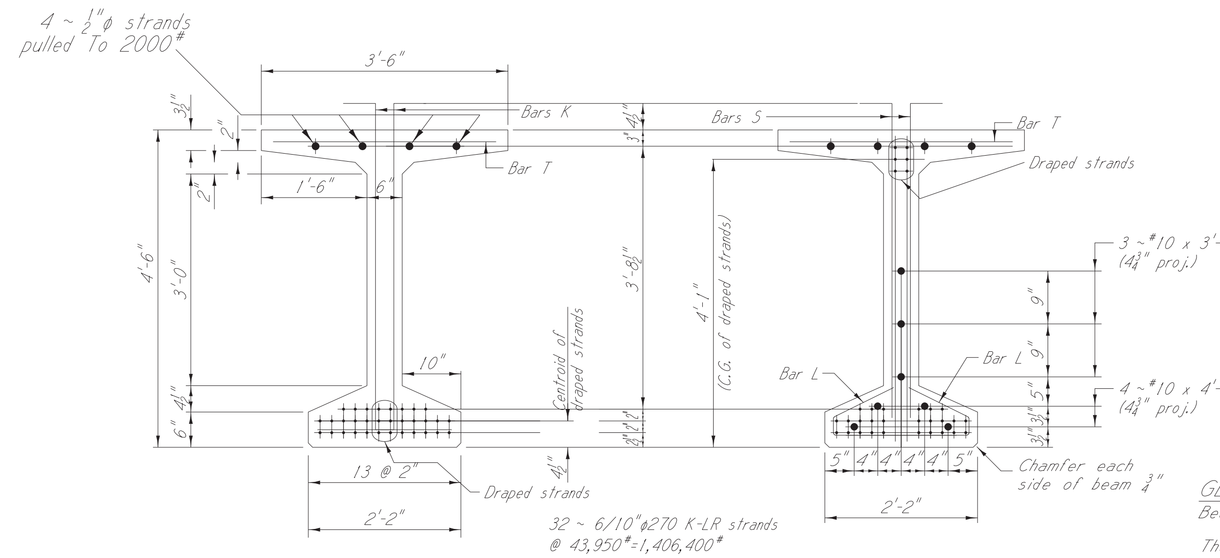
NOTE: Cut strands flush-no coating required (both ends).



PART PLAN

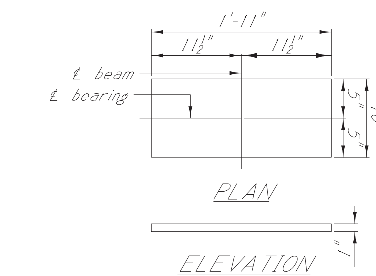
PART ELEVATION

Strands not shown for clarity



SECTION NEAR ℄ SPAN

END ELEVATION



NEOPRENE PAD DETAILS

In no case shall neoprene pads be field cut. Bearing area on top of cap shall be cast smooth and true to grade.

GENERAL NOTES:

Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2017.

The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.

Strand pattern detailed is for 6/10"φ270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.

The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.

The Fabricator shall provide camber data at release and immediately prior to shipping.

Concrete shall be class "FX" and:

(a) shall have a 28-day cylinder strength of 7700 p.s.i.

(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.

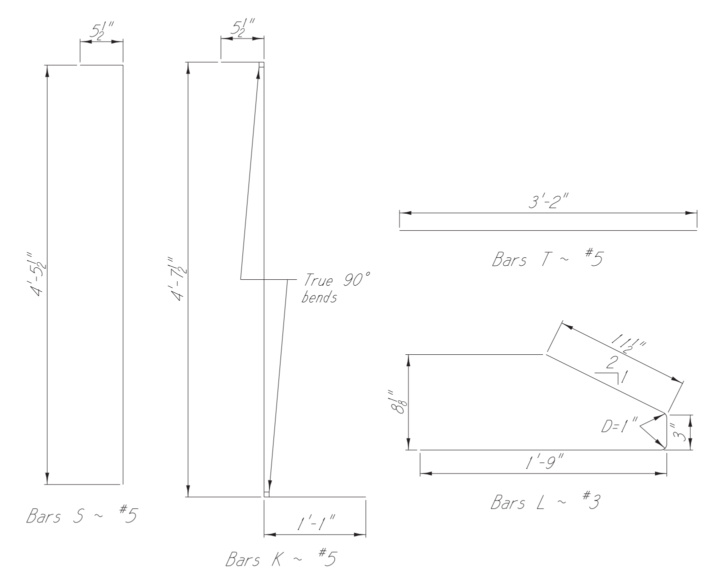
At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 7700 p.s.i. concrete.

If any cylinder tests below 7700 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

DESIGN DATA

Unit stresses are in accordance with A.A.S.H.T.O., 2014 with 2016 interims.

Stay-In-Place Metal Deck Forms......................18 ft/lb (between flanges)



BAR BENDING DETAILS

Dimensions are out to out

NOTE:
For beam general notes, beam bevels, sloped plates, bearing pad details, other beam details see sheets no. 8028, 8029.

LR indicates low-relaxation strands

PRESTRESS REQUIREMENTS

For deflection diagram, see Miscellaneous Span Details per sheet no.8021

Strand type	Minimum breaking strength (lbs/strand)	Initial tension (lbs/strand)	Required number and location of strands: Total number strands	Straight strands: Number strands	Straight strands: Centroid (in.)	Draped strands: Number strands	Draped strands: Centroid (in.) ℄ span	Draped strands: Centroid (in.) Beam end	Centroid for total number of strands (in.): At ℄ span	Centroid for total number of strands (in.): At beam end	Distance from ℄ span to hold-down point	Camber limits	Deflection diagram A	Deflection diagram B	Deflection diagram C	Minimum concrete strength at time of release (psi)
6/10"φ270 K-LR	58,600	43,950	32	26	4.19	6	4.50	49.00	4.25	12.59	10'-11"	0 to 3½"	2"	2"	0"	6300

	BY	MISSISSIPPI DEPARTMENT OF TRANSPORTATION
	REVISION	BRIDGE AT STA. 1077+37.88 110 FT BEAM DETAILS BEAM NO. 110-1 (BT-54)
		FMS: 102207 / 302000 COUNTY: Marshall PROJECT NUMBER: BR-0060-03(021)
	DATE	DESIGNER Thomas Terry CHECKER Spencer Yates DETAILER Thomas Terry ISSUE DATE 2016-07-12 DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E. DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

WORKING NUMBER 24 OF 30

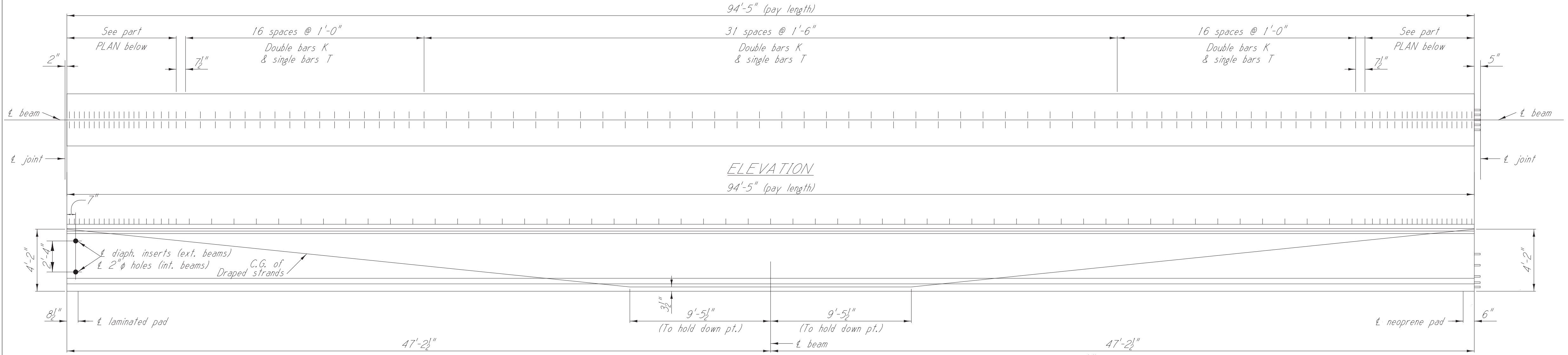
SHEET NUMBER 8026

PLAN
BRIDGE DIVISION
MISSISSIPPI DEPARTMENT OF TRANSPORTATION

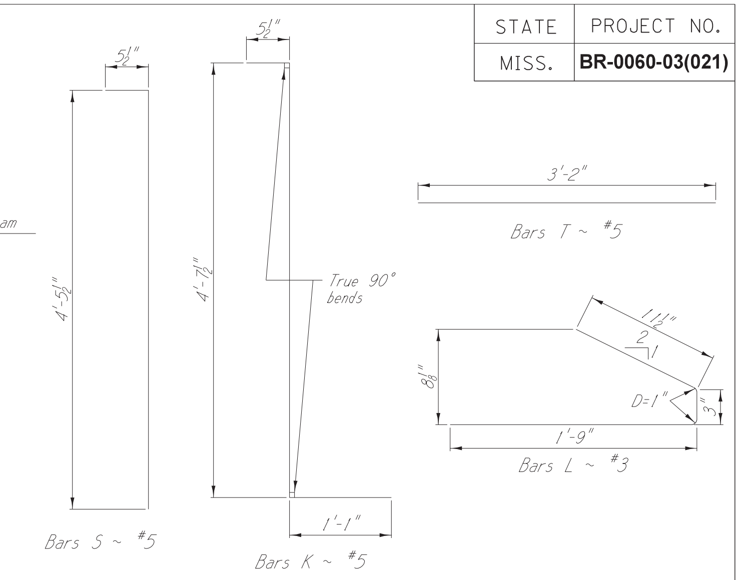
00:00AMPM DGNFILENAME

PLAN OF TOP FLANGE

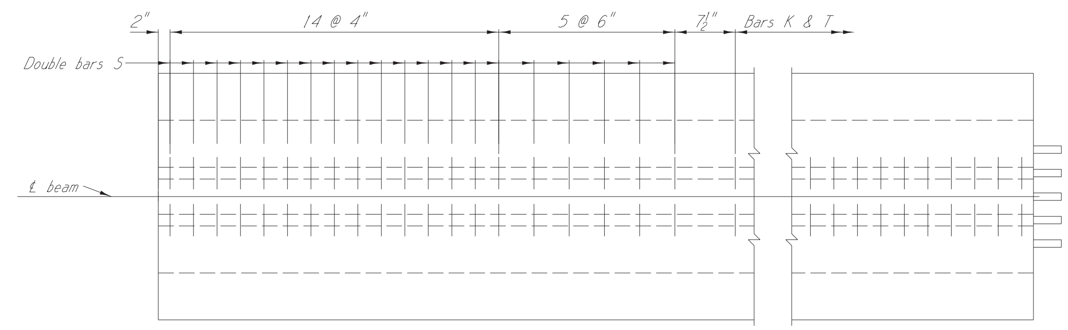
ELEVATION



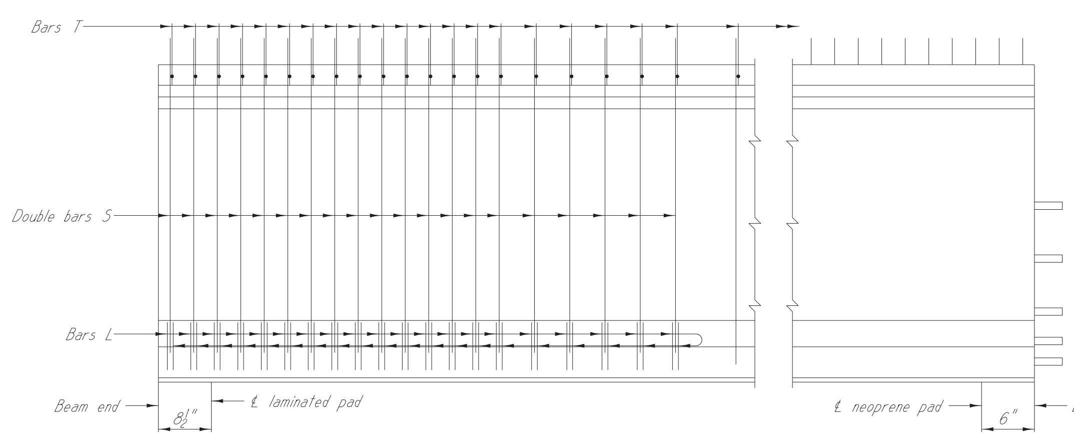
NOTE: For beam end with #10 bars projecting(Rt. End), cut strands flush-no coating required. For other beam end(Lt. End), cut strands flush and weatherproof with limestone colored "Sonolastic" (Sonneborn Building Products), "GC-9 Synthacalk" (Pecora Corp.), or approved equal, meeting the requirements of Federal Specification No. TT-S-00227E Or TT-S-00230C, applied according to Manufacturer's directions.



