Deliverable #6: Final Report

Evaluate Effects from Shored Construction on Steel Composite Bridges
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16. Abstract

Steel composite girder bridges provide economic advantages and higher load-carrying capacity than noncomposite alternatives. Over the years, a conventional sequence of unshored construction for steel girder bridges has been developed. There is an alternative construction sequence, known as shored construction, it uses temporary supports (discretely or fully supported) for the superstructure during the installation of metal forms and the pouring of concrete. Shored construction also can be used in an Accelerated Bridge Construction (ABC) practice where construction occurs at a nearby site and fully-formed elements are transported to the final site using self-propelled modular transporters (SPMTs). In order to evaluate the effect of shored construction on steel composite girder bridges, the research team performed a comprehensive review on prior researches or guidelines for shored construction of composite steel girder bridges and international codes. The research team also developed and distributed survey questionnaires to a list of U.S. and international agencies. Based on the survey results, a summary report on steel composite girder bridges constructed with shored methods in U.S. and other countries was developed, and the construction and performance of identified bridges were reviewed. Finally, an analytical study procedure to evaluate the effects of using shored method for steel composite girder construction was developed, and a sensitivity study of varying parameters using the developed analytical study procedure were performed.

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EXECUTIVE SUMMARY

Steel composite girder bridges provide economic advantages and higher load-carrying capacity than noncomposite alternatives. Over the years, a conventional sequence of **<u>unshored</u>** construction for steel girder bridges has been developed. First, the steel girders and bracing members are erected using cranes and temporary towers. Temporary towers are removed once the erection is complete and metal stay-in-place deck forms are installed. Finally, the concrete deck is placed. Unshored construction relies on noncomposite steel framing to support its self-weight, the metal deck forms, and the poured concrete of the deck. After the concrete deck has cured and achieved sufficient strength, the composite section is responsible for the superimposed dead load and live load. An alternative construction sequence, known as **<u>shored</u>** construction, uses temporary supports (discretely or fully supported) for the superstructure during the installation of metal forms and the pouring of concrete. Shored construction also can be used in an Accelerated Bridge Construction (ABC) practice where construction occurs at a nearby site and fully-formed elements are transported to the final site using self-propelled modular transporters (SPMTs). A pre-decked steel girder is another example of fully-supported construction that offers an advantage for ABC projects.

Although shored construction improves structural efficiency since the composite section resists all loads and is permitted in the United States, Article C6.10.1.1 of AASHTO Bridge Design Specification (AASHTO, 2017) states that "its use is not recommended". In AASHTO Bridge Design Specification, 7th edition (LRFD-7, AASHTO, 2014), it is stated that "there have been **_<u>no known</u>_** significant demonstration bridges built with shored construction in the U.S.". However, in the 8th edition (LRFD-8, AASHTO, 2017), it is stated that "there have been only **_<u>a limited number</u>_** of demonstration bridges built with shored construction in the U.S.". This change in Article C6.10.1.1 of AASHTO Bridge Design Specification indicates that there are existing demonstration bridges built with shored construction in the U.S. The research team was able to identify a number of demonstration bridges built with shored construction or pre-decked steel girder system. A detailed list of the identified bridges can be found in Table 5-4.

RESEARCH PROBLEM AND BACKGROUND

There are several concerns that hinder the wide implementation of shored construction for steel composite girders. These concerns include:

1. Most of the deck load is carried by the composite section, thus inducing large forces in the shear connectors. Shored composite girder bridges constructed in Germany didn't retain composite action (Grubb et al., 2015; AASHTO, 2017).
2. Time-dependent effects (creep and shrinkage of concrete) may contribute to a loss of composite action resulting in increased deflection and decreased section capacity. Article C6.10.1.1 of LRFD-8 also stated that "there has been limited research on the effects of concrete creep on composite steel girders under large dead load."
3. Increased deck tensile stresses at the intermediate support locations of continuous girders.
4. Tight camber tolerances are required for shored construction.

In light of these concerns, and prior to the widespread adoption of shored construction for bridges in Florida, there is an urgent need to investigate the behavior and performance of composite steel girder bridges using shored construction.

RESEARCH OBJECTIVES

In order to evaluate the effects of shored construction on the performance of steel composite bridges, the research team proposes a framework (Figure 1) to achieve the following objectives:

1. Understand the behavior and performance of composite steel composite girder bridges using shored construction (including pre-decked steel girders).
2. Review projects using shored construction or pre-decked steel girders to assess their design approach, detailing, construction methods, and performance history.
3. Perform a comprehensive analytical study to determine the effects and economic benefits of using shored construction for steel composite girders.
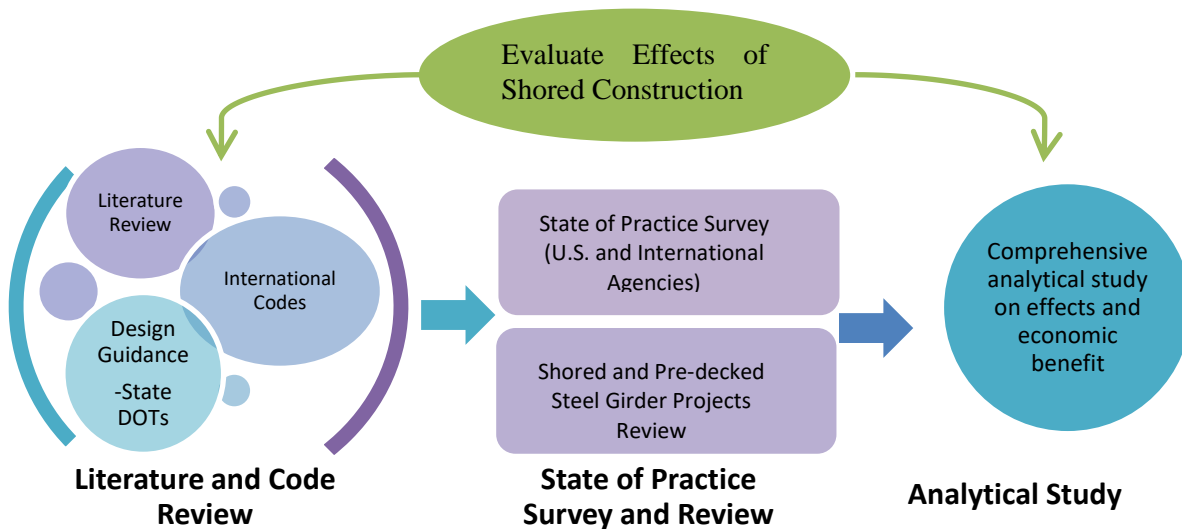
Figure 0-1. Proposed framework

The outcomes of this research effort include: (1) a comprehensive review on prior researches or guidelines for shored construction of composite steel girder bridges and international codes; (2) survey questionnaires and the distribution lists for the U.S. and international surveys and the results of the survey; (3) a summary report on steel composite girder bridges constructed with shored methods in the U.S. and other countries; (4) a review of construction and performance of identified bridges; (5) an analytical study procedure to evaluate the effects of using shored methods for steel composite girder construction; and (6) a sensitivity study of varying parameters using the developed analytical study procedure.

CONCLUSIONS AND RECOMMENDATIONS

Based on the U.S. and international questionnaire survey, the following conclusions were drawn:

1. Most of the states don't have prior experiences in shored construction.
2. The major concerns the agencies have about shored construction are:
   a. Re-decking
   b. Cost
   c. Creep and shrinkage

3. Michigan, Virginia, and West Virginia had successful experiences in shored construction. Shored constructed bridges either demonstrated less deflection or were designed with a smaller section.

The research team also reviewed the as-built construction drawings, specifications, and in-service performance information collected from respective state agencies. In addition, the inspection reports were obtained from Michigan and West Virginia. Based on the information collected, the construction and performance of these shored constructed bridges were reviewed and the findings are:

1. For shored construction in the field, the shoring methods vary among steel members, timber mat footings, pile foundations, or temporary bent. The shoring method used by the Virginia Department of Transportation (VDOT) for ABC bridges is shoring at quarter points until the concrete has attained a strength of 26.25 MPa (3,800 psi) at a nearby casting yard. There is no major construction issue with shored construction.
2. Shored constructed bridges in Michigan showed comparable performance in comparison with unshored constructed bridges of similar ages.
3. The shored constructed bridge in West Virginia demonstrated promising results as the shored constructed southbound twin showed less dead load deflection. The inspection report also proved that the shored southbound twin performs better compared to the unshored twin bridge.

The findings of the analytical study are summarized below:

(1) For a single-span steel composite girder bridge, there is a substantial reduction in the size of steel sections if the bridge girder is fully shored. If steel girder is prefabricated with the fully shored condition, a much lighter section can be used, resulting in possible savings in materials and transportation.
(2) For a three-span continuous bridge, it is observed that the reduction in steel section area averaged 9% for the positive moment region and over 17% for the negative moment region. This indicates shoring has a more significant effect on the negative moment region in comparison with the positive moment region.

(3) Based on the cost-benefit analysis, it can be concluded that only a fully shored prefabricated bridge unit will be able to provide substantial cost savings considering the relatively higher price tag for shoring.

(4) Based on the Finite Element (FE) results, it is observed that shored construction reduces the stress level significantly. It is also observed that without following the FDOT pouring sequence, the stress maintains at a similar level if the bridge was shored at 1/3 points of center span and ½ points of side spans.

(5) For bridges with the unshored condition, design values based on the 3n assumption underestimated the stress level compared with FE results (11% to 16% lower). However, for shored constructed bridges, design values based on the 3n assumption overestimated the stress level compared with FE results with a larger difference. The difference is larger for shored constructed bridges because 3n assumptions are not originally made for shored constructed steel composite girder bridges.

(6) For the three-span continuous bridge, shored construction has a more significant effect on support locations when compared with the midspan location. After the temporary shoring is removed, the reaction forces applied at the temporary shoring location cause larger strain to develop at both the positive and negative moment region. Particularly, the tensile strain on top of the concrete slab at intermediate supports exceeds 131 $\mu\varepsilon$, indicating that the concrete cracked at these intermediate supports.

(7) The long-term deflection for the unshored condition is higher than the one with the shored condition. However, the difference becomes smaller as the loading duration gets longer.

(8) Based on the FE analysis, the longitudinal stress developed on top of the steel beam is much less than the design strength of the shear studs. Thus, the composite action should hold after the temporary supports were removed.

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1 INTRODUCTION

Steel composite girder bridges provide economic advantages and higher load-carrying capacity than noncomposite alternatives. Over the years, a conventional sequence of **<u>unshored</u>** construction for steel girder bridges has been developed. First, the steel girders and bracing members are erected using cranes and temporary towers. Temporary towers are removed once the erection is complete, and metal stay-in-place deck forms are installed. Finally, the concrete deck is placed. Unshored construction relies on noncomposite steel framing to support its self-weight, the metal deck forms, and the poured concrete of the deck. After the concrete deck has cured and achieved sufficient strength, the composite section is responsible for the superimposed dead load and live load. An alternative construction sequence known as **<u>shored</u>** construction uses temporary supports (discretely or fully supported) for the superstructure during the installation of metal forms and the pouring of concrete. Shored construction also can be used in an Accelerated Bridge Construction (ABC) practice where construction occurs at a nearby site and fully-formed elements are transported to the final site using self-propelled modular transporters (SPMTs). A pre-decked steel girder is another example of fully-supported construction that offers an advantage for ABC projects.

Although shored construction improves structural efficiency because the composite section resists all loads and is permitted in the United States, Article C6.10.1.1 of AASHTO LRFD Bridge Design Specification 8th Edition (LRFD-8) (AASHTO, 2017) states that "its use is not recommended." The newly published AASHTO LRFD Bridge Design Specification 9th Edition (LRFD-9) (AASHTO, 2020) retains this stance: "While shored construction is permitted… its use is not recommended." There are several concerns that hinder the wide implementation of shored construction for steel composite girders. These concerns include:

1. Most of the deck load is carried by the composite section, thus inducing large forces in the shear connectors (Grubb et al., 2015, AASHTO, 2017). However, LRFD-9 (AASHTO, 2020) has removed the provision related to shear connectors and composite actions.

2. Time-dependent effects (creep and shrinkage of concrete) may contribute to a loss of composite action resulting in increased deflection and decreased section capacity.

Article C6.10.1.1 of LRFD-8 and LRFD-9 also stated that "there has been limited research on the effects of concrete creep on composite steel girders under large dead load".

3. Increased deck tensile stresses at the intermediate support locations of continuous girder (AASHTO, 2020).
4. Tight camber tolerances are required for shored construction (AASHTO, 2020).

In light of these concerns, the research team conducted a thorough literature search to find the most relevant literature. A comprehensive review of prior research or guidelines for shored construction of composite steel girder bridges and international codes was performed. The research team also developed and distributed survey questionnaires to a list of U.S. and international agencies. Based on the survey results, a summary report on steel composite girder bridges constructed with shored methods in the U.S. and other countries was developed and the construction and performance of identified bridges were reviewed. Finally, an analytical study procedure to evaluate the effects of using shored methods for steel composite girder construction and a sensitivity study of varying parameters using the developed analytical study procedure was performed.

2 LITERATURE REVIEW

The research team conducted a detailed literature review on various topics related to this research project including these key topics: The current state of practice on design analysis of shored constructed steel composite girders; time-dependent effects from shored construction; shear connections in shored construction; deck tensile stresses at intermediate supports; camber tolerances; shored construction in ABC; pre-decked steel girder; and FE analysis of non-linear behavior, time-dependent effects, and cracking of concrete. The findings of the literature review are summarized below:

2.1 THE CURRENT STATE OF PRACTICE ON SHORED CONSTRUCTED STEEL COMPOSITE GIRDERS

The literature review revealed that only limited research has been completed on the design and analysis of shored construction steel composite girders.

2.1.1 AASHTO and FHWA

In AASHTO LRFD-9 (AASHTO, 2020), provisions related to composite sections are presented in article 6.10.1. Regarding sequence of loading in 6.10.1.1.1a under 6.10.1.1.1-stresses, both unshored and shored construction were specified: "For unshored construction, permanent load applied before the concrete deck has hardened or is made composite shall be assumed carried by the steel section alone; permanent load and live load applied after this stage shall be assumed carried by the composite section. For shored construction, all permanent load shall be assumed applied after the concrete deck has hardened or has been made composite and the contract documents shall so indicate." However, in C6.10.1.1.1a, it is stated "While shored construction is permitted according to these provisions, its use is not recommended. Also, these provisions may not be sufficient for shored construction where close tolerances on the girder cambers are important. " It also stated "There has been limited research on the effects of concrete creep on composite steel girders under large dead loads. There have been only a very limited number of demonstration bridges built with shored construction in the U.S. Furthermore, there is an increased likelihood of significant tensile stresses occurring in the concrete deck at permanent support points when shored construction is used". Compared to LRFD-8 (AASHTO, 2017), the statements

"Unshored construction generally is expected to be more economical" and "Shored composite bridges that are known to have been constructed in Germany did not retain composite action" have been removed.

Besides article 6.10.1, there are other provisions related to shored construction in LRFD-9:

Article C6.10.1.7 on minimum longitudinal deck reinforcement: To prevent nominal yielding of longitudinal deck reinforcement and control concrete deck cracking, the use of longitudinal deck reinforcement with a specified minimum yield strength not less than 60 ksi may be taken for shored construction where the steel section utilizes steel with a specified minimum yield strength less than or equal to 50 ksi in either flange.

Article 6.10.4.2.2 and C6.10.4.2.2 on flexure permanent deformations under service limit state: For compact composite sections in positive flexure utilized in shored construction, the longitudinal compressive stress in the concrete deck due to the Service II loads, determined as specified in Article 6.10.1.1.1d, shall not exceed 0.6f'c. This is to ensure the linear behavior of the concrete.

Article C6.10.6.2.2 and C6.11.6.2.2: Compact composite sections in positive flexure must also satisfy the provisions of Article 6.10.7.3 to ensure a ductile mode of failure. Noncompact sections must also satisfy the ductility requirement specified in Article 6.10.7.3 to ensure a ductile failure. Satisfaction of this requirement ensures an adequate margin of safety against the premature crushing of the concrete deck for sections utilizing up to 100 ksi steels and/or for sections utilized in shored construction.

Article C6.10.7.2.1 on noncompact sections: The longitudinal stress in the concrete deck is limited to $0.6\,f_c'$ to ensure linear behavior of the concrete which is assumed in the calculation of the steel flange stresses for noncompact sections. This condition may govern for shored construction with geometries causing the neutral axis of the short-term and long-term composite section to be significantly below the bottom of the concrete deck.

In the steel bridge design handbook published by the FHWA Office of Bridges and Structures (White, 2015), the aforementioned discussions were also mentioned. This document states that unshored construction is generally considered to be more economical, but the overall discussion on the matter is limited.

Culmo (2011), Culmo et al. (2013a, 2013b) published several manuals for accelerated bridge construction. It is stated that temporary shoring should be designed using the AASHTO Guide Design Specifications for Bridge Temporary Works and the AASHTO LRFD Bridge Construction Specifications. However, only temporary shoring for transport of Prefabricated Bridge Elements and Systems was discussed.

Publication G13.1 Guidelines for Steel Girder Bridge Analysis, by the AASHTO/NSBA Steel Bridge Collaboration (AASHTO/NSBA, 2011), states that shored construction of steel girder bridges is rarely if ever, undertaken and is generally discouraged. However, the reason for such a statement was not specified in the guideline.

In the NHI LRFD for Highway Bridge superstructures reference manual (Grubb et al., 2015), section 6.4.2.2 offered additional discussion regarding unshored vs. shored construction. The authors discussed another situation that can be considered shored construction, at least to a certain degree, which is the re-decking of a bridge under traffic. During re-decking, some of the girders are composite when the deck load is added to the adjacent girders. When cross-frames are connecting the composite and noncomposite girders, the bridge acts as shored constructed to a certain degree. In addition, the disadvantage of shored composite construction was also discussed. They stated that the major disadvantage of shored composite construction is that most of the dead load is carried by the composite section, which puts large forces in the shear connectors and the concrete deck, and increases deflections due to the creep of the concrete. This increased deflection might affect the rideability of the bridge over time and tends to put much of the stress saved in the original design back into the steel girders. Since it is difficult to predict the amount of creep, shored composite construction is not popular in bridge construction. They also discussed camber for shored construction. The camber is often very high at the time of construction if girders are cambered for final elevation. If they are not cambered properly for creep, the roadway may deflect too much as the structure ages. However, no analytical or experimental results were provided to support the aforementioned discussions.

2.1.2 <u>State Transportation Agencies</u>

The research team also did a search on the current state of practice among various state agencies in the United States. Only limited information was found during this search. However,

more information regarding shored construction practices was collected through a comprehensive survey. Preliminary observations by State include the following:

Florida: The Florida Department of Transportation does not specify the use of shored construction in its structure design guideline (FDOT, 2021).

California: Provisions from AASHTO LRFD related to shored construction were used. However, no design details or examples were provided for shored constructed bridges.

Georgia: The Georgia Department of Transportation does not specify the use of shored construction in its bridges and structures design manual (GDOT, 2019).

North Carolina: The NC Department of Transportation does not specify the use of shored construction in its structures management unit manual (NCDOT, 2020).

South Carolina: The SC Department of Transportation does not specify the use of shored construction in its bridge design manual (SCDOT, 2006).

Wisconsin: The Wisconsin Department of Transportation Bridge Manual (WisDOT, 2013) states: "temporary shoring is not used in Wisconsin".

Louisiana: Design and detailing of shored construction shall not be used by the designer unless prior approval by the Bridge Design Engineer Administrator is granted. In C6.10.1.1.1a of the Louisiana Department of Transportation and Development (LaDOTD) Bridge Design Manual (LaDOTD, 2014), it is stated that LADOTD concurs that this practice is not recommended and would allow shored permanent construction only in unique circumstances. Other states have utilized shoring of this type with prestressed concrete girders with success; however, the importance of the shoring being placed and maintained at critical loading and elevation levels is such that construction can be complicated with no easy method for correction if problems occur.

Pennsylvania: The use of shored system requires the prior approval of the Chief Bridge Engineer (PennDOT, 2019).

Alaska: Design steel superstructures without intermediate falsework during the placing of the concrete deck slab. Shored construction is not permitted (ALDOT, 2017).

Nebraska: The Nebraska Department of Transportation does not specify the use of shored construction in its structure design guideline (Nebraska DOT, 2017).

Texas: In a study performed for the Texas Department of Transportation, Hueste et al. (2016) concluded that (1) unshored construction (no shoring towers) is preferred because it saves significant time during construction and reduces the construction costs, and (2) the required footprint for temporary shore towers is typically not available. Freeby (2005) developed two new prefabricated bridge superstructure systems for TxDOT including one steel tub-girder and a prestressed concrete pre-topped U-beam. Both systems were developed for maximum span lengths of 115 ft and a total super structure depth of 39 in. In order to achieve a shallow superstructure depth, the beams were designed to be shored during the placement of the concrete deck to make them composite for all loads. After slab placement, the beam will be hauled to the bridge site and erected on the piers/abutments. The final adoption and use of this new prefabricated bridge super structure system were not presented in this paper.

2.2 <u>TIME-DEPENDENT EFFECTS IN SHORED CONSTRUCTION</u>

The literature search revealed that very little has been published to address the time-dependent effects of shored construction. Article C6.10.1.1 of AASHTO Bridge Design Specification (AASHTO, 2017) also states that "there has been limited research on the effects of concrete creep on composite steel girders under large dead load".

Creep is a time-dependent effect due to permanent loads applied to the structure. AASHTO Article 6.10.1.1.1a (AASHTO, 2017) addresses the influences of creep on the steel stresses by transforming the elastic concrete section into an equivalent steel section with a *3n* modular ratio. Oehlers and Bradford (1999) discussed the accuracy of this type of approximation. AASHTO (AASHTO, 2017) Article 6.10.1.1.1d specifies the short-term modular ratio $n=E_s/E_c$ for calculation of longitudinal flexural stresses in the concrete for determining where sufficient longitudinal reinforcement should be provided in the concrete deck to control cracking (AASHTO Articles 6.10.3.2.4 and 6.10.1.7). AASHTO (AASHTO, 2017) Article C6.10.1.1.1a also indicates that the above method for handling creep effects may not be appropriate for shored construction where close tolerances on the final camber of the girder are important. Shrinkage is another time-dependent effect that affects structural behavior. Tests have indicated that the shrinkage strain of

the slab in composite beams may be taken as *0.0002* and the corresponding stress in steel can be estimated as an eccentrically loaded column with a load of *0.0002E_{cn}A_c* (Viest et al., 1958). Frank (2005) concluded that a refined analysis of shrinkage effects also may be important if the structure requires close tolerances on girder cambers.

During this literature review, various shrinkage and creep models (e.g. Bazant (1972), CEB-FIP (1993), and AASHTO (2017)) have been reviewed. These models and other models presented in this section have been evaluated for the modeling of long-term effects on the concrete deck. Kwak and Seo (2000) developed an analytical model to predict the long-term behavior of composite girder bridges. The proposed model considered the effects of creep and shrinkage of concrete and the cracking of concrete slabs in the negative moment regions. Based on the principle of superposition, total uniaxial concrete strain $\varepsilon_c(t)$ at any time t is assumed to be composed of the mechanical strain $\varepsilon_c^m(t)$ caused by short-term service loads and the non-mechanical strain $\varepsilon_c^{nm}(t)$ composes of creep strain $\varepsilon_c^{cr}(t)$, and shrinkage strain $\varepsilon_c^{sh}(t)$.

$$\varepsilon_c(t) = \varepsilon_c^m(t) + \varepsilon_c^{nm}(t) = \varepsilon_c^m(t) + \varepsilon_c^{cr}(t)\varepsilon_c^{sh}(t) \qquad \text{Eq. (2-1)}$$

The shrinkage strain was calculated using the shrinkage model in ACI 318-89 (ACI Committee 318, 1989). The creep strain was modeled by the first-order algorithm based on the expansion of creep compliance proposed by Kabir (1977).

The analytical model was developed based on a layered approach and matrix analysis. Figure 2-1 shows the layered section for a composite beam. The accuracy of the analytical model was validated with experimental results. As shown in Figure 2-2, the analysis shows that both the deflection and deflection ratio for the cracked composite bridge by unshored construction is the largest, where δ_t is the long-term midspan deflection and δ_e is the instantaneous elastic deformation.
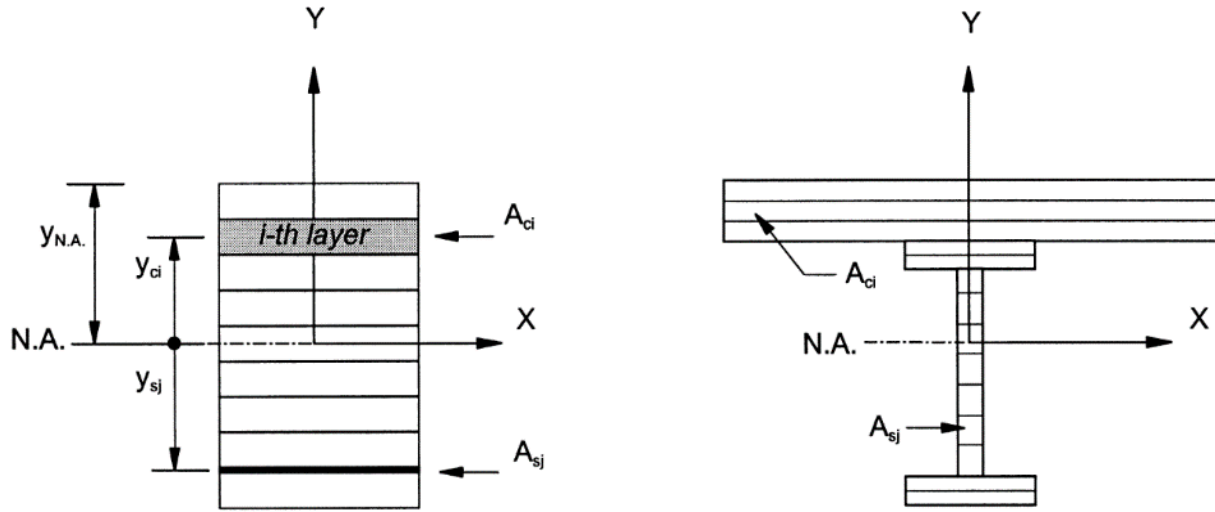
Figure 2-1. The layered section (Kwak and Seo, 2000)
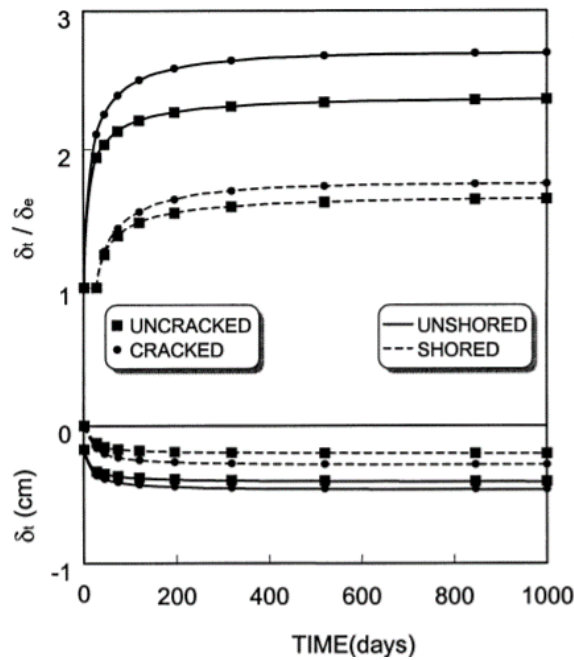
Figure 2-2. Midspan deflection vs. time (Kwak and Seo , 2000)

In the concrete deck of steel-concrete composite bridges, time-dependent shrinkage and creep effects can lead to a significant redistribution in bending moment at continuity supports as well as increase deflections. Chaudhary et al. (2009) developed a hybrid procedure to model the effect of concrete cracking and time-dependent effects of creep and shrinkage in composite beams. In this study, the age-adjusted effective modulus method (Bazant, 1972) was used for predicting creep and shrinkage effects. For a cross-section in the un-cracked zone, the total curvature ρ_{un}^t, the total

top fiber strain ε_{un}^{t}, and the total top fiber stress σ_{un}^{t} at the end of the time interval are obtained by adding the changes to their instantaneous values, respectively, as:

$$\rho_{un}^{t} = \rho_{un}^{it} + \Delta\rho_{un}^{c} + \Delta\rho_{un}^{s} + \Delta\rho_{un}^{id} \qquad \text{Eq. (2-2)}$$

$$\varepsilon_{un}^{t} = \varepsilon_{un}^{it} + \Delta\varepsilon_{un}^{c} + \Delta\varepsilon_{un}^{s} + \Delta\varepsilon_{un}^{id} \qquad \text{Eq. (2-3)}$$

$$\sigma_{un}^{t} = \sigma_{un}^{it} + E_{e}\left(\Delta\varepsilon_{un}^{c} + \Delta\varepsilon_{un}^{s} - \phi\Delta\varepsilon_{un}^{it} - \varepsilon^{sh}\right) + \Delta\sigma_{un}^{id} \qquad \text{Eq. (2-4)}$$

where the superscript "it", "c", and "s" indicate the instantaneous, creep induced, and shrinkage induced value of a quantity, respectively. Superscript "id" indicates the quantity that arises in indeterminate structures due to the redistribution of forces caused by creep and shrinkage. E_{e} is the age-adjusted elastic modulus, ε^{sh} is the strain in unrestrained concrete due to creep and shrinkage. ϕ is the creep coefficient.

Considering the effect of creep and shrinkage in a cross-section in the cracked zone, the total curvature ρ_{ts}^{t} and the total top fiber strain ε_{ts}^{t} at the end of the time interval are given as:

$$\rho_{ts}^{t} = \eta\left(\rho_{un}^{it} + \Delta\rho_{un}^{c} + \Delta\rho_{un}^{s} + \Delta\rho_{un}^{id}\right) + \xi\left(\rho_{cr}^{it} + \Delta\rho_{cr}^{id}\right) \qquad \text{Eq. (2-5)}$$

$$\varepsilon_{ts}^{t} = \eta\left(\varepsilon_{un}^{it} + \Delta\varepsilon_{un}^{c} + \Delta\varepsilon_{un}^{s} + \Delta\varepsilon_{un}^{id}\right) + \xi\left(\varepsilon_{cr}^{it} + \Delta\varepsilon_{cr}^{id}\right) \qquad \text{Eq. (2-6)}$$

where the subscript "ts" indicates that the tension stiffening effect has been taken into account, ξ is the interpolation coefficient (CEB-FIP, 1993), $\eta = 1 - \xi$, subscript "cr" indicates that the quantity is taken from a cracked section.

Nassif et al. (2008) uses the time-dependent shear and volumetric behavior of viscoelastic material to simulate the decay function of the material under constant stress or strain. The time-dependent variables can be represented in terms of a Prony Estimation series given below:

Shear Behavior:
$$g_{R}(t) = 1 - \sum_{i=1}^{N} \overline{g}_{i}^{P}\left(1 - e^{-t/\tau_{i}^{G}}\right) \qquad \text{Eq. (2-7)}$$

where N, $\overline{g}_{i}^{P}$, and τ_{i}^{G}, $i = 1, 2, \ldots, N$ are material constants.

Volumetric Behavior:
$$p = -K_{0}\left(\varepsilon^{vol} - \sum_{i=1}^{N} \varepsilon_{i}^{vol}\right) \qquad \text{Eq. (2-8)}$$

where K_0 is a material constant and assuming $\tau_i^G = \tau_i^K = \tau_i$.

$$\varepsilon_i^{vol} = \frac{\overline{k}_i^P}{\tau_i^K} \int_0^t e^{-s/\tau_i^K} \varepsilon^{vol}(t-s)\,ds \qquad\qquad \text{Eq. (2-9)}$$

Both of these equations are simply a summation of a series of exponential decays that can be used to approximate the creep properties of viscoelastic materials. Although concrete is not exactly a viscoelastic material, the Prony series provide a good approximation of creep behavior for concrete without having to develop a constitutive model. For shrinkage properties, the creep behavior is dominated by volumetric creep. Hence, only the volumetric behavior was considered in the model. As mentioned earlier, the viscoelasticity property can only be used to describe the creep behavior of concrete; the shrinkage data needs to be calibrated by back calculating the constant instantaneous stress acting on the concrete, which will cause the concrete to shrink. This is done through the use of Eq. (9), by substituting ε^{vol} to the strain at 1-day of drying from the free shrinkage result. The 1-day modulus of elasticity was used for the computation of the bulk modulus of elasticity, K_0.

Chaudhary et al. (2009) applied the hybrid procedure developed by Chaudhary et al. (2007) and Single-, three-, and five-span models were analyzed for different thicknesses and grades of concrete (Figure 2-3). Both shored and unshored construction procedures were taken into account. This paper focuses on the effects of delaying the time of mobilization of composite action between the steel section and the precast concrete deck. Creep and shrinkage in deck panels only affect the behavior of composite bridges once the shear connectors have been installed, mobilizing composite action.
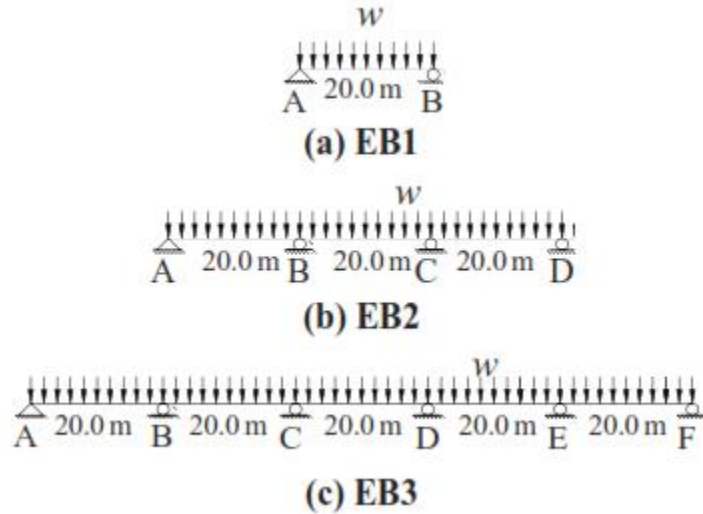
Figure 2-3. Bridges: (a) EB1, (b) EB2, and (c) EB3 (Chaudhary et al., 2009)

A numerical study was performed to evaluate the effects of creep and shrinkage of composite bridges from both shored and unshored construction. All three bridges were subjected to a uniformly distributed service load of 40 kN/m. In the shored construction, it is assumed that the load gets applied at the same time and is resisted by the composite section. For unshored construction, it is assumed that 70% of the total load is resisted by the noncomposite bare steel section while the other 30% is assumed to be applied as soon as the composite action is mobilized. Another unshored construction case named unshored-d assumed 50% of the total load resisted by the noncomposite section. Creep and shrinkage effects were not included from applied load but were simulated using the age-adjusted effective modulus method. Comité Euro International du Beton- Fédération International de la Précontrainte, Paris, (CEB-FIP, 1993), along with its update (CEP-FIP, 1999) referred to hereafter as CEB-FIP MC90-99, is used for predicting the short term as well as time-dependent properties of concrete.

