Effects of Time-Dependent Loss Models for Spliced-Girder Bridges

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ABSTRACT

In order to compare prestress losses over time, a time-dependent finite element analysis was performed on two precast spliced-girder bridges using current loss models (ACI-209, CEB-FIP, and AASHTO LRFD). The same time-dependent analysis was also performed on a simple span conventional precast/prestressed concrete bridge to compare the results using the current AASHTO prestress loss equations compared to the three loss models. CONSPLICE PT™ and Stability™ software programs were used for the design studies. Key construction stages and the comparison between prestress and post-tensioning losses over time and the critical beam stresses are highlighted using each of the three loss models independently.

Keywords: AASHTO LRFD Specifications, AASHTO Standard Specifications, Concrete Material Properties, Concrete Strength, CONSPLICE PT, Construction Stages, Dynamic Impact Effects, Factor of Safety, Future Wearing Surface, Lateral Beam Stability, Lifting, Post-Tensioning, Precast/Prestressed, Spliced-Girder Bridge, STABILITY, Superelevation Effects, Time-Dependent Analysis

INTRODUCTION

A finite element-based, time-dependent analysis using either of the three code specified loss models (e.g., ACI-209, CEB-FIP, and AASHTO LRFD) allows engineers to compare the analysis and design effects over time and more accurately take into account the variation in material behavior (e.g., increases in concrete strength and modulus of elasticity, creep and shrinkage effects, and prestress losses) over time.

A time-dependent analysis is important to owners in that a more accurate computation of camber and deflections can be obtained over time. Insight into the percent differences and the sensitivity between the loss model results as applicable to spliced girder bridges will be discussed. CONSPLICE PT™ [1] and Stability™ [2] software programs were used for the design studies.

Should a time-dependent analysis be performed on a conventional simple span precast/prestressed concrete girder? A simple span bridge was compared using the Standard AASHTO prestress loss equations[3] compared to the results of a time-dependent analysis using the ACI-209[4], CEB-FIP[5], and AASHTO LRFD models[6].

TIME-DEPENDENT MATERIALS

Concrete material properties vary with time, and various models have been developed by different code developing organizations to predict this behavior. The concrete material properties to be considered include:

- f'c = Concrete Compressive Strength
- fr = Modulus of Rupture
- E = Modulus of Elasticity
- S = Shrinkage Strain
- N = Creep Coefficient

It is well known that the behavior of the concrete material properties vary over time as shown in Figure 1. As shown, the concrete strength (f'c) varies and increases over time.

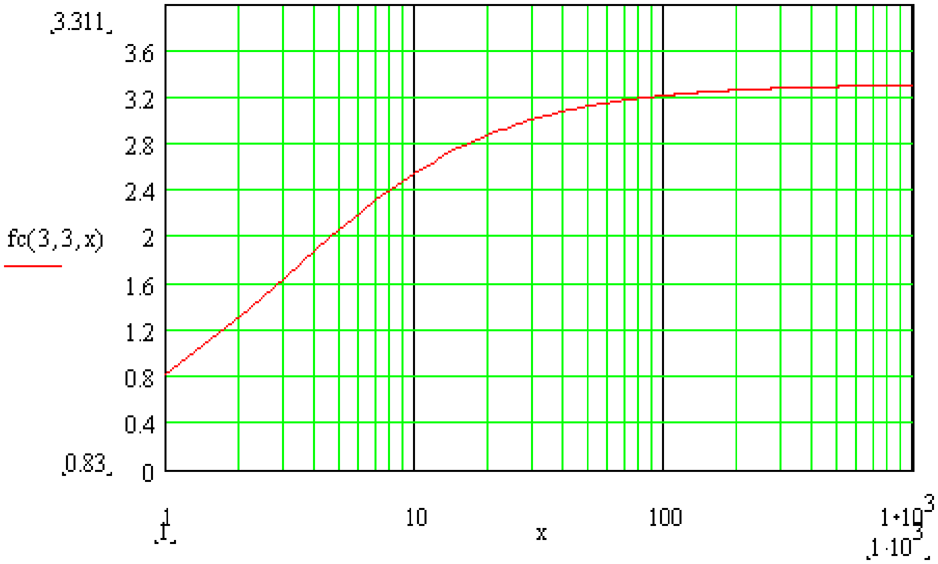

FIGURE 1 Variation of Concrete Strength (f'c) Over Time

SIMPLE SPAN CONVENTIONAL PRECAST/PRESTRESSED BRIDGE

The first design study using CONSPLICE PT[1] will be for a simple span conventional precast/prestressed bridge with the following design information:

- 40 ft. single span
- prestressing only (16-0.5" 270 ksi strands)
- precast double-T beam
- HS20 Live Load
- Construction stages, duration, elements, and applied loads (Table 1)