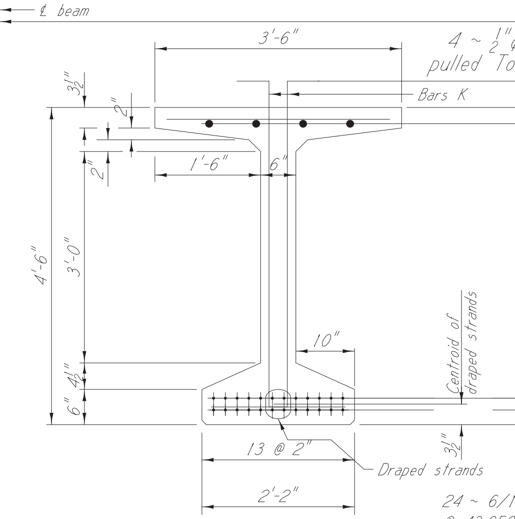
BAR BENDING DETAILS
Dimensions are out to out



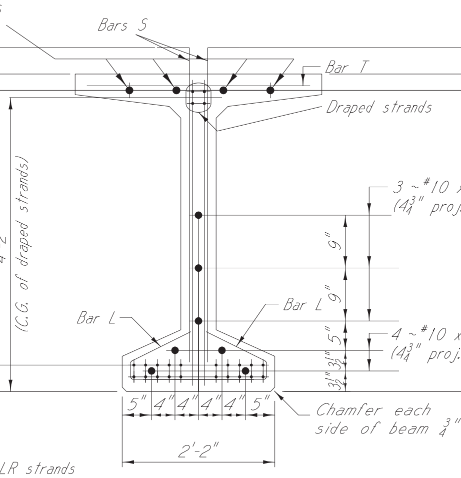
PART PLAN



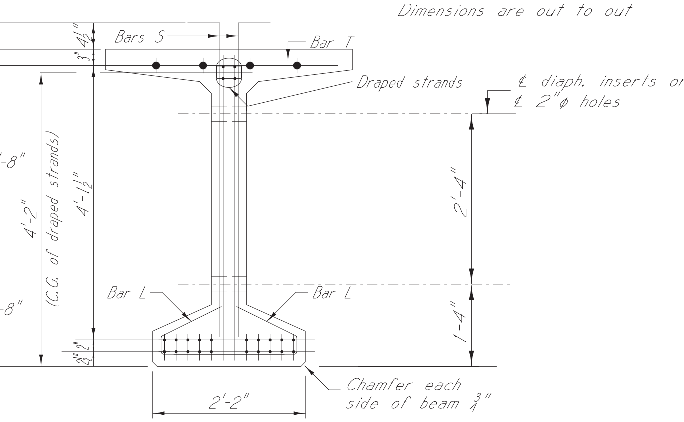
PART ELEVATION
Strands not shown for clarity



SECTION NEAR ℄ SPAN



END ELEVATION
Showing elevation at continuous end



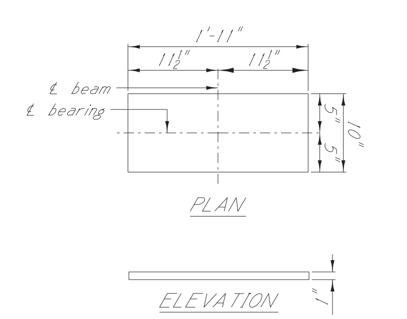
END ELEVATION
Showing elevation at simple end

GENERAL NOTES:

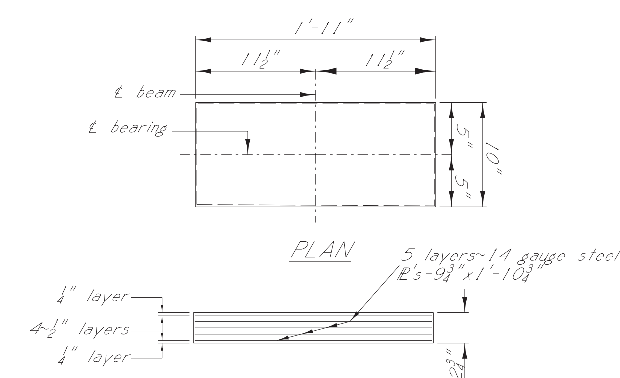
Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2017.
The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.
Strand pattern detailed is for 6/10"ϕ270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.
The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.
The Fabricator shall provide camber data at release and immediately prior to shipping.
Concrete shall be class "FX" and:
(a) shall have a 28-day cylinder strength of 6000 p.s.i.
(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.
At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 6000 p.s.i. concrete.
If any cylinder tests below 6000 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

DESIGN DATA

Unit stresses are in accordance with A.A.S.H.T.O., 2014 with 2016 interims.
Stay-In-Place Metal Deck Forms................18 ft/lb (between flanges)



NEOPRENE PAD DETAILS
In no case shall neoprene pads be field cut. Bearing area on top of cap shall be cast smooth and true to grade.



LAMINATED PAD DETAILS
Testing acceptance procedure shall be in accordance with section 714.10.6 of the Specifications.
Elastomer shall have a hardness of 50 durometer with a minimum shear modulus at 73°F of .085 k.s.i. And A maximum shear modulus at 73°F of .100 k.s.i.
Bearing area on top of cap shall be cast smooth and true to grade.

NOTE:
For beam general notes, beam bevels, sloped plates, bearing pad details, other beam details see sheets no. 8028,8029.

LR indicates low-relaxation strands

PRESTRESS REQUIREMENTS

For deflection diagram, see Miscellaneous Span Details per sheet no. 8021.

Strand type	Minimum breaking strength lbs/strand	Initial tension lbs/strand	Required number and location of strands: Total number strands	Straight strands: Number strands	Straight strands: Centroid (in.)	Draped strands: Number strands	Draped strands: Centroid (in.) ℄ span	Draped strands: Centroid (in.) Beam end	Centroid for total number of strands (in.): At ℄ span	Centroid for total number of strands (in.): At beam end	Distance from ℄ span to hold-down point	Camber limits	Deflection diagram: A	Deflection diagram: B	Deflection diagram: C	Minimum concrete strength at time of release (psi)
6/10"ϕ270 K-LR	58,600	43,950	24	20	3.50	4	3.50	50.00	3.50	11.25	9'-5½"	0 to 2 5/16"	1 3/16"	1 3/16"	0"	5200

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 1077+37.88

95FT BEAM DETAILS
BEAM NO. 95-3 (BT-54)

FMS: 102207 / 302000
COUNTY: Marshall
PROJECT NUMBER: BR-0060-03(021)

WORKING NUMBER 23 OF 30
SHEET NUMBER 8025

DESIGNER Thomas Terry CHECKER Spencer Yates
DETAILER Thomas Terry ISSUE DATE 2016-07-12
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

BY | REVISION | DATE

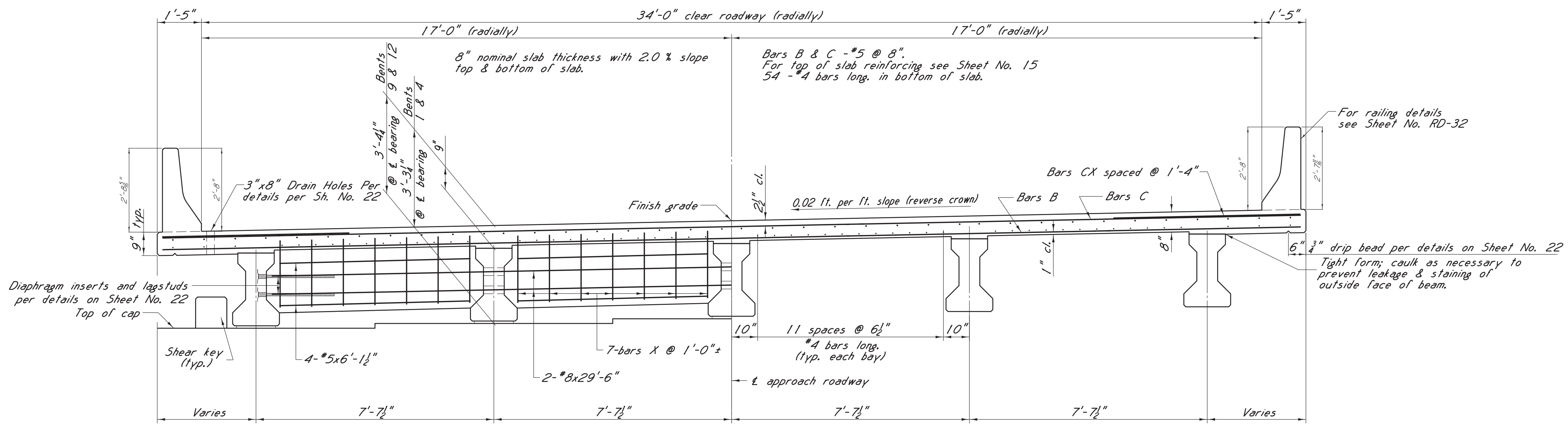
PLAN BRIDGE DIVISION MISSISSIPPI DEPARTMENT OF TRANSPORTATION

00:00 AMPM DGNFILENAME

D-3

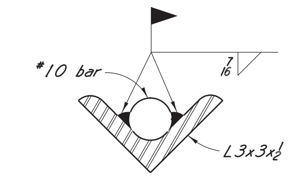
Sample Plans for MDOT Project in Leake County

STATE	PROJECT NO.
MISS.	BR-0521-00(006)

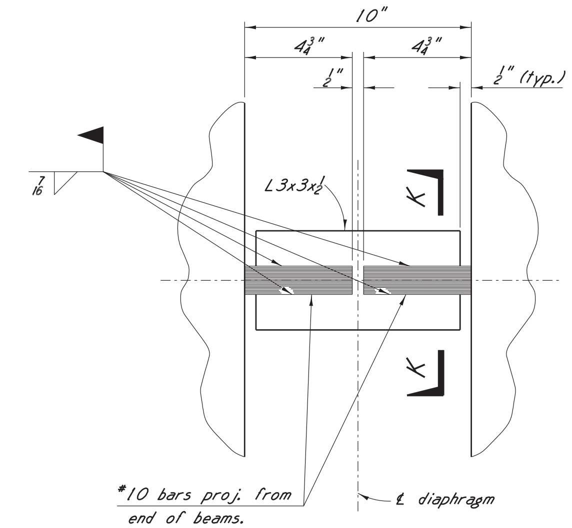


PART SECTION NEAR BRIDGE END
Showing end diaphragm details

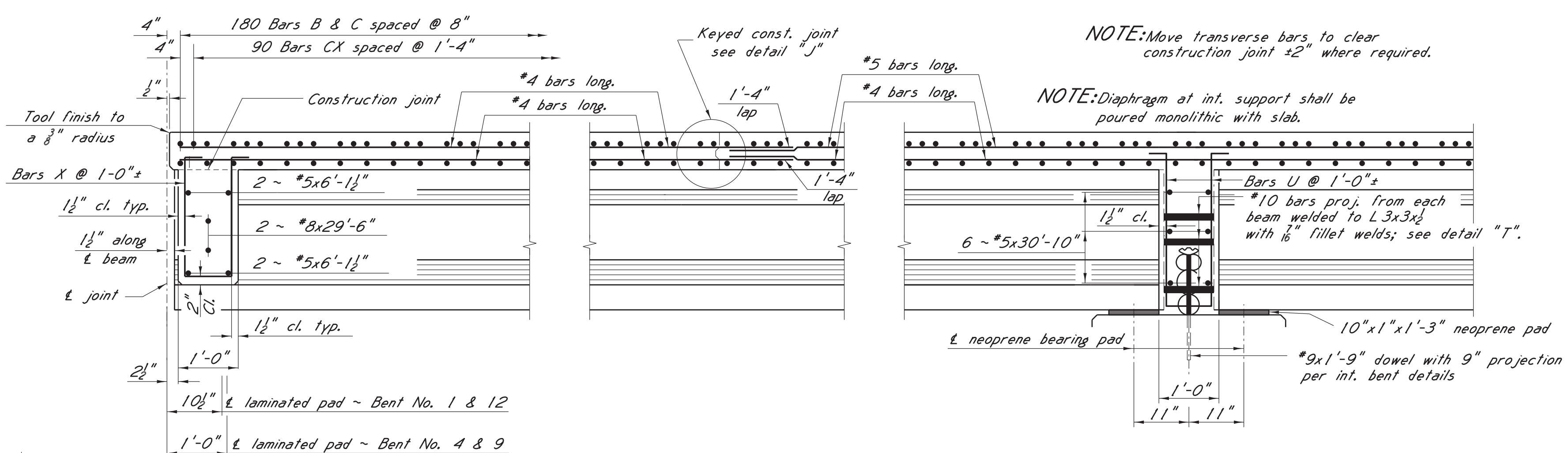
PART SECTION NEAR MIDSPAN
Showing slab reinforcing