As shown in Figure 2-4, for EB2, a significant redistribution of bending moments was observed when both creep and shrinkage effects were analyzed for both shored and unshored constructions. In particular, a significant increase in bending moment at supports accompanied by a decrease in bending moment at the midspan. However, there is only a marginal difference in both maximum positive and negative moments between shored and unshored constructions. Please note that the same load was applied to both shored and unshored constructions.
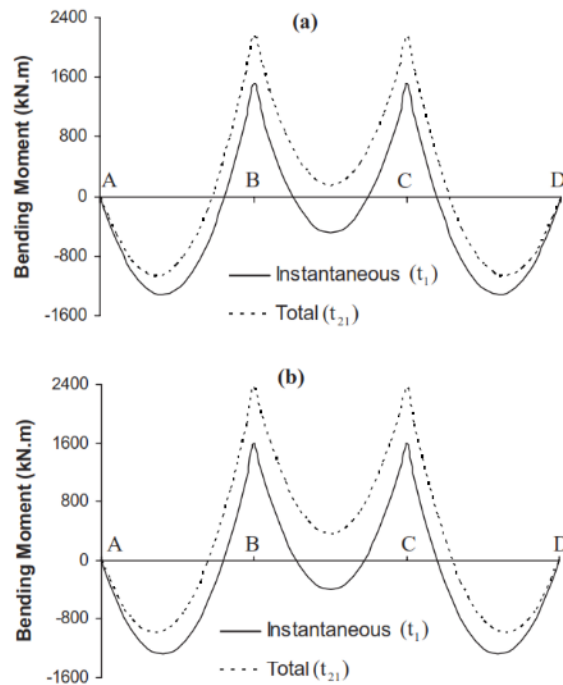
Figure 2-4. Bending moments of composite section for Bridge EB2 (a) shored construction; (b) unshored construction (Chaudhary et al., 2009)

The stresses developed at the top and bottom fibers were also compared. As shown in Figure 2-5, stresses in the top and bottom of the steel section were higher for unshored construction. However, creep and shrinkage have a more significant effect on the shored construction as the change in stress in the top fiber resulting from creep and shrinkage is higher in the shored construction. Moreover, a sharp increase along the span near the continuity supports was observed for shored construction, which resulted from the cracking of concrete near the supports and the consequent transferring of the stress to the steel section. No such sharp increase is observed for the unshored construction since the cracking does not take place.

Interesting results were observed for midspan deflection. As shown in Figure 2-6, both creep and shrinkage contributed to the time-dependent changes in midspan deflections. The instantaneous and final deflections of the unshored construction are significantly higher than those of the shored construction.
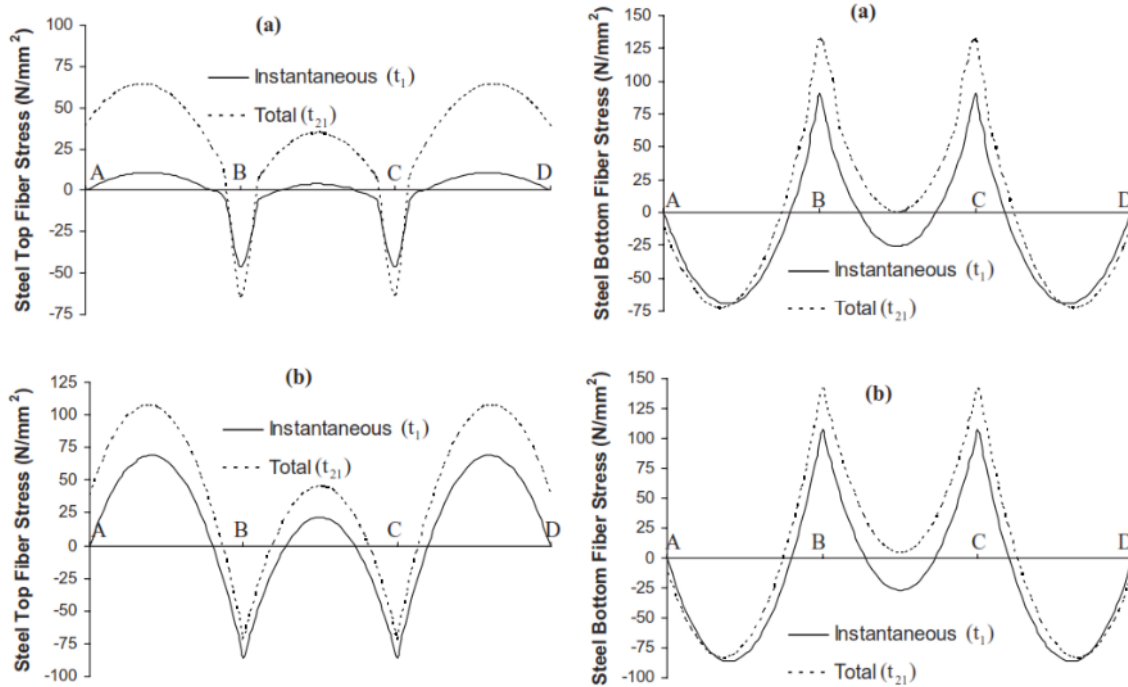
Figure 2-5. Left: Top fiber stresses in steel section for Bridge EB2; Right: Bottom fiber stresses in steel section for Bridge EB2 (a) shored construction; (b) unshored construction (Chaudhary et al., 2009)
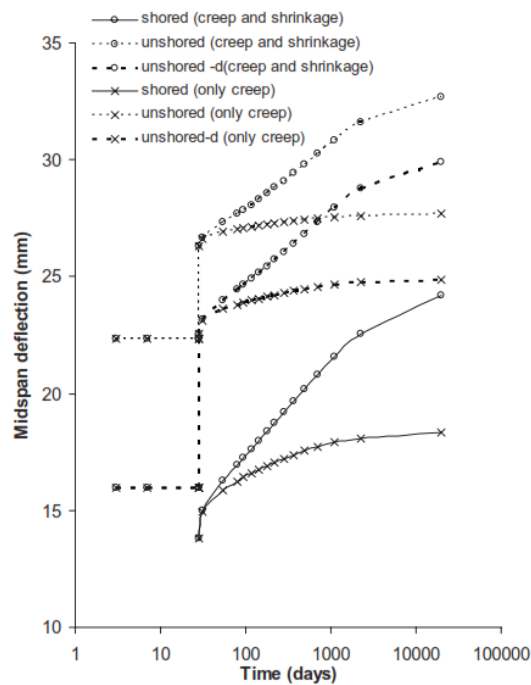
Figure 2-6. Time-dependent variation of midspan deflection of Span AB of Bridge EB2 for shored construction and unshored construction (Chaudhary et al., 2009)

Varshney et al. (2013) performed a study on the control of time-dependent effects of creep and shrinkage in steel-concrete composite frames with precast concrete slabs for both shored and unshored construction. Although this study was focused on composite frames, they found some interesting and relevant results. They concluded that while the type of construction has an insignificant effect on bending moment, the percentage change in mid-span deflection due to creep and shrinkage is significantly higher for shored construction.

2.3 SHEAR CONNECTIONS IN SHORED CONSTRUCTION

Previous bridge design specifications (AASHTO, 2017) and LRFD reference manual (Grubb et al., 2015) mentioned large forces may be induced in the shear connectors since most of the dead load is carried by the composite section. However, the newly published LRFD-9 (AASHTO, 2020) has removed the provision related to composite action for shored construction.

Through the literature review, there is only a very limited number of demonstration bridges built with shored construction in the U.S. and no study has been found on the composite action of composite steel girder bridge constructed using shored construction. However, if the girder is a fully composite section, the total force applied on the shear connectors $\Sigma Q_n = F_y A_s$ (F_y is the yield stress of steel and A_s is the area of the steel section) usually depends on the steel section only, regardless of the load carried by the composite section. If the girder is partially composite, the total force would be even less. Thus, although the composite section carries all the dead load for shored construction, it should not cause concern on the composite action of the section. In order to fully understand the behavior of composite action for shored constructed composite bridges, the research team conducted a survey with the practitioners whether composite action failure has occurred and whether they have concerns on the composite action for shored constructed composite bridges.

2.4 DECK TENSILE STRESSES AT INTERMEDIATE SUPPORTS

In Article C6.10.1.1.1a of LRFD-9 (AASHTO, 2020), it is stated that "there is an increased likelihood of significant tensile stresses occurring in the concrete deck at permanent interior supports of continuous spans when shored construction is used." It might be just speculation since no reference is provided in LRFD-9. Article 5.4.2.6 of LRFD-9 specifies the modulus of rupture

may be taken as $0.24\sqrt{f_c'}$ for normal weight concrete and Article C5.4.2.7 specifies the tensile strength may be estimated as $0.23\sqrt{f_c'}$. There is no tensile stress limit provided for shored construction. There is a very limited number of studies on deck tensile stresses at intermediate supports for shored construction. As presented earlier in section 2-B, Chaudhary et al. (2009) conducted a FE study to evaluate the effects of creep and shrinkage of composite bridges from both shored and unshored construction. The authors didn't present the results in the concrete deck but presented the steel stresses for both shored and unshored conditions. As shown in Figure 2-7, the tensile stresses of top fiber in the steel section at the intermediate support are higher for unshored construction in comparison with shored construction. Furthermore, the stresses are lower in unshored-d in comparison with the unshored case.
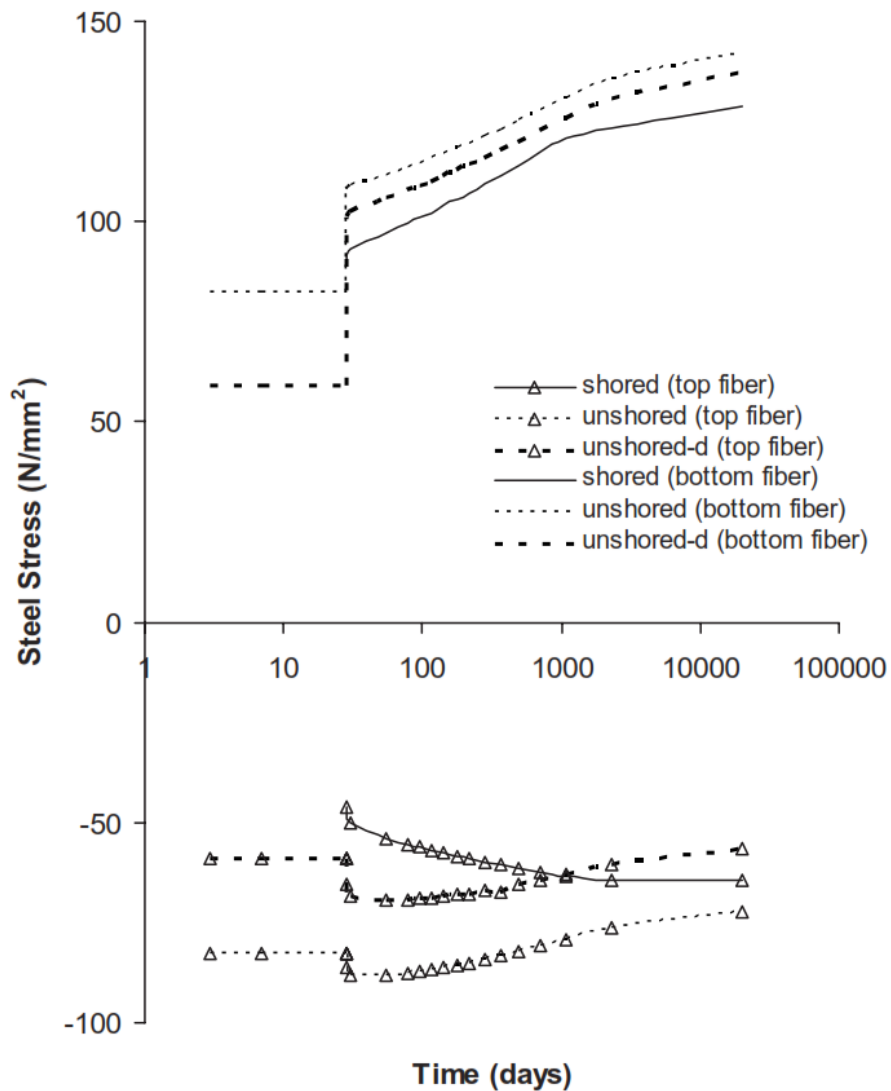
Figure 2-7. Time-dependent variation of top and bottom fiber stresses in steel section at support B of Bridge EB2 for shored construction and unshored construction (Chaudhary et al., 2009)

2.5   CAMBER TOLERANCES FOR SHORED CONSTRUCTION

In Article 6.7.2 of AASHTO LRFD-9 (AASHTO, 2020), it is stated that "steel structures should be cambered during fabrication to compensate for dead load deflection and vertical alignment." The tolerances for induced camber are provided in Article 6.4.4 of the American Institute of Steel Construction's (AISC) Code of Standard Practice for Steel Buildings and Bridges (AISC, 2016):

(a) For beams that are equal to or less than 50 ft in length, the variation shall be equal to or less than minus zero / plus ½ in.

(b) For beams that are greater than 50 ft in length, the variation shall be equal to or less than minus zero / plus ½ in. plus 1/8 in. for each 10 ft or fraction thereof in excess of 50 ft in length.

The research team didn't find other references that support shored construction should have closer camber tolerance. In fact, if shoring is performed at an offsite plant, camber should be easily monitored and controlled compared to site conditions.

2.6   SHORED CONSTRUCTION IN ABC

There is a very limited number of studies on shored construction in ABC. Freeby (2005) developed two new prefabricated bridge superstructure systems for TxDOT: A steel tub-girder and a prestressed concrete pre-topped U-beam. Both systems were developed for maximum span lengths of 115 ft and a total superstructure depth of 38 in. In this project, the beams were designed to be shored during the placement of the concrete deck to make them composite for all loads and to achieve a shallow superstructure depth. After slab placement, the beam will be hauled to the bridge site and erected on the piers/abutments. Figure 2-8 shows the steel girder superstructure. The steel tub-girder was designed with a 29.5 in deep steel section and an 8.5 in the slab, resulting in a section with 38 in total depth. The author mentioned that the design of this element was challenging because the aspect ratio was around 48:1. The author also mentioned that the service limit state was controlled not by allowable strength but by the TxDOT as well as AASHTO imposed live load deflection of L/800. Furthermore, due to the fact that the steel section is unusually shallow, the girder had to be proportioned so that the deck would not crush before the steel tub reached yield. The final adoption and use of this new prefabricated bridge super structure system were not presented in this paper. Since this is one of the first proposals to use shored construction in ABC, the research team reached out to the TxDOT bridge division and they stated that they have not adopted this prefabricated steel composite girder system yet.
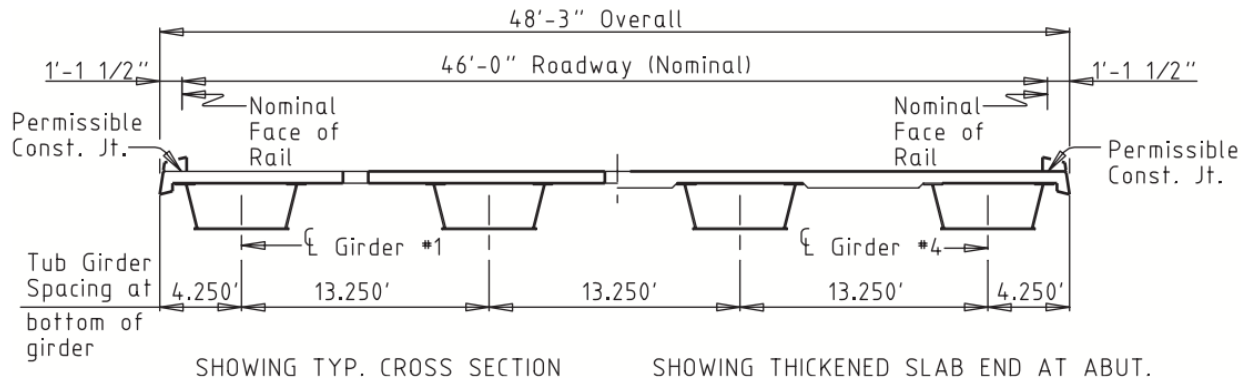
Figure 2-8. Steel girder superstructure (Freeby, 2005)

The Manual on Use of Self-Propelled Modular Transporters to Remove and Replace Bridges (FHWA, 2007) mentions that shored construction provides resistance of the deck self-weight by the entire superstructure cross-section and can increase girder efficiency by 30% or more. This allows for the elimination of a beam or two per span or the use of shallower beams for lower fill heights. Figure 2-9 shows a composite design concept with beams shored at midspan during deck casting. The manual also states that the elevation tolerances should be specified and the temporary shoring should be designed using the AASHTO Guide Design Specifications for Bridge Temporary Works. However, in this manual, only general information is provided without detailed guidance or examples.
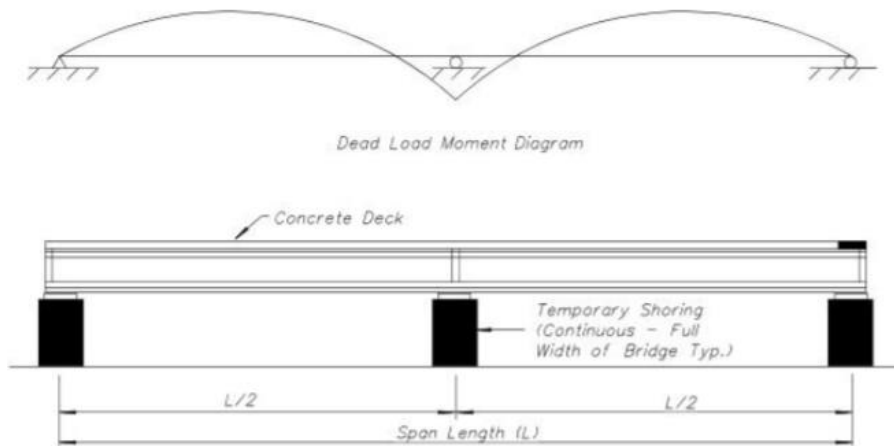
Figure 2-9. Composite dead load design concept for improved cross-section efficiency (FHWA, 2007)

In a recent Strategic Highway Research Program 2 (SHRP2) report, *Innovative Bridge Designs for Rapid Renewal* (HNTB&SHRP, 2013), the following advantages related to shored construction were presented:

- For steel stringers or girders with precast decks, the overall efficiency of the section is improved and lighter steel beams may be used.
- If the deck is precast under conventional shored conditions, the modular system will provide the benefit of shored construction where the dead load is carried by the composite section. However, beams should be designed for noncomposite dead loads in consideration of future deck replacement.
- Casting of the deck can be completed under fully-shored conditions where the beams are ground supported for the decked steel girder systems. Fully-shored conditions will provide advantages such as ease of construction, worker safety, and enhanced structural resistance of the system because it avoids buildup of noncomposite stresses.

On the contrary, a recently published SHRP2 report (NRC 2013) states that all formwork for the deck will be supported from the longitudinal girders similar to conventional deck construction for a decked-stringer system (i.e., shored construction will not be assumed). This provision ensures that future deck replacements can be carried out without shoring. It is worth noting that the ability to perform deck replacements is mainly a concern for states that utilize road salts. However, this concern can be eased if stainless rebar is required to be used. In the SHRP report, it is also recommended to add the following language to AASHTO 6.10.1.1.1a: "Shored construction as allowed in the last sentence of this section is not allowed for spans assembled using steel modular systems." However, no detailed research work has been done to compare shored vs. unshored construction for pre-decked steel modular systems.

2.7   PRE-DECKED STEEL GIRDER

The design and construction of pre-decked steel girders are well documented in several manuals published by FHWA (FHWA, 2007, Culmo et al., 2011 and Culmo et al., 2013a). SHRP 2 Renewal Project R04 (HNTB, 2013) discussed many different options for ABC, including pre-decked composite steel girder systems. Culmo et al. (2013) stated that the design of the deck for modular deck or beam elements is typically the same as with a conventional deck design which follows the provisions of Chapter 4 and Chapter 9 of LRFD-8 (AASHTO, 2017).

Burgueño and Pavlich (2008) evaluated a prefabricated composite steel box girder system for rapid bridge construction. The objective of this project was to evaluate the feasibility of an entirely

prefabricated composite box girder bridge system through numerical simulations. The authors concluded that the prefabricated steel/concrete composite girder/deck units are a safe and viable system for short-span highway bridges. However, shored construction was not considered in this study.

Phares et al. (2013) conducted a laboratory and field testing of an accelerated bridge construction demonstration bridge: US Highway 6 Bridge over Keg Creek. The new bridge is a three-span 204.5-ft-long steel/precast modular structure (Figure 2-10). The performance of the UHPC transverse joints and global bridge behavior was evaluated through laboratory and live load field testing. However, the time-dependent effects and long-term performance were not discussed in this study.
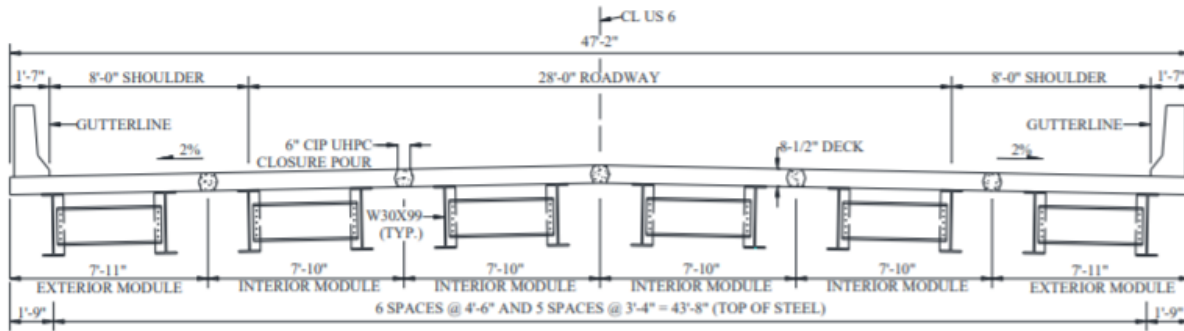
Figure 2-10. Bridge cross-section view (Phares et al., 2013)

In addition, through this literature search, the research team was able to identify a number of steel girder bridges constructed using pre-decked systems. These bridges can be candidates for further evaluation (Road to the Future, 2009, Gilley, 2009, Bhajandas et.al, 2011, Littleton, 2013, Mallela et al., 2014, Ruzzi and Bedillion, 2014, and Bhajandas, 2015). The basic information of these bridges is summarized in Table 2-1. There are two typical pre-decked systems that have been used, one is a single pre-decked steel composite girder and another one is a multi-stringer/beam with a precast concrete deck. All these bridges were investigated further by interviewing the respective owner of the bridges.

Table 2-1. Steel girder bridges built with shored construction (Pre-decked system)

Bridge	Location	Year of Construction	Number of Spans	Construction Method
U.S. 15/29 Bridge Over Broad Run Near Gainesville, VA (Gilley, 2009)	Gainesville, VA	2008	Three span	Two rolled steel beams made composite with the high-performance lightweight concrete deck
I-95 James River Bridge (Road to the Future, 2009)	Richmond, Virginia	2009	Single Span	3 steel plate girders with an 8.75 in deck
Eastern Avenue Bridge Over Kenilworth Avenue (Bhajandas et.al, 2011)	Washington, DC	2010	Two span	Precast superstructure unit (two W16x100 steel beams supporting lightweight concrete deck.)
US Highway 6 Bridge over Keg Creek (Littleton, 2013)	Pottawattamie County, Iowa	2011	Three span continuous	Pre-decked system with UHPC closure pour
Martin Luther King (MLK) Jr. Memorial Bridge (Ahmad and Mongi, 2014)	Bluefield, WV	2011	Single-span	Two steel girders with precast concrete deck
Fourteen Bridges on I-93 in Medford (Mallela et al., 2014)	Medford, Massachusetts	2011	Single-span	Precast superstructure unit (two weathering steel beams and a precast concrete deck)
I-190 Bridges over Buffalo Avenue, Niagara Falls, NY (Bhajandas, 2015)	Niagara Falls, NY	2013	Single-span	Pre-decked steel beam modules with high early strength concrete pour
SR 288 Main St. Bridge, Wampum (Ruzzi and Bedillion, 2014)	Wampum, Pennsylvania	2014	Single-span	Pre-decked system with UHPC closure pour

2.8   FE ANALYSIS OF NONLINEAR BEHAVIOR, TIME-DEPENDENT EFFECTS, AND CRACKING OF CONCRETE

The creep and shrinkage of concrete is an intricate phenomenon involving time-dependent effects. It requires an accurate creep and shrinkage model and sophisticated FE modeling techniques to successfully simulate and analyze creep and shrinkage. Bazant (1972, 1982, 1988) discussed modeling concrete as an aging viscoelastic material using compliance functions and an age-adjusted effective modulus.

Bradford and Gilbert (1991) developed a number of analytical approaches for the calculation of short- and long-term strains and deflections of composite beams. They also conducted an experimental test to validate their proposed method. There are four simply supported steel composite beams that were tested and monitored for 250 days under controlled environmental conditions.

White and Dutta (1993) performed a series of numerical studies on moment-rotation behavior in steel and composite steel-concrete bridge girders. The focus of this study is on the inelastic moment-rotation behavior of continuous-span non-compact bridge girders at interior-pier locations. Four component tests were performed with one specimen as a composite design. In the testing of the composite specimen, the applied loading simulated the loading under unshored construction. Similarly, for the analysis of this test, the simulated dead load is applied to the steel girder alone while all the additional loading was applied to the cracked composite section. The authors found out that although the composite specimen and Specimen D have similar proportions (Figure 2-11), the moment-rotation curves for the composite specimen, both from the analysis and from the experiment, showed slightly greater rotation capacity than the one for Specimen D (Figure 2-12). They attributed this difference to the mode of construction. They stated that when the composite girder is analyzed for shored construction, it exhibits moment-rotation characteristics closer to those of Specimen D. This statement is a bit confusing since the loading applied was simulated as an unshored construction condition. The authors didn't provide further discussion or clarification about the results.
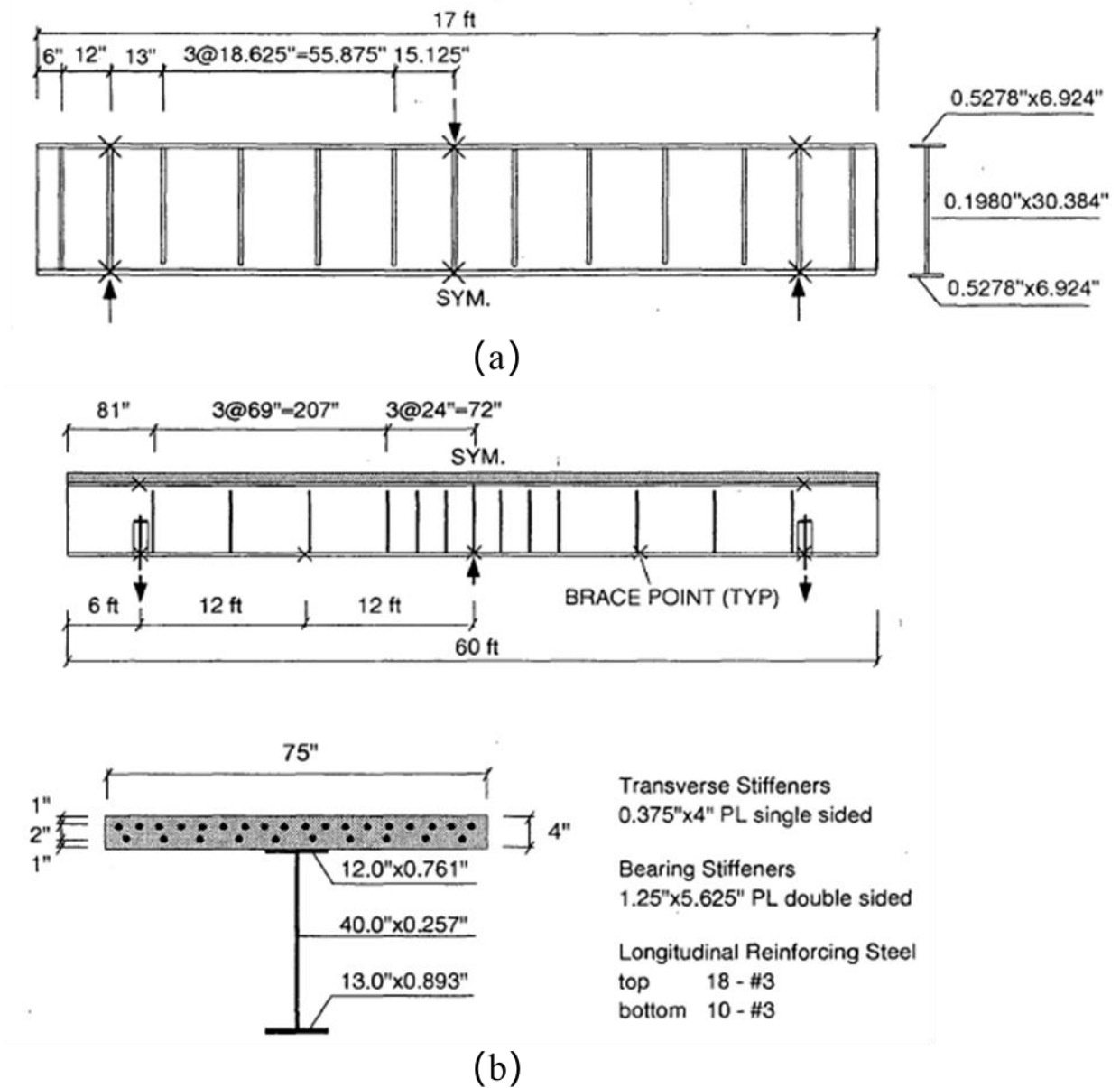
Figure 2-11. (a) Steel girder specimen D and (b) Composite test specimen (White and Dutta, 1993)
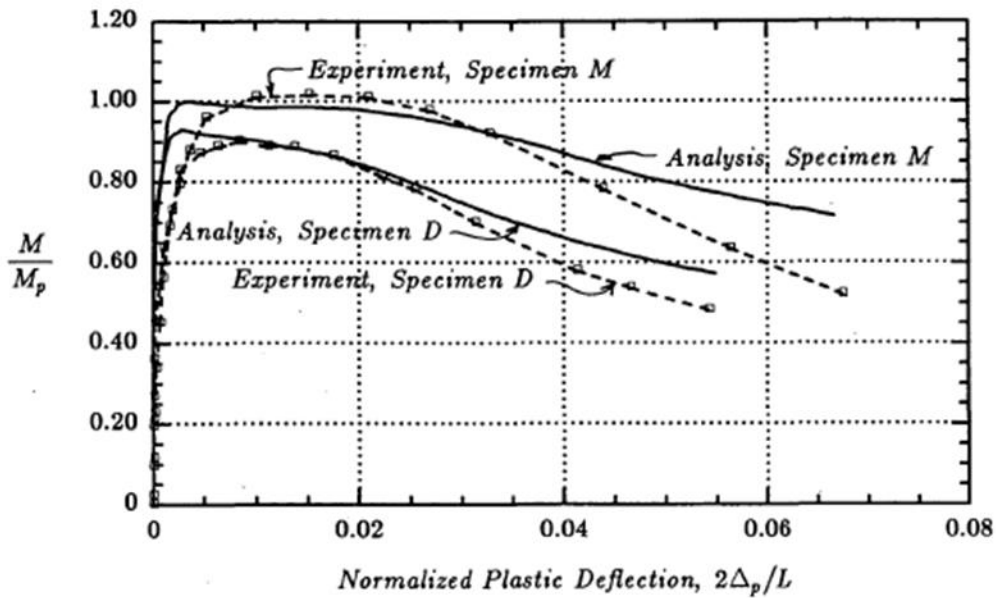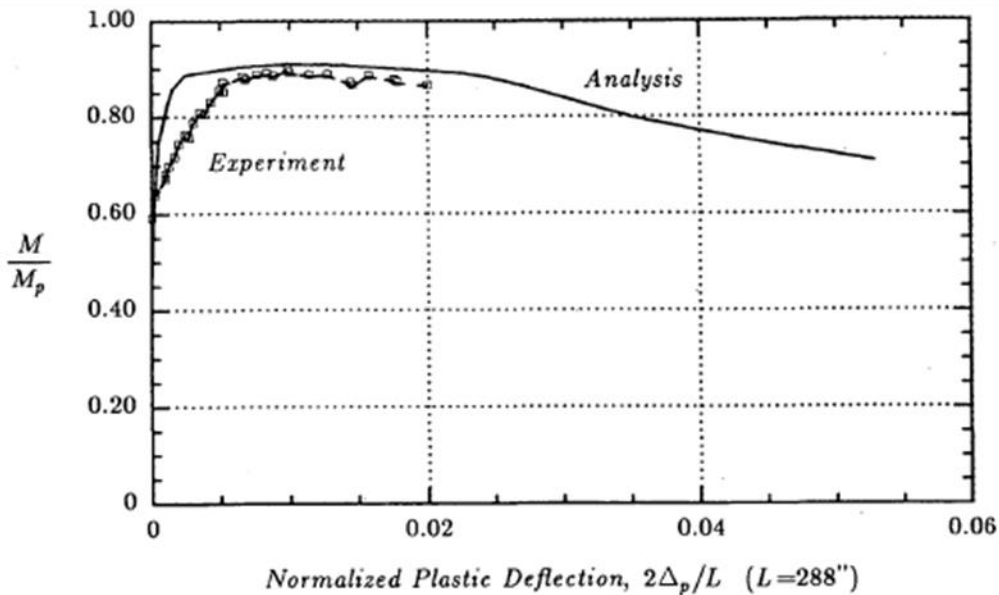
Figure 2-12. Normalized moment versus plastic deflection at midspan: (a) Steel girder Specimen D and (b) Composite test specimen (White and Dutta, 1993)

Grubb (1993) reviewed the alternate load factor design method. In his review, he discussed a series of tests that were completed on a composite bridge specimen. At that time, the main reason for the tests was to validate the concerns about the effect of permanent deformations on concrete

cracking over interior piers and on the fatigue life of the steel beams. Thus, a half- scale composite girder model of an interior support region of a continuous composite bridge was tested in negative bending to (1) demonstrate the fatigue strength after inelastic rotation, (2) observe the amount, pattern, and width of concrete cracks, (3) determine the number of cycles to shakedown, and (4) to observe steel-beam yielding during inelastic rotation. Some interesting results were reported: (1) the measured concrete crack width never exceeded 0.01 in. at any time during the test and (2) the cracks closed to about 0.003 in. when unloaded, (3) no significant web and compression-flange buckling were observed at the largest overload moment; and (4) the total linear length of cracks had increased by 23% for the test specimen shored during casting of composite deck slab but the amount of linear cracking at the LFD and maximum overload moments would be less than these amounts for an unshored specimen because the concrete strains would be lower. However, the author didn't provide any evidence to support this statement since only one specimen was tested.