TABLE 1 Construction Stages for Conventional Prestressed Bridge

Stage #	Description	Duration (days)	Total Duration (days)	Elements Activated	Elements Removed	Loads
1	Construct abutments and install supports, Place Beams (beam age = 2)	0	0	supports/ beams	n/a	beam self wt.
2	Store beams on site	60	60	n/a	n/a	n/a
3	Time Step (form deck)	20	80	n/a	n/a	n/a
4	Pour Deck + Added DL	0	80	slab	n/a	slab self wt.
5	Time Step	30	110	n/a	n/a	n/a
6	Add Live Load	0	110	n/a	n/a	Live Load
7	Time Step	4000	4110	n/a	n/a	n/a
8	Add Future Wearing Surface + Live Load	0	4110	n/a	n/a	SDL + Live Load

Typically, the future wearing surface (FWS) is included in the analysis up front to account for any additional future load applied to the structure due to resurfacing maintenance over time, whereas in reality, the FWS load is not applied until some time into the future (i.e., after the bridge is open to traffic). What happens to the prestress losses when the FWS is applied in the future rather than when the bridge is open to traffic?

As Figure 2 indicates, at mid-span, the prestress force decreases from approximately 400 kips (stage 6) to 350 kips (stage 7). The AASHTO equation method[3] predicts a constant prestress force of approximately 400 kips over time, whereas the ACI-209[4], AASHTO LRFD [6], and CEB-FIP[5] loss models all indicate that there is an additional loss of prestress over time as shown in stages 7 and 8. These results also indicate that, for this example, using a time-dependent analysis with either loss model will predict more losses and less prestress force over time.

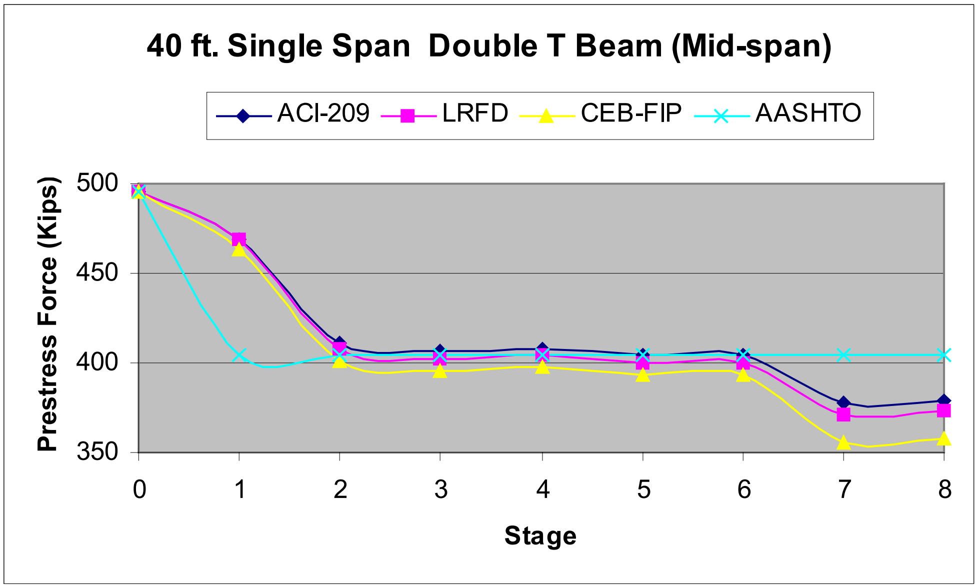

FIGURE 2 Prestress Force During Each Construction Stage (Simple Span Conventional Prestressed Bridge)

SPLICED-GIRDER PROJECT DETAILS

Example 1: Twisp River Bridge

Location: State of Washington

- 192 ft. single span
- 3 individual precast segments
- Temporary supports
- Combination prestressing and post-tensioning
- Option for 1- or 2-stage stressing sequence for post-tensioning
- Elevation view (Figure 3)

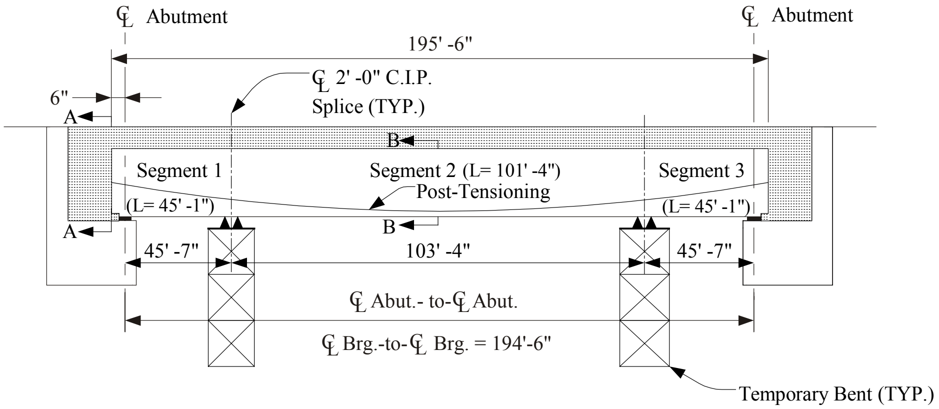

FIGURE 3 Elevation View of Twisp River Spliced-Girder Bridge

- Cross section details (Figure 4)