SECTION K-K



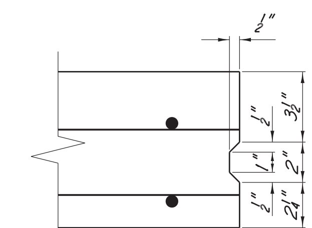
DETAIL "T"



SECTION A-A

SECTION B-B

SECTION C-C



DETAIL "J"

NOTE:
Contractor should be aware of possible tilting of exterior beams during construction of the superstructure and should take precautionary steps to prevent such tilting of beams.

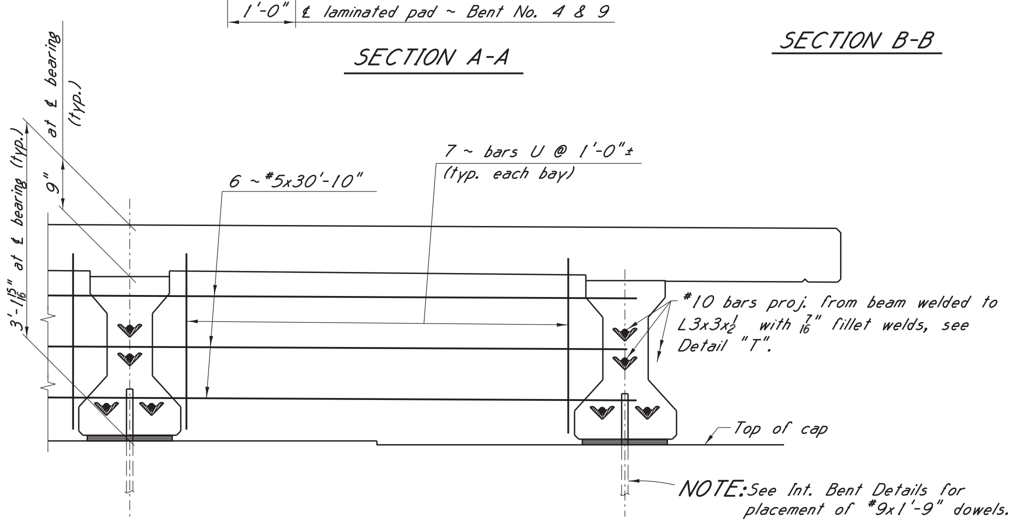
NOTE:
Transverse bars B & C are placed on radial lines spaced along ℄ approach roadway arc. Longitudinal bars are placed concentric to ℄ approach roadway arc.

NOTE:
Insure that holes in beam webs are completely filled with diaphragm concrete.
Prestressed concrete beams and bearing details per sheets no. 43-44

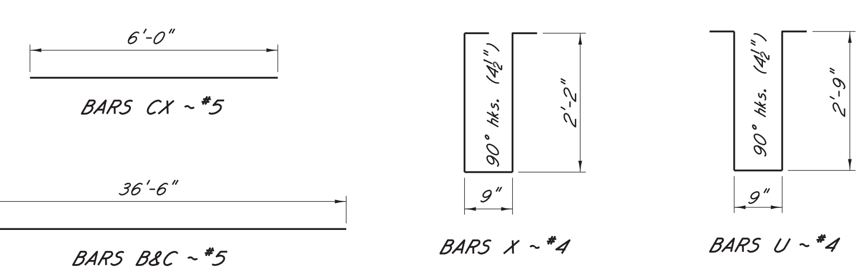
NOTE:
The volume of concrete in the fillets between the bottom of the nominal slab and the top of the beam has been estimated by using one half (1/2) of the fillet height, at the bearing, multiplied by the top flange width and the full length of the beam. This volume shall be used for final pay quantity.

NOTE:
For GENERAL NOTES, Railing Details and other Typical Span Details see Sheets No. 1, 22 & RD-32

DESIGN DATA:
Specifications . . . . A.A.S.H.T.O., LRFD 2012 and Int. thru 2014
Loading HL93
Slab Stresses . . . . fs=24,000 p.s.i.; f'c=1,600 p.s.i.; n=8
Prestressed Beam Details . . . . See Sheets No. 23 & 24



PART SECTION NEAR 1'-0" INT. DIAPHRAGM



BAR BENDING DETAILS
Dimensions are out to out

DEPARTMENT OF TRANSPORTATION MDOT MISSISSIPPI

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21

40 FT. SPAN DETAILS

PROJECT 100592/301000
BR-0521-00(006)

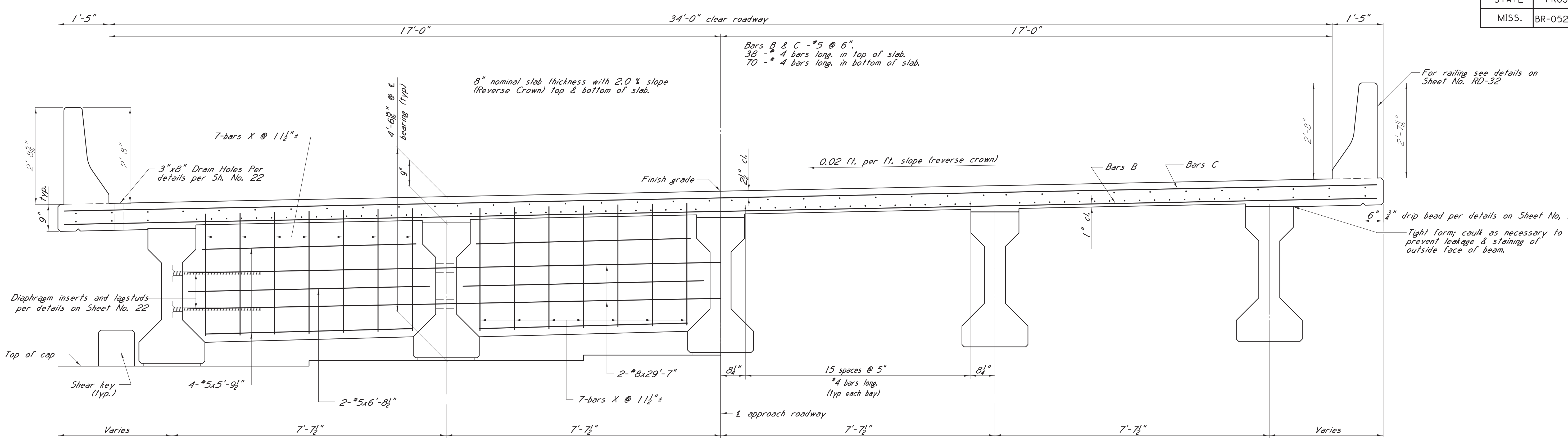
LEAKE COUNTY

WORKING NUMBER 16 OF 31
SHEET NUMBER 8019

REVISIONS BY DATE

DESIGNER Spencer Yates CHECKER Kevin Champney
DETAILER Korey Beckman ISSUE DATE 01/14/2015
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASSIST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

STATE	PROJECT NO.
MISS.	BR-0521-00(006)



PART SECTION NEAR BRIDGE END

Showing diaphram details and slab reinforcement

PART SECTION NEAR MIDSPAN

Showing slab reinforcement

NOTE:
Contractor should be aware of possible tilting of exterior beams during construction of the superstructure and should take precautionary steps to prevent such tilting of beams.

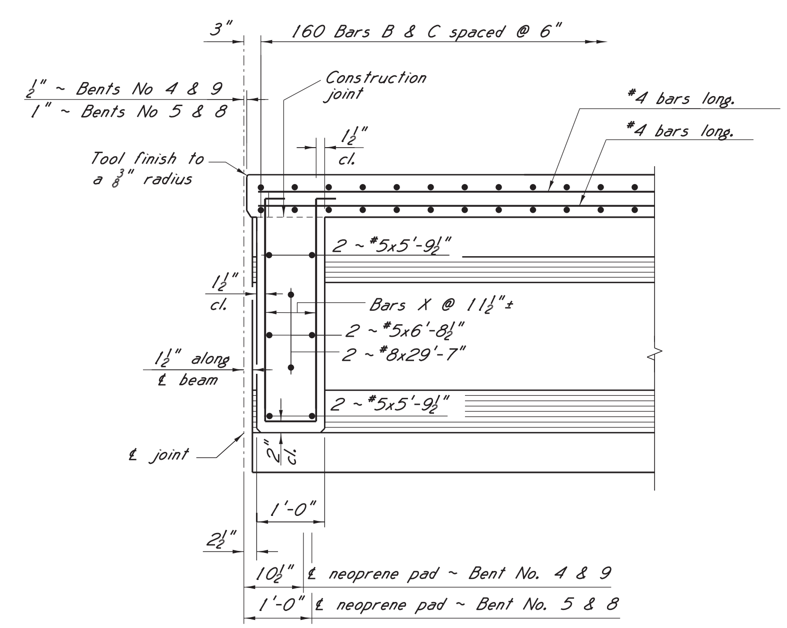
NOTE:
Transverse bars B & C are placed on radial lines spaced along ℄ approach roadway arc. Longitudinal bars are placed concentric to ℄ approach roadway arc.

NOTE:
Insure that holes in beam webs are completely filled with diaphragm concrete.
Prestressed concrete beams and bearing details per sheets no. 43-44

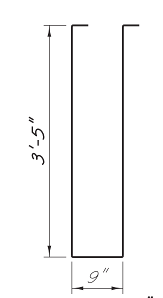
NOTE:
The volume of concrete in the fillets between the bottom of the nominal slab and the top of the beam has been estimated by using one half (1/2) of the fillet height, at the bearing, multiplied by the top flange width and the full length of the beam. This volume shall be used for final pay quantity.