De Borst and Den Boogaard (1994) developed a general approach for numerically simulating the non-linear behavior due to thermal strains, creep, and cracking. The time-dependent effects were accommodated in a finite element analysis using a smeared-crack model. Fragiacomo et al. (2004) developed a numerical procedure for analyzing steel-concrete composite beams with regards to long-term behavior under service loads. Both creep and shrinkage, as well as non-linear behavior of material properties, are adequately considered. Maxwell's generalized rheological model is utilized through a step-by-step time increment procedure in order to accurately model creep effects. A new method called the "modified secant stiffness method" is used to account for the nonlinear behavior of component materials. The model is validated with experimental results to test its accuracy. To validate the model, results were compared to a mid-span vertical displacement test on two, two-span continuous composite beams with rigid connections tested by Gilbert and Bradford (1995). A different distributed load was applied to each beam that caused cracking in the slab near the intermediate support. A fairly good agreement can be seen between the FE model's predictions and the actual measured experimental value over time.

Fragiacomo & Ceccotti (2006) performed a study on the finite element modeling of composite timber-concrete beams under long-term loading. Things affecting long-term behavior such as the connection system, creep, mechanosorptive creep, shrinkage/swelling, and temperature variations are all considered. The structural problem is solved using a uniaxial finite element model with

flexible connections and a step-by-step numerical procedure over time. The proposed numerical procedure is then validated on two long-term experimental tests in outdoor conditions. The finite element model consisted of a lower timber beam linked to an upper concrete flange by the means of a continuous spring system. This represents the connection by hypothesizing the connectors as smeared along the beam axis. The model was then verified using data from a previous long-term loading study done at the EMPA Laboratory (Kenel and Meierhofer, 1998). This experiment used shores to support the structure while the concrete deck hardened. The shores were removed 21 days after the concrete casting. The proposed model/numerical solution agreed well with the experimental results.

Sakr and Sakla (2008) developed a uniaxial nonlinear finite element procedure for modeling the long-term behavior of composite steel-concrete beams. The finite element procedure follows a displacement-based approach. Nassif et al. (2008) performed a comprehensive study of bridge deck cracking and composite action analysis. The study indicated that concrete cracking can be attributed to three important factors: (1) concrete shrinkage, (2) thermal loads, and (3) preliminary construction loads. The authors concluded that higher cracking potential is expected at the end restraints. It was also observed that truck loads traveling in adjacent lanes have a significant effect on cracking potential in the fresh concrete deck. The extent of these effects depends on the concrete pouring sequence and the magnitude of the live load.

Kim (2014) carried out research to identify a simple method for analyzing the long-term deformations of steel-concrete composite members based on existing models to predict the creep and shrinkage and to estimate the time-varying deflection of the member for design purposes. Four previously established models to predict creep and shrinkage were first reexamined, then an analytical approach using the age-adjusted effective modulus method (AEMM) was used to calculate the long-term deflection of a simply supported composite beam. A large advantage of using the AEMM is that it can cope with the variations in stresses and strains with time due to creep and shrinkage in the composite cross-section. Experimental test data from Bradford and Gilbert (1991) was one of the experimental data sets the model's predictions were compared to. This test included 4 beams that were monitored for 200 days to analyze the effects of creep and shrinkage. Beams 1 and 2 were designed for nearly full composite action, while Beams 3 and 4, were designed with pairs of studs spaced at 600 mm (substantial slip is likely to occur). Beams 1

and 3 were subjected to a superimposed sustained uniformly distributed load, while Beams 2 and 4 experienced self-weight only. The beams were moist-cured for 10 days and were fully propped during this time. The paper only discusses their model's predictions to beams 1 and 3, which can be seen by the graph on the next page. The model's predictions were only off by 2% of the experimental value for both beams. Finally, a parametric study was conducted to analyze the variation of time-dependent deflection with a variety of combinations of creep coefficient, shrinkage strain, beam size, and span length. The paper states research shows the long-term deflection due to creep and shrinkage could be 1.5 to 2.5 times higher than the short-term deflection.

Su et al. (2018) performed a comprehensive FE analysis of a steel composite girder bridge considering various factors such as curing and restraint shrinkage, thermal gradient effects, and parapet load effects. In this study, a typical 3-span continuous bridge was selected for investigation. The bridge is consisted of 3 spans and is supported by 5 girders spaced at 2.36 m. Due to the length of the bridge, the deck construction cannot be finished in a single segment. Therefore, the deck construction was arranged into 3 stages as shown in Fig. 1. Stage I: Positive moment region of Span 1 was poured from south to north. Stage II: Positive moment region of Span 2 was poured first (Stage II-A), followed by the positive moment region of Span 3 from south to north. Stage III: Concrete was poured over Pier 1 and Pier 2 at this stage. Figure 2-13 shows the structural and instrumentation plan for the selected bridge.
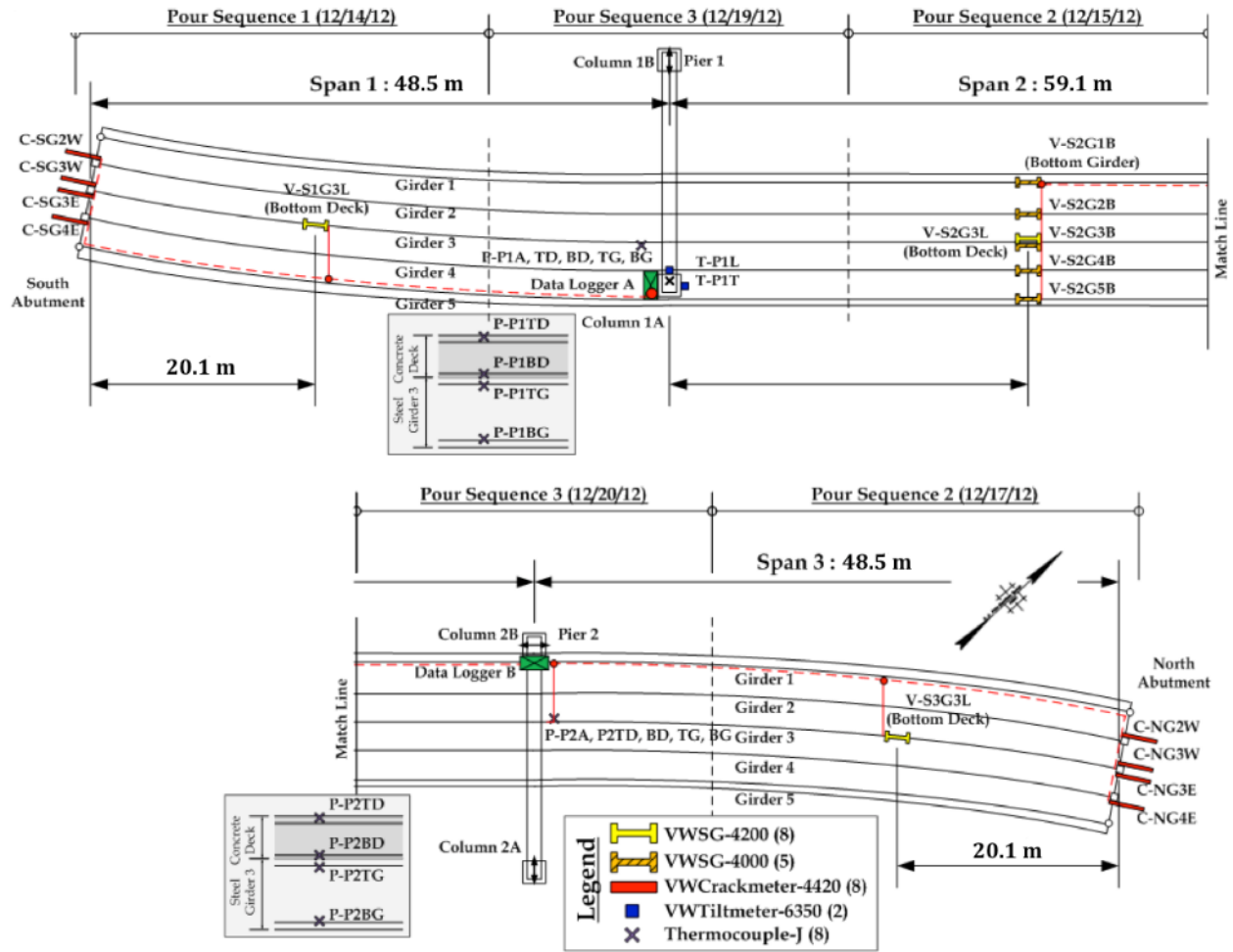
Figure 2-13. Structural and instrumentation plan for a typical 3-Span continuous steel girder bridge (Su et al., 2018)

As shown in Figure 2-16 (b), five months after the deck construction, cracks were found in Span 1 and Span 2 while no crack was observed in Span 3. Many cracks were observed in Span 1 and Span 2 with an average spacing of 1.7 m (5.6 ft.) and an average length of 2.1 m (7.0 ft.). A few cracks passed through the bridge deck transversely (4.9 m (16 ft.) or longer). Therefore, in order to evaluate the current practice and to evaluate the effect of various factors including creep and shrinkage, temperature gradient, and staging on cracking of new concrete deck, a comprehensive experimental and analytical study was performed. A detailed FE model was developed and validated with experimental data. Figure 2-14 shows the FE model developed for the investigated bridge and Figure 2-15 presents the comparison between the experimental data and FE analysis results for various construction stages. For the stage with large strains, e.g., Stage

II-A, the average error is less than 7%; and for the stages generating small strains, the average error of less than 15% was observed. Figure 2-16 shows the comparison between the strain contour map from the FE analysis and an actual crack map for Span 1 after Stage II-A. For region [B] with concrete tensile strain ranging from 40 to 60 με, few cracks were observed in the crack map while excessive cracks were observed in the region [A] where the tensile strain ranges from 60 to 236 με. This also indicates the accuracy of the FE model. Using the validated FE model, a comprehensive parametric study was performed to investigate the reason for excessive concrete cracking. Based on the analysis results, an optimized deck construction staging practice was recommended for future use.
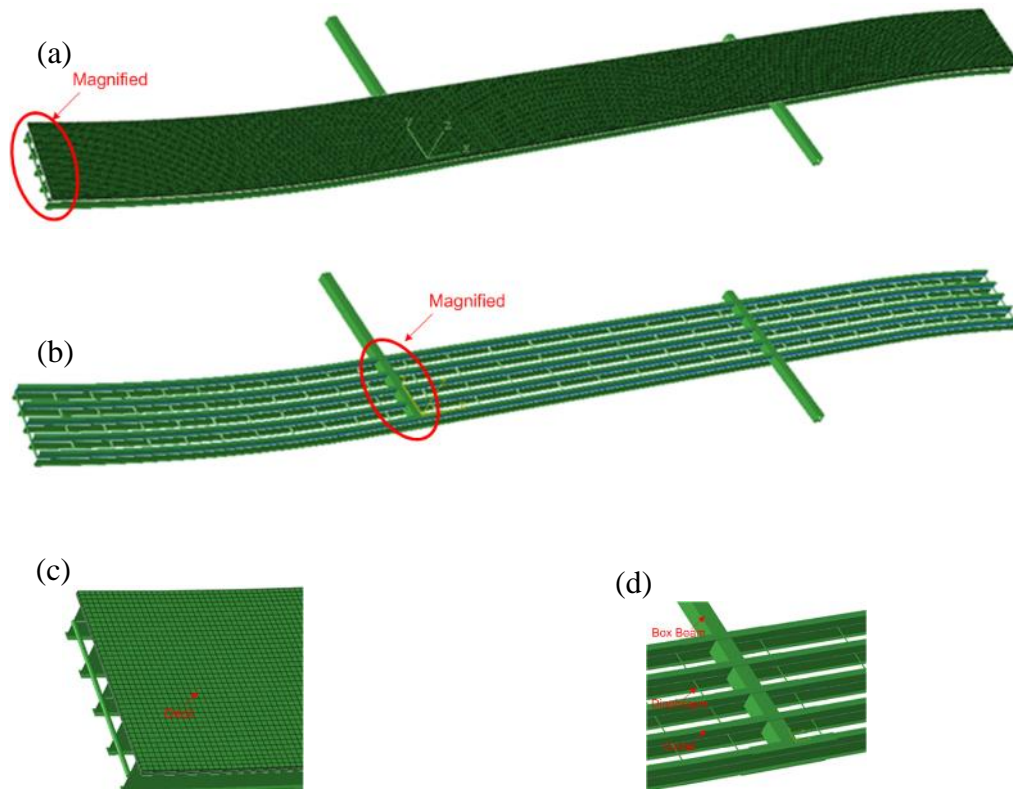
Figure 2-14. FE model of the investigated bridge; (a) Whole Structure, (b) Steel Frame, (c) Magnified abutment and (d) Magnified box-girder connections (Su et al., 2018)
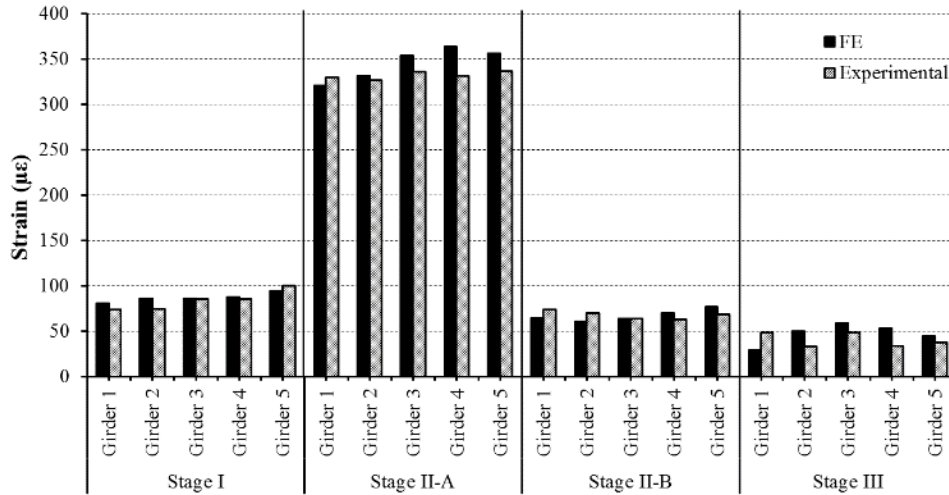
Figure 2-15. Comparison of experimental data and FE analysis results (Su et al., 2018)

(a)   **(b)**
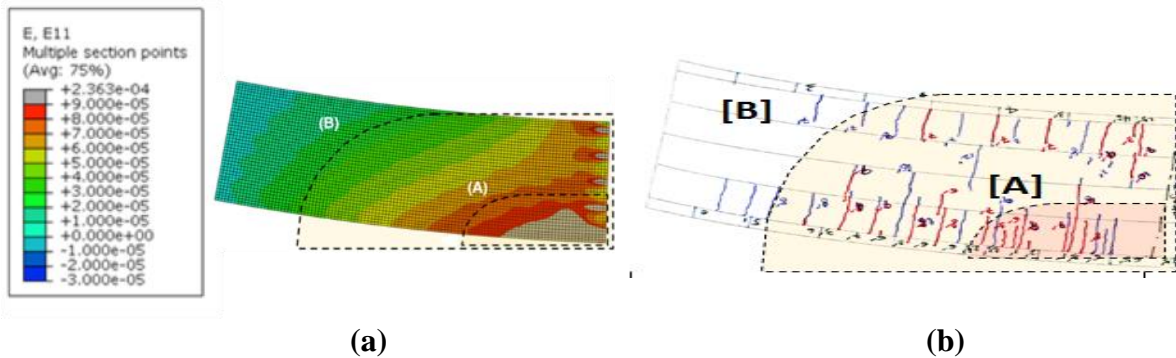
Figure 2-16. (a) Strain contour map for Span 1 after Stage II-A and (b) Crack map of Span 1 (5 months after construction) (Su et al., 2018)

2.9   KEY FINDINGS

The research team conducted a detailed literature review on various topics related to this research project including these key topics: The current state of practice on design analysis of shored constructed steel composite girders; time-dependent effects from shored construction; shear connections in shored construction; deck tensile stresses at intermediate supports; camber tolerances; shored construction in ABC; pre-decked steel girders; and FE analysis of non-linear behavior, time-dependent effects, and cracking of concrete. The findings of the literature review are summarized below:

(1) Based on the literature review, the primary concerns for shored construction of composite steel girder bridges are: (a) large forces induced to the shear connectors may cause the

failure of composite actions; (b) Time-dependent effects (creep and shrinkage of concrete) may contribute to a loss of composite action resulting in increased deflection and decreased section capacity; (c) Increased deck tensile stresses at the intermediate support locations of continuous girders; and (d) the tight camber tolerances required for shored construction. However, the validity of these concerns needs to be further investigated. Through literature review, no evidence has been found to support (a) and the new AASHTO LRFD-9 has removed provision related to composite action in C6.10.1.1.1a.

(2) Another concern for shored construction to be used in pre-decked composite stringer system in ABC is future deck replacements may need shoring if shored construction was used in original deck placement. However, there are scenarios this concern can be eliminated: (1) If prefabricated modular is used for deck replacement; (2) If staged construction is used, the adjacent composite girder will provide shoring support to the sections under replacement; (3) if shoring is available for re-decking. The shored construction has several obvious advantages for ABC including shallower steel sections, cost efficiency, worker safety, etc.

(3) AASHTO and other FHWA design manuals are not recommending the use of shored construction for composite steel girder bridges based on the aforementioned concerns. Most of the states either don't allow shored construction or require such practice with prior approval from State Bridge Engineers. It was also mentioned that the required footprint may not be available for shored construction at the bridge construction site.

(4) For time-dependent effects including creep and shrinkage of concrete, the previous research indicated that both the deflection and deflection ratio for the cracked composite bridge by unshored construction is larger in comparison with shored construction. A significant redistribution of bending moment resulting from creep and shrinkage was observed for both the shored and unshored constructions. However, there is only a marginal difference in both maximum positive and negative moments between shored and unshored constructions with the same load applied. Stresses in the top and bottom of the steel section were higher for unshored construction but creep and shrinkage have more significant effects for the shored construction.

(5) There are several models that have been developed to evaluate the time-dependent effects of creep and shrinkage in composite beams (Bazant (1972), CEB-FIP (1993), Kwak and

Seo (2000), Chaudhary et al. (2007), Nassif et al. (2008), and AASHTO (2017)). The research team has further evaluated these models and has chosen the most accurate model for the analytical study.

The quantity of previously conducted research is limited. No detailed study has been performed to evaluate the performance, especially the long-term performance, of shored constructed steel composite girder bridges. No comprehensive comparison has been performed to compare shored vs. unshored construction. Furthermore, no parametric study and no lifecycle cost-benefit analysis have been performed for shored construction vs. unshored construction, especially for bridges using ABC. Thus, it is deemed necessary and important to develop a design guideline for composite steel girder bridges using shored construction, for a general construction scenario as well as for a predecked steel composite girder unit commonly deployed in ABC.

3 INTERNATIONAL CODE REVIEW

Upon the completion of the literature review, the research team gained a better understanding of the current state of the practice with regards to the shored construction for composite steel bridges. It was found that there is a significant knowledge gap in shored construction for composite bridges. The research team has reviewed various current international design codes and specifications from different countries. In addition, the related reference manuals and design guidelines for the respective code were also reviewed. The codes and standards that have been reviewed by the research team are summarized in Table 3-1. The sections that relate to composite steel girder bridge design and construction were also identified for each international code. In addition, the related articles from other sections were also reviewed and summarized. Section 2 presents a detailed review of each code/standard.

Table 3-1. List of international codes for review

Code	Publisher/Agency	Current Edition	Sections
Eurocode 4: Design of composite steel and concrete structures	European Committee for Standardization	EN1994	EN 1994-2
Canadian Highway Bridge Design Code (CHBDC)	Canadian Standards Association	CSA S6-14	Section 5, 8, 10
CISC Code of Standard Practice	Canadian Institute of Steel Construction	Eighth Edition	Chapter 6 & 7
Australian Standard: Bridge design	Standards Australia's technical committee BD-090, Bridge Design	AS 5100.6:2017	Part 6
New Zealand Standard: Steel structures standard	Standards New Zealand	NZS 3404: Part 1: 2009	No specific section
New Zealand Standard: Concrete structures standard	Standards New Zealand	NZS 3101.1: 2006 NZS 3101.2: 2006	Section 6 and 18
JRA Specifications for Highway Bridges, Part 2 Steel Highway Bridges	Japan Road Association	2012	No specific section
JSCE Standard specifications for steel and composite structures	Japan Society of Civil Engineers	2009	Chapter 15

3.1 <u>EUROCODE 4: DESIGN OF COMPOSITE STEEL AND CONCRETE STRUCTURES (EN1994)</u>

EN 1994 is the design code for the design of composite steel and concrete structures conducted within the European Union. It contains three documents: (1) EN 1994-1-1 (CEN, 2004): General rules and rules for buildings, (2) EN1994-1-2 (CEN, 2005): General rules-Structural fire design, and (3) EN 1994-2 (CEN, 2005): General rules and rules for bridges. The research team has reviewed all three documents but mainly focused on EN 1994-2 because it is the design code for composite steel and concrete bridges. In addition, the other companion references and publications related to composite bridge design using Eurocodes were also reviewed, including bridge design to Eurocodes worked examples (Bouassida, et al., 2012), composite beam design to Eurocode 4 (Lawson and Chung, 1994), composite beam design manual Eurocode 4-2004 for ETABS® 2016 (CSI, 2016), and designers' guide to EN 1994-2 (Hendy and Johnson, 2006). It is worth noting that Eurocodes don't use the terms "shored/unshored" but uses "propped/un-propped" instead. The articles related to the shored construction of composite steel bridges are summarized below.

(1) Stresses in structural steel:

Article 6.2.1.4 (5) of EN 1994-2: The stresses in structural steel in compression or tension should be derived from the bi-linear diagram given in EN 1993-1-1, 5.4.3(4) and should take account of the effects of the method of construction (e.g. propped or un-propped). Figure 3-1 shows the bi-linear stress-strain relationship specified in EN 1993-1-1 (CEN, 2005).
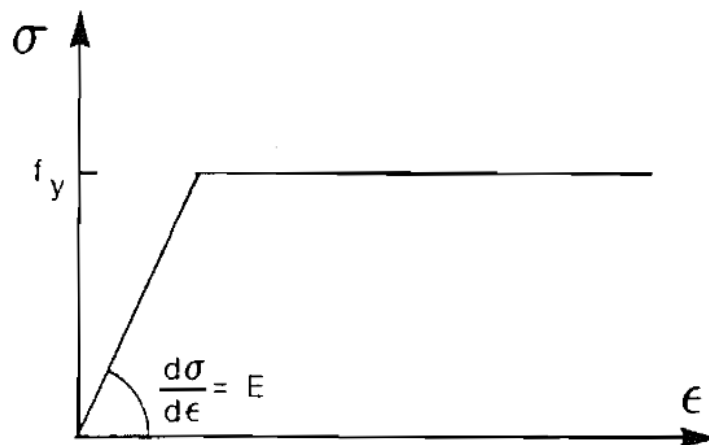
Figure 3-1. Bilinear stress-strain relationship (CEN, 2005)

(2) Non-linear resistance bending moment:

Article 6.2.1.4 (6) of EN 1994-2: For Class 1 and Class 2 composite cross-sections with concrete flange in compression, the non-linear resistance moment M_{Rd} may be determined as a function of the compressive force in the concrete N_c using equations below:

$$M_{Rd} = M_{a,Ed} + \left(M_{el,Rd} - M_{a,Ed}\right)\frac{N_c}{N_{c,el}} \qquad \text{when } N_c \leq N_{c,el} \qquad \text{Eq. (3-1)}$$

$$M_{Rd} = M_{el,Rd} + \left(M_{pl,Rd} - M_{el,Rd}\right)\frac{N_c - N_{c,el}}{N_{c,f} - N_{c,el}} \quad \text{when } N_{c,el} \leq N_c \leq N_{c,f} \qquad \text{Eq. (3-2)}$$

$$M_{el,Rd} = M_{a,Ed} + kM_{c,Ed} \qquad \text{Eq. (3-3)}$$

where:

$M_{a,Ed}$ is the design bending moment applied to the structural steel section;

$M_{el,Rd}$ is the design value of the elastic resistance moment of the composite section;

N_c is the design value of the compressive normal force in the concrete flange;

$N_{c,el}$ is the compressive normal force in the concrete flange corresponding to $M_{el,Rd}$;

$M_{pl,Rd}$ is the design value of the plastic resistance moment of the composite section with full shear connection;

$N_{c,f}$ is the design value of the compressive normal force in the concrete flange with full shear connection;

$M_{a,Ed}$ is the design bending moment applied to structural steel section before composite behavior;

$M_{c,Ed}$ is the part of the design bending moment acting on the composite section;

k is the lowest factor such that a stress limit in Article 6.2.1.5 (2) is reached; where un-propped construction is used, the sequence of construction should be taken into account;

$N_{c,el}$ is the compressive force in the concrete flange corresponding to the moment $M_{el,Rd}$

.

For composite sections, the classification system defined in Article 5.5.2 of EN 1993-1-1 (CEN, 2005) applies. The role of cross-section classification is to identify the extent to which the resistance and rotation capacity of the cross-section is limited by its local buckling resistance. There are four classes of cross-sections, as follows:

Class 1: Cross sections are those which can form a plastic hinge with the rotation capacity required from the plastic analysis without reduction of the resistance;

Class 2: Cross sections are those which can develop their plastic moment resistance, but have limited rotation capacity because of local buckling;

Class 3: Cross sections are those in which the stress in the extreme compression fiber of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent the development of the plastic moment resistance;

Class 4: Cross sections are those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Figure 3-2 shows the idealized moment-rotation relationships for sections in Class 1 to 4. It is noted Article 6.2.1.4 (6) only applies to Class 1 and Class 2 sections due to the fact that Class 3 and 4 sections are subject to local buckling before the development of the plastic moment of resistance. Thus elastic resistance was used as bending resistance for Class 3 and Class 4 sections.
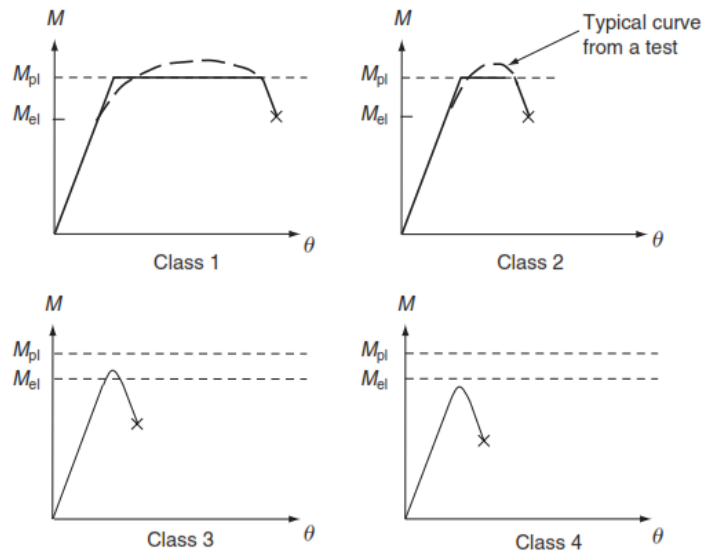
Figure 3-2. Idealized moment–rotation relationships for sections in Classes 1 to 4 (Hendy and Johnson, 2006)

For cross-sections where article 6.2.1.2 (2) applies, the reduced value $\beta M_{pl,Rd}$ should be used in Eq. (2) and in Figure 3-3 instead of $M_{pl,Rd}$.
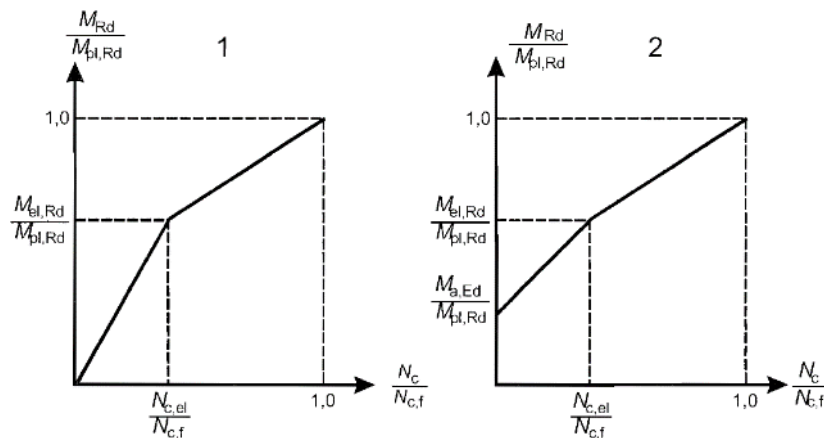
Figure 3-3. Simplified relationship between M_{Rd} and N_c for sections with concrete slab in compression: 1. Propped construction and 2. Unpropped construction (CEN, 2005)

In Article 6.2.1.5 (2), it is stated that the limiting stresses should be taken as f_{cd} for concrete in compression; f_{yd} for structural steel in tension or compression; f_{sd} for

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reinforcement in tension or compression but alternatively, reinforcement in compression in a concrete slab may be neglected. where f_{cd} is the design value of the cylinder compressive strength of concrete, f_{yd} is the design value of the yield strength of structural steel, and f_{sd} is the design value of the yield strength of reinforcing steel.

In Article 6.2.1.2 (2), it is stated that for composite sections with structural steel grade S420 (Gr. 60 equivalent) or S460 (Gr. 65 equivalent), where the distance x_{pl} between the plastic neutral axis and the extreme fiber of the concrete slab in compression exceeds 15% of the overall depth h of the member, the design resistance moment M_{Rd} should be taken as $\beta M_{pl,Rd}$ where β is the reduction factor given in Figure 3-4.
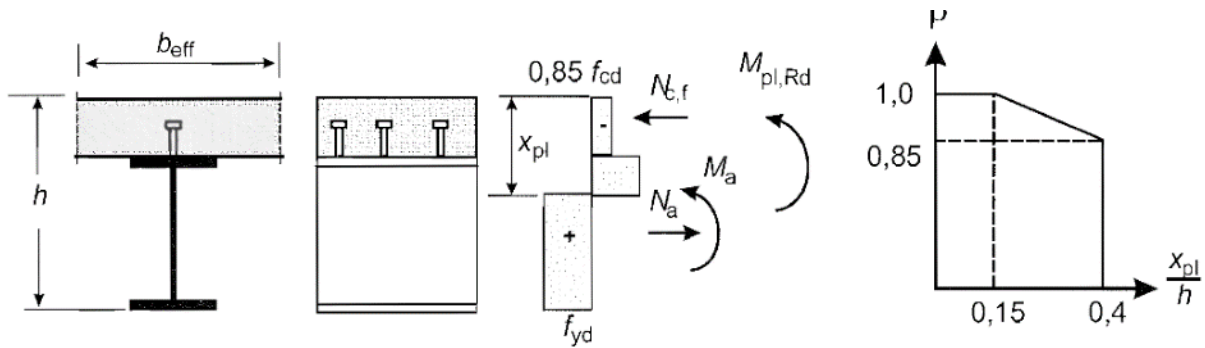
Figure 3-4. Reduction factor β for $M_{pl,Rd}$ (CEN, 2005)

(3) Cracking of concrete:

Article 7.4.1(4) and 9.8.1(1) of EN 1994-1-1: In cases where beams in buildings are designed as simply supported although the slab is continuous and the control of crack width is of no interest, the longitudinal reinforcement provided within the effective width of the concrete slab according to 6.1.2 should be not less than 0.2% of the "cross-sectional area of concrete above the ribs" for unshored construction, and 0.4% for shored construction. Please note this provision is only specified for beams in buildings. No similar provision is specified for beams in bridges.

3.2 CANADIAN HIGHWAY BRIDGE DESIGN CODE (CSA S6:19)

The Canadian Highway Bridge Design Code, CSA S6:19 is the twelfth edition of CSA S6. CSA S6:19 published in Nov. 2019 is limit states based design code for all Canadian provinces and territories. In CSA S6:19, there are several sections related to shored construction for composite bridges, including section 5 "Methods of analysis", section 8 "Concrete structures", and section 10 "Steel structures". However, other sections that related to shored construction were also reviewed. In addition, the companion reference: Commentary on CSA S6:19, Canadian Highway Bridge Design Code (CSA, 2019) has been reviewed and the supplemental information was extracted.

(1) Analysis for dead load

Article 5.6.3 Analysis for dead load: with regards to dead load analysis, for skewed bridges with $\psi \leq 45°$, the longitudinal vertical shear forces at the obtuse corner shall be magnified by the skew factor F_s for slab on girder bridges, calculated as follows:

    i.   no consideration needs to be taken for skew effects due to dead load in unshored construction conditions; and

    ii.   for shored construction or for superimposed dead loads, for the exterior girder at the obtuse corner:

$$F_s = 1.2 - \frac{2.0}{(\varepsilon + 10)} \quad \text{where} \quad \varepsilon = \frac{Length\,of\,the\,bridge}{Girder\,Spacing} \tan \psi \qquad \text{Eq. (3-4)}$$

(2) Use of shored construction

Article 10.11.1 General under Section 10.11 Composite beams and girders: if the beams are shored during casting of the deck, the design methods used shall be subject to approval by the owner. Article C10.11.1 mentioned that composite bridges are generally unshored during the placement of the slab.

Other than these two articles, shored construction was not mentioned in CSA S6:19.

3.3 CISC CODE OF STANDARD PRACTICE (CISC, 2015)

The research team also reviewed another Canadian Code: CISC CODE OF STANDARD PRACTICE (CISC, 2015). Similar to the AISC manual, this publication is included in Part 7 of the CISC Handbook of Steel Construction, 11th Edition. Unfortunately, no article was found related to the shored construction of composite bridges.

3.4 AUSTRALIAN STANDARD: BRIDGE DESIGN (AS 5100:2017)

The AS 5100 bridge design code (Australian Standard, 2017) is a series of bridge design codes including nine parts. The research team mainly focused on reviewing Part 6: Steel and composite construction but other parts including Part 2: Design loads, Part 5: Concrete, and Part 7: Assessment. The research team also reviewed the commentary document for AS 5100.

After a thorough review of AS 5100 bridge design code, no provisions have been found related to the shored construction of composite bridges. However, during the review process, it is found that the New Zealand Standards include some articles related to shored construction. Thus, the New Zealand standard is added and discussed in the next section.

3.5 NEW ZEALAND STANDARD (NZS 3404:2009; NZS 3101:2006, REVISED 2017)

There are two New Zealand standards related to shored construction of composite bridges: (1) NZS 3404:2009 Steel structures standard and (2) NZS 3101:2006 Concrete structures standard. Although these two standards were published in the 2000s, they were revised with amendments and are still current. The related articles have been summarized by the research team, as follows.