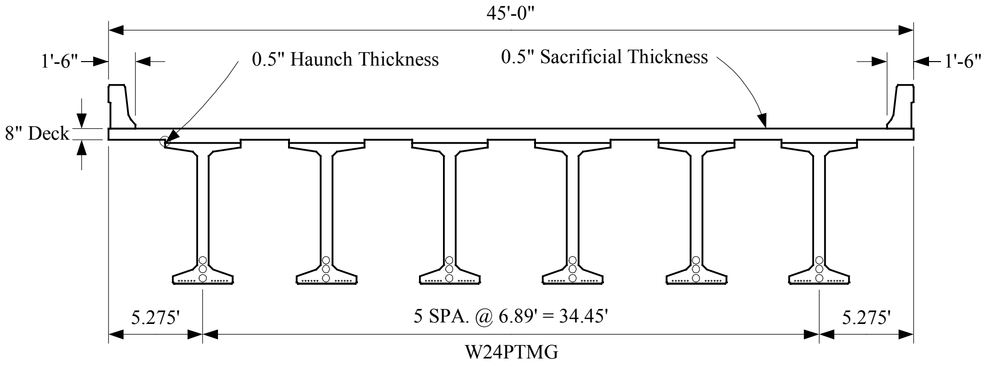

FIGURE 4 Cross Section of Twisp River Spliced-Girder Bridge

- Pre-tension and post-tension details (Figure 5)

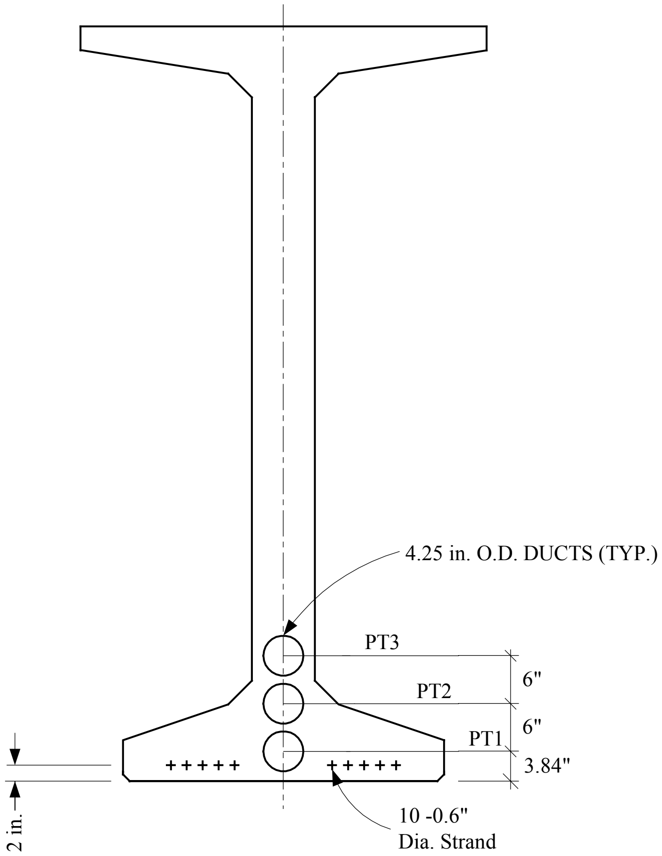

FIGURE 5 Pre-Tension & Post-Tension Details of Twisp River Spliced-Girder Bridge

- Construction stages, duration, elements, and loads (Table 2)

TABLE 2 Construction Stages for Twisp River Spliced-Girder Bridge

Stage #	Description	Duration (days)	Total Duration (days)	Elements Activated	Elements Removed	Loads
1	Construct abutments & install temporary bents, active beams	0	0	n/a	n/a	beam self wt.
2	Place Beams #1, 2, 3 in storage	60	60	n/a	n/a	n/a
3	Time Step (forming)	20	80	n/a	n/a	n/a
4	Pour Cast-in-Place Splices + Diaphragms	0	80	splice	n/a	self wt. Diaphragms
5	Time Step (splice curing)	10	90	n/a	n/a	n/a
6	Stress PT1 & PT2	0	90	tendons	strongback	n/a
7	Time Step (form deck)	20	110	n/a	n/a	n/a
8	Pour Deck + Supplemental Thickness 0.5”	0	110	slab + suppl.	n/a	self wt. of slab
9	Time Step (slab curing)	14	124	n/a	n/a	n/a
10	Stress PT3	0	124	tendons	n/a	n/a
11	Time Step	11	135	n/a	n/a	n/a
12	Remove Temporary Bents	0	135	n/a	temporary bents	n/a