NOTE:
For GENERAL NOTES, Railing Details and other Typical Span Details see Sheets No. 1, 22 & RD-32

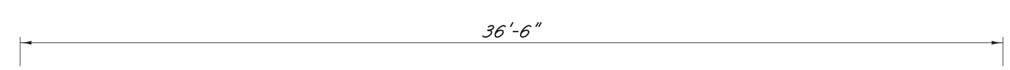
DESIGN DATA:
Specifications . . . . A.A.S.H.T.O., LRFD 2012 and Int. thru 2014
Loading HL93
Slab Stresses . . . . fs=24,000 p.s.i.; fc= 1,600 p.s.i.; n=8
Prestressed Beam Details . . . . See Sheets No. 25



SECTION A-A



BARS X ~ #4



BARS B&C ~ #5

BAR BENDING DETAILS

Dimensions are out to out

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21

80 FT. SPAN DETAILS

PROJECT 100592/301000
BR-0521-00(006)

LEAKE COUNTY

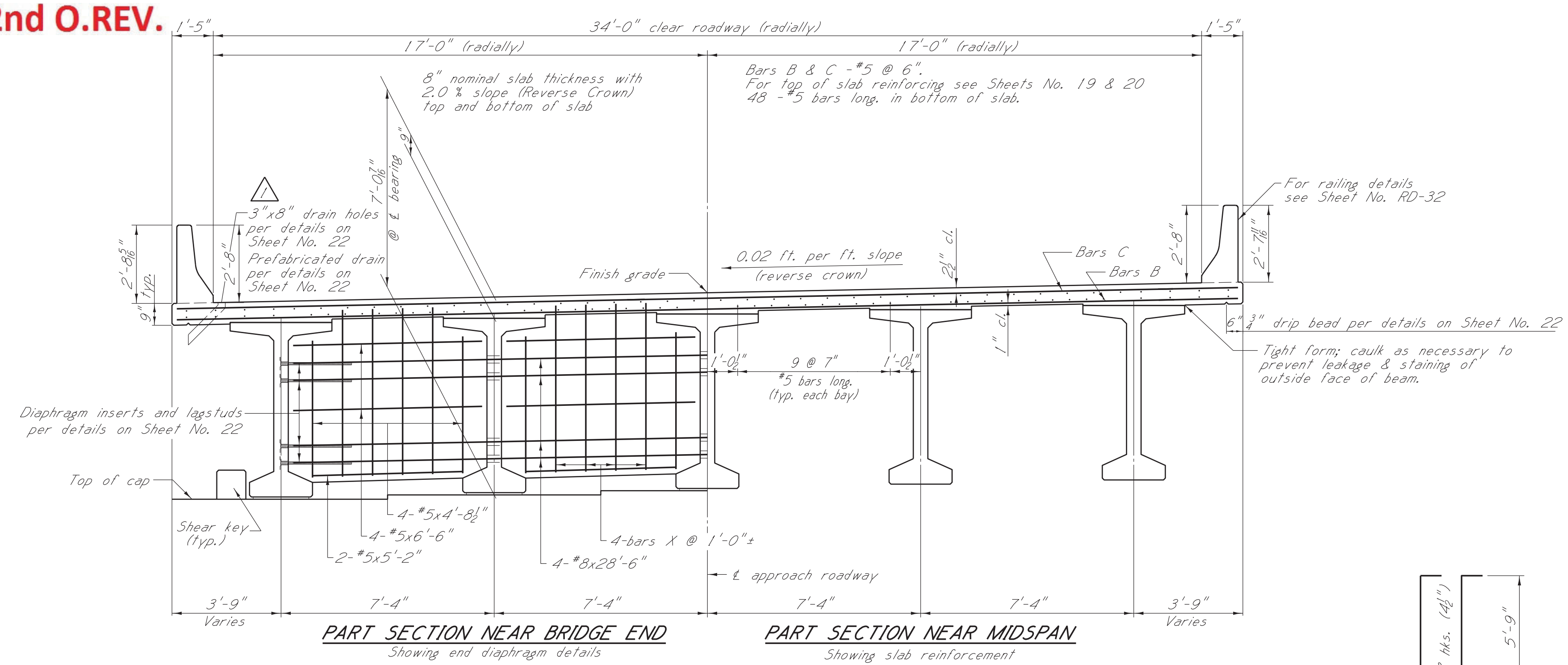
WORKING NUMBER 18 OF 31

SHEET NUMBER 8021

BY | REVISIONS | DATE

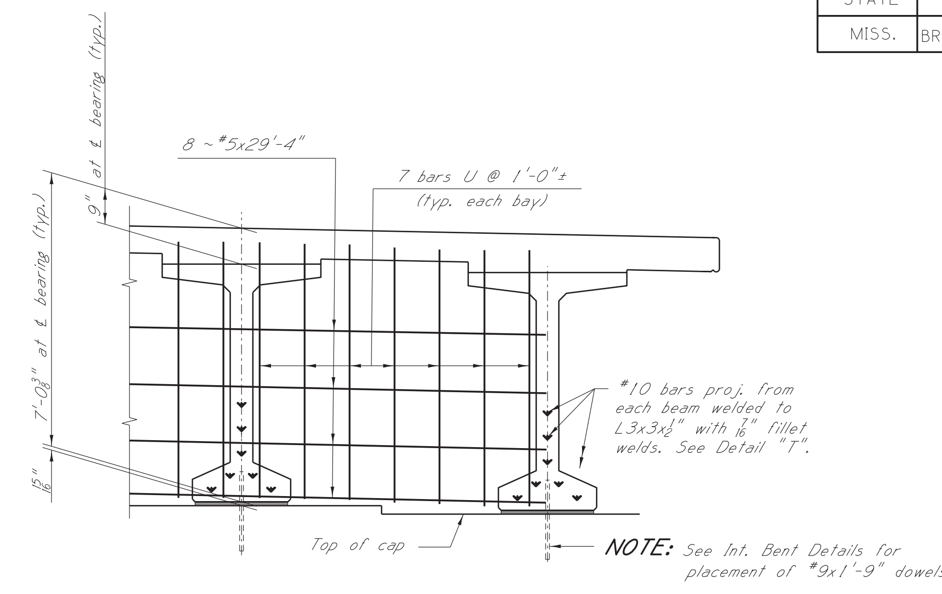
DESIGNER Spencer Yates CHECKER Kevin Champney
DETAILER Josh Wiltshire ISSUE DATE 01/14/2015
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASSIST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

2nd O.REV.



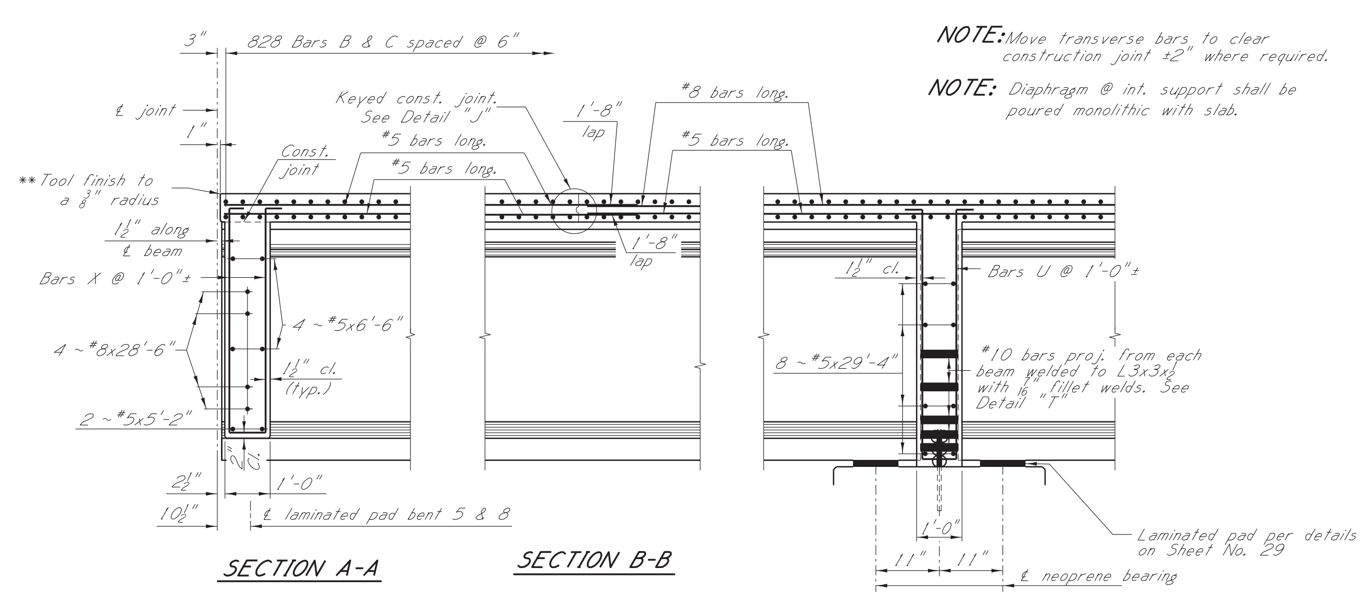
PART SECTION NEAR BRIDGE END
Showing end diaphragm details

PART SECTION NEAR MIDSPAN
Showing slab reinforcement



NOTE: See Int. Bent Details for placement of "9x1'-9" dowels.

PART SECTION NEAR 1'-0" INT. DIAPHRAGM
Dimension "T" = compressed pad thickness
For compressed pad thickness see Sheet No. 29.



NOTE: Move transverse bars to clear construction joint ±2" where required.

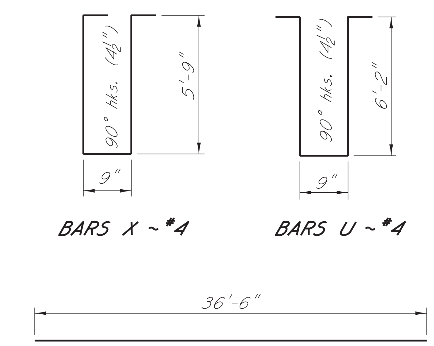
NOTE: Diaphragm @ int. support shall be poured monolithic with slab.

SECTION A-A

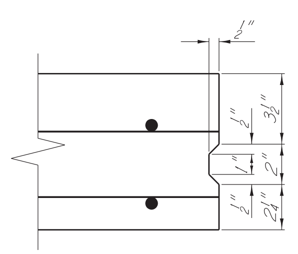
SECTION B-B

SECTION C-C

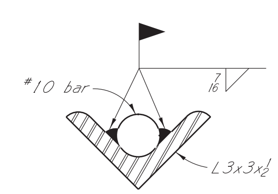
**NOTE: ½" seat required at joint. See sealing details on Sheet No. 22



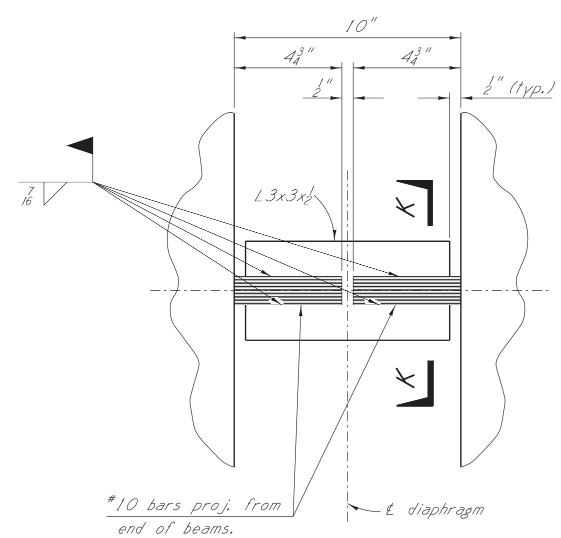
BAR BENDING DETAILS
Dimensions are out to out



DETAIL "J"



SECTION K-K



DETAIL "T"

NOTE:
Contractor should be aware of possible tilting of exterior beams during construction of the superstructure and should take precautionary steps to prevent such tilting of beams.

NOTE:
Transverse bars B & C are placed on radial lines spaced along ℄ approach roadway arc. Longitudinal bars are placed concentric to ℄ approach roadway arc.

NOTE:
Insure that holes in beam webs are completely filled with diaphragm concrete.
Prestressed concrete beams and bearing details per sheets no. 43-44

NOTE:
The volume of concrete in the fillets between the bottom of the nominal slab and the top of the beam has been estimated by using one half (1/2) of the fillet height, at the bearing, multiplied by the top flange width and the full length of the beam. This volume shall be used for final pay quantity.

NOTE:
For GENERAL NOTES, Railing Details and other Typical Span Details see Sheets No. 1, 22 & RD-32

DESIGN DATA:
Specifications . . . . A.A.S.H.T.O., LRFD 2012 and Int. thru 2014
Loading HL93
Slab Stresses . . . . fs=24,000 p.s.i.; fc=1,600 p.s.i.; n=8
Prestressed Beam Details . . . . See Sheets No. 26 & 27

STATE	PROJECT NO.
MISS.	BR-0521-00(006)

DATE	REVISIONS	BY
12/01/15	Revised prefab drain detail	SEY

DEPARTMENT OF TRANSPORTATION MDOT MISSISSIPPI

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21

138 FT. SPAN DETAILS

PROJECT 100592/301000 BR-0521-00(006)

LEAKE COUNTY

WORKING NUMBER 21 OF 31

SHEET NUMBER 8024

DESIGNER Spencer Yates
DETAILER Korey Beckman
CHECKER Kevin Champney
ISSUE DATE 01/14/2015
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASSIST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

2nd O.REV.