(1) Calculation of Deflection

Article 6.8.5.1 Deflection after the removal of supports under Section 6.8.5 Shored composite construction (New Zealand Standard, 2006): if composite flexural members are supported during construction so that, after removal of temporary supports, the dead load is resisted by the full composite section, the composite member may be considered equivalent to a monolithically cast member for calculation of deflection. The curvatures resulting from differential shrinkage of precast and cast-in-place components and of the axial creep effects in a prestressed concrete member should be taken into account.

Article 6.8.5.2 Deflection of non-prestressed composite members under Section 6.8.5 Shored composite construction (New Zealand Standard, 2006): the long-term deflection of the precast member shall be investigated including the magnitude and duration of load prior to the beginning of effective composite action.

(2) Member Design

Article 18.5.2.1 Shored and unshored members under section 18.5.2 Composite concrete flexural members (New Zealand Standard, 2006): No distinction shall be made between shored and unshored members in the design for flexural strength of composite members for the ultimate limit state.

Please note that the articles mentioned in this section only apply to shored constructed concrete composite members are not intended to be used for steel composite members. Thus, the applicability needs to be further investigated.

3.6 JAPANESE SPECIFICATIONS (JRA SPECIFICATIONS FOR HIGHWAY BRIDGES, 2012 AND JSCE STANDARD SPECIFICATIONS FOR STEEL AND COMPOSITE STRUCTURES, 2009)

There are two Japanese specifications that related to shored construction of composite bridges: (1) JRA Specifications for Highway Bridges, Part 2 Steel Highway Bridges (JRA, 2002, translated to English in 2017), and (2) JSCE Standard specifications for steel and composite structures (JSCE, 2009, in English). Both of these specifications are current. The research team has reviewed both specifications. No specific article that discusses the shored construction was found in these two specifications. Shored construction of composite steel girder was mentioned in Chapter 15 of JSCE Standard specifications for steel and composite structures (JSCE, 2009). However, only a general description of two construction methods was included, written as "There are two types of the composite girder, each which has different stress distribution inside the steel girder and concrete deck. One is the shored construction in which the whole dead load and live load are resisted with the composite cross-section. Another is unshored construction." The research team also reviewed several journal publications that discussed the Japanese bridge specifications and recent

development of steel composite bridges in Japan (Nagai, 2005, Tamura, 2002, Fukui et al., 2005). However, no discussion related to shored construction was found in these researches.

3.7 COMPARISON AND KEY FINDINGS

As shown in Section A through Section E, the considerations on shored construction of composite bridges can be very different in different codes from different counties. It shows that there is no uniformity in practice for the design of composite bridges using shored construction. However, the review of these international codes provides different perspectives and it is valuable to develop a rational design guideline to be used in the U.S.

Based on the reviews presented in this chapter, the research team performed a qualitative comparison between these international codes, and the results are shown in Table 3-2.

Table 3-2. Comparison of international codes

Design Considerations	International Codes				
	EN 1994-2	**CSA S6:19**	**AS 5100:2017**[1]	**NZS 3101:2006**	**JRA 2012 and JSCE 2009**[2]
Restrictions on using shored construction	N[3]	Y	N	N	N
Stresses in structural steel	Y	N	N	N	N
Non-linear analysis	Y	N	N	N	N
Deflection	N	N	N	Y	N
Cracking of concrete	Y	N	N	N	N
Skew factor	N	Y	N	N	N
Member design	N	N	N	Y	N
Addressed loss of composite action	N	N	N	N	N

1: Australian code AS 5100:2017 doesn't include any article related to shored construction.
2: Japanese specifications only generally mentioned about shored construction but no specific provision was included.
3: "Y" denotes the consideration is included while "N" denotes the consideration is not included.

There are several key findings that can be concluded based on the international code review and the comparison between various international codes:

(1) Similar to AASHTO, CSA S6:19 includes the restriction on using shored construction. It is stated in CSA S6:19 that if the beams are shored during casting of the deck, the design methods used shall be subject to approval by the owner. All the other international codes don't have restrictions on using shored construction.

(2) The provisions in EN 1994-2 that related to shored construction mainly about the non-linear analysis of shored constructed composite section. These provisions can be a useful reference for developing a design guideline for the U.S.

(3) Cracking of concrete was also considered in EN 1994-1-1 and a minimum reinforcement ratio was specified for both shored and unshored construction. Although this consideration is taken for beams in buildings, it can be an approach to control tensile stress of concrete as well as cracking at intermediate supports.

4 U.S. AND INTERNATIONAL QUESTIONNAIRES AND SURVEY RESULTS

A survey is a "means for gathering information about the characteristics, actions, or opinions of a large group of people" (Pinsonneault and Kraemer, 1993). As shown in the literature review, the degree of acceptance and experience in shored construction varies from state to state and from country to country. Thus, a comprehensive survey is an efficient way to gather valuable information from bridge personnel from all over the U.S. and also from different countries.

In this report, the main goal is to design a survey questionnaire that can be used to collect data and information related to design policies, construction specifications, and construction experiences of shored construction of composite steel girder bridges. The research team has developed a State of Practice survey that was distributed to the practitioners and bridge personnel from domestic and international practitioners. The research team also has compiled a distribution list that includes personnel from various relevant agencies including State DOT bridge/structures offices, FHWA, and transportation agencies from other countries.

The research team also has distributed the U.S. and international questionnaires to the contacts on the distribution list developed in this report. After receiving all the responses, the researcher compiled and investigated the survey results. This state of practice survey has provided insights into the design policies, construction specifications, and construction experiences of shored construction of composite steel girder bridges. It also provided valuable information on existing steel bridges constructed with shored construction, which helped the research team to perform shored and pre-decked steel girder projects review.

4.1 SURVEY QUESTIONNAIRE

Introduction: This brief survey questionnaire is part of a research project sponsored by the Florida Department of Transportation (FDOT) that seeks to better understand the performance of composite steel girder bridges using shored construction. The answers provided helped the research team to determine the design policies, construction specifications, and experiences of transportation agencies relevant to this structure type and construction method.

Shored construction for beam-slab bridges uses temporary supports (discretely or fully supported) for the superstructure during the installation of metal forms and the pouring of concrete. Shored construction can also be used in an Accelerated Bridge Construction (ABC) practice where construction occurs at a nearby site and fully-formed elements are transported to the final site using self-propelled modular transporters (SPMTs). A pre-decked steel girder that is shored during the pouring of the deck is another example that offers an advantage for ABC projects.

Part 1: Pre-interview questions

1. Have any slab on steel girder bridges been built in your state/country/region with a fully or discretely shored deck placement process?
2. Have any slab on concrete beam bridges been built in your state/country/region with a fully or discretely shored deck placement process?
3. Is shoring (fully or discretely) of a slab on steel girder during deck placement permitted in your agency?
    a. If yes, what types of construction have shored construction been approved for, e.g., concrete and/or steel beams; pre-decked girders and/or cast-in place deck; simple and/or continuous spans?
    b. If no, what is the driving factor in preventing your agency from permitting it (e.g. cost, concerns on composite action, long-term creep and shrinkage effects, redecking)?
4. Please provide contact information so we may follow up as necessary.

Part 2: Post-survey follow up interview (If the agencies answer yes to question 1 and 3 in pre-interview):

General

1. What types of construction have shored construction been approved for, e.g., accelerated bridge construction, and/or cast-in place?
2. Please provide a list of bridges where shored construction was implemented with a) bridge number, b) girder material (concrete or steel), c) simple or continuous span.
3. What was the performance of these bridges?
    a. More/less cracks observed
    b. Loss of composite action
    c. More/less deflection
    d. Other measures?
4. Were there any engineering and/or construction issues on these bridges?
5. What is your department's major concern with shored construction?

Design

6. Did you make any modifications to the AASHTO Design Specification to accommodate for shored construction? If yes, what are the modifications you made?

a. Was there a modification in the design of shear connectors to carry additional load from the composite section?
b. How are time-dependent effects considered? Was there a modification to the modular ratio used?
c. Others (Please specify your modifications)
7. In your design, were you able to use a lighter steel beam for shored construction comparing to unshored construction?
8. Are there any tensile stress limits for the concrete deck at the intermediate supports?
9. Did you require a tight camber tolerance for shored construction?
10. How were dead load deflections calculated?
11. What computer tools (software) were used in the analysis?
12. Was future deck replacement considered in the design? If so, what were the assumptions?

Construction

13. What type of shoring was used on the shored construction project?
14. When was the shoring removed?
15. Describe any construction issues that may have occurred.
Do you feel that shored construction could be beneficial for accelerated bridge construction?

4.2 DISTRIBUTION LIST

The research team has compiled a distribution list that consists of both domestic and international agencies. There are a total of fifty-eight agencies in this list including 52 agencies from the U.S. and six international agencies. Among the 52 agencies from the U.S., most of them are bridge divisions from fifty states and the District of Columbia, and the Office of Bridges and Structures from FHWA. Six international agencies represent the European Union, Canada, England, Australia, and New Zealand.

(1) Dayi Wang
   Office of Bridges and Structures, FHWA
   202-366-5604
   E-mail: dayi.wang@dot.gov
(2) David J. Welch, P.E.
   Design Bureau, Alabama Department of Transportation
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   Email: welchd@dot.al.us
(3) Richard Pratt, P.E.
   Bridge Design Office, Alaska Department of Transportation
   907-465-8890
   Email: richard.pratt@alaska.gov
(4) David Eberhart
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   602-712-7481

Email: DEberhart@azdot.gov

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(6) Caltrans Design Office
California Department of Transportation
916-657-0081
Email: hq.design.webmaster@dot.ca.gov

(7) Staff Bridge Branch
Colorado Department of Transportation
303-757-9309

(8) Bartholomew P. Sweeney, P.E.
Division of Bridges, Connecticut Department of Transportation
(860) 594-3272

(9) Bridge Design Office
Delaware Department of Transportation
302.760.2299

(10) Dawit Muluneh
Infrastructure Project Management Division, District of Columbia Department of Transportation
Email: dawit.muluneh@dc.gov

(11) Robert Robertson, P.E.
Structures Design Office, Florida Department of Transportation
850-414-4255
Email: Andre.Pavlov@dot.state.fl.us

(12) Bill DuVall
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(404) 631-1985

(13) Karen Chun
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(14) Matthew M. Farrar, P.E.
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(16) Stephanie Wagner
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Email: SWagner2@indot.in.gov

(17) James Nelson
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515-239-1206

Email: James.S.Nelson@iowadot.us

(18) Shawn Schwensen
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(785) 296-6449
Email: <u>Shawn Schwensen</u>

(19) Bridge Maintenance and Design Branch
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(20) Zhengzheng "Jenny" Fu
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225-379-1321
Email: zhengzheng.fu@la.gov

(21) Richard Crawford, PE
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207-624-3400
Email: projectdev.mainedot@maine.gov

(22) Bridge Administrative Section
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(23) Jonathan Gulliver
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(24) Bradley Wagner
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(25) MnDOT Bridge Office
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(26) Justin Walker
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(32) Bridge Division
New Jersey Department of Transportation
609-530-4231

(33) Kathy Crowell, P.E.
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(34) James Flynn, P.E.
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4.3   SURVEY RESULTS

4.3.1   Results from Pre-Interview Survey

The research team has received responses from 31 states and the Ontario Ministry of Transportation of Canada. Figure 4-1 shows the states that have responded in yellow.
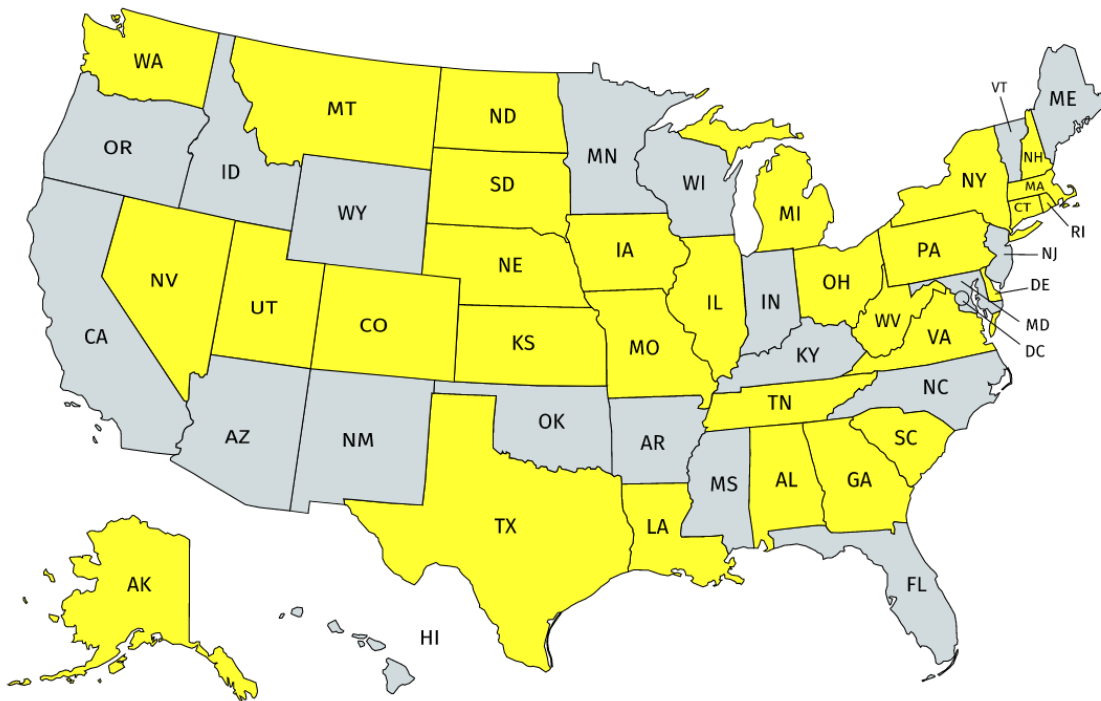
Figure 4-1. State agencies responded to the pre-interview survey (highlighted in yellow)

4.3.1.1 *Question 1: Have any slab on steel girder bridges been built in your state/country/region with a fully or discretely shored deck placement process?*

As shown in Figure 4-2, among all the responses, Michigan, Rhode Island, West Virginia, Utah, and Nebraska answered yes.

1. Have any slab on steel girder bridges been built in your state/country/region with a fully or discretely shored deck placement process?
   More Details

● Yes	5
● No	28

Figure 4-2. Responses to Question 1

4.3.1.2 *Question 2: Have any slab on concrete beam bridges been built in your state/country/region with a fully or discretely shored deck placement process?*

As shown in Figure 4-3, among all the responses, Georgia, Utah, Nebraska, and Louisiana answered yes.

2. Have any slab on concrete beam bridges been built in your state/country/region with a fully or discretely shored deck placement process?
   More Details

● Yes	4
● No	29

Figure 4-3. Responses to Question 2

4.3.1.3 Question 3: Is shoring (fully or discretely) of a slab on steel girder during deck placement permitted in your agency?

As shown in Figure 4-4, among all the responses, Nevada, Maryland, New Hampshire, West Virginia, Montana, Utah, and Georgia answered yes.

Figure 4-4. Responses to Question 3

4.3.1.4 Question 4: If you answered yes to question 3, what types of construction have shored construction been approved for (e.g., concrete and/or steel beams; pre-decked girders and/or cast-in-place deck; simple and/or continuous spans)?

As shown in Table 4-1, among seven states that responded yes to question 3, Nevada, Maryland, New Hampshire, and Montana don't prohibit the use of shored construction but have not been used. West Virginia, Michigan, Georgia, Utah, and Nebraska have prior experiences in shored construction thus the follow-up survey was sent to them.

Table 4-1. Responses to Question 4

State	Response
Nevada	Shoring is not prohibited by any specification but has not been used.
Maryland	Shoring is not prohibited by any specification but has not been used.
New Hampshire	Shoring is not prohibited by any specification but has not been used.
Montana	Shoring is not prohibited by any specification but has not been used.

Table 4 1. Responses to Question 4, cont'd

State	Response
West Virginia	Steel beams with cast-in-place deck.
West Virginia	Steel beams with cast-in-place deck.
Michigan	Shored construction was done in the past on some bridges in Michigan, with thin steel superstructure sections, and shoring required to support the dead load of deck concrete, until composite section is achieved.
Georgia	We had one project that constructed a CIP concrete deck on Bulb Tees on temporary supports at the project site then were moved via trailers to the bridge site and set on final substructure. The bridge consisted of 3 single-spans. This was ABC construction with a road closure duration of 60 days.
Utah	UDOT allows steel girder and prestressed concrete girder superstructures where the complete superstructure is constructed off-site on shoring and then moved into final location using SPMTs or where the superstructure is constructed on falsework adjacent to the final location and slid into place. Other than this ABC method, UDOT does not allow shored construction.
Nebraska	All the above

4.3.1.5 *Question 5: If you answered no to question 3, what is the driving factor in preventing your agency from permitting it (e.g. cost, concerns on composite action, long-term creep and shrinkage effects, redecking)?*

As shown in Figure 4-5, based on the survey results from question 5, the driving factors that hinder using of shored construction rank from high to low are as follows: (1) redecking, (2) cost, (3) creep and shrinkage, (4) composite action, and (5) lack of design software.

5. If you answered no to question 3, what is the driving factor in preventing your agency from permitting it (e.g. cost, concerns on composite action, long-term creep and shrinkage effects, redecking)?
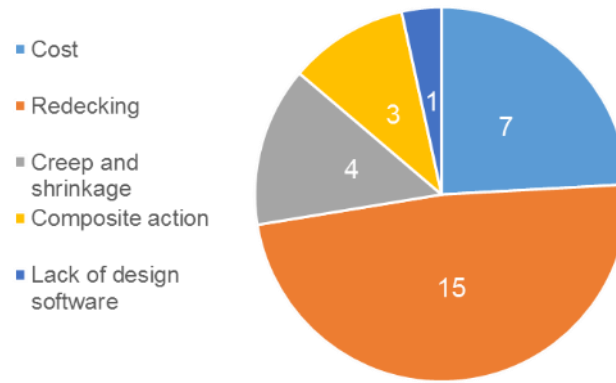
Figure 4-5. Responses to Question 5

4.3.2 <u>Results from Follow-up Survey</u>

The research team has received responses to a follow-up survey from Michigan and West Virginia. In addition, the research team has conducted email and phone interviews with Utah and Virginia. Utah clarified that in fact, they didn't use shored construction for their ABC project. The supports they used for beam-slab modular were the same as the final support conditions. Virginia has constructed two bridges using shored construction including the James River bridge and a nearby ramp bridge. The follow-up survey composes of three sections: (1) general questions, (2) design questions, and (3) construction questions. Table 4-2 presents the responses to general questions of the follow-up survey. It is worth noting that all these states stated they didn't have a problem with the performance of the shored constructed bridges. Particularly, West Virginia praised the performance of the shored constructed bridge since its deflection is less compared with the twin bridge that was unshored.

Table 4-2. Responses to the follow-up survey: general

General	State		
	Michigan	West Virginia	Virginia
What types of construction have shored construction been approved for?	Prefabricated elements, entire bridge superstructures, complex bridge superstructure skeletons (arch bridge with PT tie girder).	Deck was cast-in-place on stay-in-place forms at the bridge site on steel multi-girder superstructure.	Prefabricated elements
Please provide a list of bridges	I-96 bridges in the 1970's US-131 over Three Mile Road M-50 over I-96 2nd Ave over I-94	20-77-116.02 Southbound (20A818) - Steel plate girder - single-span	I-95 bridges over James River
What was the performance of these bridges?	Deflections of temporary substructures as the bridges were slid into place. Overall structure performance okay.	No abnormal cracking has been observed in the concrete bridge deck or the parapet walls. Negative camber has been observed in the girders of the northbound twin structure, which is of equal span length and was constructed without shored construction. This southbound twin has less dead load deflection.	bridge has been open to traffic for 20(ish) years, no significant problems that I am aware of related to positive moment.
Were there any engineering and/or construction issues on these bridges?	Yes, many issues because has to change from unshored to shored, which were worked out during shop drawing phase, construction, and submission of move and monitoring plans.	No known construction issues occurred related to shored construction on the southbound twin.	N/A
What is your department's major concern with shored construction?	Ensuring it is designed appropriately, signed and sealed by a Michigan PE.	No major general concerns with shored construction, but should be evaluated on a case-by-case basis for each bridge.	redecking

Table 4-3 shows the summary of the responses for design-related questions. It is worth noting that Michigan was able to use a lighter steel beam when shored construction was used. Virginia

was also able to reduce the top flange width from 12" to 10". Although West Virginia used the same size of the beam, but the deflection was reduced.

Table 4-3. Responses to the follow-up survey: design

Design	State		
	Michigan	**West Virginia**	**Virginia**
Did you make any modifications to the AASHTO Design Specification to accommodate for shored construction?	None	No known modifications were made to the AASHTO code (LRFD Bridge Design Specifications).	No. We just applied the deck as a superimposed dead load (n=3) the way we would for a parapet.
In your design, were you able to use a lighter steel beam for shored construction comparing to unshored construction?	In some cases, yes.	The plate girder size remained the same size between the northbound (unshored) and southbound (shored) twin structures but deflection was minimized on the southbound twin.	top flange was 10" wide instead of 12"
Are there any tensile stress limits for the concrete deck at the intermediate supports?	Just those specified by AASHTO.	No.	N/A
Did you require a tight camber tolerance for shored construction?	Yes, we specified cambers and deflections, along with tolerances, and monitored these during construction operations.	No, we only attempted to achieve the deflection that was calculated in the initial line girder design that had assumed non-shored construction.	No, we designed for no bolster (haunch) in the section properties.
How were dead load deflections calculated?	Normal dead load deflection theory - distributed load, = 5wl^4/384EI	Utilizing a line girder analysis. Afterward, we began utilizing a finite element analysis to determine dead load deflections since it was felt that system analysis better predicted dead load deflections.	N/A

Table 4-3. Responses to the follow-up survey: design, cont'd

Design	State		
	Michigan	West Virginia	Virginia
What computer tools (software) were used in the analysis?	Some proprietary, some commercial.	I believe MDX but not for certain.	N/A
Was future deck replacement considered in the design? If so, what were the assumptions?	Yes. Assumption the deck will be removed the reverse sequence it was placed.	Future deck replacement was not considered since the initial design assumed non-shored construction. However, if the design considered shored construction we would need to consider future deck replacement options.	No, but it should be considered for future projects.

Table 4-4 shows the responses to construction questions. Michigan, Virginia, and West Virginia responded that they feel shored construction could be beneficial for ABC while West Virginia stated a case-by-case basis evaluation is needed.

Table 4-4. Responses to the follow-up survey: construction

Construction	State		
	Michigan	West Virginia	Virginia
What type of shoring was used on the shored construction project?	Mainly steel members, some timber mat footings, some pile foundations.	Unsure on type of shoring used.	Shored at quarter points
When was the shoring removed?	After permanent placement.	After the deck and parapet walls achieved proper strength.	Until the concrete has attained a strength of 26.25 MPa.
Describe any construction issues that may have occurred.	Excessive deflections of sliding rail at the transition from temporary abutment to permanent abutment due to change in stiffness. Added additional foundation piles.	None, other than more deflection than anticipated on the northbound twin that did not utilize shored construction.	N/A

Table 4-4. Responses to the follow-up survey: construction, cont'd

Construction	State		
	Michigan	**West Virginia**	**Virginia**
Do you feel that shored construction could be beneficial for accelerated bridge construction?	Yes.	It could be beneficial but would need to be evaluated on a case-by-case basis.	Yes, but the benefits is minimal if consider crane cost.

4.3.3   Findings

The research team conducted a two-part survey on various topics related to shored construction. The findings of the survey are summarized below:

(1) Most of the States don't have prior experiences in shored construction.

(2) The major concerns the agencies have about shored construction are:

- Re-decking;
- Cost;
- Creep and shrinkage.

(3) Michigan, Virginia, and West Virginia had successful experiences in shored construction. Shored constructed bridge either demonstrated less deflection or was designed with a smaller section.

5 SHORED AND PRE-DECKED STEEL GIRDER PROJECTS REVIEW

5.1 IDENTIFY STEEL GIRDER BRIDGES USING SHORED CONSTRUCTION

In AASHTO LRFD-9, article C6.10.1.1.1a, it says "There have been only a very limited number of demonstration bridges built with shored construction in the U.S.". In light of this statement, the research team aimed at identifying existing steel girder bridges that have used shored construction. The detailed findings are presented below.

5.1.1 Michigan

In the follow-up survey, Michigan responded that they have constructed a series of concrete deck on steel girder bridges using shored construction in the 1970s. The research team further followed up with additional interviews regarding these bridges. It was revealed that the shored construction was proposed by the contractor during the bridge construction in the 1970s, thus there is no significant record noting these bridges were shored constructed. MDOT also responded that they had to analyze these bridges for deck placement in the unshored condition when they replaced the bridge decks, but that was all in the background without documentation of that. As shown in Figure 5-1, the blue dots are shored constructed bridges built between 1970 to 1974. They are all located in Wayne county along I-96 just northwest of Detroit. As shown in Table 5-1, overall, most of these bridges are in good or fair shape despite they are about 46 to 50 years old. The detailed information on these bridges can be found in Table 5-2.

Figure 5-1. Locations of shored constructed steel girder bridges in Michigan

Table 5-1. Condition of shored constructed steel girder bridges in Michigan

	Good	Fair	Poor
All Bridges	29 (69.05%)	12 (28.57%)	1 (2.38%)
		Total: 42	

Table 5-2. Details of shored constructed steel girder bridges in Michigan

Structure Number	Year Built	# of Spans in Main Unit	Structure Length (ft)	Maximum Span (ft)	Bridge Roadway Width (ft)	Skew Angle (degrees)
11490	1974	2	175.9	86.9	34.1	0
11498	1971	2	158.1	81.7	51.8	0
11494	1974	2	175.9	86.9	36.1	0
11506	1970	2	166	81.7	61	0
11502	1974	2	176.8	87.3	64	4
11519	1971	2	166.3	82	24	0
11515	1973	2	175.9	86.9	20	0
11523	1974	2	176.5	86.9	19.7	0
11527	1974	2	175.9	86.9	27.9	0
11977	1971	3	275.9	175.9	42	64
11972	1971	4	437	131.9	60.4	0
11488	1974	2	178.5	87.9	64	0
11496	1974	2	175.9	86.9	34.1	0
11492	1974	2	178.5	87.9	64	0
11508	1970	2	238.8	139.8	36.7	4
11504	1974	2	206.4	103.3	29.9	1
11517	1973	2	176.2	87.9	34.1	0
11521	1971	2	166.3	83	20	0
11525	1974	2	175.9	86.9	24	0
11529	1974	2	175.9	87.9	40.4	0
11974	1971	4	453.1	133.9	34.4	0
11489	1974	2	178.5	87.9	64	0
11497	1971	2	166.3	82	87.9	0
11493	1974	2	175.9	86.9	34.1	0
11505	1970	7	899.9	165	87.9	0
11514	1973	2	220.1	122.4	27.9	6
11518	1971	2	166.3	82	24	0
11522	1974	2	176.5	86.9	20	0
11526	1974	2	175.9	86.9	18	0
11530	1974	2	175.9	86.9	18	0
11975	1971	3	311	192.9	42	63
11487	1972	3	377	161.4	29.5	45
11495	1974	2	176.5	86.9	64	0
11491	1974	2	182.1	89.6	51.8	11
11507	1970	2	166	81.7	61	0

Table 5-2. Details of shored constructed steel girder bridges in Michigan, cont'd

Structure Number	Year Built	# of Spans in Main Unit	Structure Length (ft)	Maximum Span (ft)	Bridge Roadway Width (ft)	Skew Angle (degrees)
11503	1974	2	200.1	98.8	40	0
11499	1971	2	177.5	87.9	34.1	0
11516	1973	2	175.9	86.9	20	0
11520	1971	2	166.3	82	20	0
11528	1974	2	175.9	86.9	36.1	0
11524	1974	2	175.9	86.9	24	0
11973	1971	4	453.1	133.9	53.8	0

5.1.2 <u>West Virginia</u>

In the follow-up survey, West Virginia responded that they have built a southbound twin bridge on I-77 using shored construction in 2007. Originally shored construction was not assumed. The northbound twin structure was built first and more deflection was observed in the steel girders than anticipated when pouring the concrete bridge deck. Therefore, the southbound structure utilized shored construction in order to minimize deflection when it was constructed. No known construction issues occurred related to shored construction on the southbound twin and the southbound twin showed less dead load deflection. Figure 5-2 and Figure 5-3 show the location and rating history of bridge #20A818 in West Virginia, respectively. The superstructure maintained a rating of 8 since the bridge was built in 2007. The deck condition rating decreased from 8 to 7 in 2015 but is still in good condition. On a similar note, the substructure rating decreased from 8 to 7 in 2015 as well indicating some minor problems but the rating was rescored to 8 in 2018 likely due to repair.
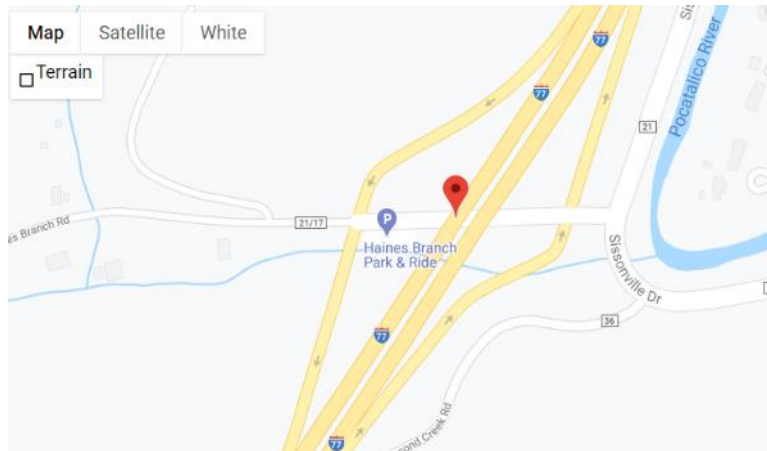
Figure 5-2. Location of a shored constructed steel girder bridge (#20A818, WV)
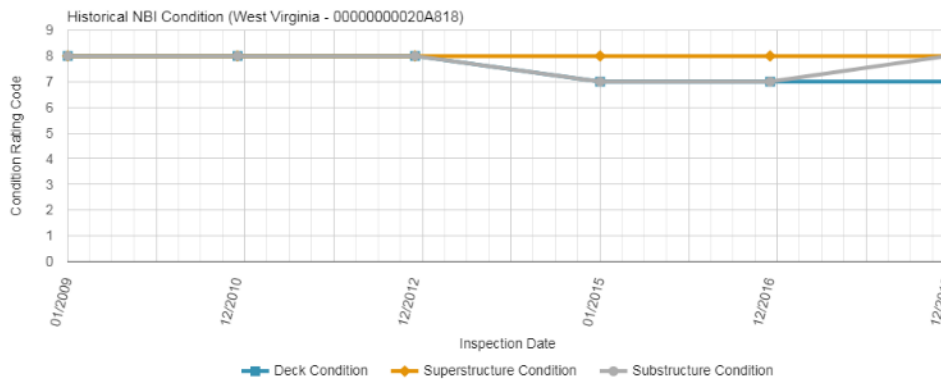
Figure 5-3. Condition rating history for bridge #20A818, WV

General condition ratings (GCRs) are provided by the National Bridge Inventory (NBI) in the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* by the FHWA (FHWA, 1995), shown in Table 5-3. Structures with GCR of 7 or higher are in good condition. Those rated a 5 or 6 are in fair condition, while 4 and under are in poor condition. Note that transitions between good, fair, and poor conditions are substantial and ultimately change the bridge functionality and management procedure. Those in poor condition can no longer be fixed through maintenance and must undergo rehabilitation or replacement. As shown in Figure 5-3, the deck, superstructure, and substructure are all in good condition after 13 years of service.

Table 5-3. NBI general condition ratings (FHWA, 1995)

Code	Condition	Description	Actions
9	Excellent	-	Preservation Cyclic Maintenance
8	Very Good	No problems noted.	
7	Good	Some minor problems.	
6	Satisfactory	Structural elements show some minor deterioration.	Preservation Condition-Based Maintenance
5	Fair	All primary structural elements are sound but may have minor section loss, cracking, spalling, or scour.	
4	Poor	Advanced section loss, deterioration, spalling, or scour.	Rehabilitation or Replacement
3	Serious	Loss of section, deterioration, spalling, or scour have seriously affected primary structural components. Local Failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.	
2	Critical	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.	
1	"Imminent" Failure	Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic, but corrective action may put back in light service.	
0	Failed	Out of service. Beyond corrective action.	

5.1.3 New York

RT. 9 bridge over New York State Thruway (I-87) was built with a prefabricated decked stringer system using the shored construction method in 2004. This bridge (#1004939) has a span of 120.5 feet and a skew of 45 degrees. The units weighing as much as 128 tons were shipped to the site on special hauling rigs.