TABLE 2 Construction Stages for Twisp River Spliced-Girder Bridge (Continued)

Stage #	Description	Duration (days)	Total Duration (days)	Elements Activated	Elements Removed	Loads
13	Add Superimposed Dead Loads (Barrier)	0	135	n/a	n/a	superimposed dead load
14	Time Step	30	165	n/a	n/a	n/a
15	Add Live Load + Uniform Temp + Temp Gradient	0	165	n/a	n/a	live load + temperature
16	Time Step	4000	4165	n/a	n/a	n/a
17	Sacrificial Wearing Surface	0	4165	sacrificial thickness	n/a	n/a
18	Time Step (Infinity) + Live Load	6000	10165	n/a	n/a	n/a
19	Deck Removal	0	10165	n/a	slab	n/a
20	Redeck + Live Load	0	10165	Redeck	n/a	n/a

The construction stages for the Twisp River Bridge are more involved than the previous example for a conventional precast/prestressed concrete girder. The spliced-girder stages as shown account for two stages of post-tensioning (see stages 6 and 10). There are a total of 3 tendons; PT1 and PT2 are stressed during stage 6 prior to the deck pour and the final PT3 tendon is stressed after the deck has cured during stage 10. If should be noted that the State of Washington showed two options for post-tensioning in the contract bid documents. The first option, as depicted in this example, utilizes two stages of post-tensioning. The second option, which was selected by the contractor, only utilized one stage of post-tensioning. Also, note that during stage 15, a uniform temperature and temperature gradient was included according to AASHTO specifications[3].

Figures 6 and 7 show the prestress and post-tensioning losses/force (at mid-span) over time using the three loss models independently. As shown, the prestress and post-tension losses/force and corresponding beam stresses are similar.

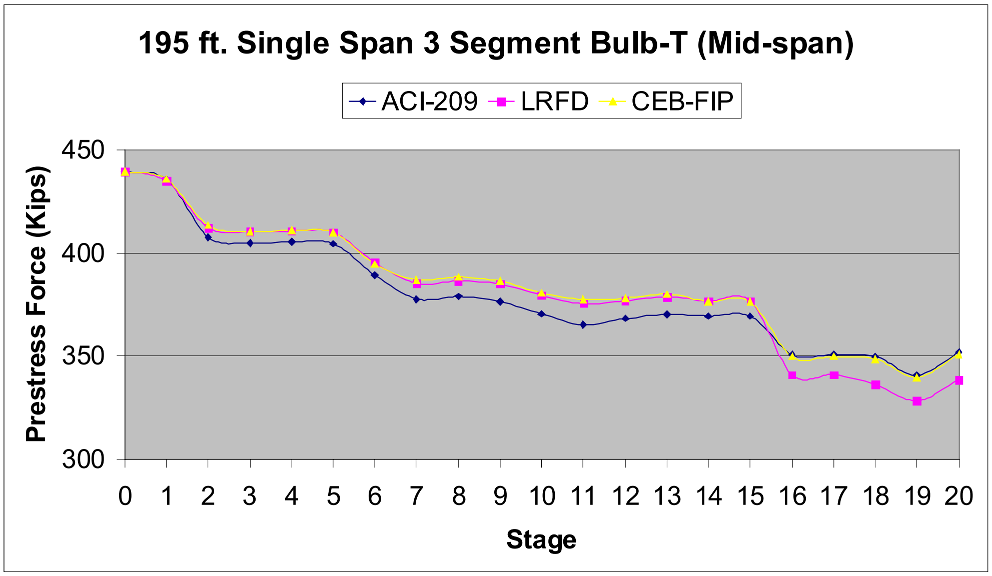

FIGURE 6 Prestress Force During Each Construction Stage (Twisp River Spliced-Girder Bridge)