STATE	PROJECT NO.
MISS.	BR-0521-00(006)

GENERAL NOTES:

Specifications: Mississippi Standard Specifications for Road and Bridge Construction, 2004.
No change of plans will be permitted except by written approval of the Director of Structures, State Bridge Engineer. Minor changes in detail of design or construction procedure may be authorized by the Director of Structures, State Bridge Engineer provided such changes will not be cause for contract price adjustment.
The final surface texture of the bridge deck shall be mechanically transverse grooved in accordance with Sections 501 and 907-804 of the specifications. See Misc. Span Details for limits of transverse grooving on bridge deck.
Bridge concrete shall be class "AA".
Railing expansion joint material shall be bituminous fiber type unless otherwise noted.
No payment will be allowed for excavation incidental to the construction of end bents.
Bar bending details shall be in accordance with "Manual of Standard Practice for Detailing Reinforced Concrete Structures" (ACI 315R-94).
Reinforcement order lists and required placing plans shall be furnished in accordance with Section 805 of the Mississippi Standard Specifications. Partial submittals are not acceptable.
Shop drawings of prestressed beams, including an erection plan, shall be submitted in duplicate to the Director of Structures, State Bridge Engineer for approval prior to the manufacture of beams.
The Fabricator shall provide camber data at release and immediately prior to shipping.
The Contractor shall provide camber data after erection. The Contractor should be aware that the deflection diagram may be modified based on the provided camber data. Therefore, deck grades should be set only after notification from the Director of Structures, State Bridge Engineer.
Concrete surfaces shall receive a Class 2 rubbed or spray finish in accordance with the specifications.
Reinforcing steel shall be ASTM A615, Grade 60, unless otherwise noted.
Work for which no pay item is provided in the proposal will not be paid for directly and compensation therefor will be included in the prices and payments for bid items.
All riprap and geotextile fabric shown on the Bridge Plans are included in the Bridge Quantities

NOTE:

The girder deflection diagrams shown in these plans were prepared and intended for design and estimation purposes only. Actual bridge girder deflections may differ from the deflection diagrams shown in these plans.
It is the Contractor's responsibility to construct the bridge to meet the requirements of the plans and specifications including, but not limited to, the requirements for bridge deck smoothness.
Prior to formwork construction, the Contractor shall submit three (3) copies of a proposed bridge superstructure construction plan to the Director of Structures, State Bridge Engineer for review, through the Project Engineer. This submittal shall include all calculations, assumptions and parameters used by the Contractor to determine bridge girder deflections and form grade elevations. This submittal shall also include an erection and construction procedure that addresses the construction means and methodologies used by the Contractor and shall consider effects including, but not limited to, construction phasing, pouring schedules, applied permanent and construction loading, and shall include calculations and details of temporary girder bracing systems used to ensure girder stability and to counter the effects of girder tilt.
After girder erection and prior to deck construction, the Contractor shall submit deck thickness verification calculations for each girder. These calculations shall include a comparison of the erected girder top flange profiles versus the plan deck grade elevations over each girder plus the anticipated girder deflection due to applied permanent dead load and creep.
Three (3) copies of the deck thickness verification calculations and any proposed remediation measures to correct for thin deck areas shall be submitted to the Director of Structures, State Bridge Engineer for review, through the Project Engineer.
The bridge superstructure construction plan and the deck thickness verification calculations shall be prepared and stamped by a Mississippi Registered Professional Engineer.

SPECIAL PROVISIONS REQUIRED:

Concrete Bridges And Structures . . . . . No. 907-804

CONCRETE PILE NOTES:

Test piles shall be driven as permanent piles at the location shown in the PDA TEST PILE SCHEDULE and will be paid for as test piles only.
The Director of Structures, State Bridge Engineer may authorize test piles driven outside the structural limits.
Test piles shall be driven as a continuous operation, to the bearing capacity and the minimum ground penetration shown in the PDA TEST PILE SCHEDULE, unless otherwise directed by the Director of Structures, State Bridge Engineer.
Permanent piles shall be driven to an elevation no higher than the elevation shown in the REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE.
The tip elevation of piling, for hydraulic structures, may be determined by the scour line.
When feasible, bearing piles shall be driven full length and be spliced, only, as approved by the Director of Structures, State Bridge Engineer.
When loading tests are required, the maximum test load shall be one and one half ($1\frac{1}{2}$) times the minimum pile bearing capacity.
All piles shall be prestressed type per details on Sheets No. CPD.
Prestressed concrete piling shall not be driven until the concrete has reached a minimum compressive strength of 5,000 psi and is at least 7 days old.
PDA test piles shall require a 1 day and 7 day restrike unless otherwise directed by the Engineer.
Pile lengths and driving criteria shall be provided based on the results of the PDA test piles.
The required ultimate pile bearing shown in the REQUIRED ULTIMATE PILE BEARING AND TIP ELEVATION SCHEDULE includes the LRFD resistance factor for PDA of 0.65.
Pile hammer leads used for all PDA test piles and PDA restrikes shall be large enough to provide a minimum of 3" clearance on each side of the pile in order to properly place and protect PDA gages.

STEEL PIPE PILE NOTES: △1

PDA test piles all be driven with an approved impact hammer as an indicator test pile or production pile at the location shown in the PDA TEST PILE SCHEDULE and will be paid for as test piles only.
The first PDA test pile driven shall be an indicator PDA test pile as shown on the Foundation Plan. The indicator PDA test pile shall be driven continuously using an approved impact hammer. The full length of the indicator PDA test pile shall e monitored using PDA.
Remaining test piles all be driven as a continuos operation, to the bearing capacity and the minimum ground penetration shown in the PDA TEST PILE SCHEDULE, unless otherwise directed by the Director of Structures, State Bridge Engineer.
PDA test piles shall require a 1 day and 7 day restrike unless otherwise directed by the Engineer.
Pile lengths and driving criteria shall be provided based on the results of the PDA test piles.
Permanent piles shall be driven to an elevation no higher than the elevation shown in the REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE.
The required ultimate pile bearing shown in the REQUIRED ULTIMATE PILE BEARING AND TIP ELEVATION SCHEDULE includes the LRFD resistance factor for PDA of 0.65.
The Director of Structures, State Bridge Engineer may authorize test piles driven outside the structural limits.
The tip elevation of piling, for hydraulic structures, may be determined by the scour line.
When feasible, bearing piles shall be driven full length and be spliced, only, as approved by the Director of Structures, State Bridge Engineer.
When loading tests are required, the maximum test load shall be one and one half ($1\frac{1}{2}$) times the minimum pile bearing capacity.
Welding shall be done by the ELECTRIC ARC process. Welders shall be certified and electrodes shall be approved.
Pile hammer leads used for all PDA test piles and PDA restrikes shall be large enough to provide a minimum of 3" clearance on each side of the pile in order to properly place and protect PDA gages.
Pipe piles shall receive a protective coating begining at the bottom of the cap and extending to the 100 yr. scour elevation as shown on the Layout Sheet. The coating shall be one of the following, applied according to the manufacturer's specifications in two coats of 16mil minimum dry film thickness:
A. Bitumastic 300-M Coal Tar Epoxy
manufactured by Carboline Compnay in St. Louis, MO
www.carboline.com
B. Corotech Coal Tart Epoxy
manufactured by INSL-X Company in Montvale, NJ
www.carotechcoatings.com
C. Series 46-143 TNEME-Tar
manufactured by TNEMEC Co Inc in Kansas City, MO
www.tnemec.com
Any areas of coating above the ground line that become damaged during shipping or driving shall be repaired per the manufacturer's specifications. Any areas of coating affected by pipe pile splicing shall be repaired per the manufacture's specification. Protective coating, including surface preparation and application, will be paid for as Steel Pipe Piling, (not a seprate pay item).
All Steel Pipe Piles shall be ASTM A253, Grade 3 (F_y = 45,000 psi.).
Steel Pipe Piles are intended to be opend ended.
Welding shall comply with ANSI/AWS D1.5 Bridge Welding Code and be performed by a certified welder.

REQUIRED ULTIMATE PILE BEARING CAPACITY AND TIP ELEVATION SCHEDULE

Bent No.	Pile Type	Pile Size	Required Ultimate Bearing (Tons)	Min. Tip Elevation	Est. Length (ft.)	Controlling Limit State	LRFD Resistance Factor
1	Prest. conc.	16"x16"	115	323.0	55	Strength I	0.65
2	Prest. conc.	18"x18"	183	321.0	55	Strength I	0.65
3	Prest. conc.	18"x18"	188	328.0	55	Strength I	0.65
4	Prest. conc.	18"x18"	217	327.0	55	Strength I	0.65
5	Steel pipe pile	24"	347	329.0	80	Strength I	0.65
6	Steel pipe pile	30"	463	285.0	90	Strength I	0.65
7	Steel pipe pile	30"	463	285.0	90	Strength I	0.65
8	Steel pipe pile	24"	347	328.0	80	Strength I	0.65
9	Prest. conc.	18"x18"	217	329.0	55	Strength I	0.65
10	Prest. conc.	18"x18"	188	329.0	55	Strength I	0.65
11	Prest. conc.	18"x18"	183	318.0	55	Strength I	0.65
12	Prest. conc.	16"x16"	115	318.0	55	Strength I	0.65

PDA TEST PILE SCHEDULE

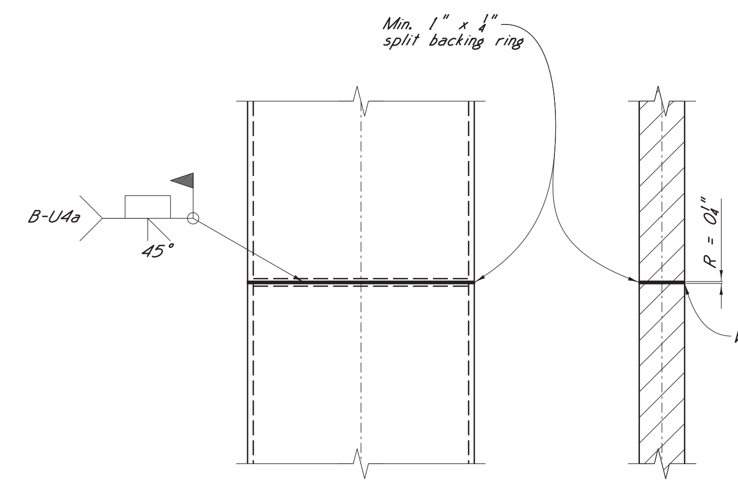
Bent No.	Min. Length (ft.)	Tip Elevation
1	65	297.5
4	65	297.3
6	△2 110**	△2 250.9
8	△2 100**	△2 259.7
10	65	296.9
12	65	295.6
Sta. 803+64*	△2 110***	△2 250.9
Sta. 806+40*	△2 100***	△2 259.7

△1 Note: *Indicator steel pipe pile
**Includes additional uncoated 10' above planned top of pile
△2 ***Indicator piles shall be uncoated

PILE HAMMER REQUIREMENTS*

Pile Type	Pile Size	Min. Ram Weight (lb.)	Max Ram Weight (lb.)	Min Energy (ft.-lb.)	Max Energy (ft.-lb.)
Prest. conc.	All	N/A	6,600	N/A	75,000
Steel pipe pile	All	10,000	N/A	100,000	N/A