Figure 5-4 and Figure 5-5 show the location and rating history of bridge #1004939 in New York, respectively. Since the reconstruction in 2004, the bridge maintained good condition. Although the rating decreased from 9 to 8 in 2013 and again decreased from 8 to 7 for superstructure and substructure, the ratings are still above 6 indicating there are only minor problems that cyclic maintenance should be sufficient.
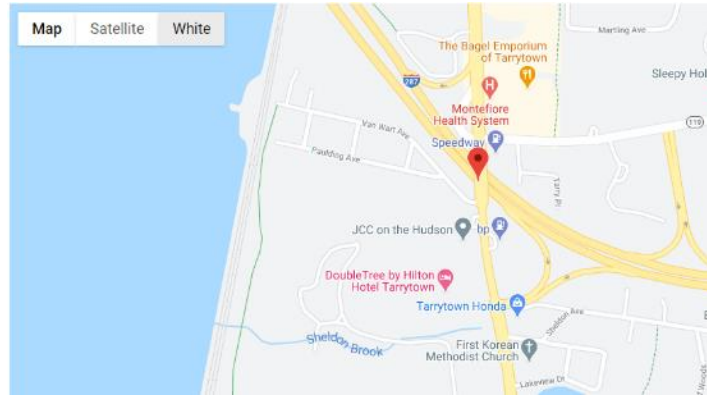
Figure 5-4. Location of bridge #1004939, NY
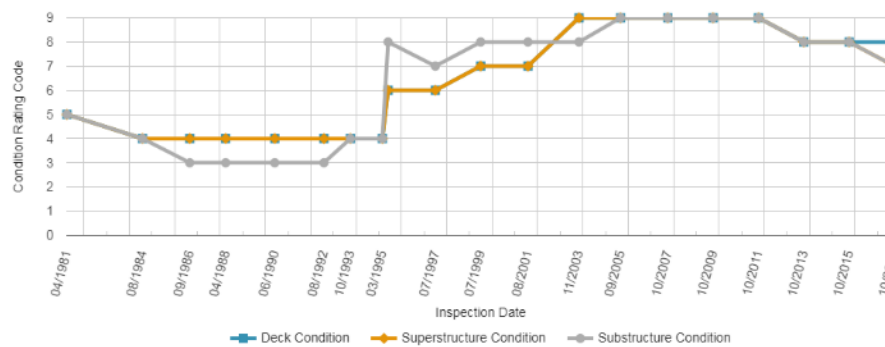
Figure 5-5. Condition rating history for bridge #1004939, NY (rebuilt in 2004)

5.1.4   Virginia

I-95 James River Bridge was built with full span length prefabricated composite units (PCU) up to 114 ft long for 102 superstructure spans in 2002. All these 102 spans are approach spans with 51 spans for each direction. The prefabricated composite units were fabricated at a nearby casting yard using shored construction. As shown in Figure 5-6, Each unit composes of two or three plate steel girders with a W-section diaphragm. The minimum concrete slab is 225 mm (8.86 in.)The beams and other framings for each unit were placed on bearing supports, then the framing was shimmed at ¼ points (steel DL was cambered for), then the deck/parapet was placed. The shoring was removed until the concrete has attained a strength of 26.25 MPa (3,800 psi). Beams were designed so that all concrete was treated as a superimposed dead load under the Standard Specifications. Figure 5-7 and Figure 5-8 show the location and rating history of bridge #21494 in Virginia, respectively. As shown in Figure 5-8, after the completion of the bridge in 2002, the rating decreased from 8 to 6 and 7 to 6 for deck and for superstructure and substructure,

respectively. Again the rating of superstructure and substructure decreased to 5 in 2007 and 2010, respectively. The changes in ratings at an early age (less than 10 years in service) indicate some deteriorations of the structure.
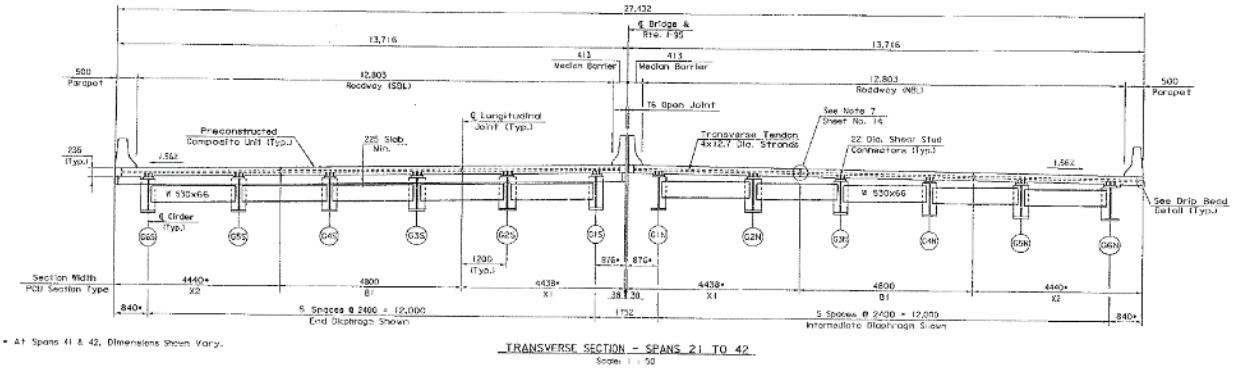
Figure 5-6. Transverse section of bridge # 21494 (spans 21 to 42), Virginia

Figure 5-7. Location of bridge # 21494, Virginia
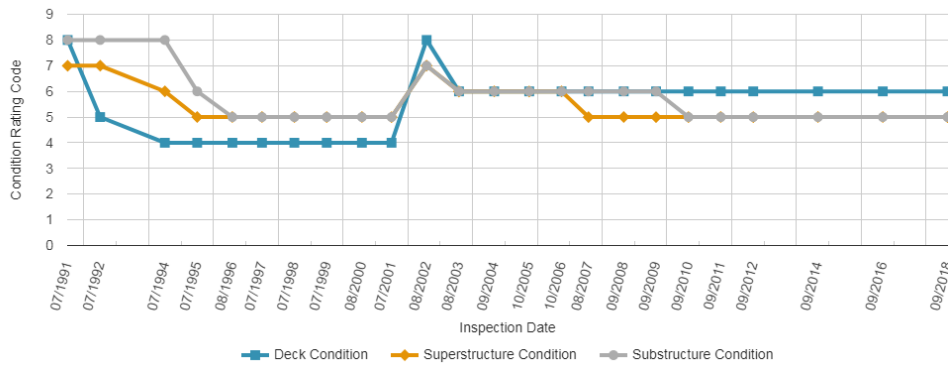
Figure 5-8. Condition rating history for bridge # 21494, Virginia (rebuilt in 2002)

5.1.5 Inverset System

Inverset-type concrete deck and steel composite systems have had a good track record for rapid superstructure replacement. For inverset system, the unit is cast in an inverted position at a prefabrication yard so that the deck is in compression in the final condition. Although inverse system doesn't use shoring during concrete pouring, the dead load is carried by the composite beam section. Thus essentially the behavior of inverse system should be similar to the ones using conventional shored conditions.

Inverset was at one time patented but the patent has run out. Fort Miller Co. rebranded it as Prefabricated Bridge Units and is a major fabricator of this type of system. Fort Miller Co. has produced spans up to 126-ft long with skews over 45 degrees. There are multiple bridges successfully used the inverset system:

1. US-15/29 bridge over Broad Run, Virginia;
2. Three Route 1 bridges, New Jersey;
3. Tappan Zee Bridge, Tarrytown, New York;
Sagamore Resort Bridge, Bolton Landing, New York.

5.1.6 Summary

Through the survey questionnaire, phone interview, and literature review, shored constructed steel composite bridges were identified. As shown in Table 5-4, these bridges represent a wide range of ages from just over 10 years to 50 years. They also represent both conventional and ABC construction methods. Besides these bridges, there are bridges built with the inverset system that can be treated as shored constructed.

Table 5-4. List of shored constructed bridges

State	Bridge Name	Structure #	Bridge Geometry	Year of Built/Reconstruction	Conventional/ABC
Michigan	I-96 bridges	Table 1	Table 1	1970 to 1974	Conventional
West Virginia	I-77 over 21/17	20A818	Width: 42 ft Span length: 136.8 ft number of spans: 1	2007	Conventional
New York	RT. 9 bridge over New York State Thruway (I-87)	1004939	Width: 66.9 ft Span length: 121.1 ft number of spans: 2	2004	Inverset/ABC
Virginia	I-95 James River Bridge	21494	Width: 89.9 ft Span length: 44 ft to 114 ft number of spans: 112	2002	ABC
Virginia	US-15/29 bridge over Broad Run	14189	Width: 42 ft Span length: 33.1 ft number of spans: 1	2013	Inverset
New Jersey	Three Route 1 bridges	N/A[1]		N/A	Inverset
New York	Tappan Zee Bridge	N/A[1]		N/A	Inverset
New York	Sagamore Resort Bridge	N/A[1]		N/A	Inverset

No detailed information has been obtained for these three bridges.

5.2 CONSTRUCTION AND PERFORMANCE

Through survey questionnaires, phone interviews, and literature reviews, shored constructed steel composite bridges were identified. Among the forty-five (45) bridges identified, most of them are single-spans. There is one two-span continuous bridge (RT. 9 bridge over New York State Thruway in New York), and one three-span continuous bridge (US-24 over I-96). The as-built construction drawings, specifications, and in-service performance information have been collected from respective state agencies. In addition, the inspection reports were obtained from Michigan and West Virginia. Based on the information collected, the construction and performance of these shored constructed bridges were reviewed and the findings were summarized.

The performance of these bridges was evaluated through a review of the inspection report and email and phone interviews. The quantitative measures of performance of these bridges were

obtained from the Long Term Bridge Performance (LTBP) program database using general condition ratings (GCRs). GCRs are provided by the National Bridge Inventory (NBI) in the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* by the FHWA (FHWA, 1995). Structures with a GCR of 7 or higher are in good condition. Those rated a 5 or 6 are in fair condition, while 4 and under are in poor condition. Note that transitions between good, fair, and poor conditions are substantial and ultimately change the bridge functionality and management procedure. Those in poor condition can no longer be fixed through maintenance and must undergo rehabilitation or replacement.

5.2.1 <u>Construction</u>

Table 5-5 shows the responses to construction-related questions from Michigan, West Virginia, and Virginia. Michigan's bridges were built in the 1970s and the shored construction was proposed by the contractor during the bridge construction. However, there is no significant record noting these bridges were shored constructed. MDOT also responded that they had to analyze these bridges for deck placement in the unshored condition when they replaced the bridge decks but that was all in the background without documentation. The shorings used in Michigan bridges were mainly steel members, some timber mat footings, and some pile foundations. For bridges in West Virginia, temporary bent was used for shoring. However, the details of shoring were not recorded in the as-built drawings for bridges in Michigan and West Virginia. Bridges in Virginia are ABC bridges and they were shored at a precast yard at quarter points until the concrete attained a strength of 3,800 psi (26.25 MPa). There is no information on construction details for the New York bridge. The research team only learned that Rt. 9 bridge over New York State Thruway (I-87) was a shored constructed Inverset bridge from a Fort Miller's brochure. However, NYSDOT later stated they didn't use shoring for their Inverset bridges.

Table 5-5. Responses to follow-up survey: construction

Construction	State		
	Michigan	**West Virginia**	**Virginia**
What type of shoring was used on the shored construction project?	Mainly steel members, some timber mat footings, some pile foundations.	Temporary bent	Shored at quarter points
When was the shoring removed?	After permanent placement of concrete slab.	After the deck and parapet walls achieved proper strength.	Until the concrete has attained a strength of 26.25 MPa.
Describe any construction issues that may have occurred.	Excessive deflections of sliding rail at the transition from temporary abutment to permanent abutment due to change in stiffness. Added additional foundation piles.	None, other than more deflection than anticipated on the northbound twin that did not utilize shored construction.	N/A

5.2.2   Performance

5.2.2.1   Michigan

Figure 5-9 shows the overall condition rating history for bridges in Michigan. It shows almost half of these 42 bridges deteriorated to poor condition in 1998 with about 25 to 28 years of service. Through repair and maintenance, over 90% of bridges were restored to fair condition in 2004. Then after another 10 years of service (40 to 45 years total), nearly 65% of the bridges (27 of 42) were reconstructed so the rating was restored to good condition.

Compared with other bridges built using unshored construction methods between 1970 and 1974 in Wayne county, the condition rating history is comparable. As shown in Figure 5-10, for 257 bridges, almost 50% of bridges deteriorated to poor condition in 1999, which is very similar to shored constructed bridges shown in Figure 5-9. Then the percentage of bridges in poor conditions decreased over the years with the percentage of bridges in good conditions jumping in 2013 and 2015 due to bridge reconstruction. Thus, by comparing the rating history of shored constructed bridges with unshored constructed bridges, no significant difference has been identified between shored constructed and unshored constructed bridges.
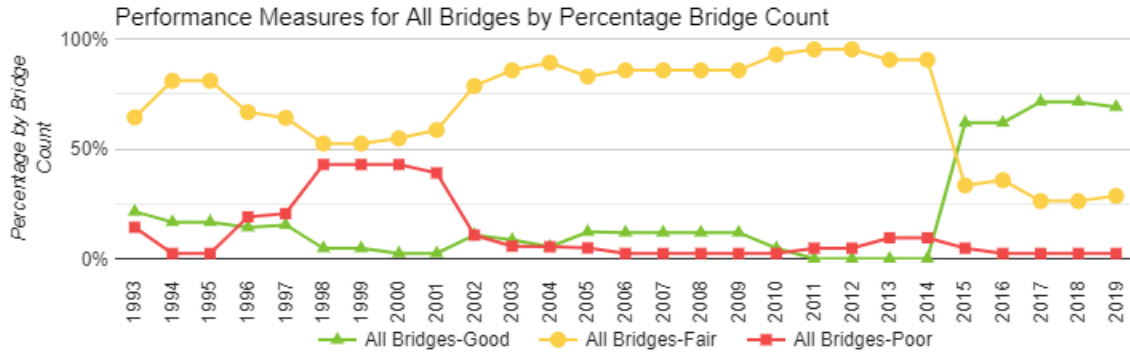
Figure 5-9. Condition rating of shored construction bridges in Michigan (1993-2019)
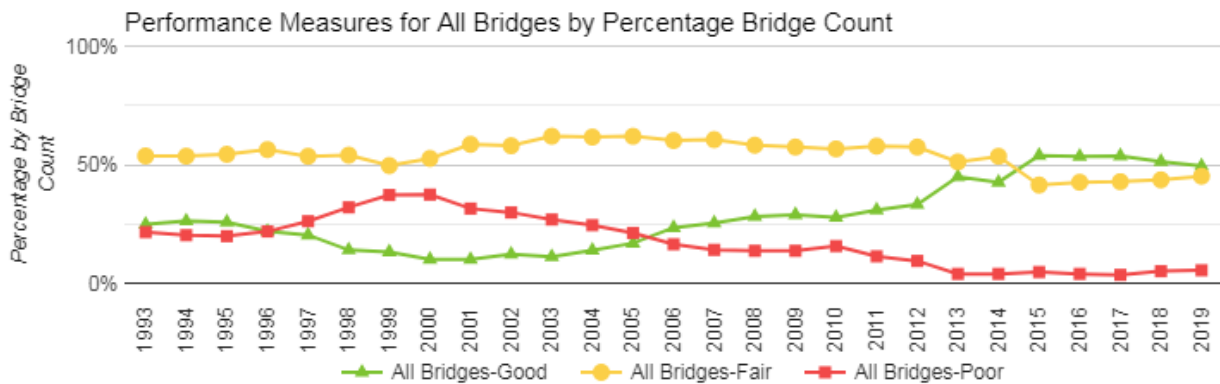
Figure 5-10. Condition rating of bridges built between 1970 and 1974 in Wayne County, Michigan (1993-2019)

The research team also further examined the performance of each individual bridge. The span lengths of shored constructed bridges in Michigan range from 80 ft to 200 ft. The majority of them (32 bridges) range from 80 ft to 100 ft while six bridges range from 100 ft to 150 ft and four bridges are above 150 ft. Thus, the rating histories of three typical bridges were extracted to represent bridges with span lengths of 80 to 100 ft, 100 to 150 ft, and 150 ft up.

Figure 5-11 shows the rating history of an 87 ft single-span bridge, the rating of superstructure dipped to 6 after 10 years of service in 1985 and again deteriorated to 6 in 2010 until reconstruction in 2014. The deck rating deteriorated to 5 in 1995 after about 20 years of service. Repairs and maintenance kept the deck rating at 6 until 2008.

Figure 5-12 shows the rating history of a 134 ft single-span bridge. Compared to Figure 5-11, the deck was maintained at a satisfactory rating of 6 while the deterioration of the superstructure

is more severe. The superstructure rating dropped to 3 in 1997 after 26 years of service. After a major rehabilitation, the rating was able to maintain at 7 and above.

Figure 5-13 presents the rating history of a 161 ft single-span bridge. Compared to shorter span bridges presented in Figure 5-11 and Figure 5-12, the deterioration of superstructure took place faster with rating dropping from 8 to 4 in 5 years (1983 to 1987). On the other hand, the deck rating was well maintained at a satisfactory rating of 6 until 2010 and the deck was replaced in 2014 after 42 years of service.
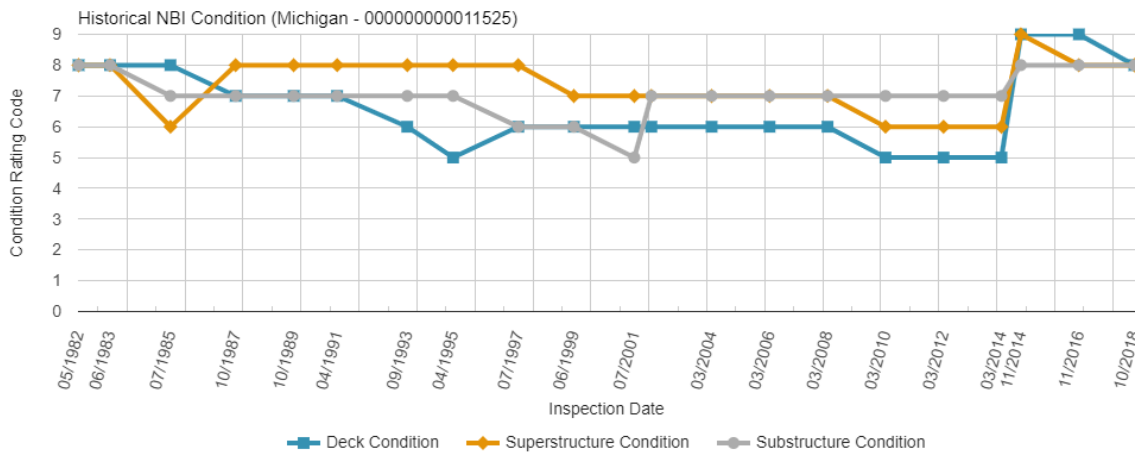
Figure 5-11. Condition rating of bridge #11525 in Michigan (87 ft)
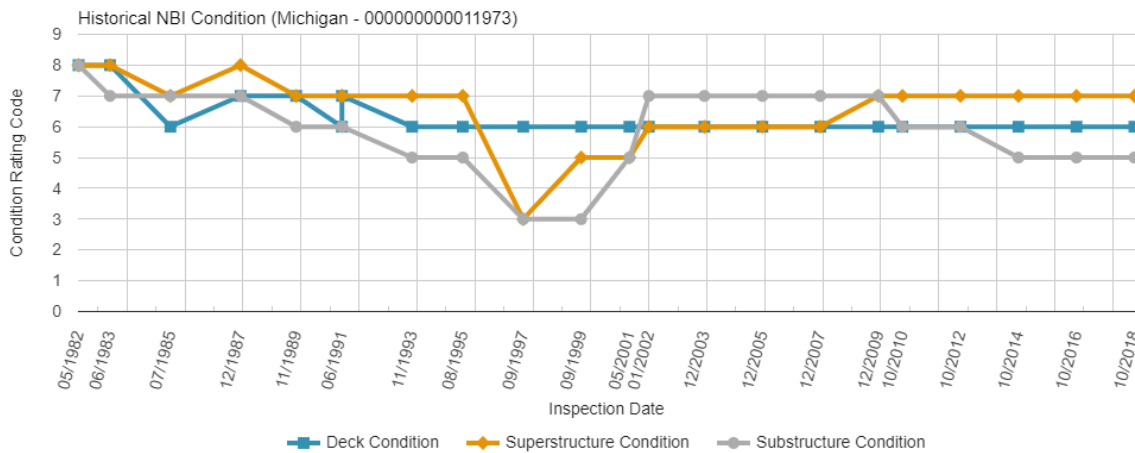
Figure 5-12. Condition rating of bridge #11973 in Michigan (134 ft)
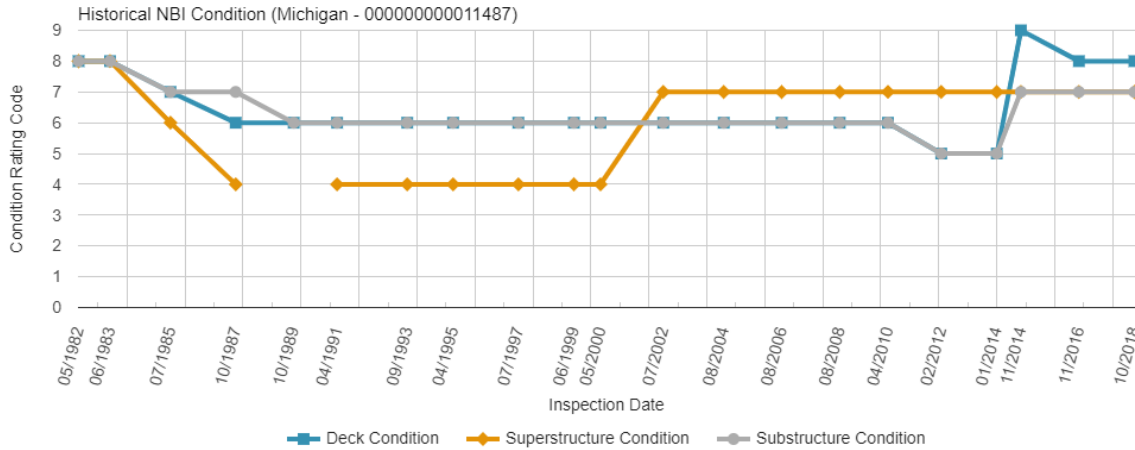
Figure 5-13. Condition rating of bridge #11487 in Michigan (161 ft)

There is only one continuous bridge among shored constructed bridges in Michigan, Rt. 24 bridge over I-96 (structure #11505). It is a seven-span bridge with the middle spans is a three-span continuous. The research team has obtained the as-built drawings and most recent inspection report for this bridge. As shown in Figure 5-14, the deck condition of this bridge deteriorated to rating 3 in 1997 after 27 years of service. Thus the deck was replaced in 2003. However, the superstructure maintained a fair condition after over 50 years of service. Figure 5-15 shows the field photos taken during the most recent inspection (04/20). It was observed that there are light areas of corrosion of cross frames and flanges near joints. There is approximately 25% section loss of diaphragms a pier 4N and minor to moderate rusting at some beam ends.
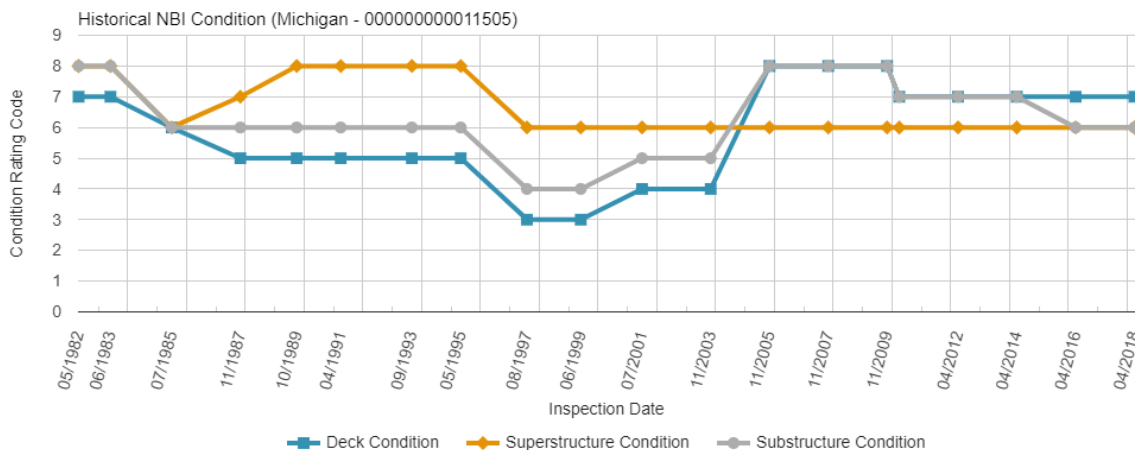
Figure 5-14. Condition rating of bridge #11505 in Michigan

Figure 5-15. Photos from inspection report (Left: Span 3s, right: Span 4s)

5.2.2.2 *West Virginia*

Shored construction in West Virginia demonstrated promising results as the shored constructed southbound twin showed less dead load deflection in comparison with the unshored northbound twin. No known construction issues occurred related to shored construction on the southbound twin. Figure 5-16, Figure 5-17, and Figure 5-18 show the plan, elevation, and typical cross-section of bridge #20A818, respectively. Bridge #20A818 is a single-span bridge with a span length of 136.38 ft and a skew angle of 35°. Figure 5-19 and Figure 5-20 show the rating history of bridge #20A818 (shored) and #20A831 (unshored) in West Virginia, respectively. The rating history is almost identical for these two bridges. The superstructure maintained a rating of 8/9 since the bridge was built in 2007. The deck condition rating decreased from 8 to 7 in 2015 but is still in good condition. On a similar note, the substructure rating decreased from 8 to 7 in 2015 as well indicating some minor problems but the rating was restored to 8 in 2018 likely due to repair for #20A818.
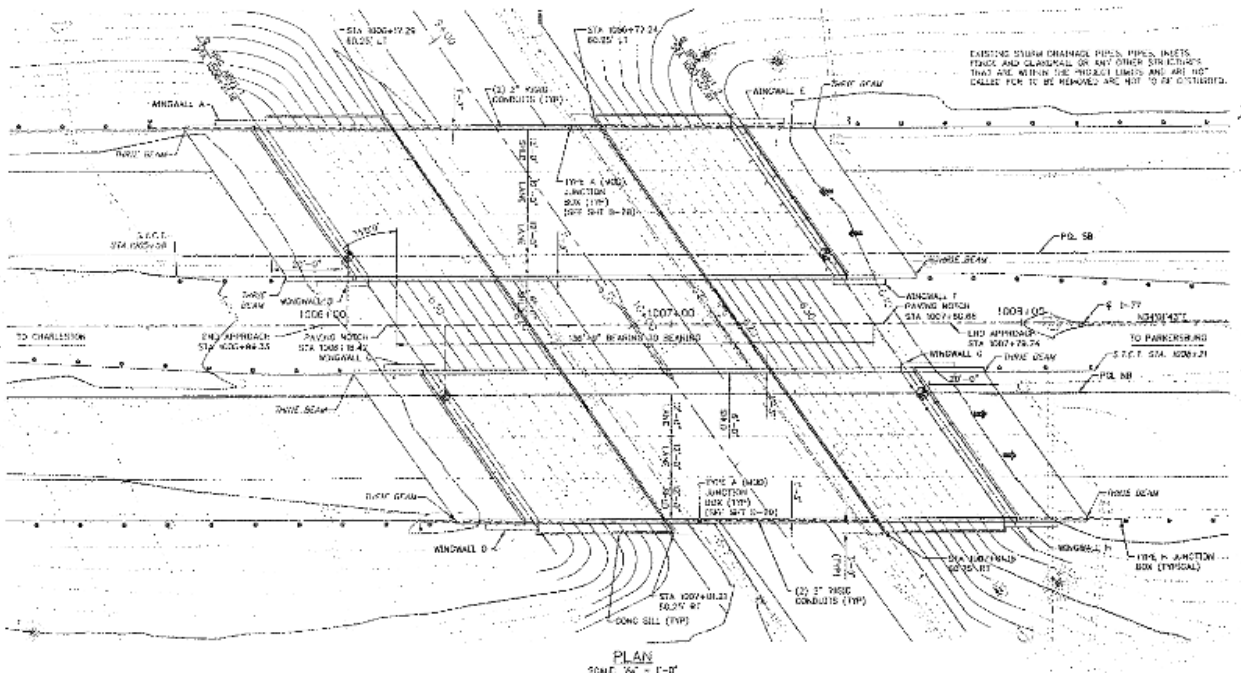
Figure 5-16. Plan of bridge #20A818 (shored)
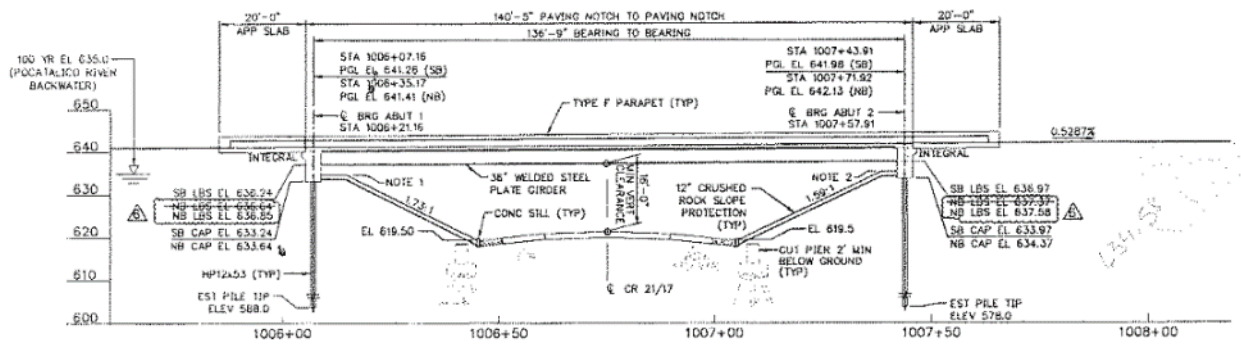
Figure 5-17. Elevation of bridge #20A818 (shored)
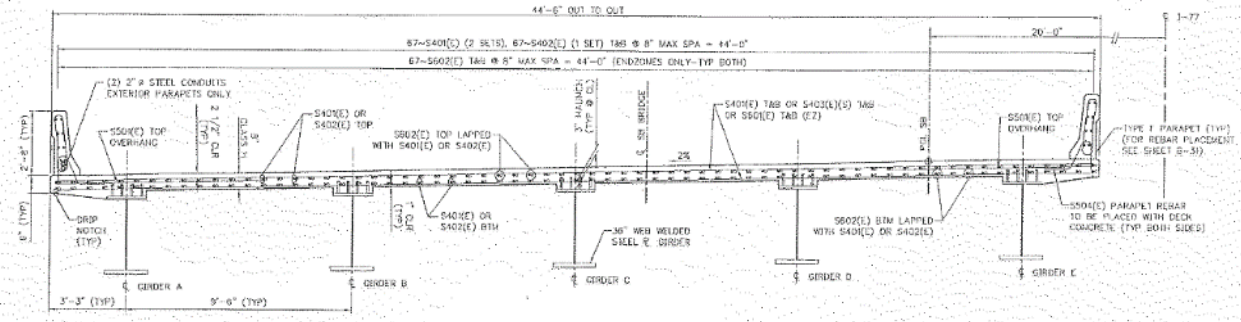
Figure 5-18. Typical cross-section of bridge #20A818 (shored)

The research team also has reviewed the most recent inspection reports for both bridges. As shown in Table 5-6, since it was built in 2007, both bridges are still in good condition and have no history of fatigue cracking. However, the inspection report revealed that the girders of the unshored #20A831 bridge display substantial negative camber due to construction error compared to only slight negative camber for the shored #20A818 bridge. Since both bridges use the same size of girders, the shored constructed bridge shows better performance after about 13 years of service.
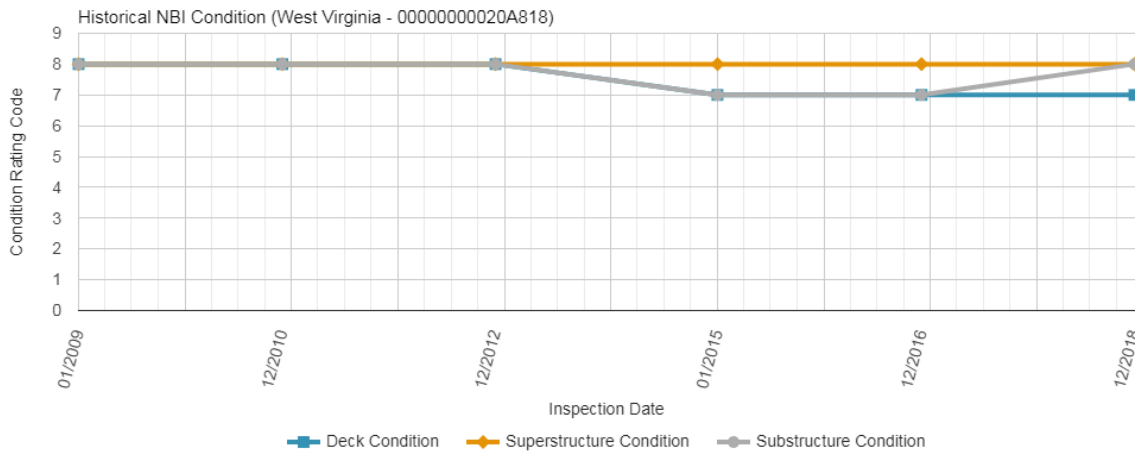
Figure 5-19. Condition rating history for bridge #20A818 (shored), WV
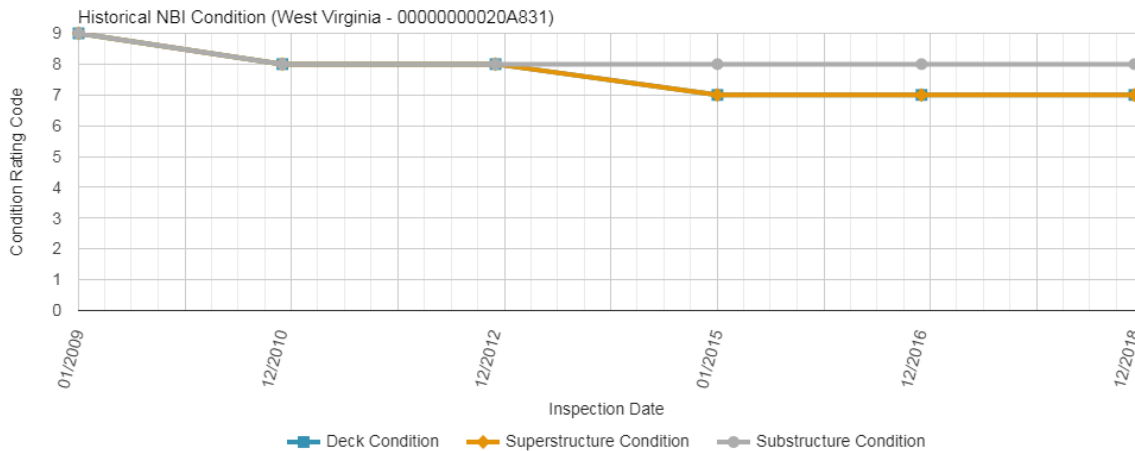
Figure 5-20. Condition rating history for bridge #20A831 (unshored), WV

Table 5-6. Inspection results for bridges #20A818 and #20A831

Criteria	Bridge	
	#20A818 (shored)	**#20A831 (unshored)**
Fracture Critical Members	This structure has no fracture critical members.	This structure has no fracture critical members.
Bearings	There are no visible or functioning bearing devices due to the fully-integral design of the structure.	There are no visible or functioning bearing devices due to the fully-integral design of the structure.
Girders and Diaphragms	These members, which are constructed of weathering steel, are still in good condition. We again observed only a slight negative camber in the girders. No other noteworthy deficiencies are present in the superstructure members.	These members, which are constructed of weathering steel, are still in good condition. The girders display substantial negative camber due to construction error. The superstructure was intended to be supported by a temporary bent (falsework) while the deck was poured, but the bent was not utilized. No other deficiencies were observed in the girders and/or diaphragms.
Overall	The simple steel plate girder superstructure is still in good overall condition, and the structure has no history of fatigue cracking.	The simple steel plate girder superstructure is still in good overall condition, and the structure has no history of fatigue cracking.

5.2.2.3 _Virginia_

I-95 James River Bridge was built with full span length prefabricated composite units (PCU) up to 114 ft long for 102 superstructure spans in 2002. All these 102 spans are approach spans with 51 spans for each direction. The prefabricated composite units were fabricated at a nearby casting yard using shored construction. As shown in Figure 5-21, Each unit composes of two or three plate steel girders with a W-section diaphragm. The minimum concrete slab is 8.86 in. (225mm). Beams were designed so that all concrete was treated as a superimposed dead load under the Standard Specifications. Figure 5-22 shows the rating history of bridge #21494 in Virginia. As shown in Figure 5-8, after the completion of the bridge in 2002, the rating decreased from 8 to 6 and 7 to 6 for the deck and the superstructure and substructure, respectively. Again the rating of

superstructure and substructure decreased to 5 in 2007 and 2010, respectively. The changes in ratings at an early age (less than 10 years in service) indicate some deteriorations of the structure.