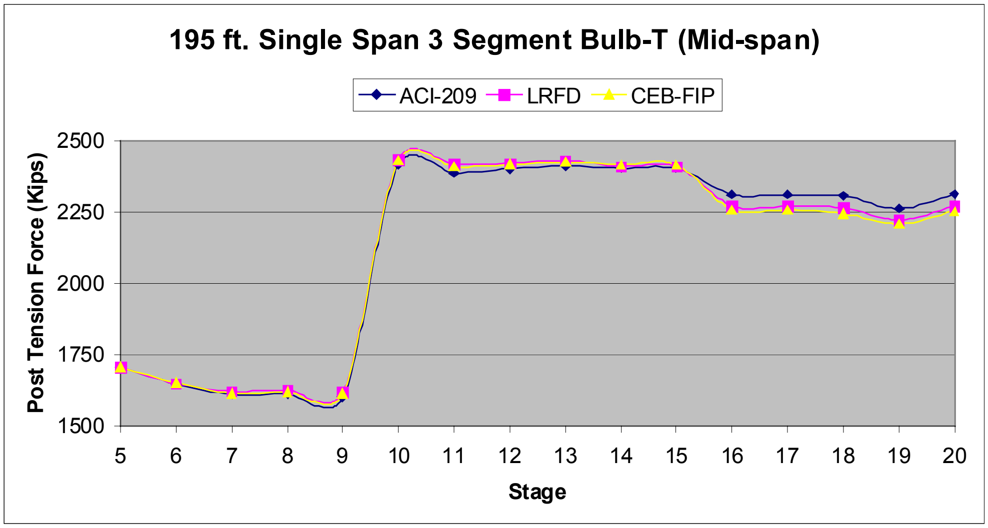

FIGURE 7 Post-Tension Force During Each Construction Stage (Twisp River Spliced-Girder Bridge)

SPLICED-GIRDER PROJECT DETAILS:

Example 2: Black Warrior Parkway Bridge (As Shown in Figure 8)

Location: State of Alabama

FIGURE 8 Black Warrior Parkway, An Example of a Spliced-Girder Bridge

- 3-span (208-260-208 ft) continuous with drop-in segment
- Combination prestressing and post-tensioning
- Variable depth 10 ft. pier segments using constant web height
- Elevation view (Figure 9)

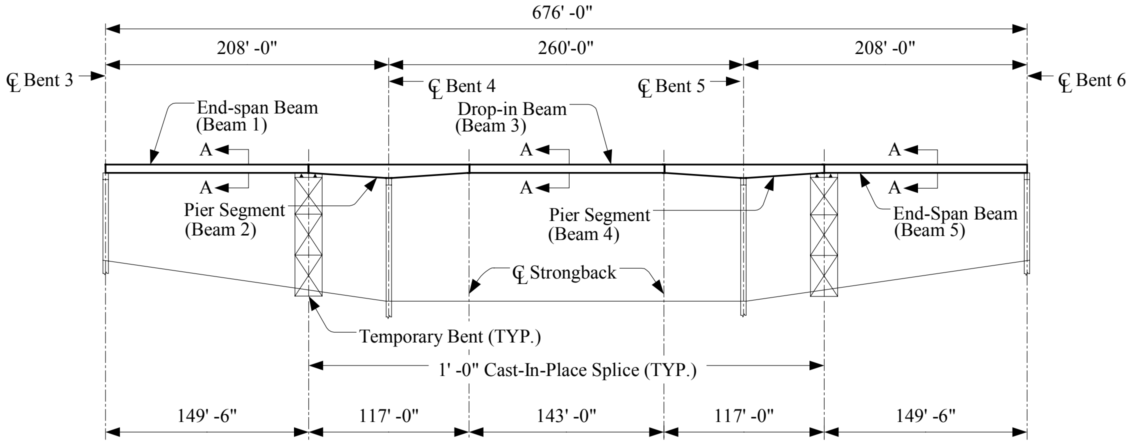

FIGURE 9 Elevation View of Black Warrior Parkway Spliced-Girder Bridge

- Cross section details (Figure 10)

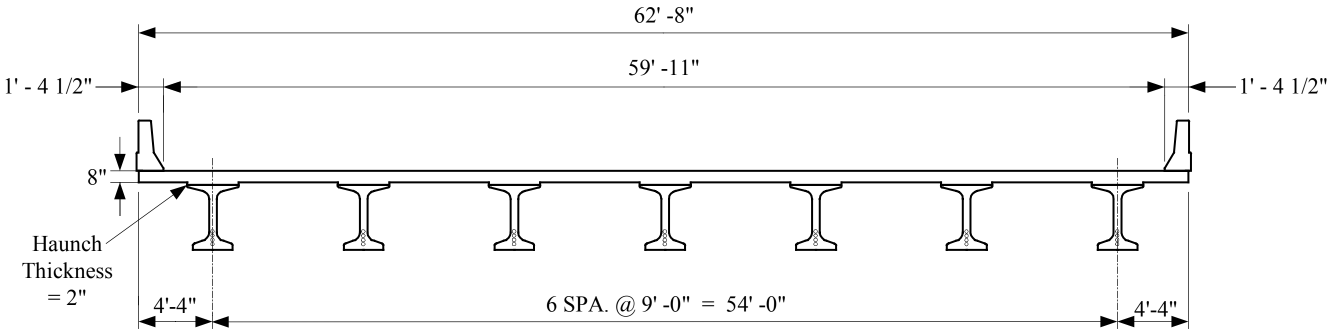

FIGURE 10 Cross Section of Black Warrior Spliced-Girder Bridge

- End-span and drop-in beam details (Figure 11)