*NOTE: Based on preliminary drivability analysis



PIPE PILE SPLICING DETAIL
24" Diam. steel pipe piles
30" Diam. steel pipe piles

ESTIMATED QUANTITIES

Item / Location	Trans. Grooving	Conventional Static Pile Load Test	16"x16" Prest. Conc. Piling	18"x18" Prest. Conc. Piling	PDA Test Pile	Pile Restrike	24" Diam. Steel Pipe Piling	30" Diam. Steel Pipe Piling	Bridge Concrete Class "AA"	40 Ft. Prest. Conc. Beams Type-I+2	80 Ft. Prest. Conc. Beams Type-III	138 Ft. Prest. Conc. Beams BT-72	Reinforcement	Concrete Railing, 32"	Loose Riprap (300#)	Geotextile Fabric Under Riprap
	S.Y.	Each	L.F.	L.F.	Each	Each	L.F.	L.F.	C.Y.	L.F.	L.F.	L.F.	Lb.	L.F.	Ton	S.Y.
Spans	2,713.33								823.31	1,180.83	797.52	2,062.39	214,930	1,628.00		
End bents			880.0		2	1			42.44				8,433	3.00	2,076.0	1,942.0
Int. bents		1		2,090.0	6	1	720.0	810.0	155.38				18,676			
Totals	2,713.33	1	880.0	2,090.0	8	2	720.0	810.0	1,021.13	1,180.83	797.52	2,062.39	242,039	1,631.00	2,076.0	1,942.0

△1

Date	Revisions	By
6/22/15 △1	Added Indicator Steel Pipe Pile	SEY
8/27/15 △2	Test Pile Lenghts Revised	SEY

DEPARTMENT OF TRANSPORTATION MDOT MISSISSIPPI

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21
ESTIMATED QUANTITIES
& GENERAL NOTES

PROJECT 100592/301000
BR-0521-00(006)

LEAKE COUNTY

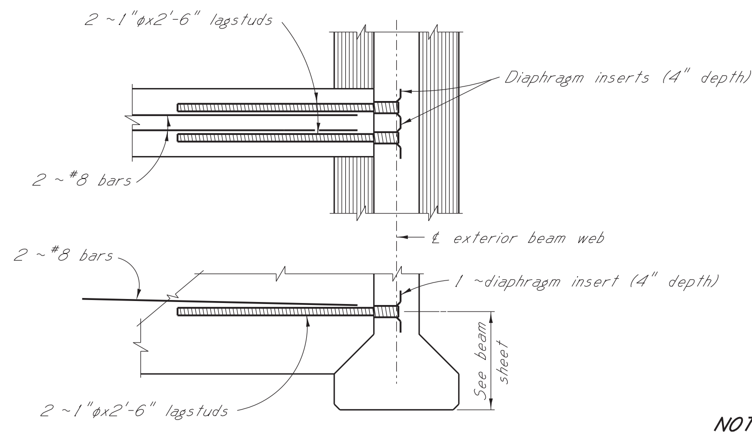
WORKING NUMBER 1 OF 31

SHEET NUMBER 8004

DESIGNER Spencer Yates CHECKER Kevin Champney
DETAILER Spencer Yates ISSUE DATE 01/14/2015
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASSIST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

2nd O.REV.

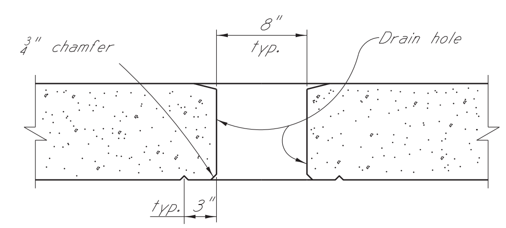
STATE	PROJECT NO.
MISS.	BR-0521-00(006)



PART SECTION

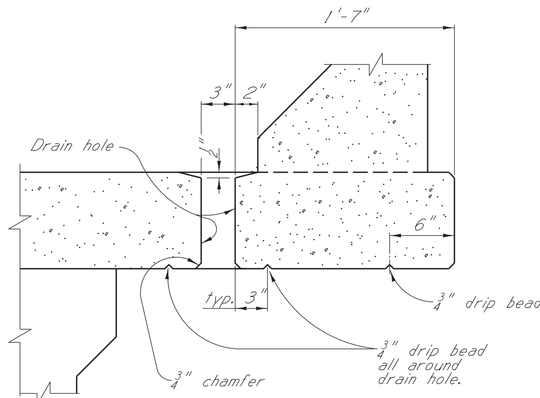
DIAPHRAGM INSERT AND LAGSTUD DETAILS

NOTE: Continuous threaded lagstuds and diaphragm inserts shall be as manufactured by the Richmond Screw Anchor Co., Inc., Atlanta, GA; By Meadow Steel Products Co., Inc., Birmingham, AL Or Dayton Superior Co., Inc., Birmingham, AL.



SECTION T-T

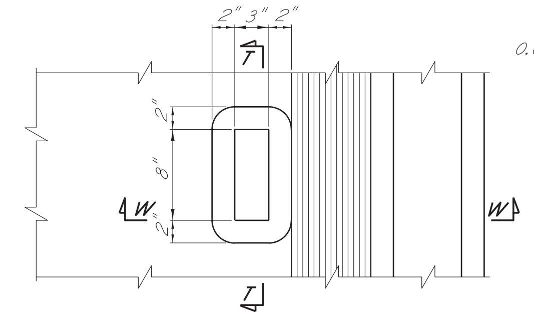
NOTE: Drain holes shall be located so that bars B & C will not be cut.



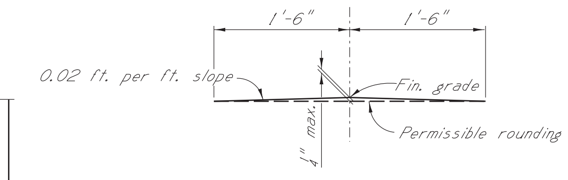
SECTION W-W

DRAIN HOLE DETAILS

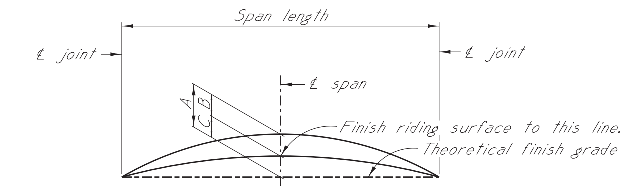
Use where shown on the Span Detail sheet.



DETAIL "D"



CROWN DETAILS

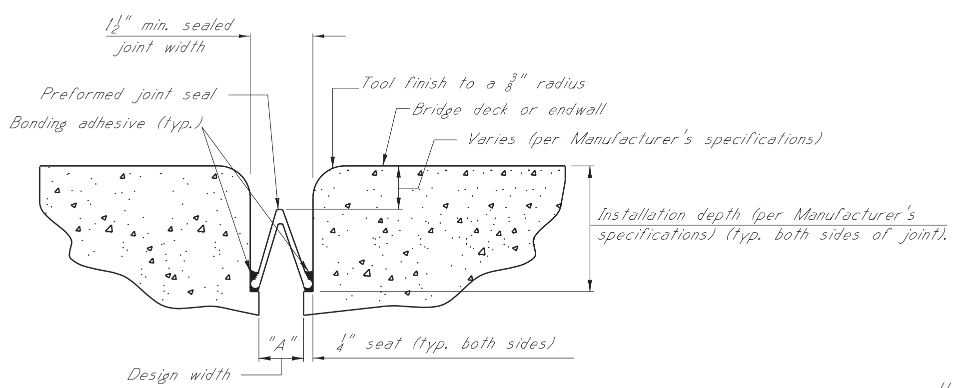


DEFLECTION DIAGRAM

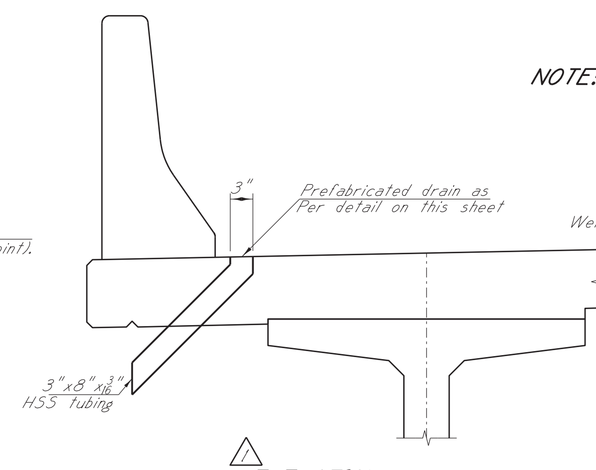
A = total recommended allowance for deflection.
B = estimated deflection due to dead load of slab & rail.
C = A-B = net initial camber in riding surface, which includes an allowance or creep.

NOTE: For values of A, B & C, see Beam Detail sheets.

NOTE: The Girder Deflection Diagrams shown in these plans were prepared and intended for design and estimation purposes only. Actual bridge girder deflections may differ from the deflection diagrams shown in these plans. It is the Contractor's responsibility to construct the bridge to meet the requirements of the plans and specifications including, but not limited to, the requirements for bridge deck smoothness. Prior to formwork construction, the Contractor shall submit three (3) copies of a proposed BRIDGE SUPERSTRUCTURE CONSTRUCTION PLAN to the Director of Structures, State Bridge Engineer for review, through The Project Engineer. This submittal shall include all calculations, assumptions and parameters used by the Contractor to determine bridge girder deflections and form grade elevations. This submittal shall also include an erection and construction procedure that addresses the construction means and methodologies used by the Contractor and shall consider effects including, but not limited to, construction phasing, pouring schedules, applied permanent and construction loading, and shall include calculations and details of temporary girder bracing systems used to ensure girder stability and to counter the effects of girder tilt. After girder erection and prior to deck construction, the Contractor shall submit deck thickness verification calculations for each girder. These calculations shall include a comparison of the erected girder top flange profiles versus the plan deck grade elevations over each girder plus the anticipated girder deflection due to applied permanent dead load and creep. Three (3) copies of the deck thickness verification calculations and any proposed remediation measures to correct for thin deck areas shall be submitted to the Director of Structures, State Bridge Engineer for review, through the Project Engineer. The BRIDGE SUPERSTRUCTURE CONSTRUCTION PLAN and the deck thickness verification calculations shall be prepared and stamped by a Mississippi Registered Professional Engineer.

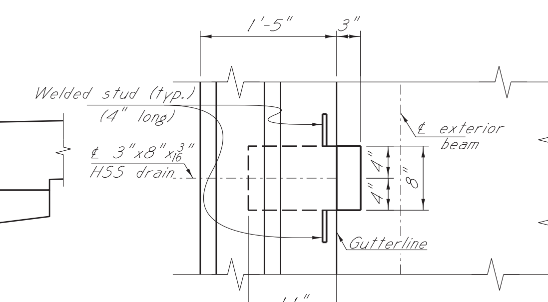


TYPICAL SECTION PREFORMED JOINT SEAL



ELEVATION

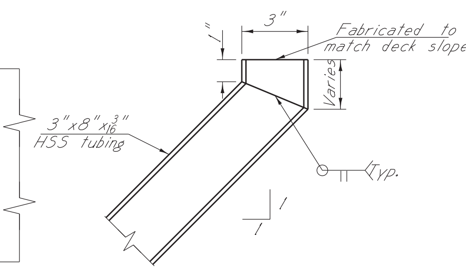
NOTE: 3"x8"x3/16" HSS drains shall be in accordance with ASTM A53 and shall be hot dipped galvanized in accordance with ASTM A153



PLAN

PRE FABRICATED DRAIN HOLE DETAILS

Use where shown on the Span Detail sheet.



PRE-FABRICATED DRAIN DETAILS

NOTES:

1. The preformed joint seal shall be one of the following, installed according to the Manufacturer's specifications:

   A. Silicoflex Joint Sealing System, manufactured by R.J. Watson, Inc in Alden, NY
   www.rjwatson.com
   Type I: Model SF-150
   Type II: Model SF-225

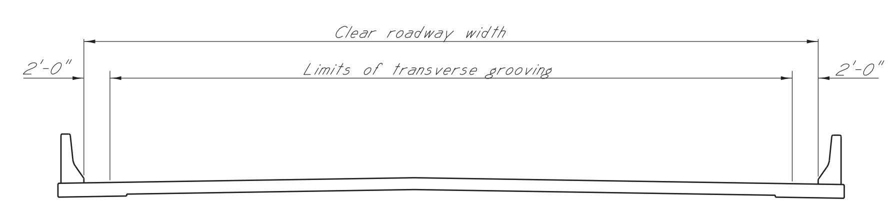
   B. Wabo SPS Joint System manufactured by Watson Bowman Acme Corporation in Amherst, NY
   www.wbacorp.com
   Type I: Model SPS-225
   Type II: Model SPS-400

   C. V-Seal Expansion Joint System manufactured by The D.S. Brown Company in North Baltimore, OH
   www.dsbrown.com
   Type I: Model V-300
   Type II: Model V-400

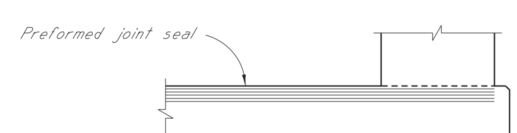
2. For estimating purposes, The RJ Watson Silicoflex Joint Sealing System was selected. However, should another supplier be chosen, it is the Contractor's responsibility to ensure that the Manufacturer's recommendations are followed for joint preparation, installation depths and widths, adhesive setting times, and any other variances between the specifications provided by the Manufacturers. A Manufacturer representative shall be present at the time joint sealing begins to ensure that the Contractor is properly schooled in installation of the joint material. All open joints shall be sealed at their design widths, dimension "A", as indicated on the end bent and span details.