Through the interview with VDOT, the deterioration of the bridge is mainly due to the change during the construction process. The original concept included a bent threaded PT bar for connecting single spans. The contractor proposed a prestressed 7 wire strand solution since they could not thread the PT bars. The strand was very short and there was a relatively large percentage of prestress losses. The jacked length was also very short and the jacked force may not have been correctly applied. Overall, there is no significant problem for the positive moment area. The problem related to the negative moment area was due to insufficient post-tensioning as mentioned above. Please note the continuity detail of this bridge was designed to carry live load only while the beams themselves were designed to function as single-spans.
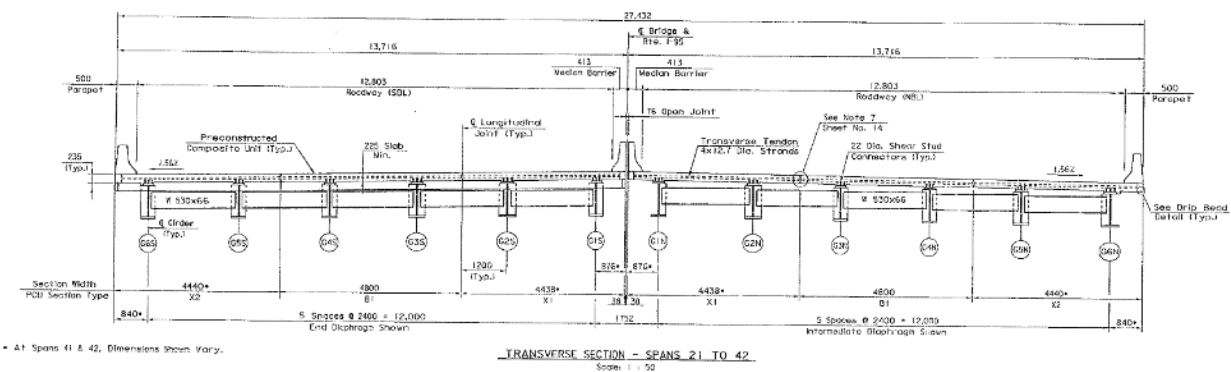
Figure 5-21. Transverse section of bridge # 21494 (spans 21 to 42), Virginia
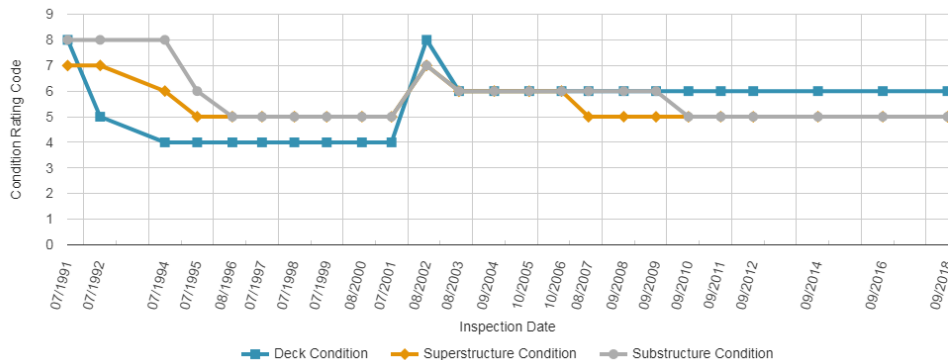
Figure 5-22. Condition rating history for bridge # 21494, Virginia (rebuilt in 2002)

5.2.2.4   *New York*

RT. 9 bridge over New York State Thruway (I-87) was re-built with a prefabricated Inverset stringer system in 2004. The original bridge was built in 1954. This reconstructed bridge (#1004939) has a span of 120.5 feet and a skew of 45 degrees. The units weighing as much as 128 tons were shipped to the site on special hauling rigs. There is still doubt about whether this bridge is shored constructed or not. The Research team learned this bridge was shored constructed from a Fort Miller's brochure but NYSDOT stated they didn't use shoring for their Inverset bridges. Thus, no other details other than the condition rating history of this bridge were obtained. Figure 5-23 shows the rating history of bridge #1004939 in New York. Since the reconstruction in 2004, the bridge maintained good condition. Although the rating decreased from 9 to 8 in 2013 and again decreased from 8 to 7 for superstructure and substructure, the ratings are still above 6 indicating there are only minor problems that cyclic maintenance should be sufficient.
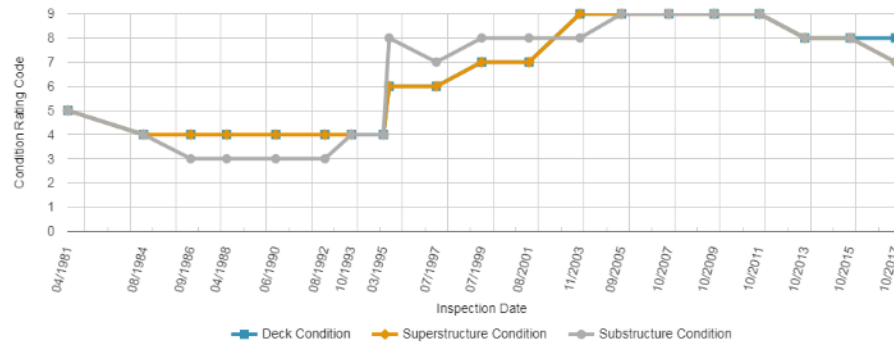
Figure 5-23. Condition rating history for bridge #1004939, NY (rebuilt in 2004)

5.2.2.5   *Summary*

The research team conducted a thorough review of construction details and performance for all shored constructed bridges identified earlier. Construction drawings and inspection reports (if available) have been collected and email/phone interviews have been conducted to collect as much information as possible to evaluate the construction and performance of shored constructed bridges. Overall, there are a total of 44 bridges have been reviewed. Based on the study, the following conclusions are drawn:

1. For shored construction in the field, the shoring methods vary from steel members, timber mat footings, pile foundations, or temporary bent. The shoring method used by

VDOT for the ABC bridge is shoring at quarter points until the concrete has attained a strength of 26.25 MPa (3,800 psi) at a nearby casting yard. There is no major construction issue about shored construction.

2. Shored constructed bridges in Michigan showed comparable performance in comparison with unshored constructed bridges of similar ages.

3. Shored constructed bridge in West Virginia demonstrated promising results as the shored constructed southbound twin showed less dead load deflection. The inspection report also proved that the shored southbound twin performs better compared to the unshored twin bridge.

4. There was no significant problem observed for the positive moment area for James River Bridge in Virginia. Due to insufficient prestressing, there were some issues in the negative moment but were not related to shored construction.

6 ANALYTICAL STUDY

Due to the fact that there are only a limited number of shored constructed steel girder bridges available for investigation, it is difficult to evaluate the effect of shored construction in different design conditions directly with varied parameters. Thus, a comprehensive analytical study is needed to perform a sensitivity study with varied parameters. As shown in Figure 6-1, based on findings from previous chapters, design parameters can be extracted for the design of example bridges that were used for finite element modeling and analysis. Coordinating with the FDOT project manager, example bridges have been designed and study parameters were selected. Then the finite element models were developed and validated. Finally, the sensitivity study was performed and the long-term effects were evaluated. This chapter summarizes the details of the example bridges, the intended design, and analysis methods, and a matrix of the parameters to be varied.
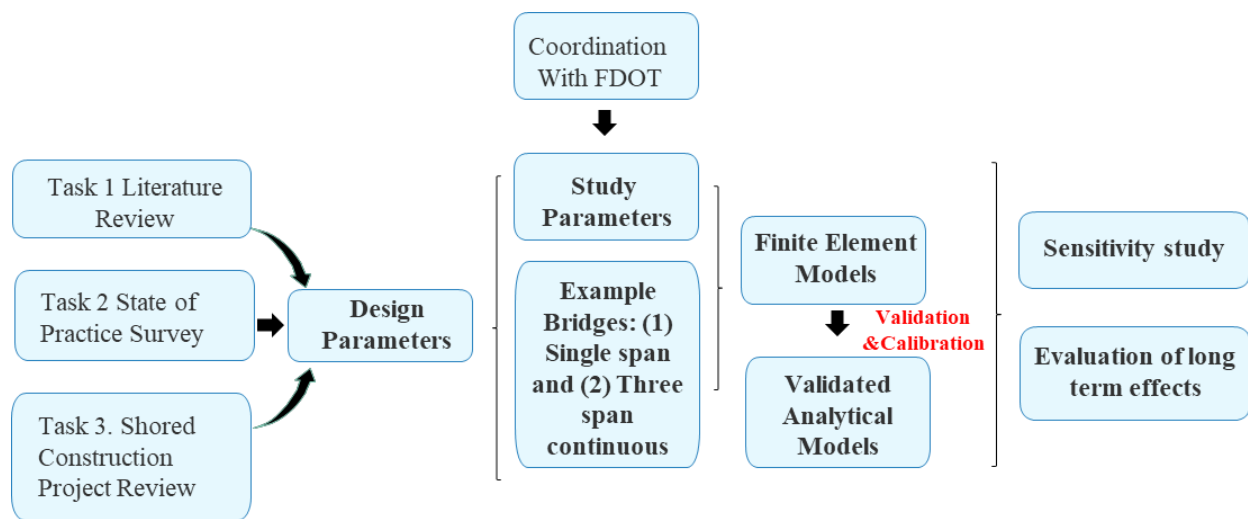
Figure 6-1. Flowchart for the analytical study

6.1 DEVELOP ANALYTICAL STUDY

6.1.1 Selected Example Bridges

Coordinating with the Project Manager, a list of steel composite bridges built since the year 1990 were obtained including 263 single-span bridges and 226 multi-span bridges. As shown in Figure 6-2, the deck width ranges from 10 ft to 204.7 ft while 55% of the bridges have a deck

width of 30 ft to 60 ft. Figure 6-3 shows the histogram of span length for single-span steel composite bridges. 174 out of 263 single-span bridges range from 100 ft to 220 ft. For three-span continuous steel composite girder bridges, as shown in Figure 6-4, excluding ten bridges with a center span length less than 100 ft, there are 18 out of 36 bridges that fall into span length of 124 ft to 204 ft. The L_1/L_2 Ratio for three-span continuous steel composite girder bridges was also investigated (Figure 6-5). It is observed that the L_1/L_2 ratios of 0.66 to 0.79 were most used (17 bridges). These statistics were used as the basis to choose study parameters and example bridges.
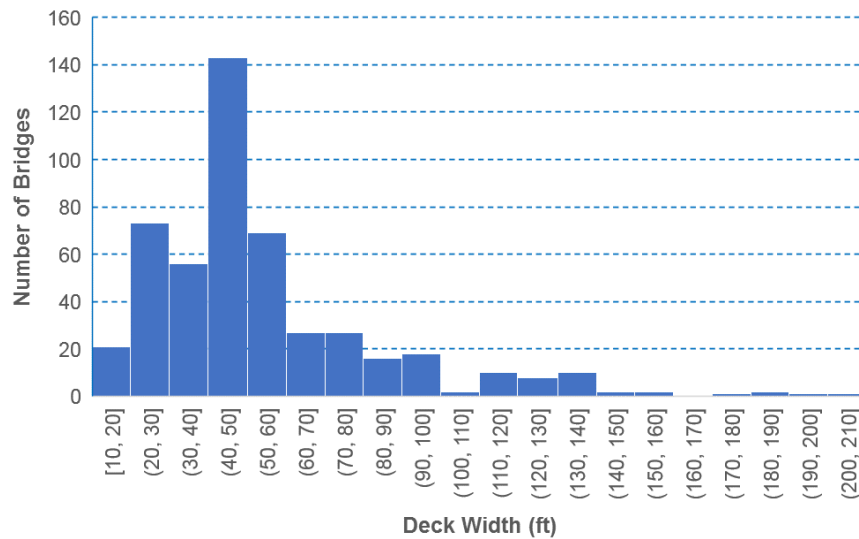
Figure 6-2. Histogram of deck width for steel composite bridges (Since 1990)
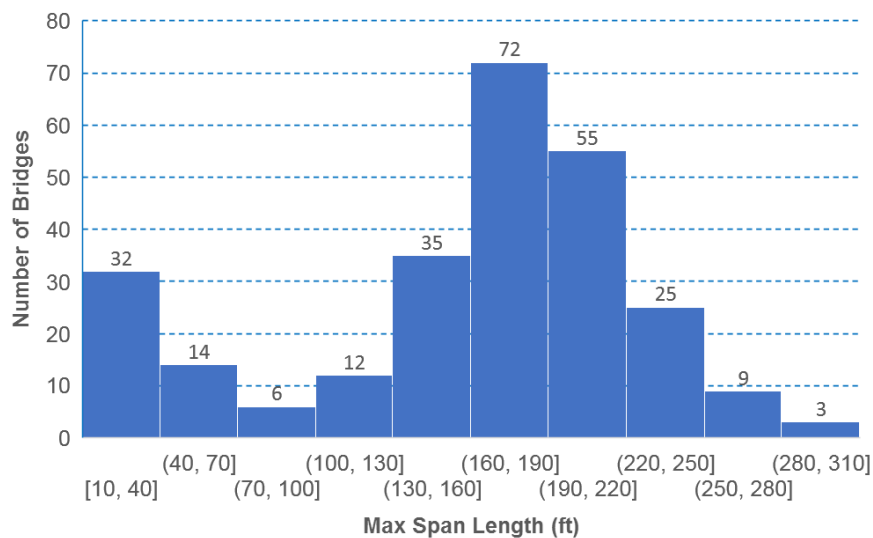
Figure 6-3. Histogram of span length for single-span steel composite bridges (Since 1990)
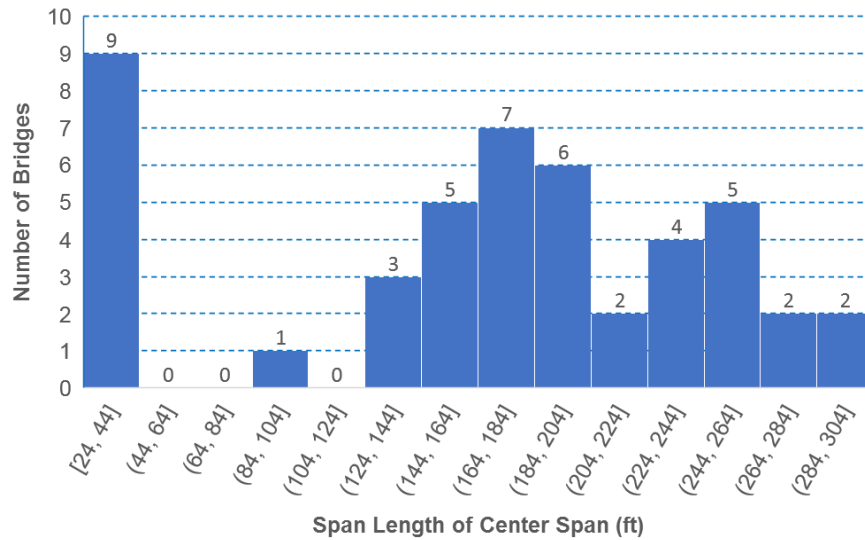
Figure 6-4. Span length histogram of center span for three-span continuous steel composite bridges (Since 1990)
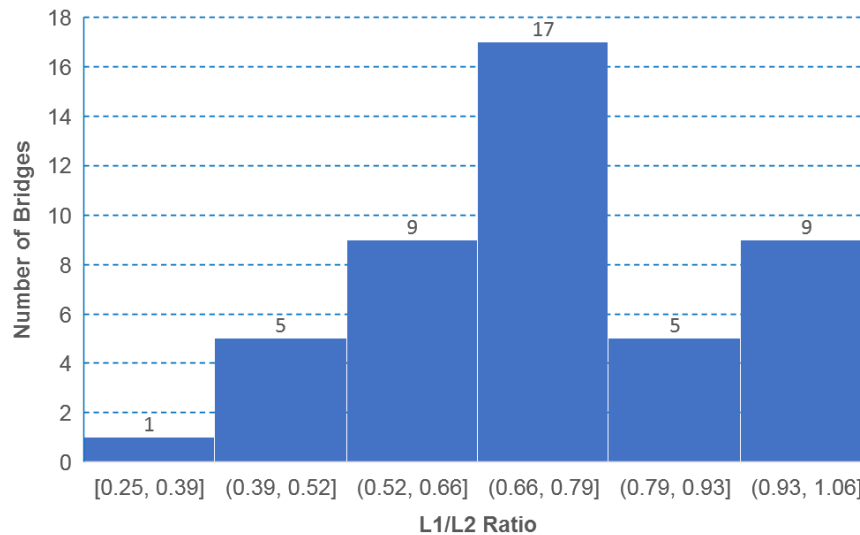
Figure 6-5. Histogram of L_1/L_2 ratio for three-span continuous steel composite bridges (Since 1990)

6.1.1.1   *Example Single-Span Steel Composite BridgeS*

After meeting with the Project Manager and FDOT engineers, a series of example single-span steel composite bridges have been designed and further investigated in finite element modeling and analysis. Figure 6-6 shows the proposed example of single-span steel composite bridges. It is proposed to design and study the following bridges: (1) Rolled beam with a span

length of 80 ft, 100 ft, and 120 ft, and (2) Plate girder with a span length of 80 ft, 100 ft, 120 ft, 140 ft, 160 ft, 180 ft, and 200 ft. The reason that the research team proposes to design the plate girder bridge with a span length longer than 120 ft is 162 out of 263 bridges (62%) of the single-span steel composite bridges built by FDOT since 1990 are within the span range of 130 ft to 220 ft.

Moreover, both girder spacing of 8 ft and 10 ft have been considered. For all these bridges, four shoring schemes were considered including (a) unshored, (b) fully shored, (c) discretely shored at 1/3 points, and (d) discretely shored at quarter points. There are other characteristics considered for example single-span steel composite bridges and three-span continuous steel composite bridges, including (a) 0° skew angle, (b) straight alignment, (c) homogeneous girder with Grade 50W steel, and (d) no lateral bracing. In addition, there are other parameters that were considered including pouring sequences and creep and shrinkage models, which were discussed in detail in Chapter 6.

Rolled Beam: 80 ft, 100 ft, 120 ft
Plate Girder: 80 ft, 100 ft, 120 ft, 140 ft, 160 ft, 180 ft, 200 ft
Girder Spacing: 8 ft and 10 ft

(a) Unshored

(b) Fully Shored

(c) Discretely Shored (1/3 points)

(d) Discretely Shored (quarter points)
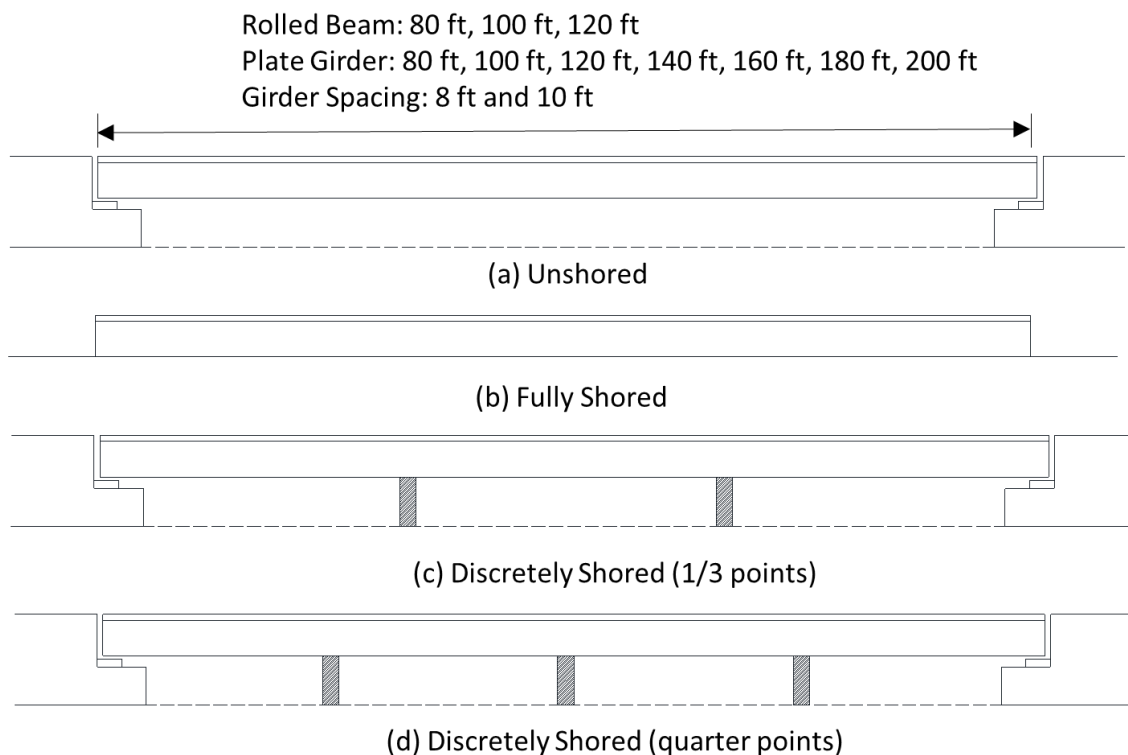
Figure 6-6. Proposed example of single-span steel composite bridges

6.1.1.2 *Example Three-Span Continuous Steel Composite Bridges*

Similar to single-span steel composite bridges, the criteria for example three-span continuous steel composite bridges were discussed with the Project Manager and FDOT engineers. Based on these discussions, statistics of the existing bridge database (section 2-A), and guidelines from National Steel Bridge Alliance (NSBA) Continuous Span Standards (NSBA, 2015), a set of three-span continuous steel composite girder example bridges are proposed to be used as example bridges for design and further finite element modeling and analysis. As shown in Figure 6-7, the span length combinations of 140'-180'-140', 176'-225'-176', and 199'-255'-199' are proposed to be used. It is also proposed to apply girder spacing of 10'-6" and 12' for these three-span continuous steel composite bridges. All of these bridges have an end span to center span ratio (L_1/L_2) of 0.78, as presented in the NSBA Continuous Span Standards. There are four shoring schemes proposed as shown in Figure 6-7, (a) unshored, (b) discretely shored at 1/3 points for center span only, (c) discretely shored at 1/3 points for center span and midspan point for end spans, and (d) discretely shored at 1/4 points for center span and 1/3 points for end spans. The intention of testing case (b) is to compare with case (a) for traditional pouring sequences (positive moment regions first, then negative moment regions).

It is worth noting that no hybrid girders were considered in this study. Only homogeneous A709-50W steel was used in developing these example bridges. Other parameters including pouring sequences and creep and shrinkage models were also considered.

Span Length: 140'-180'-140', 176'-225'-176', 199'-255'-199' (L_1/L_2=0.78)
Girder Spacing: 10'-6" and 12'

(a) Unshored

(b) Discretely Shored (1/3 points, Center Span only)

(c) Discretely Shored (1/3 points, Center Span, and Midspan point, End Spans)

(d) Discretely Shored (1/4 points, Center Span, and 1/3 points, End Spans)
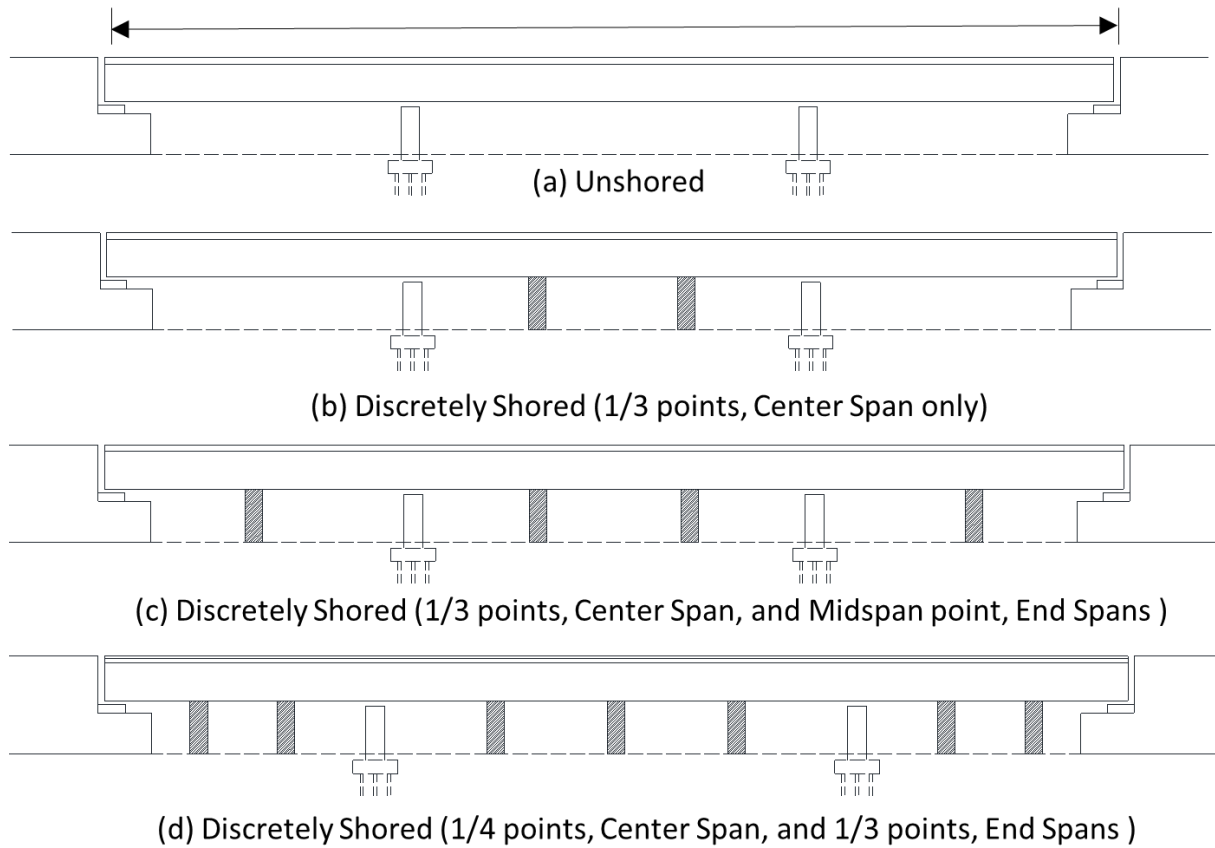
Figure 6-7. Proposed example of three-span continuous steel composite bridges

6.1.2   Design of Steel Composite Girder Bridges

All bridge configurations proposed earlier have been designed using an in-house developed excel spreadsheet. In addition, for bridges designed with the unshored condition, the eSPAN140 program developed by short span steel bridge alliance (SSSBA) was used to check and validate the design for single-span bridges and three-span continuous bridges, respectively. The design procedure laid out in the Steel Bridge Design Handbook (USDOT and FHWA, 2015) was used to develop the design spreadsheet. The most recent AASHTO LRFD Bridge Design Specification (AASHTO, 2019) and FDOT Structures Manual (FDOT, 2021) were adopted in the designs as well.

Assuming a moderately aggressive or extremely aggressive environmental condition, Class IV concrete as specified in the FDOT Structures Manual was used in the designs. A minimum 28-day compression strength of 5.5 ksi is assumed for Class IV concrete. As stated earlier, homogeneous A709-50W steel was used for the design of all bridges. The designs of shored constructed bridges were compared with the ones designed with the unshored condition, and potential cost savings by using shored construction were estimated. This potential cost saving was combined with the cost of temporary shoring, including labor and material costs to evaluate the feasibility of shored construction for steel composite girder bridges, presuming comparable performance was observed for shored and unshored constructed bridges.

The specific design considerations are summarized in the sections below.

6.1.2.1 *Design Considerations for Shored Construction*

Through the survey and follow-up interview, only one bridge was intentionally designed as a shored constructed steel composite girder bridge, I-95 James River Bridge in Virginia. The only modification that VDOT implemented was using a modular ratio of *3n* when computing stresses due to the self-weight of the concrete slab. Thus, the research team implemented this modification and all other related design provisions provided in the current AASHTO LRFD Design Specification (AASHTO, 2020) for the design of shored constructed steel composite girder bridges.

The design considerations that have been taken into account for shored steel composite bridge design are listed below.

1. AASHTO article 6.10.1.1.1a

For unshored construction, the permanent load applied before the concrete deck has hardened or is made composite shall be assumed carried by the steel section alone; permanent load and live load applied after this stage shall be assumed carried by the composite section. For shored construction, all permanent load shall be assumed to be applied after the concrete deck has hardened or has been made composite and the contract documents shall so indicate.

Thus, the long-term modular ratio, *3n*, will be used when calculating the stresses for sections in positive flexure due to DC_1 load (steel girder, haunch, overhang taper, cross frames, and stay-in-place forms).

2. AASHTO article C6.10.1.7

To prevent nominal yielding of longitudinal deck reinforcement and control concrete deck cracking, the use of longitudinal deck reinforcement with a specified minimum yield strength not less than 60 ksi may be taken for shored construction where the steel section utilizes steel with a specified minimum yield strength less than or equal to 50 ksi in either flange.

3. AASHTO Article 6.10.4.2.2 and C6.10.4.2.2

For compact composite sections in positive flexure utilized in shored construction, the longitudinal compressive stress in the concrete deck due to the Service II loads, determined as specified in Article 6.10.1.1.1d, shall not exceed 0.6f'c. This is to ensure the linear behavior of the concrete.

4. AASHTO Article C6.10.6.2.2 and C6.11.6.2.2

Compact composite sections in positive flexure must also satisfy the provisions of Article 6.10.7.3 to ensure a ductile mode of failure. Noncompact sections must also satisfy the ductility requirement specified in Article 6.10.7.3 to ensure a ductile failure. The satisfaction of this requirement ensures an adequate margin of safety against the premature crushing of the concrete deck for sections utilizing up to 100-ksi steels and/or for sections utilized in shored construction.

5. AASHTO Article C6.10.7.2.1

The longitudinal stress in the concrete deck is limited to $0.6\,f_c'$ to ensure the linear behavior of the concrete which is assumed in the calculation of the steel flange stresses for noncompact sections. This condition may govern for shored construction with geometries causing the neutral axis of the short-term and long-term composite section to be significantly below the bottom of the concrete deck.

6. Dimensional Limits

All bridge girders have been designed in accordance with the dimensional limits specified in FDOT Structures Design Guidelines (SDG) (FDOT, 2021), AASHTO LRFD9 (AASHTO, 2020) and G12.1 Guidelines to Design for Constructability (AASHTO, 2016). Table 6-1 shows a summary of dimensional limits that were considered in this study.

Table 6-1. Dimensional limit for steel girder design

Dimension	Dimensional Limit	Reference
Minimum flange thickness	0.3125"	LRFD 6.7.3
Minimum flange thickness	¾"	G12.1 Section 1.3
Minimum web thickness	½"	G12.1 Section 1.3
Minimum web thickness	7/16"	SDG 5.5
Minimum flange size	¾"×12"	SDG 5.5
Minimum stiffener thickness	½"	SDG 5.5
Minimum Overall Depth (Including Deck)	0.04L for simple spans 0.032L for continuous spans	LRFD 2.5.2.6.3-1
Minimum I-beam Portion of Composite I-beam	0.033L for simple spans 0.027L for continuous spans	LRFD 2.5.2.6.3-1
Thickness increment for steel plates	1/8" (1/16" for web plate) for plate thickness up to 2-1/2" ¼" for plate thickness more than 2-1/2"	SDG 5.5

6.1.2.2 *Validation of Design Procedure*

The design procedure developed has gone through a two-step validation process. First, the design procedure was validated with the design example from FHWA Steel Bridge Design Handbook (USDOT and FHWA, 2015). Then the design results of unshored single-span bridges were validated with design outcomes from the eSPAN140 program and the design results of unshored three-span continuous bridges were validated with results from NSBA Continuous Span Standards. This two-step validation process ensures the accuracy of the design procedure and design results.

6.2   DESIGN AND ANALYSIS RESULTS

6.2.1   Design of Steel Composite Bridges

Coordinating with the Project Manager, the general design considerations were developed for the design of steel composite bridges including (a) 0° skew angle, (b) straight alignment, (c) homogeneous girder with Grade 50W steel, and (d) no lateral bracing. Figure 6-8 shows a typical section of a 5-girder steel composite girder bridge.
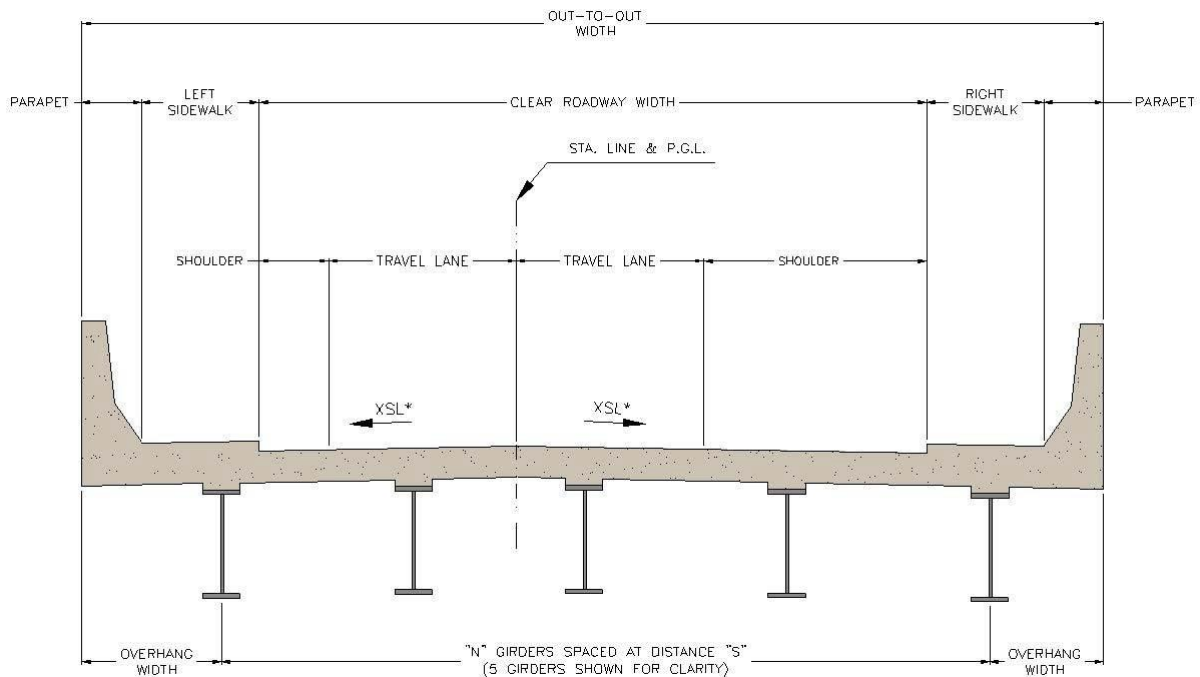
Figure 6-8. Typical section for a 5-girder steel composite girder bridge

Through the survey and follow-up interview, only one bridge was intentionally designed as a shored constructed steel composite girder bridge, I-95 James River Bridge in Virginia. The only modification that VDOT implemented was using a modular ratio of 3n when computing stresses due to the self-weight of the concrete slab based on AASHTO LRFD 6.10.1.1.1b. Besides this modification, the following modifications were made to incorporate the shored construction condition into the design.