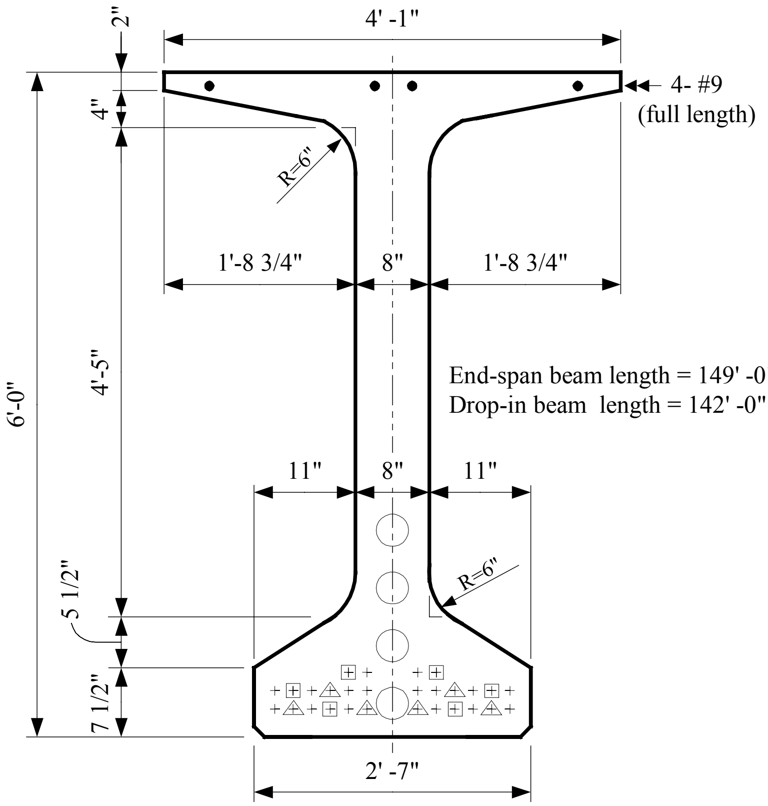

FIGURE 11 End-Span and Drop-In Beam Details (FLBT-72) of Black Warrior Spliced-Girder Bridge

- Construction stages, duration, elements, and loads (Table 3)

TABLE 3 Construction Stages for Black Warrior Parkway Spliced-Girder Bridge

Stage #	Description	Duration (days)	Total Duration (days)	Elements Activated	Elements Removed	Loads
1	Install temporary bents in end spans, add Pier Segment beams	0	0	supports/ beams	n/a	beam self wt.
2	Storage Time for Pier Segment Beams	20	20	n/a	n/a	n/a
3	Place End Span Beams (assumed time for cutting strands on these beams)	0	20	beam	n/a	beam self wt.
4	Time Step (storage of end beams)	20	40	n/a	n/a	n/a
5	Place Drop-In Beams (assumed time for cutting strands on this beam)	0	40	beam	n/a	beam self wt.
6	Time Step (storage of drop-in beam)	10	50	n/a	n/a	n/a
7	Pour Cast-In-Place Splices and Diaphragms	0	50	splice	n/a	dead load + diaphragms
8	Time Step (splice curing)	14	64	n/a	n/a	n/a
9	Stress PT1 & PT2 & PT3	0	64	tendons	strongback	n/a
10	Time Step (form deck using stay-in-place forms)	30	94	n/a	n/a	n/a
11	Pour Deck	0	94	slab	n/a	slab self wt.
12	Time Step (slab curing)	7	101	n/a	n/a	n/a

TABLE 3 Construction Stages for Black Warrior Parkway Spliced-Girder Bridge (Continued)

Stage #	Description	Duration (days)	Total Duration (days)	Elements Activated	Elements Removed	Loads
13	Stress PT4	0	101	tendons	n/a	n/a
14	Remove Temporary Bents	0	101	n/a	temporary bents	n/a
15	Add Superimposed Dead Loads (Barrier)	2	103	n/a	n/a	superimposed dead load
16	Time Step	30	133	n/a	n/a	n/a
17	Add Live Load	0	133	n/a	n/a	live load
18	Time Step	4000	4133	n/a	n/a	n/a
19	Add Future Wearing Surface + Live Load	0	4133	n/a	n/a	superimposed dead load + live load
20	Time Step	4000	8133	n/a	n/a	n/a
21	Full depth deck replacement/removal	0	8133	n/a	Deck	n/a
22	Pour Deck	0	8133	redeck	n/a	dead load
23	Time Step (infinity)	3000	11133	n/a	n/a	n/a
24	Live Load (infinity)	0	11133	n/a	n/a	live load

One important note to mention in looking at the construction stages is that a full depth deck replacement was considered in the analysis because of the two stages of post-tensioning. Since the last tendon (PT4) was added to the composite section (i.e., beam plus deck), engineers have to investigate the impacts of removing the deck in the future on beam stresses since there will be 4 post-tensioning tendons acting on the non-composite (i.e., beam only) section. Instead of plotting prestress losses/force over time at mid-span, the bottom of beam stresses are shown in Figure 12.