3. Dimension "A" is defined as the design width of the joint opening, which does not account for the ¼" seat required on both sides of the joint. Preformed Joint Seal, Type I, shall be used for design widths less than 2". Preformed Joint Seal, Type II, shall be used for design widths greater than or equal to 2", with the maximum design width being 2½". In cases where design widths are greater than 2½", another type of expansion material shall be required as directed by the Director of Structures, State Bridge Engineer.

4. Joints in newly constructed bridge decks shall be protected from damage until accepted for maintenance by the State. Damaged joints shall be repaired at no additional cost to the State.



LIMITS OF TRANSVERSE GROOVING



ELEVATION AT END OF SPAN

GENERAL NOTES:

All concrete in span and railing shall be class "AA".
Chamfer all edges ¾", unless otherwise noted.
See Layout sheet for finishing of concrete surfaces.
Placing dimensions for reinforcing steel to concrete surfaces are clear distances.
To determine the dimension from finish grade to cap, the assumption is made that the compressed thickness of the neoprene pad is as shown in table, and that the original camber of the beams will be within the limits shown on the Beam Detail sheets. The Director of Structures, State Bridge Engineer shall be notified if the cambers are not within these limits.

DATE	REVISIONS	BY
12/01/15	Revised prefab drain detail	SEY

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21

MISCELLANEOUS SPAN DETAILS

PROJECT 100592/301000
BR-0521-00(006)

LEAKE COUNTY

WORKING NUMBER 22 OF 31

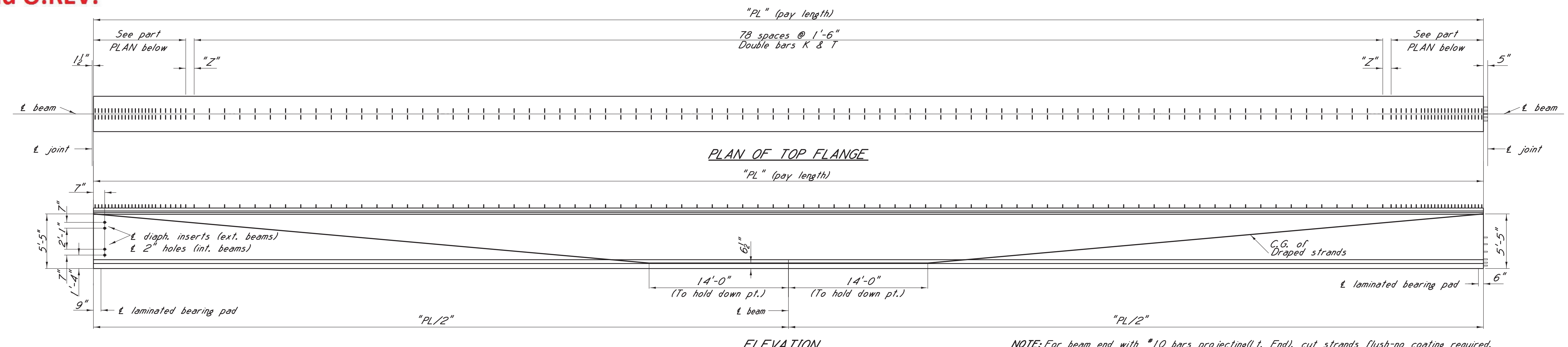
SHEET NUMBER 8025

DESIGNER Spencer Yates CHECKER Kevin Chamoney
DETAILER Korey Reckman ISSUE DATE 01/14/2015
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASSIST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

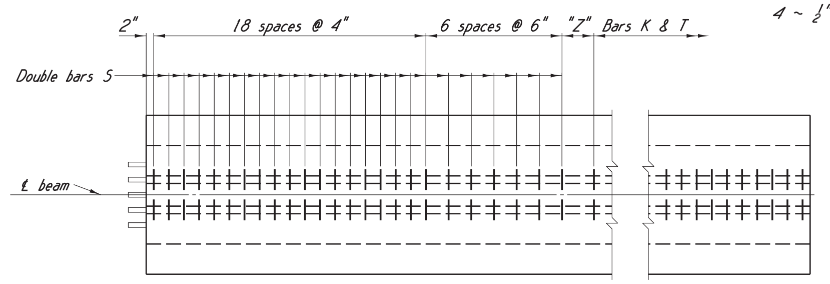
DEPARTMENT OF TRANSPORTATION MDOT MISSISSIPPI

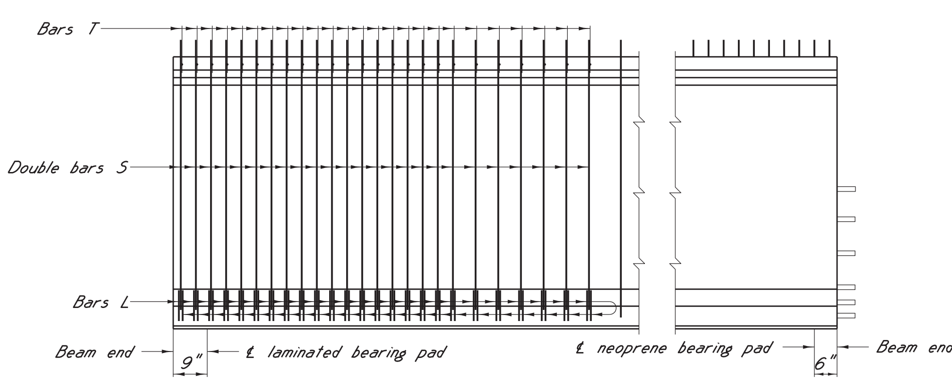
2nd O.REV.

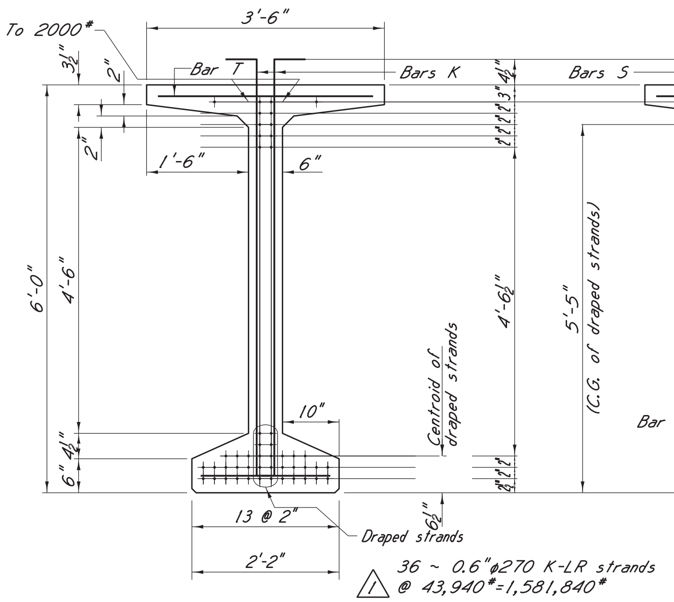
STATE	PROJECT NO.
MISS.	BR-0521-00(006)



NOTE: For beam end with #10 bars projecting(Lt. End), cut strands flush-no coating required. For other beam end(Rt. End), cut strands flush and weatherproof with limestone colored "Sonolastic" (Sonneborn Building Products), "GC-9 Synthacalk" (Pecora Corp.), or approved equal, meeting the requirements of Federal Specification No. TT-S-00227E Or TT-S-00230C, applied according to Manufacturer's directions.







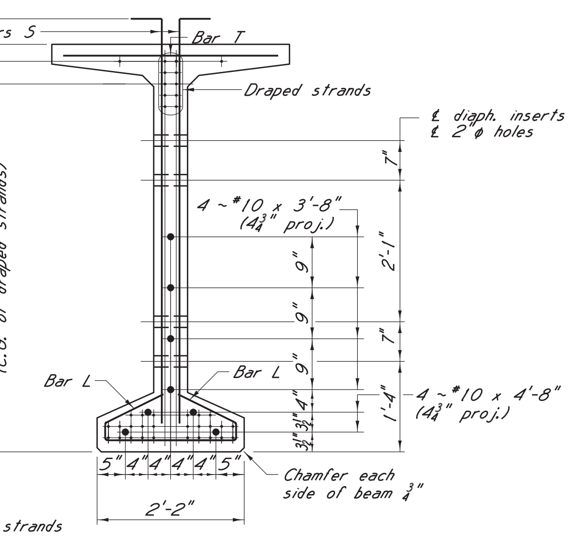


TABLE OF BEAM DIMENSIONS

BEAM NO.	"PL"	"PL/2"	"Z"	*BEVEL
138-1	137'-2¼"	68'-7⅛"	11³⁄₁₆"	None
138-2	137'-3⅞"	68'-7¹⁵⁄₁₆"	11¹⁵⁄₁₆"	None
138-3	137'-5½"	68'-8¾"	1'-0¾"	None
138-4	137'-7⅛"	68'-9⁹⁄₁₆"	1'-1½"	None
138-5	137'-8⅝"	68'-10⁵⁄₁₆"	1'-2⁵⁄₁₆"	None
138-6	137'-2¼"	68'-7⅛"	11³⁄₁₆"	½"
138-7	137'-3⅞"	68'-7¹⁵⁄₁₆"	11¹⁵⁄₁₆"	½"
138-8	137'-5½"	68'-8¾"	1'-0¾"	½"
138-9	137'-7⅛"	68'-9⁹⁄₁₆"	1'-1½"	½"
138-10	137'-8⅝"	68'-10⁵⁄₁₆"	1'-2⁵⁄₁₆"	½"

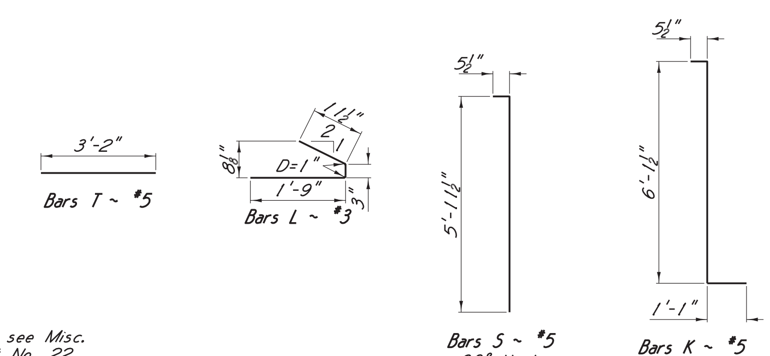
***NOTE:** See Sht. No. 29 for BEAM BEVEL DETAILS

GENERAL NOTES:

Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2004.
The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.
Strand pattern detailed is for ½"φ270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.
The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.
The Fabricator shall provide camber data at release and immediately prior to shipping.
Concrete shall be class "FX" and:
(a) shall have a 28-day cylinder strength of 7500 p.s.i.
(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.
At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 7500 p.s.i. concrete.
If any cylinder tests below 7500 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

DESIGN DATA

Unit stresses are in accordance with A.A.S.H.T.O., LRFD 2012.
Stay-in-place metal forms............18 psf (between flanges)