(1) Fully Shored Condition

a. Equation D6.2.2-1, use long-term section modulus for moment due to D1 load. The revised equation is shown below:

$$F_{yt} = (M_{D1}/S_{LT}) + (M_{D2}/S_{LT}) + (M_{AD}/S_{ST})$$

b. Article 6.10.4.2, use long-term section modulus for calculating flange stress:

$$f_f = (M_{DC1} / S_{lt}) + ((M_{DC2} + M_{DW}) / S_{lt}) + (1.3\, M_{LL+IM} / S_{st})$$

(2) Discretely Shored Condition

As shown inFigure 6-9, since shoring towers provide additional intermediate supports for discretely shored condition, the equations for the following sections from AASHTO LRFD BDS were updated:

a. Equation D6.2.2-1, uses long-term section modulus for the moment due to D1 load. The revised equation is shown below:

$$F_{yt} = (M_{D1}'/S_{NC}) + (M_{D1}/S_{LT}) + (M_{D2}/S_{LT}) + (M_{AD}/S_{ST})$$

b. Article 6.10.4.2, the stress equations were updated. For positive moment region,

$$f_f = (M_{DC1}' / S_{nc}) + (M_{DC1} / S_{lt}) + ((M_{DC2} + M_{DW}) / S_{lt}) + (1.3\, M_{LL+IM} / S_{st})$$

For negative moment region,

$$f_f = (M_{DC1} / S_{lt}) + ((M_{DC2} + M_{DW}) / S_{lt}) + (1.3\, M_{LL+IM} / S_{st})$$
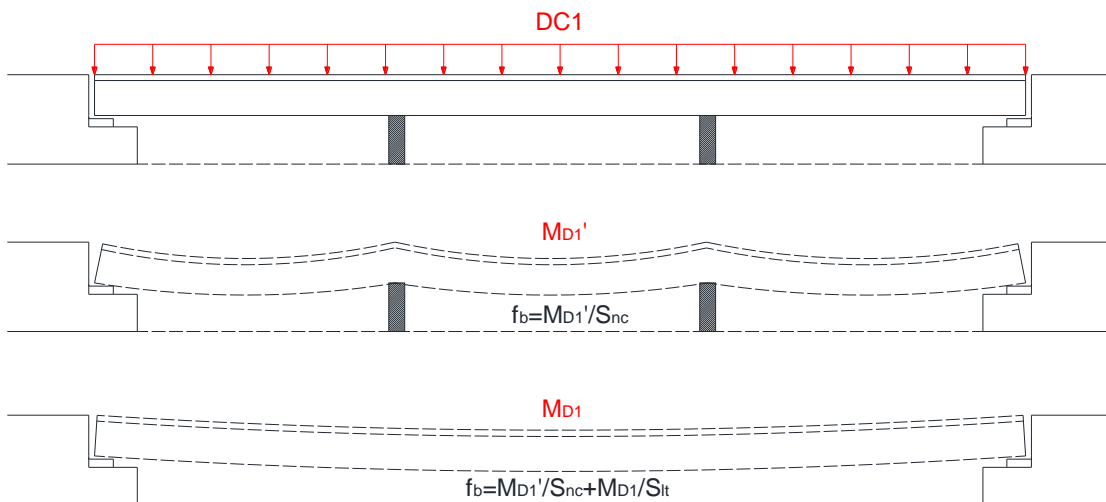
Figure 6-9. Scheme for discretely shored construction condition

The designs of composite steel girder bridges were carried out for both shored and unshored construction conditions. Figure 6-10 shows a summary of bridges that were considered in this study. It included eighty single-span bridges and twenty-four (24) three-span continuous span bridges. All bridges were designed using an in-house developed excel spreadsheet. In addition, for bridges designed with the unshored condition, the eSPAN140 program developed by the short span steel bridge alliance (SSSBA) was used to check and validate the design for single-span bridges. The continuous span standards developed by National Steel Bridge Alliance (NSBA) were used to compare with our designs. The design procedure laid out in the Steel Bridge Design Handbook (USDOT and FHWA, 2015) was used to develop the design spreadsheet. The most recent AASHTO LRFD Bridge Design Specification (AASHTO, 2019) and FDOT Structures Manual (FDOT, 2021) were adopted in the designs as well.

Assuming a slightly aggressive environmental condition, Class II concrete as specified in FDOT Structures Manual was used in the designs. A minimum 28-Day Compression Strength of 4.5 ksi is assumed for Class II concrete. As stated earlier, homogeneous A709-50W steel was used for the design of all bridges.
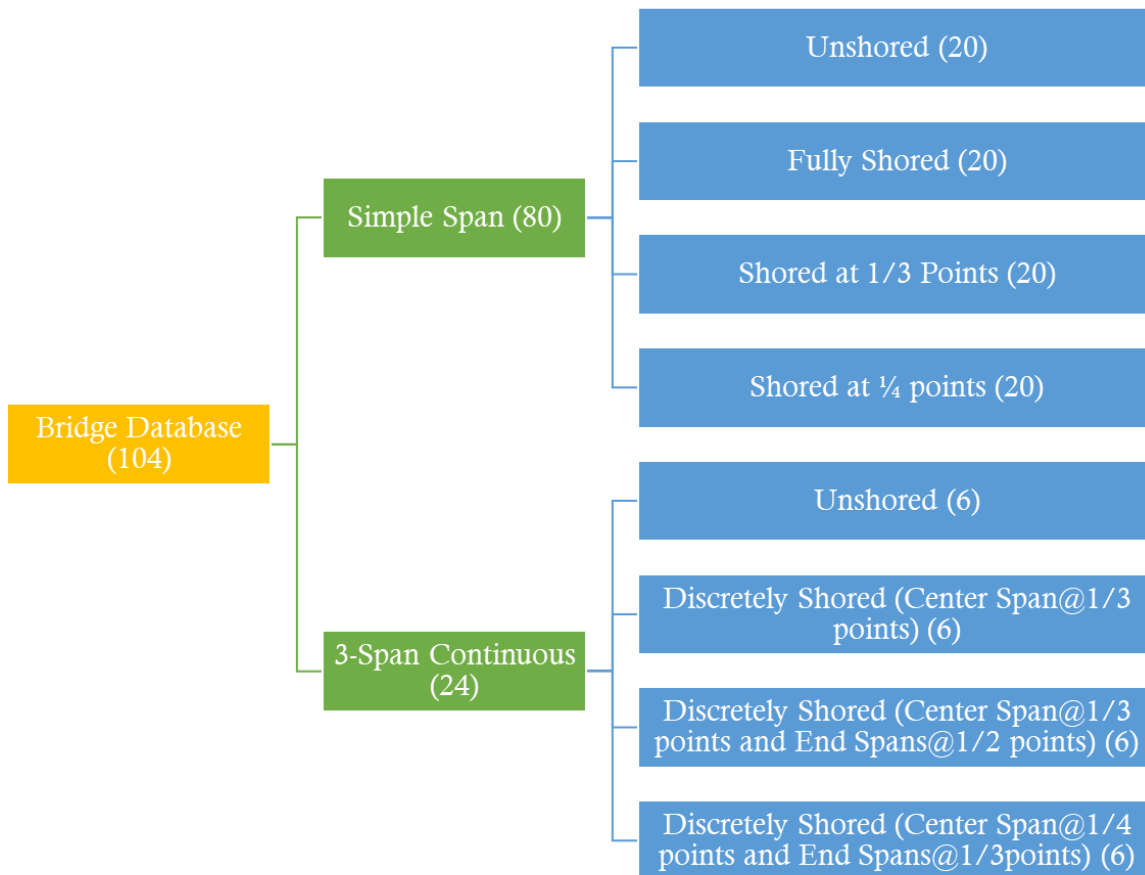
Figure 6-10. Composition of bridge database considered in this study

6.2.1.1 *Single-Span Steel Composite Bridges*

Figure 6-11 shows the example of single-span steel composite bridges. The following bridges have been considered: (1) Rolled beam with a span length of 80 ft, 100 ft, and 120 ft, and (2) Plate girder with a span length of 80 ft, 100 ft, 120 ft, 140 ft, 160 ft, 180 ft, and 200 ft. The reason that the research team decided to design the plate girder bridge with a span length longer than 120 ft is 162 out of 263 bridges (62%) of the single-span steel composite bridges built by FDOT since 1990 are within the span range of 130 ft to 220 ft.

Moreover, both girder spacing of 8 ft and 10 ft were considered. For all these bridges, four shoring schemes were considered including: (a) unshored, (b) fully shored, (c) discretely shored at 1/3 points, and (d) discretely shored at quarter points.

Rolled Beam: 80 ft, 100 ft, 120 ft
Plate Girder: 80 ft, 100 ft, 120 ft, 140 ft, 160 ft, 180 ft, 200 ft
Girder Spacing: 8 ft and 10 ft

(a) Unshored

(b) Fully Shored

(c) Discretely Shored (1/3 points)

(d) Discretely Shored (quarter points)
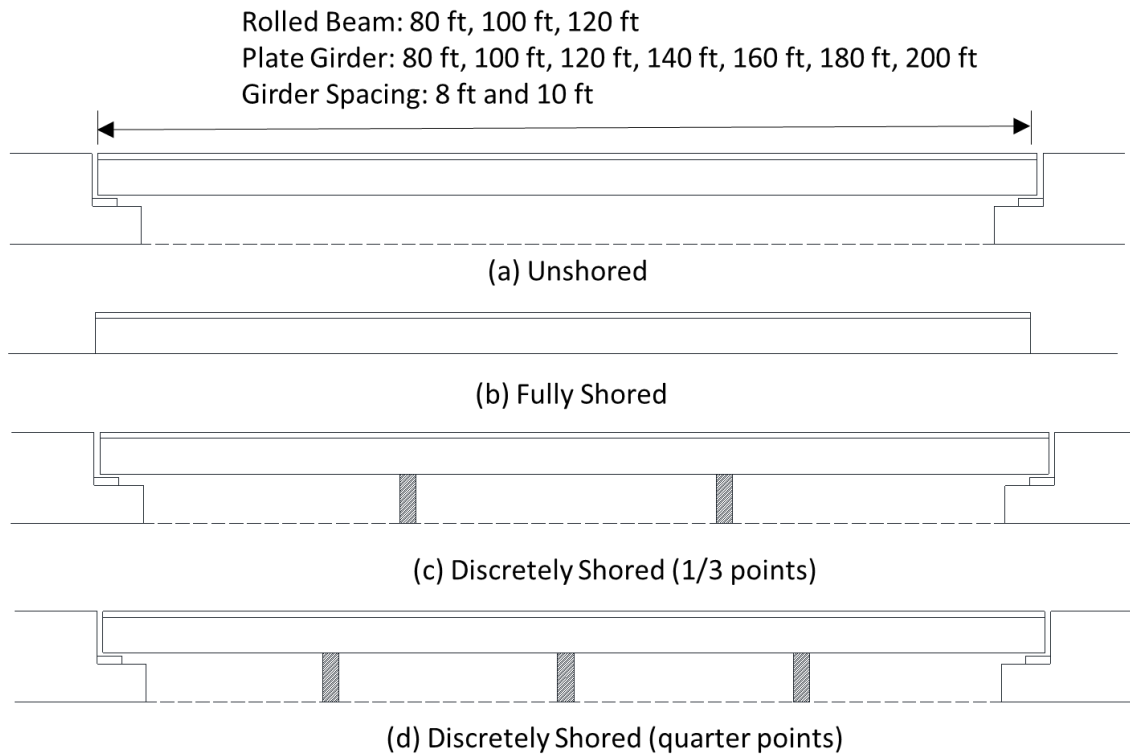
Figure 6-11. Example of single-span steel composite bridges

Table 6-2 shows the design results for single-span bridges with rolled beam sections and 8 ft spacing. During the design process, different limit states were checked and satisfied including Service II limit state (AASHTO 6.10.4), Fatigue and Fracture limit state (AASHTO 6.6.1.2 and 6.10.5.3), and Strength I limit state (AASHTO 6.10.6). It was observed that the main limiting factor in reducing the steel section by using shored construction is AASHTO 6.10.4.2.2. The limitations on flange stresses were usually the governing factor in the design of shored constructed steel composite girders. As shown in Table 6-2, there is a substantial reduction in the size of steel sections if the bridge girder is fully shored. It is indicated if the steel girder is prefabricated with the fully shored condition, a much lighter section can be used, resulting in possible savings in materials and transportation. Similar results were also observed for the rolled beam with 10 ft spacing.

Table 6-2. Single-span bridges with rolled beam (8 ft spacing with 6 girders)

Span Length (ft)	Girder Spacing (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	8	W30x191	W30x173	W30x173	W30x148	23%
100	8	W36x231	W36x231	W36x231	W36x194	17%
120	8	W40x324	W40x297	W40x297	W40x249	23%

Table 6-3. Single-span bridges with rolled Beam (10 ft spacing with 5 girders)

Span Length (ft)	Girder Spacing (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	10	W30x235	W30x211	W30x211	W30x191	19%
100	10	W36x302	W36x282	W36x262	W36x247	19%
120	10	W40x372	W40x362	W40x362	W40x297	20%

Table 6-4 and Table 6-5 show the design results for single-span plate girder bridges with 8 ft and 10 ft spacing, respectively. Although significant reductions were also observed for plate girder sections with an average of 11% and 16% for 8 ft and 10 ft spacing, respectively, the reduction is a little less compared to rolled section. The main reason is for a longer span designed with plate girder section, the reduction is smaller compared to short spans.

Table 6-4. Single-span bridges with plate girder (8 ft spacing with 6 girders)

Span Length (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	Top : 16x3/4" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x7/8"	14%
100	Top : 16x3/4" Web: 40x1/2" Bottom: 16x1.25"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1.125"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1.125"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1"	15.4%
120	Top : 18x3/4" Web: 48x1/2" Bottom: 18x1.25"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1"	10.1%
140	Top : 18x3/4" Web: 54x1/2" Bottom: 18x1.5"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.25"	8.3%
160	Top : 18x3/4" Web: 60x1/2" Bottom: 18x1.75"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.625"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.625"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.5"	8%
180	Top : 18x3/4" Web: 68x1/2" Bottom: 18x2"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.875"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.875"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.75"	10.7%
200	Top : 20x3/4" Web: 74x1/2" Bottom: 20x2"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.875"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.875"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.75"	11%

Table 6-5. Single-span bridges with plate girder (10 ft spacing with 5 girders)

Span Length (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	Top : 16x1″ Web: 32x1/2″ Bottom: 16x1.5″	Top: 16x3/4″ Web: 32x1/2″ Bottom: 16x1.25″	Top: 16x3/4″ Web: 32x1/2″ Bottom: 16x1.125″	Top: 16x3/4″ Web: 32x1/2″ Bottom: 16x1.125″	18%
100	Top : 18x1″ Web: 40x1/2″ Bottom: 18x2″	Top : 18x3/4″ Web: 40x1/2″ Bottom: 18x1.25″	Top : 18x3/4″ Web: 40x1/2″ Bottom: 18x1.25″	Top : 18x3/4″ Web: 40x1/2″ Bottom: 18x1.25″	19%
120	Top : 18x1″ Web: 48x1/2″ Bottom: 20x2″	Top: 18x3/4″ Web: 48x1/2″ Bottom: 20x1.375″	Top : 18x3/4″ Web: 48x1/2″ Bottom: 20x1.375″	Top : 18x3/4″ Web: 48x1/2″ Bottom: 20x1.25″	23%
140	Top : 20x1″ Web: 54x1/2″ Bottom: 20x2″	Top : 18x3/4″ Web: 54x1/2″ Bottom: 20x1.625″	Top : 18x3/4″ Web: 54x1/2″ Bottom: 20x1.625″	Top : 18x3/4″ Web: 54x1/2″ Bottom: 20x1.5″	19%
160	Top : 20x1″ Web: 60x1/2″ Bottom: 20x2″	Top : 18x3/4″ Web: 60x1/2″ Bottom: 20x2″	Top : 18x3/4″ Web: 60x1/2″ Bottom: 20x1.875″	Top : 18x3/4″ Web: 60x1/2″ Bottom: 20x1.75″	13%
180	Top : 20x1″ Web: 68x1/2″ Bottom: 20x2.25″	Top : 18x3/4″ Web: 68x1/2″ Bottom: 20x2.25″	Top : 18x3/4″ Web: 68x1/2″ Bottom: 20x2.125″	Top : 18x3/4″ Web: 68x1/2″ Bottom: 20x2″	11%
200	Top : 20x1.25″ Web: 74x1/2″ Bottom: 20x2.5″	Top : 18x3/4″ Web: 74x1/2″ Bottom: 20x2.625″	Top : 18x3/4″ Web: 74x1/2″ Bottom: 20x2.375″	Top : 18x3/4″ Web: 74x1/2″ Bottom: 20x2.25″	15%

6.2.1.2   *Three-Span Continuous Steel Composite Bridges*

Similar to single-span steel composite bridges, the criteria for example three-span continuous steel composite bridges were discussed with the Project Manager and FDOT engineers. Based on these discussions, statistics of the existing bridge database, and guidelines from National Steel Bridge Alliance (NSBA) Continuous Span Standards (NSBA, 2015), a set of three-span

continuous steel composite girder example bridges were designed. As shown in Figure 6-12, the span length combinations of 140'-180'-140', 176'-225'-176', and 199'-255'-199' were used. It includes girder spacing of 10'-6" and 12' for these three-span continuous steel composite bridges. All of these bridges have an end span to center span ratio (L_1/L_2) of 0.78, as presented in the NSBA Continuous Span Standards. There are four shoring schemes that have been used as shown in Figure 6-7, (a) unshored, (b) discretely shored at 1/3 points for center span only, (c) discretely shored at 1/3 points for center span and midspan point for end spans, and (d) discretely shored at 1/4 points for center span and 1/3 points for end spans. It is worth noting that no hybrid girders were considered in this study. Only homogeneous A709-50W steel was used in developing these example bridges.

Span Length: 140'-180'-140', 176'-225-176', 199'-255'-199' (L_1/L_2=0.78)
Girder Spacing: 10'-6" and 12'
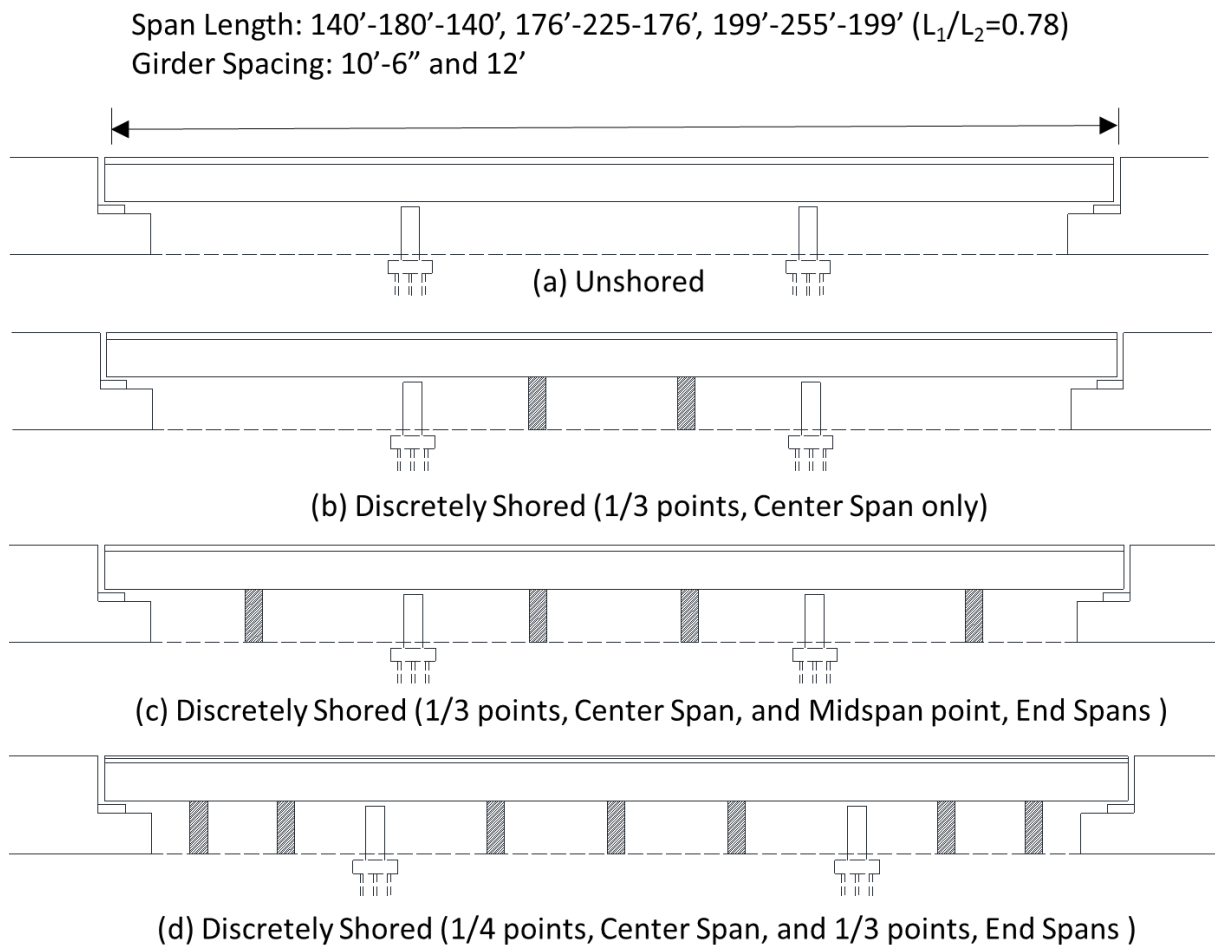
Figure 6-12. Example of three-span continuous steel composite bridges

Table 6-6 and Table 6-7 summarized the design results for the three-span continuous bridges with 10.5 ft and 12 ft spacing, respectively. Similar to Single-span bridges, for the design of positive moment region, different limit states were checked and satisfied including Service II limit state (AASHTO 6.10.4), Fatigue and Fracture limit state (AASHTO 6.6.1.2 and 6.10.5.3), and Strength I limit state (AASHTO 6.10.6). On the other hand, limiting criteria listed in AASHTO Appendix A6 were checked for the negative moment region. The aforementioned shoring conditions shown in Figure 6-7 were considered for the designs. For bridges with 10.5 ft spacing, it is interesting to see that the reduction in steel section area is averaging 9% for the positive moment region while is averaging over 17% for the negative moment region. It indicates shoring has a more significant effect on the negative moment region in comparison with the positive moment region.

Table 6-6. Three-span continuous bridges with plate girder (10'6" spacing with 5 girders)

Main Span Length (ft)	Region	Unshored	Shored at 1/3 points	Shored at 1/4 points	% Diff. if fully shored
180	M+	Top : 16x1" Web: 66x1/2" Bottom: 18x1.5"	Top : 16x3/4" Web: 66x1/2" Bottom: 18x1.25"	Top : 16x3/4" Web: 66x1/2" Bottom: 18x1.125"	12%
180	M-	Top : 18x1.5" Web: 66x1/2" Bottom: 20x2.5"	Top : 16x1.5" Web: 66x1/2" Bottom: 20x1.75"	Top : 16x1.375" Web: 66x1/2" Bottom: 20x1.75"	18%
225'	M+	Top : 16x1" Web: 76x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 74x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 74x0.5625" Bottom: 16x1.5"	7%
225'	M-	Top : 18x1.5" Web: 76x0.5625" Bottom: 20x2.5"	Top : 16x1.375" Web: 74x0.5625" Bottom: 20x1.75"	Top : 16x1.375" Web: 74x0.5625" Bottom: 20x1.75"	18%

Table 6-6. Three-span continuous bridges with plate girder (10'6" spacing with 5 girders), cont'd

Main Span Length (ft)	Region	Unshored	Shored at 1/3 points	Shored at 1/4 points	% Diff. if fully shored
255'	M+	Top : 16x1" Web: 83x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 81x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 81x0.5625" Bottom: 16x1.5"	7%
255'	M-	Top : 18x1.5" Web: 83x0.5625" Bottom: 20x2.5"	Top : 16x1.375" Web: 81x0.5625" Bottom: 20x1.75"	Top : 16x1.375" Web: 81x0.5625" Bottom: 20x1.75"	16%

Table 6-7. Three-span continuous bridges with plate girder (12 ft spacing with 5 girders)

Main Span Length (ft)	Region	Unshored	Shored at 1/3 points	Shored at 1/4 points	% Diff. if shored at ¼ points
180	M+	Top : 16x3/4" Web: 72x0.5625" Bottom: 20x1.125"	Top : 16x3/4" Web: 68x1/2" Bottom: 20x1.125"	Top : 16x3/4" Web: 68x1/2" Bottom: 20x1.125"	9%
180	M-	Top : 18x1.375" Web: 72x0.5626" Bottom: 20x1.75"	Top : 18x1.25" Web: 68x1/2" Bottom: 20x1.75"	Top : 18x1.25" Web: 68x1/2" Bottom: 20x1.75"	10%
225'	M+	Top : 16x3/4" Web: 74x0.5625" Bottom: 20x1.125"	Top : 16x3/4" Web: 74x1/2" Bottom: 20x1.125"	Top : 16x3/4" Web: 74x1/2" Bottom: 20x1.125"	6%
225'	M-	Top : 18x1.5" Web: 74x0.625" Bottom: 20x1.75"	Top : 18x1.25" Web: 74x0.5625" Bottom: 20x1.75"	Top : 18x1.25" Web: 74x0.5625" Bottom: 20x1.75"	9%

Table 6-7. Three-span continuous bridges with plate girder (12 ft spacing with 5 girders), cont'd

Main Span Length (ft)	Region	Unshored	Shored at 1/3 points	Shored at 1/4 points	% Diff. if shored at ¼ points
255′	M+	Top : 16x3/4″ Web: 87x0.75″ Bottom: 20x1.125″	Top : 16x3/4″ Web: 87x0.625″ Bottom: 20x1.125″	Top : 16x3/4″ Web: 87x0.625″ Bottom: 20x1.125″	11%
255′	M-	Top : 18x1.5″ Web: 87x0.75″ Bottom: 22x1.75″	Top : 18x1.25″ Web: 87x0.625″ Bottom: 22x1.75″	Top : 18x1.25″ Web: 87x0.625″ Bottom: 22x1.75″	12%

6.2.2 Cost-Benefit Analysis

The designs of shored constructed bridges were compared with the ones designed with the unshored condition and potential cost savings by using shored construction were estimated. This potential cost saving was combined with the cost of temporary shoring including labor and material costs to evaluate the feasibility of shored construction for steel composite girder bridges, presuming comparable performance was observed for shored and unshored constructed bridges. Consulting with the project manager and practitioners, the shoring cost was estimated at $50,050~$59,045 per location if the shoring towers are purchased. However, this cost can be significantly lower if the contractor uses the tower in multiple projects. On the other hand, the cost of renting towers is considerably cheaper. It is estimated about $24,000 ($3,200 rental for each tower plus labor cost).

As shown in Figure 6-13, according to AISC data, the average mill price of A709-50W is about $0.61/lb including $0.78/lb RMS Surcharge and 32% transportation surcharge, the total price becomes $1.8/lb According to FDOT SDG 9.2B, the cost of a straight plate girder is $1.65/lb, which is very similar to the estimation based on AISC data. Thus, $1.65/lb was used in the following studies.
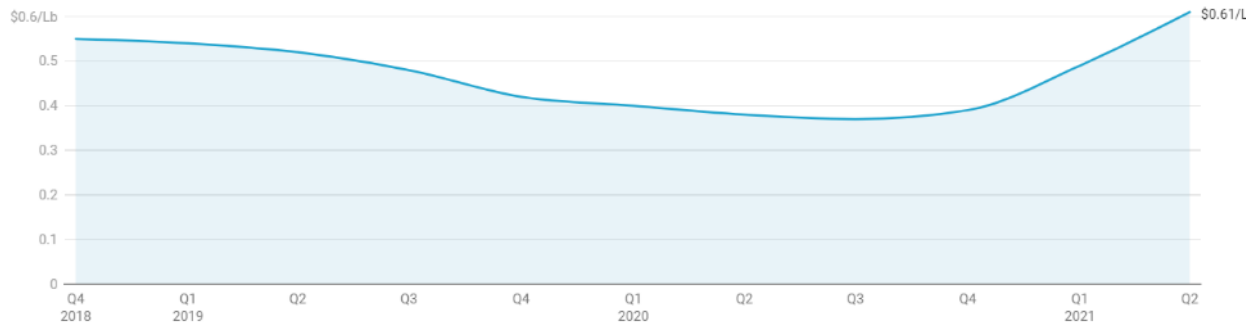
Figure 6-13. Average mill price of A709-50W plate (https://www.aisc.org/economics/)

6.2.2.1 _Single-Span Steel Composite Bridges_

Table 6-8 and Table 6-9 show the cost-benefit analysis for single-span bridges designed with rolled beam and plate girder, respectively. It is shown that only when the bridge is constructed with a fully shored condition, there will be significant savings. If the bridge only shored at 1/3 points, the cost of purchasing shoring towers will offset the savings, which makes shoring not feasible from the cost point of view. Even for the rental option, the saving would not be significant. In general, for fully shored construction which can be achieved for Prefabricated Bridge Elements and Systems (PBES), the saving increases as the span length increases.

Table 6-8. Cost-benefit analysis: Single-span with rolled section (8 ft spacing with 6 girders)

Span Length (ft)	Unshored	Shored at 1/3 points	Fully Shored	Saving if fully shored	Savings if Shored at 1/3 points
80	W30x191	W30x173	W30x148	$34,056	No significant savings
100	W36x231	W36x231	W36x194	$29,304	No significant savings
120	W40x324	W40x297	W40x249	$59,400	No significant savings

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Table 6-9. Cost-benefit analysis: Single-span with plate girder (8 ft spacing with 6 girders)

Span Length (ft)	Unshored	Shored at 1/3 points	Fully Shored	Saving if fully shored	Savings if Shored at 1/3 points
80	Top : 16x3/4" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x7/8"	$16,601	No significant savings
100	Top : 16x3/4" Web: 40x1/2" Bottom: 16x1.25"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1.125"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1"	$26,977	No significant savings
120	Top : 18x3/4" Web: 48x1/2" Bottom: 18x1.25"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1"	$24,498	No significant savings
140	Top : 18x3/4" Web: 54x1/2" Bottom: 18x1.5"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.25"	$26,423	No significant savings
160	Top : 18x3/4" Web: 60x1/2" Bottom: 18x1.75"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.625"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.5"	$32,340	No significant savings
180	Top : 18x3/4" Web: 68x1/2" Bottom: 18x2"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.875"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.75"	$54,177	No significant savings
200	Top : 20x3/4" Web: 74x1/2" Bottom: 20x2"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.875"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.75"	$68,184	No significant savings

6.2.2.2 *Three-Span Continuous Steel Composite Bridges*

Similar to the single-span bridges, the cost-benefit analysis was also performed for all designed three-span continuous steel composite bridges. The saving from the reduction in steel section and associated cost averages $68,643 combines positive and negative moment regions. However, the two shoring towers would cost about $100,100, which offsets the possible savings. If the rental option was used for shoring towers, two shoring towers would cost about $48,000, which means a saving of $20,643 from shored construction.

Table 6-10. Cost-benefit analysis: Three-span continuous (10.5 ft spacing with 5 girders)

Span Length (ft)	Region	Unshored	Shored at 1/3 points	Savings if Shored at 1/3 points	Cost of Shoring
180	M+	Top : 16x1" Web: 66x1/2" Bottom: 18x1.5"	Top : 16x3/4" Web: 66x1/2" Bottom: 18x1.25"	$21,476	$100,100
180	M-	Top : 18x1.5" Web: 66x1/2" Bottom: 20x2.5"	Top : 16x1.5" Web: 66x1/2" Bottom: 20x1.75"	$50,026	
225'	M+	Top : 16x1" Web: 76x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 74x0.5625" Bottom: 16x1.5"	$14,635	$100,100
225'	M-	Top : 18x1.5" Web: 76x0.5625" Bottom: 20x2.5"	Top : 16x1.375" Web: 74x0.5625" Bottom: 20x1.75"	$54,460	
255'	M+	Top : 16x1" Web: 83x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 81x0.5625" Bottom: 16x1.5"	$15,331	$100,100
255'	M-	Top : 18x1.5" Web: 83x0.5625" Bottom: 20x2.5"	Top : 16x1.375" Web: 81x0.5625" Bottom: 20x1.75"	$50,001	

6.2.3   Finite Element Analysis

Based on bridges deigned in section 6.2.1. Finite element models have been developed for typical single span and three span continuous bridges. These bridges include:

a) 80', 100', 120', and 160' single span bridges with unshored condition.

b) 80', 100', 120', and 160' single span bridges with shoring at 1/3 points.

c) 80', 100', 120', and 160' single span bridges with fully shored condition.

d) 140'-180'-140' and 199'-255'-199' three span continuous bridge with unshored condition.

e) 140'-180'-140' and 199'-255'-199' three span continuous bridge with shoring at 1/3 points of center span and ½ points of side spans with FDOT specified pouring sequence.

f) 140'-180'-140' and 199'-255'-199' three span continuous bridge with shoring at 1/3 points of center span and ½ points of side spans without FDOT specified pouring sequence.

The research team used finite element analysis (FEA) software ANSYS for finite element modeling and analysis. ANSYS is a well-known FEA software with strong modeling and analysis capability in non-linear material models, damage models, non-linear geometry, and time-dependent effects. The FEA models utilize tridimensional discretization of structures using the 3D beam, shell, and solid elements without simplification of any degree of freedom. Tridimensional discretization is essential in the case of nonlinear simulation of concrete structures. Shrinkage and creep models specified in the current AASHTO LRFD BDS (AASHTO (2020)) were used to determine the input for the modeling of the long-term effects on the concrete deck.

6.2.3.1 *Model Idealization*

The research team used beam, shell, and solid elements for 3D FE models. The model details such as connections and boundary conditions were under special consideration to ensure a proper modeling idealization. The cross-frames were connected to the girder with moment releases at the ends.