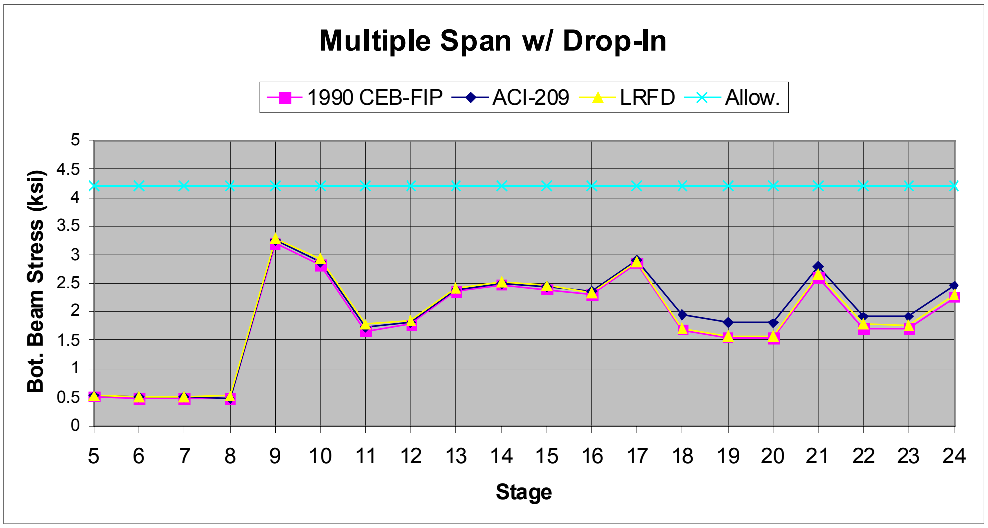

FIGURE 12 Bottom of Beam Stresses During Each Construction Stage (Black Warrior Parkway Spliced-Girder Bridge)

Bottom of Beam Stresses at Mid-Span (Compression Positive)

As a comparison, the allowable compressive stress of 4.2 ksi is shown in Figure 12. All computed/actual bottom of beam stresses over time and for every construction stage are less than the allowable.

It should be noted, that the controlling stage of construction on the final design may not necessarily be the final construction stage. This depends on the material properties, loads, and cross section (composite or non-composite). Also the design control could be due to shear, moment, or stresses for any construction stage, therefore a time-dependent, finite element, analysis provides an engineer this information.

LATERAL BEAM STABILITY

One very important design aspect that was not addressed in the AASHTO Standard specification[2] was lateral beam stability. AASHTO LRFD[5] now addresses lateral beam stability as stated in Article 5.5.4.3; "Buckling of precast members during handling, transportation, and erection shall be investigated."

The critical beam stresses and corresponding factors of safety associated with lateral beam stability shall be investigate for the following conditions:

- **Superelevation Effects**

  Lateral beam stability calculations take into account the superelevation effects. It is up to the engineer to design for a given cross slope during transportation and/or at the job site, as shown in Figure 13.

FIGURE 13 Superelevation Effects: Lateral Beam Stability (AASHTO LRFD Art. 5.5.4.3)

- **Dynamic Impact Effects**

The effects of dynamic impact (as shown in Figure 14) and the different stiffness conditions of the supports during transportation compared with lifting have to be accounted for in the lateral beam stability calculations. Also, the lateral unsupported beam length may differ during transportation compared with lifting depending on the location of the beam of the tracker and dollies.

FIGURE 14 Dynamic Impact Effects: Lateral Beam Stability (AASHTO LRFD Art. 5.5.4.3)

- **Lifting**

Some factors to consider during lifting include a different concrete strength at release compared with the 28-day strength, as shown at the job site. Stresses will be different if the beam is picked using two cranes with vertical lifting locations or using a single crane with an inclined pick (as shown in Figure 15).

FIGURE 15 Lifting Effects: Lateral Beam Stability (AASHTO LRFD Art. 5.5.4.3)

As shown in Figure 16, by varying the lifting loop location from the precast beam end, the factor of safety during lifting varies. Two results are apparent; one, that the F.S. decreases for a given lifting loop location as the beam length increases and, second, that the F.S. increases as the distance of the lifting loop increases from the beam end.