BAR BENDING DETAILS
Dimensions are out to out

LR indicates low-relaxation strands

PRESTRESS REQUIREMENTS

For deflection diagram, see Misc. Span Details per Sheet No. 22

Strand type	Minimum breaking strength (lbs/strand)	Initial tension (lbs/strand)	Required number and location of strands: Total number strands	Straight strands: Number strands	Straight strands: Centroid (in.)	Draped strands: Number strands	Draped strands: Centroid (in.) ℄ span	Draped strands: Centroid (in.) Beam end	Centroid for total number of strands (in.): At ℄ span	Centroid for total number of strands (in.): At beam end	Distance from ℄ span to hold-down point	Camber limits	Deflection diagram: A	Deflection diagram: B	Deflection diagram: C	Minimum concrete strength at time of release (psi)
0.6"φ270 K-LR	45,000	△1 43,940	36	26	4.19	10	6.50	65.00	4.83	21.08	14'-0"	0 to 5¾"	5¹⁄₁₆"	2⁵⁄₁₆"	2¾"	6000

9/22/15	Revised initial tension	SEY		
DATE	REVISIONS	BY		

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21
138 FT. BEAM DETAILS
BEAMS NO.
138-1 THRU 138-5 (BT-72)

PROJECT 100592/301000
BR-0521-00(006)

LEAKE COUNTY

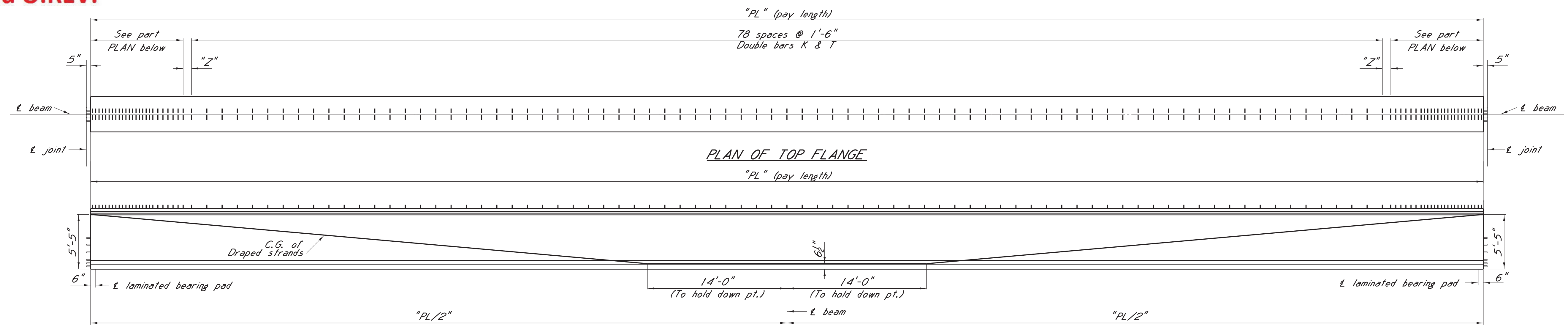
WORKING NUMBER 26 OF 31

SHEET NUMBER 8029

DESIGNER Spencer Yates CHECKER Kevin Champney
DETAILER Korey Beckman ISSUE DATE 01/14/2015
DIRECTOR OF STRUCTURES, STATE BRIDGE ENGINEER - JUSTIN WALKER, P.E.
DEP. DIR. OF STRUCTURES, ASSIST. STATE BRIDGE ENGINEER - SCOTT WESTERFIELD, P.E.

2nd O.REV.

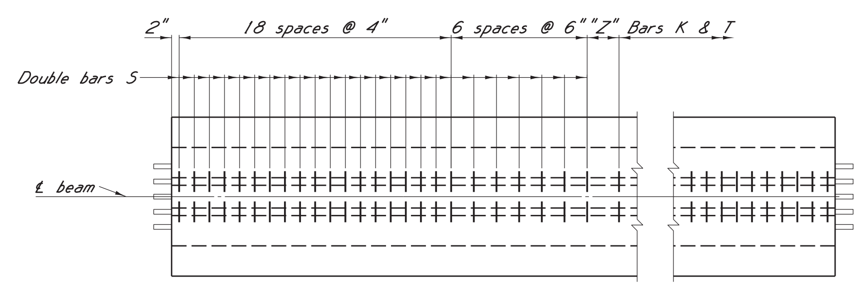
STATE	PROJECT NO.
MISS.	BR-0521-00(006)



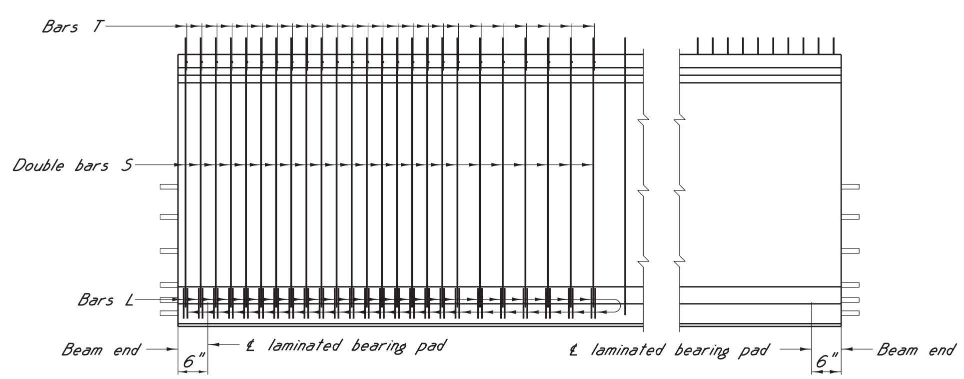
PLAN OF TOP FLANGE

ELEVATION

NOTE: Cut strands flush-no coating required (both ends).

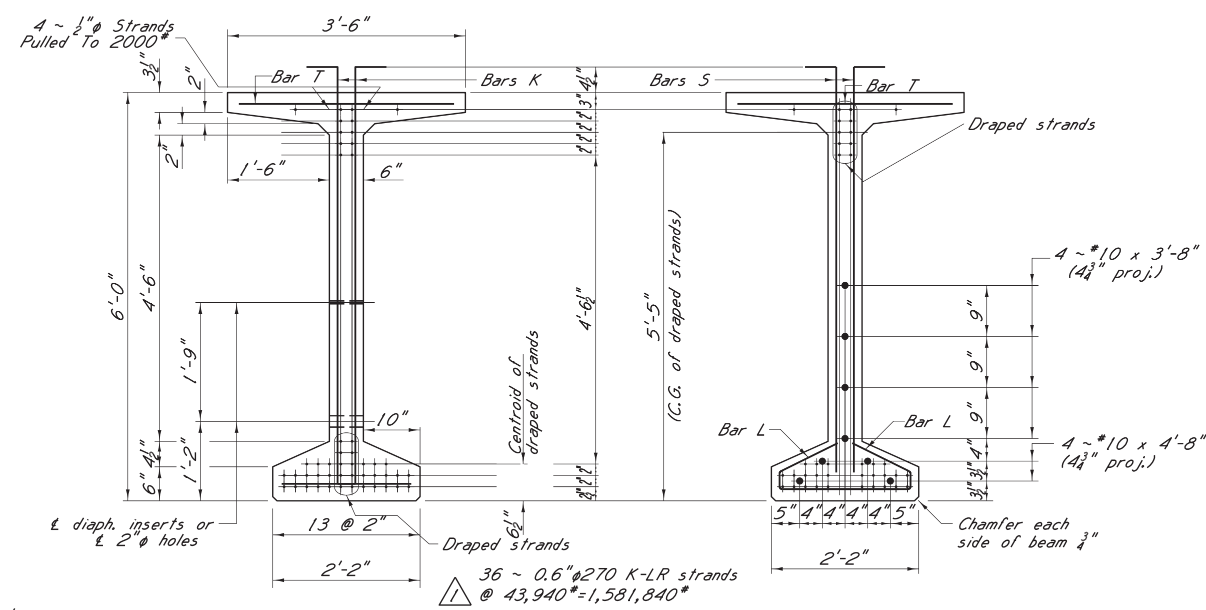


PART PLAN



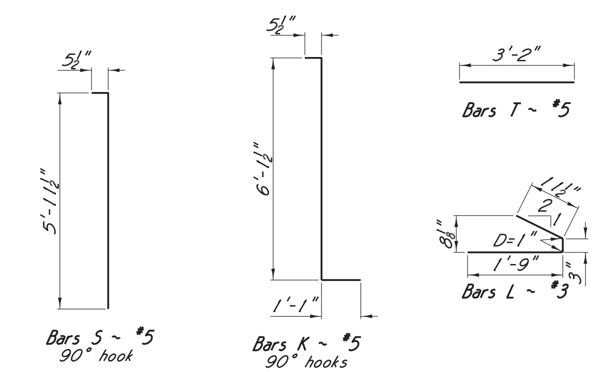
PART ELEVATION

Strands not shown for clarity



SECTION NEAR ℄ SPAN

END ELEVATION



BAR BENDING DETAILS

Dimensions are out to out

GENERAL NOTES:

Beams shall be manufactured in accordance with Mississippi Standard Specifications for Road & Bridge Construction, 2004.

The tops of beams shall be rough floated. At approximately the time of initial set the entire tops of beams shall be scrubbed transversely with a coarse wire brush to remove all laitance and produce a roughened surface for bonding slab. Other surfaces shall be finished per specifications.

Strand pattern detailed is for ½"φ270 K-LR strands. Shop drawings of prestressed beams shall include the type and location of all strands.

The Director of Structures, State Bridge Engineer shall be notified if the camber of the beam is not within the limits shown in table.

The Fabricator shall provide camber data at release and immediately prior to shipping.

Concrete shall be class "FX" and:

(a) shall have a 28-day cylinder strength of 7500 p.s.i.

(b) at transfer of the tensioning load, the cylinder strength of the concrete shall be as shown in table.

At the Contractor's request a suggested concrete design mix will be furnished with the understanding that it is the Contractor's responsibility to maintain 7500 p.s.i. concrete.

If any cylinder tests below 7500 p.s.i., the beam represented will be held on the yard until the 28-day strength is determined and acceptance or rejection has been established.

DESIGN DATA

Unit stresses are in accordance with A.A.S.H.T.O., LRFD 2012 & Int. thru 2014

Stay-in-place metal forms............18 psf (between flanges)

PRESTRESS REQUIREMENTS

LR indicates low-relaxation strands

For deflection diagram, see Misc. Span Details per Sheet No. 22

Strand type	Minimum breaking strength (lbs/strand)	Initial tension (lbs/strand)	Required number and location of strands: Total number strands	Straight strands: Number strands	Straight strands: Centroid (in.)	Draped strands: Number strands	Draped strands: Centroid (in.) ℄ span	Draped strands: Centroid (in.) Beam end	Centroid for total number of strands (in.): At ℄ span	Centroid for total number of strands (in.): At beam end	Distance from ℄ span to hold-down point	Camber limits	Deflection diagram A	Deflection diagram B	Deflection diagram C	Minimum concrete strength at time of release (psi)
0.6"φ270 K-LR	45,000	△1 43,940	36	26	4.19	10	6.50	65.00	4.83	21.08	14'-0"	0 to 5¾"	4 11/16"	2¼"	2 7/16"	6000

TABLE OF BEAM DIMENSIONS

BEAM NO.	"PL"	"PL/2"	"Z"
138-6	136'-10¾"	68'-5⅜"	9⅜"
138-7	137'-0⅜"	68'-6 3/16"	10 3/16"
138-8	137'-2"	68'-7"	11"
138-9	137'-3⅝"	68'-7 13/16"	11 13/16"
138-10	137'-5⅛"	68'-8 9/16"	1'-0 9/16"

DEPARTMENT OF TRANSPORTATION MDOT MISSISSIPPI

SEY					BY
9/22/15 Revised initial tension					REVISIONS / DATE

△1

MISSISSIPPI DEPARTMENT OF TRANSPORTATION
BRIDGE AT STA. 800+25.21
138 FT. BEAM DETAILS
BEAMS NO.
138-6 THRU 138-10 (BT-72)

PROJECT 100592/301000
BR-0521-00(006)

LEAKE COUNTY

WORKING NUMBER 27 OF 31

DESIGNER Spencer Yates
DETAILER Korey Beckman
CHECKER Kevin Champney
ISSUE DATE 01/14/2015
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SHEET NUMBER 8030