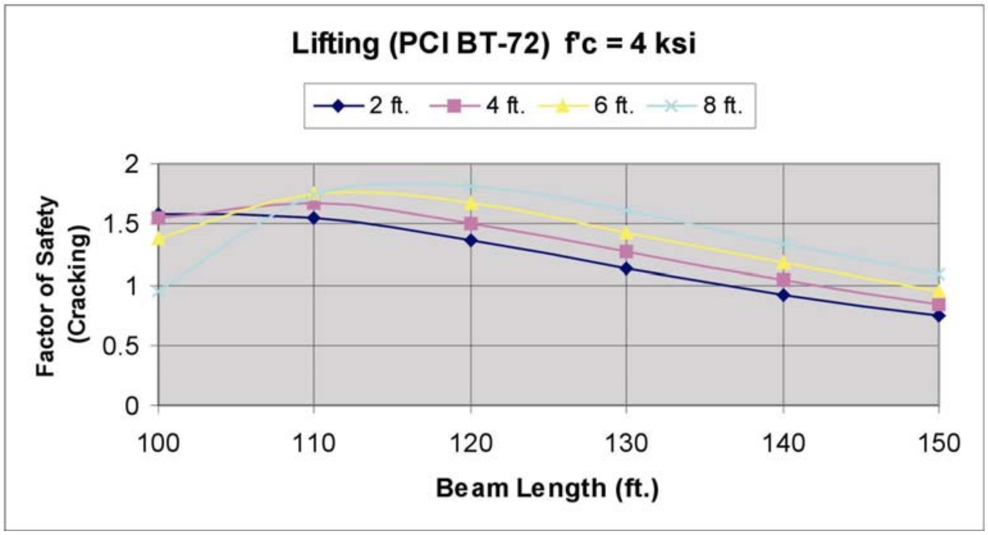

FIGURE 16 Variation of Lifting Loop Location on the Factor of Safety Against Cracking

Figure 17 shows the effect that concrete strength has on the factor of safety. For this example, the concrete strength at release is 4 ksi, and the 28-day strength is 7 ksi. Therefore for a 120 ft. beam length, assuming a PCI BT-72 section, the factor of safety varies from 1.5 (4 ksi) to 2.0 (7 ksi). The Stability™ software program[2] was used for this design study.

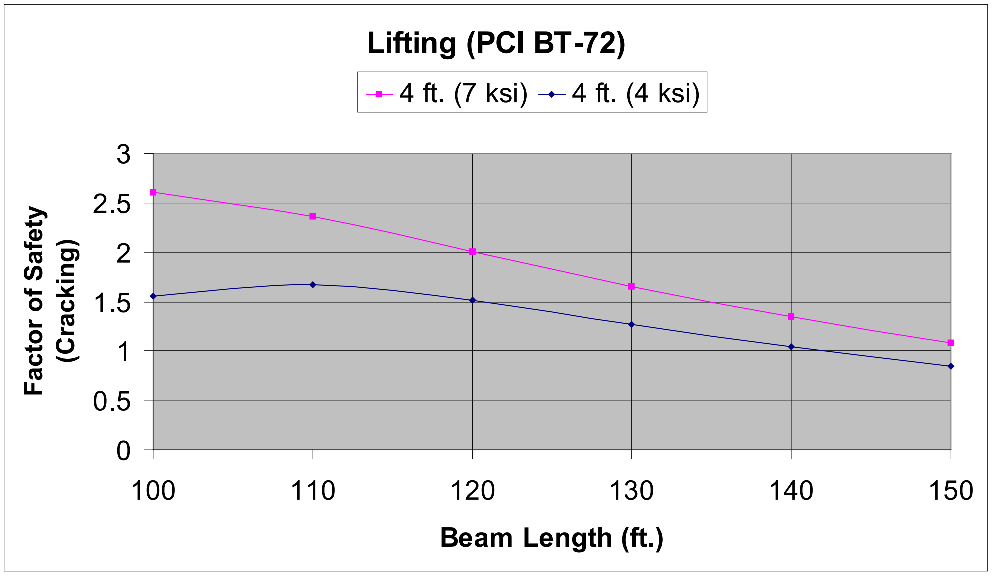

FIGURE 17 Variation of Concrete Strength on the Factor of Safety Against Cracking Conclusions

SUMMARY

A time-dependent analysis is warranted for spliced-girder bridges in order to more accurately account for the time-dependent material behavior and properties of concrete and for computing more accurate camber and deflections. For the Twisp River and Black Warrior Parkway spliced-girder bridges, the ACI-209[4], CEB-FIP[5], and AASHTO LRFD material models[6] produced similar prestress and post-tensioning loss results over time. The 40 ft. simple span conventional precast/prestressed concrete example study showed that a time-dependent analysis predicts more losses and less prestress force over time compared to the current AASHTO equations[3] for losses. Factors of safety and critical beam stresses shall be investigated for precast concrete beams to ensure that lateral beam stability is maintained in accordance with AASHTO LRFD specifications Article 5.5.4.3[6].

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