Figure 6-14 and Figure 6-15 show the meshing of steel girders and the entire bridge from a sample steel composite girder bridge model. The flanges and web of the girder are modeled using shell elements. The size of the element can be adjusted to achieve the desired accuracy while minimizing the model running time. Figure 6-16 shows the assembly of the model using the contact function. There are various methods to assemble the model. For rigid connection, the CPINTF command can be used to couple two coincident nodes with defined degrees of freedom. The supports were assumed as elastomeric bearings with a nominal stiffness of 100 kips/ft in the lateral and longitudinal directions (White et al., 2020). Shoring supports were simulated as deflection-controlled supports with adjustable stiffness. Based on AASHTO Guide Design Specifications for Bridge Temporary Works (AASHTO, 2007), the maximum vertical deflection shall not exceed 1/240 of their span under the dead load of the concrete only, regardless of the fact that deflection may be compensated for by camber strips.
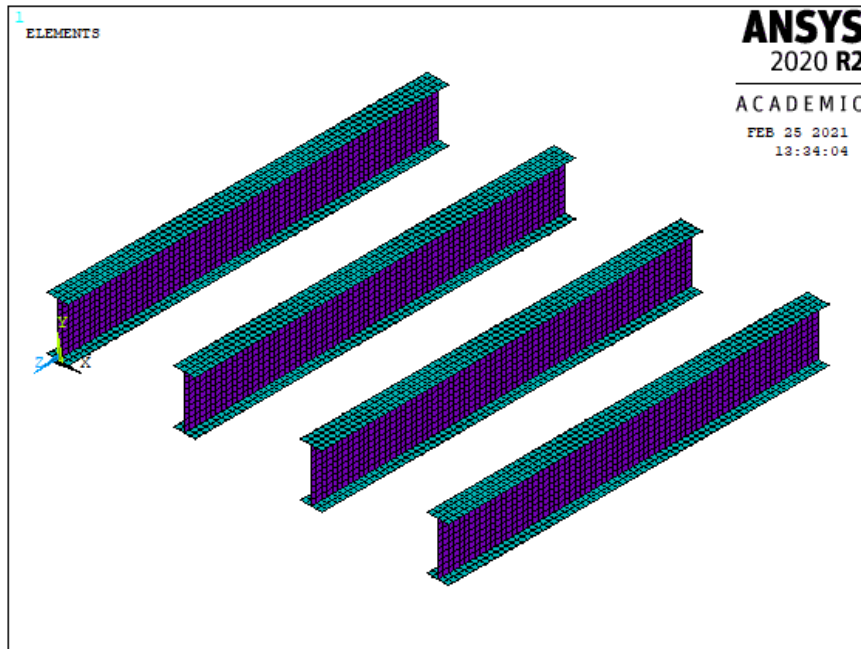
Figure 6-14. The meshing of steel girders
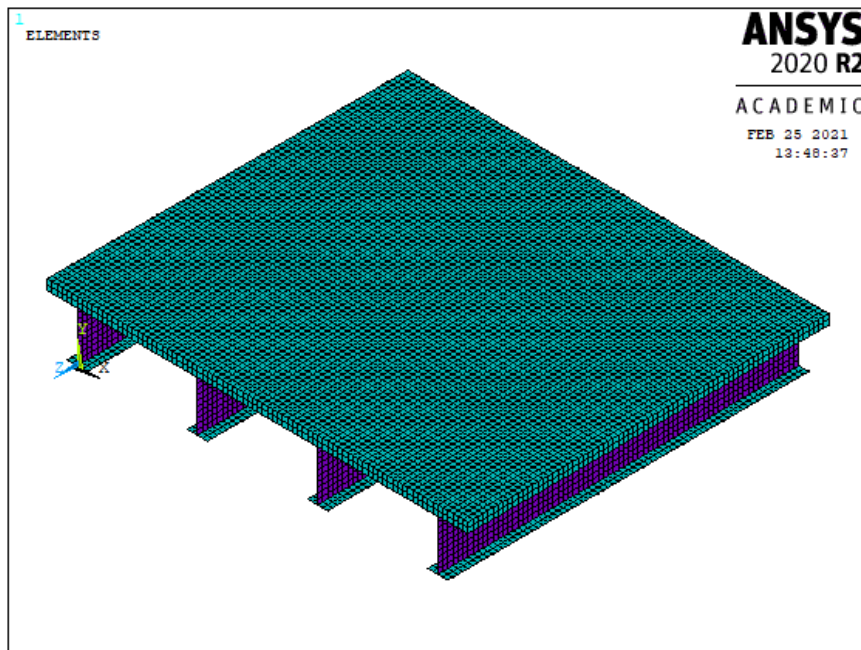
Figure 6-15. The meshing of a typical steel composite girder bridge model
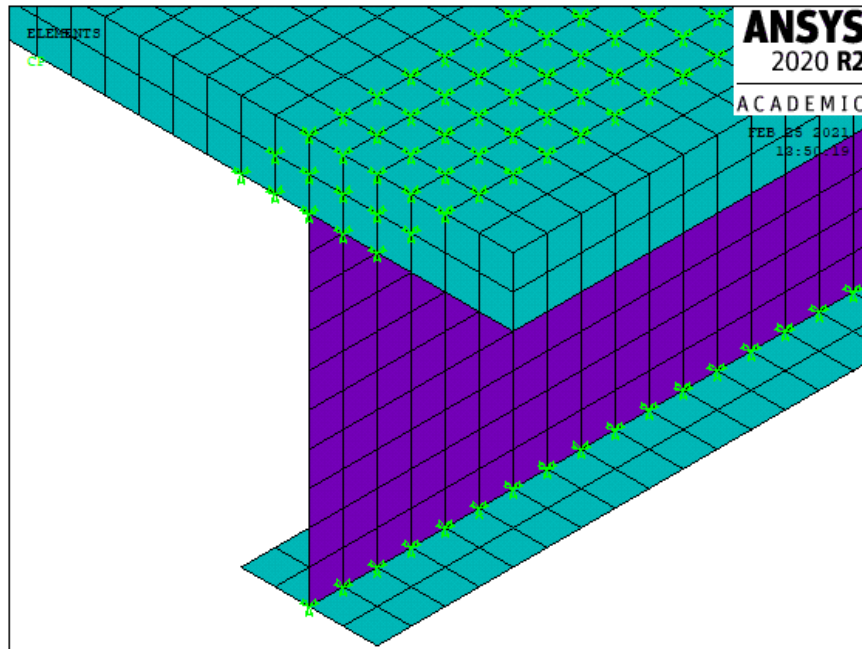
Figure 6-16. Model assembly using contact function

6.2.3.2 *Concrete Modeling*

The concrete material was modeled using Drucker-Prager Concrete Model as specified in Ansys material reference manual (Ansys, 2021). The tension and tension-compression Drucker-Prager yield surface was defined in the FE model. The input used for concrete modeling was based on the modified Hognestad stress-strain curve (Wight, 2012). Figure 6-17 shows the stress-strain curve that was used for concrete input in this project.

Concrete stress-strain relations
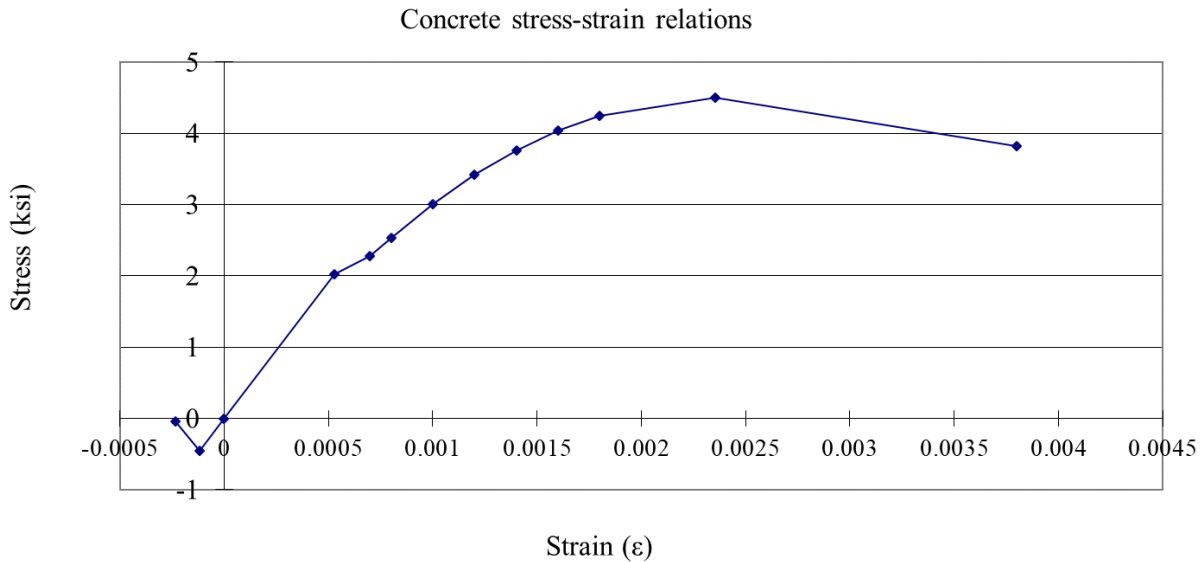
Figure 6-17. Stress-strain curve for concrete material (f'c=4.5ksi)

SOLID65 element was used to model concrete material in this project. SOLID65 is used for the 3-D modeling of solids with or without reinforcing bars (rebar). The solid is capable of cracking in tension and crushing in compression. In concrete applications, the solid capability of the element may be used to model the concrete while the rebar capability is available for modeling reinforcement behavior. Whenever the rebar capability of the element is used, the rebar are assumed to be "smeared" throughout the element.

6.2.3.3 *Time-Dependent Effects (Creep and Shrinkage)*

The long-term time-dependent effects such as creep and shrinkage are very important in evaluating the impact of shored construction on steel composite girder bridges. Thus, creep and shrinkage effects were included in the finite element analysis. As shown in Table 6-10 and Figure 6-18, the creep coefficients and shrinkage strains were calculated based on AASHTO 5.4.2.3.2. Based on ACI 209.2R-08 "Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete", it is assumed that the age at loading is 14 days for unshored condition. If shored construction is used, it is assumed that the shoring supports were removed at 28 days thus the age at loading is 28 days. Figure 6-18 shows the calculated creep coefficients for shored and unshored conditions at different ages.

It was noted that for unshored constructed bridges, only DC2 loads (Parapet loads plus wearing surface load) were used for creep analysis. However, total dead load including steel girder self-weight and concrete slab were used for shored constructed bridges. The reason for applying different loads for shored and unshored constructed bridges is the concrete slab is only subjected to DC2 loads for unshored constructed bridges while it is subjected to all dead loads for shored constructed bridges.

Table 6-11. Creep and shrinkage parameters

t, days	ktd, unshored	ktd, shored	$\Psi(t,ti)$, unshored	$\Psi(t,ti)$, shored	εsh ($\mu\varepsilon$)
7	0	0	-	-	0
14	0	0	-	-	52
28	0.391	0	0.353	-	125
30	0.408	0.408	0.390	0.059	134
50	0.535	0.535	0.657	0.449	196
100	0.697	0.697	0.964	0.834	277
200	0.821	0.821	1.177	1.068	338
300	0.873	0.873	1.260	1.153	364
365	0.893	0.893	1.292	1.185	374
500	0.920	0.920	1.333	1.225	387
700	0.941	0.941	1.365	1.257	398
1000	0.958	0.958	1.391	1.281	406
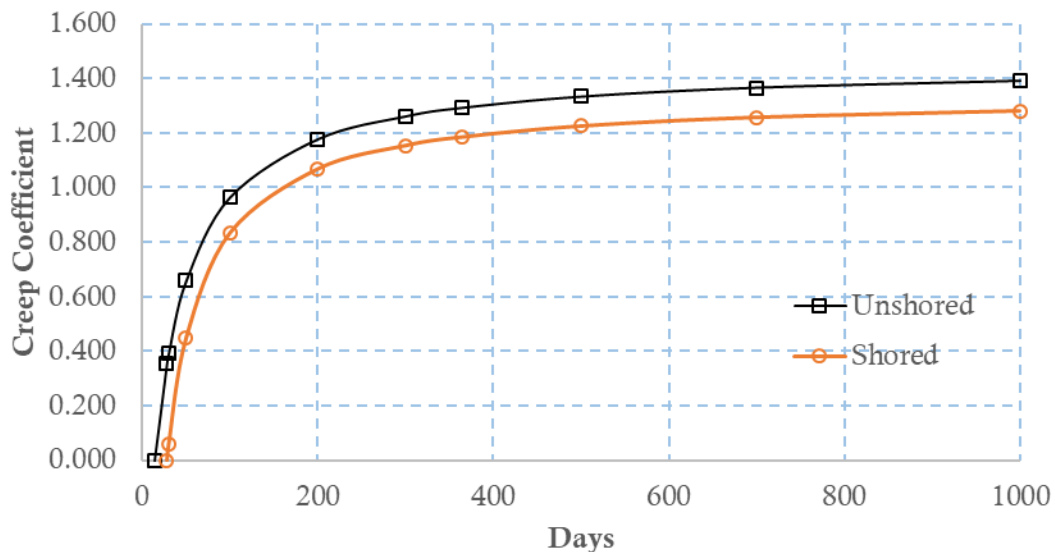
Figure 6-18. Creep coefficients for shored and unshored condition

A total of 13 different creep models were considered including strain hardening model, time hardening model, generalized exponential, combined time hardening etc. After comparing results with AASHTO predictions, the modified time hardening model was chosen to simulate the creep effect. The shrinkage effect was simulated by applying the temperature change to the model.

To ensure the accuracy of creep modeling, the creep strains obtained from FE modeling were compared with strains predicted with AASHTO 5.4.2.3.2 equations. As shown in Figure 6-19, the FE model prediction is very close to AASHTO prediction during the early age. From age of 90 days to 200 days, as concrete ages older, the discrepancy becomes larger. However, from 200 days to 1000 days, the discrepancy becomes smaller and smaller, and the predications at 1000 days are almost identical. Please note that this graph was plotted for a specific location. However, the concrete creep model was applied to the FE model in general and similar results were observed for different locations.
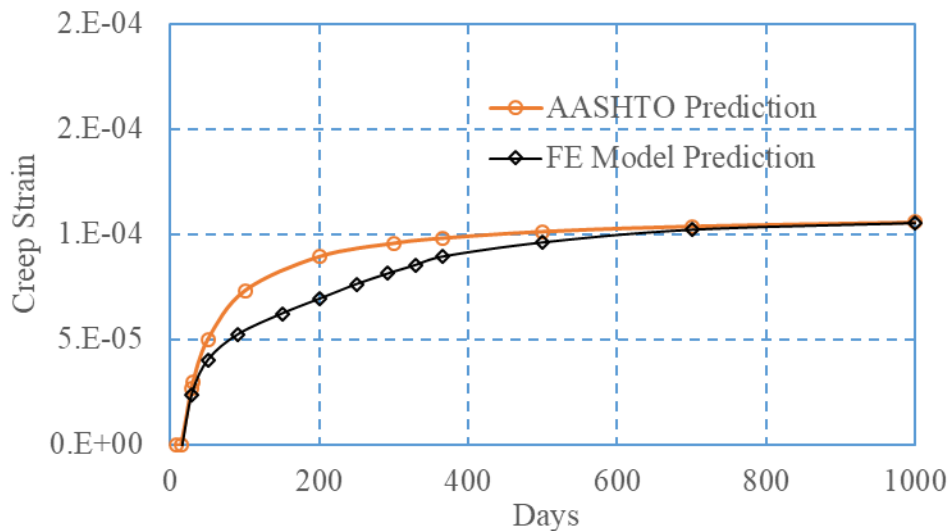
Figure 6-19. AASHTO predictions vs. FE model predictions

6.2.3.4   *FE Analysis Results and Parametric Study*

After the creep results were verified with AASHTO prediction as shown in Figure 6-19, a series of FE models were developed and a parametric study was performed with varied parameters. Figure 6-20 presents a matrix of parameters that have been considered. Although both rolled beam and plate girder were used in the design of single span steel composite bridge, only plate girder

single span bridges were used in the FE analysis. Furthermore, for single span bridge, girder spacing of 10' and deck width of 45' were used in the FE analysis. For 3-span continuous bridge, girder spacing of 10'6" and deck width of 48' were used in the FE analysis. The span lengths considered in this study are presented in section 6.2.1 of this report. For shored constructed bridges, various shoring schemes presented in section 6.2 of the report were considered. Furthermore, different concrete deck pouring sequences were considered for continuous span bridges. As shown in Figure 6-20, the current pouring sequence used by FDOT (FDOT SDM Vol. 2) is assumed for the unshored constructed bridges, with 72 hours gap in between each concrete pour. For shored construction, it is assumed that span 1 was poured first, followed by span 2 and span 3, with 72 hours waiting gap waived. However, different age/strength was considered for different spans. For instance, if it took 5 hours to pour span 1 and 5 hours for span 2, when pouring span 3, the concrete strength in span 1 was assigned as 10 hours concrete. Deck concrete strength gain values from FDOT SDG Table 4.2.4-1 were used to calculate concrete strength at different ages. Based on the parametric study, the long-term effects and the performance of the steel composite girder bridge using shored construction can be evaluated.
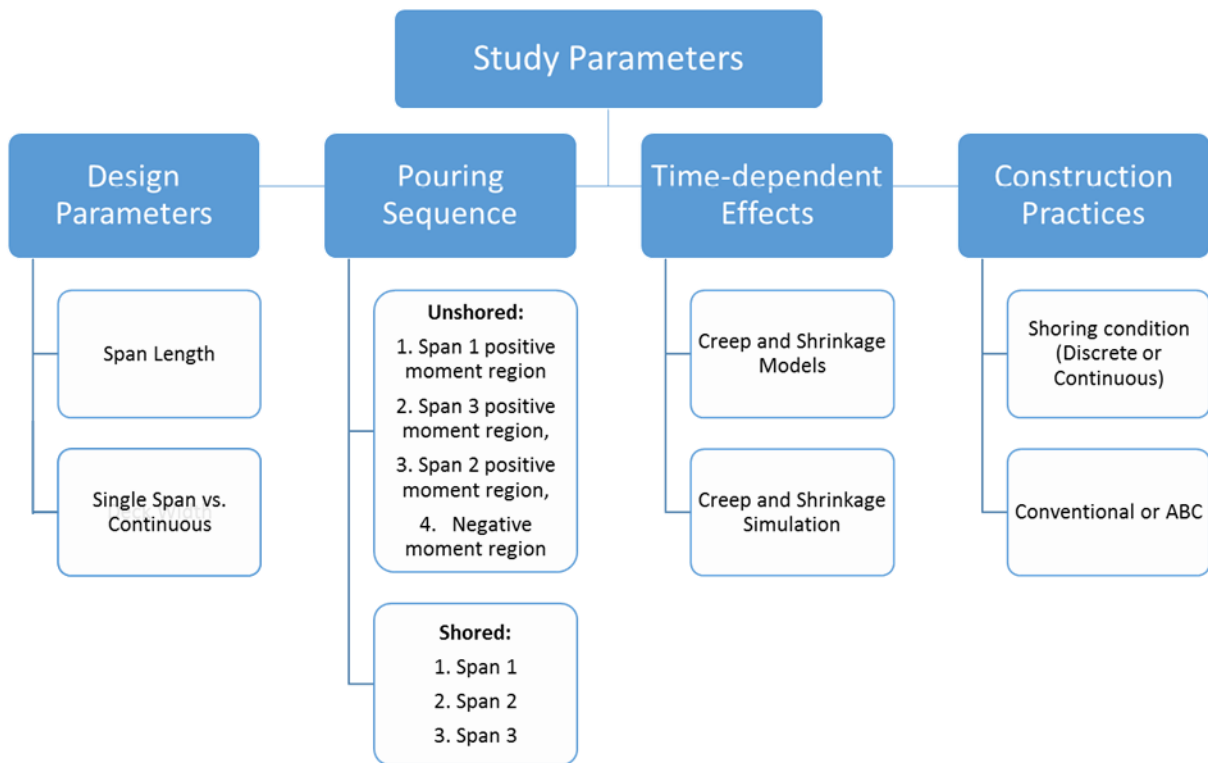
Figure 6-20. Matrix of parameters for parametric study

As described in the scope of work for this research project, there are several concerns with regards to shored construction including the loss of composite action, increased deck tensile stresses at intermediate support of continuous girder superstructures, and increased long-term deflection. These concerns were investigated in the following FE analysis studies. Moreover, the stresses extracted from FE analysis are also compared with the design stresses based on the 3n assumption.

6.2.3.4.1 Single-span Bridges (80' Span with 10' Spacing)

As shown in Table 6-12, various load cases were considered in the FE analysis starting with self-weight of the girder, followed by wet concrete load, DC2 load including parapet and wearing surface, creep effect, and shrinkage effect. Both creep and shrinkage were considered until 1000 days after concrete casting.

Table 6-12. Load cases for single-span bridges

#	Load Case
1	Self-weight of the girder
2	Wet concrete
3	DC2
4	Creep (1000 days)
5	Shrinkage (1000 days)

Table 6-13 through Table 6-15 show the stresses for an 80 ft single-span bridge with different shoring conditions at midspan of girder #3, which is highest among all girders. The stress at the bottom fiber of the steel section is about 21.999 ksi for the unshored condition compared to 18.595 ksi for shored at 1/3 points. It indicates that shored construction reduces the stress level by about 15%. The stress results from FE analysis were also compared with design values calculated based on "3n" assumption. As shown in Table 6-13, for unshored conditions, without considering shrinkage, the total stress at the bottom flange is 20.06 ksi, which is very close to 3n based result of 19.24 ksi. For bridges with the shored condition as shown in Table 6-14and Table 6-15. The stress at the top flange from FE would be -24.47 ksi without considering shrinkage, which is comparable to 3n based result of -20.65 ksi too. Based on the AASHTO shrinkage model, at 1000 days, the shrinkage strain is about 406 microstrains (validated with ACI 209 numerical example). This amount of shrinkage strain resulted in higher stresses as shown in Table 6-13 through Table 6-15.

Table 6-13. Stresses for an 80 ft single span plate pirder bridge with 10' spacing, five-girder system (girder #3, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.188	-3.178	0.000
2	Wet concrete	steel section	12.030	-14.756	0.000
3	DC2	composite section	4.473	-0.445	-0.177
4	Creep(1000 days)	composite section	1.365	-6.087	0.402
	Subtotal		20.056 (19.24)*	-24.467 (-20.65)*	0.225
5	Shrinkage (1000 days)	composite section	1.943	-9.040	0.115
	Total		21.999	-33.507	0.340

* In Table 12 through Table 14, the values in parenthesis are design values based on "3n" calculations.

Table 6-14. Stresses for an 80 ft single span plate pirder bridge with 10' spacing, five-girder system (girder #3, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	Removel of temporary support (28 days)	composite section	16.605	-1.407	-0.650
4	Creep(1000 days)	composite section	0.971	-4.317	0.300
	Subtotal		17.576 (19.63)	-5.724 (-6.19)	-0.350
5	Shrinkage (1000 days)	composite section	1.019	-4.741	0.061
	Total		18.595	-10.465	-0.289

Table 6-15. Stresses for an 80 ft single span plate pirder bridge with 10' spacing, five-girder system (girder #3, shored at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.054	-0.090	0.000
2	Wet concrete	steel section	0.306	-0.342	0.000
3	Removel of temporary support (28 days)	composite section	16.605	-1.407	-0.650

Table 6-15. Stresses for an 80 ft single span plate pirder bridge with 10' spacing, five-girder system (girder #3, shored at 1/3 points), cont'd

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
	Subtotal		17.940 (22.21)	-6.174 (-8.77)	-0.349
4	Shrinkage (1000 days)	composite section	1.019	-4.741	0.061
	Total		18.959	-10.916	-0.288

In addition, the effect of the shoring condition on creep strain development was investigated. As shown in Figure 6-21, shored construction also lowers the creep strain level since the creep effect delays in developing as permanent loading starts later compared to the unshored condition.
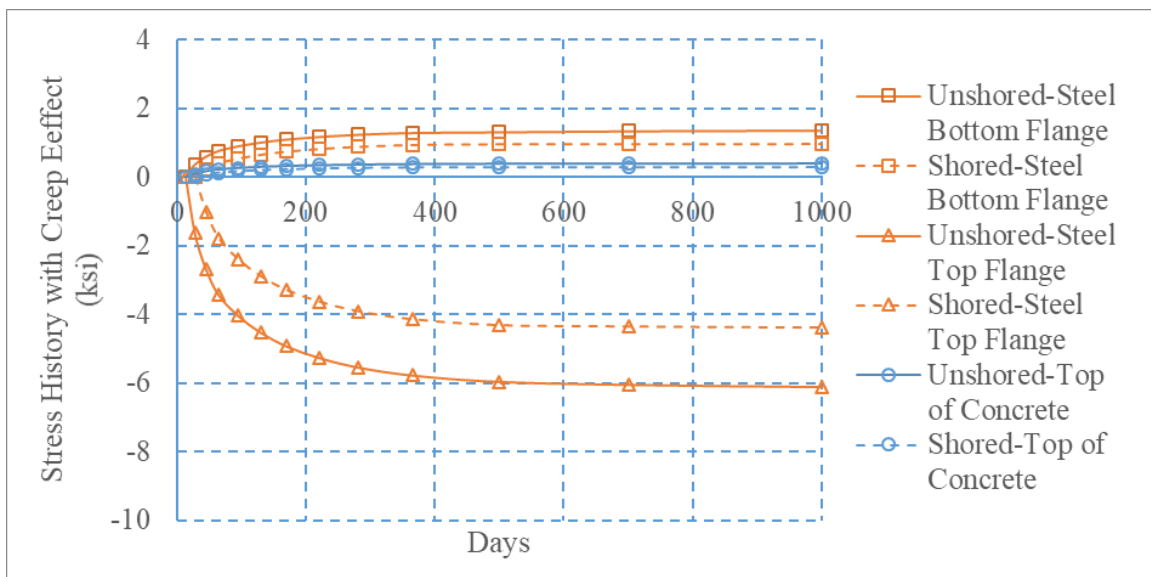
Figure 6-21. Time history for stress due to creep effect (80 ft single span plate girder bridge with 10' spacing, five-girder system, midspan of girder #3)

Figure 6-22 shows the time history of creep stresses for various span lengths. Although the initial stress level for different span lengths varies from 16.6 ksi to 24.6 ksi, the overall increases in stress due to the creep effect are very similar.
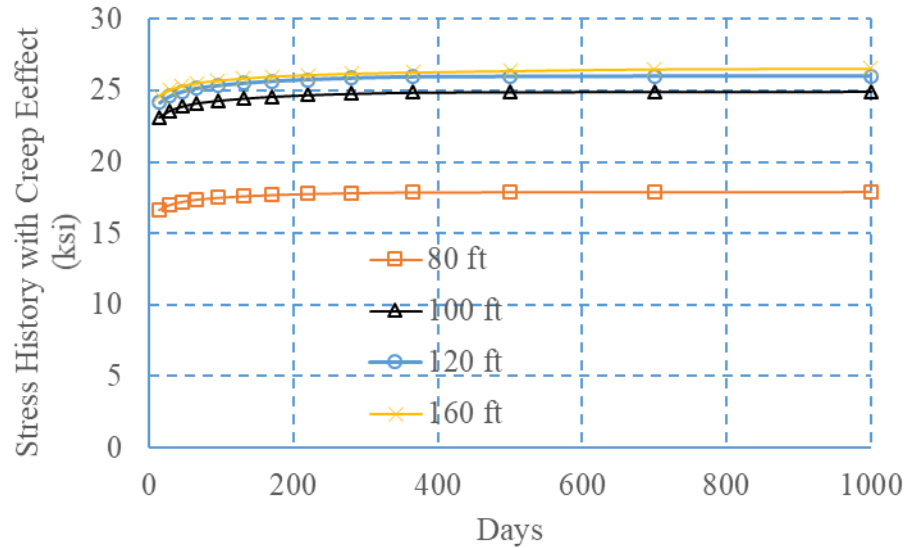
Figure 6-22. Time history for stress with creep effect (80 ft to 160 ft single span plate girder bridge with 10' spacing, five-girder system, midspan of girder #3)

6.2.3.4.2 *Three-Span Continuous Bridges (140'-180'-140' with 10'6'' Spacing)*

A 140'-180'-140' three-span continuous bridge was modeled to demonstrate the behavior of three-span continuous bridges. As shown in Table 6-16, various load cases were considered in the FE analysis starting with self-weight of the girder, followed by concrete pour #1 through #5, DC2 load including parapet and wearing surface, creep effect, and shrinkage effect. Similar to the single-span bridge, both creep and shrinkage were considered until 1000 days after concrete casting. It is worth noting that the pouring sequence was modeled based on FDOT SDM Vol. 2 provisions as shown in Figure 6-23. The length of pour #1 and #2 is the same, which is about 75% of span 1. The length of pour #4 and #5 is the same and is about 25% of span 1 plus 25% of span 2, and the length of pour #3 is about 50% of span 2. The concrete strength gain values are taken based on FDOT SDG Table 4.2.4-1 as shown in Table 6-17.

Table 6-16. Load cases for continuous span bridges

#	Load Case
1	Self-weight of the girder
2	Pour #1
3	Pour #2
4	Pour #3
5	Pour #4
6	Pour #5
7	DC2+WS
8	Creep(1000 days)
9	Shrinkage (1000 days)

DECK CASTING SEQUENCE
(3 SPAN CONTINUOUS SUPERSTRUCTURE SHOWN; OTHER CONTINUOUS SUPERSTRUCTURES SIMILAR)
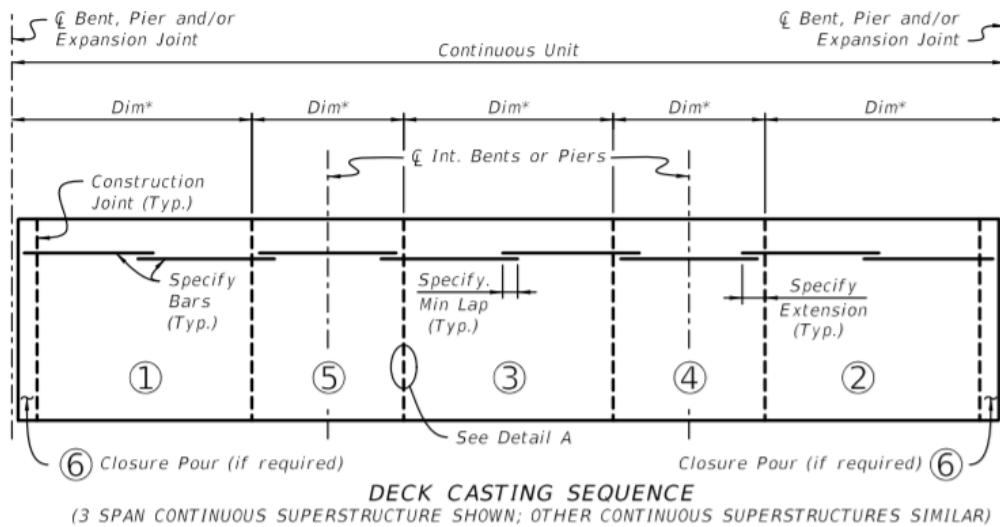
Figure 6-23. Pouring sequence (FDOT SDM Vol. 2)

Table 6-17. Deck concrete strength gain values (FDOT SDG Table 4.2.4-1)

Day	Class II (Bridge Deck) (psi)	Class IV (psi)
3	2,740	3,720
6	3,180	4,210
9	3,610	4,340
12	3,840	4,550
15	4,020	4,820
18	4,160	5,040
21	4,290	5,220
24	4,390	5,390
27	4,500	5,500

6.2.3.4.2.1 Overall behavior

Table 6-18 through Table 6-20 show the stresses at midspan of span 2 of interior girder #3 for a 140'-180'-140' three-span continuous bridge with three shoring conditions: (1) unshored, (2) shored at 1/3 points of center span and ½ points of side spans with FDOT specified pouring sequence, and (3) shored at 1/3 points of center span and ½ points of side spans without FDOT specified pouring sequence. Similar to single-span bridges, shored construction helps lower the stress level significantly. There is a reduction of 35% for stress at the top flange of the steel section. In addition, comparable results were observed between FE results and design values based on the 3n assumption. As shown in Table 6-18, design values based on the 3n assumption underestimated the stress level compared with FE results (16% lower). Similar results were also observed for the bridges designed with shored construction condition (Table 6-19). Comparing with Table 6-19, Table 6-20 shows that without following the FDOT pouring sequence, the stress maintains at a similar level if the bridge was shored with the same shoring condition.

Table 6-18. Stresses for a 140'-180'-140' three span continuous plate girder bridge with
10.5' spacing, five-girder System at midspan of span 2, girder #3 (unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.111	-2.332	0
2	Pour#1	steel section	-0.743	0.862	0
3	Pour#2	steel section	-2.948	3.236	0
4	Pour#3	steel section	15.69	-8.992	0
5	Pour#4	steel section	-0.542	3.568	-0.202
6	Pour#5	steel section	-1.006	0.206	0.021
7	DC2+WS	composite section	2.954	-0.419	-0.08
	Subtotal		15.516	-3.871	-0.261
8	Creep(1000 days)	composite section	0.428	-1.233	0.024
	Subtotal		15.944 (13.47)	-5.104 (-4.5)	-0.237
9	Shrinkage (1000 days)	composite section	-11.085	-7.12	0.584
	Total		4.859	-12.224	0.346

* In Table 17 through Table 19, the values in parentheses are design values based on 3n calculations.

Table 6-19. Stresses for a 140'-180'-140' three span continuous plate girder bridge with 10.5'
spacing, five-girder System at midspan of span 2, girder #3 (shored at 1/3 points of center span
and ½ points of side spans)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.276	-0.307	0
2	Pour#1	steel section	0.297	-0.346	0
3	Pour#2	steel section	0.323	-0.377	0
4	Pour#3	steel section	2.152	-0.862	0
5	Pour#4	steel section	-0.562	0.859	-0.043
6	Pour#5	steel section	-0.356	0.058	0.006
7	Removal of temporary support (28 days)	composite section	7.45	-2.2	-0.25
	Subtotal		9.58	-3.175	-0.287
8	Creep(1000 days)	composite section	0.414	-1.219	0.037
	Subtotal		9.994 (7.09)	-4.394 (-3.57)	-0.25
9	Shrinkage (1000 days)	composite section	-7.068	-4.682	0.384
	Total		2.926	-9.076	0.134

Table 6-20. Stresses for for a 140'-180'-140' three span continuous plate girder bridge with 10.5' spacing, five-girder System at midspan of span 2, girder #3, no pouring sequence (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.276	-0.307	0
2	Wet Concrete	steel section	1.444	-0.602	0
3	Removal of temporary support (28 days)	composite section	7.45	-2.2	-0.25
	Subtotal		9.17	-3.109	-0.25
4	Creep(1000 days)	composite section	0.415	-1.221	0.037
	Subtotal		9.59 (7.09)	-4.33 (-3.57)	-0.213
5	Shrinkage (1000 days)	composite section	-7.312	-4.697	0.385
	Total		2.273	-9.027	0.172

Table 6-21 through Table 6-23 show the results at intermediate support of girder #3 for unshored, shored at 1/3 points of center span and ½ points of side spans following FDOT specified pouring sequence, and shored at 1/3 points of center span and ½ points of side spans without following FDOT specified pouring sequence, respectively. Similar to the midspan location, shored construction lowers the stress level significantly. Comparing between midspan and support location, shored construction has a more significant effect on support locations. The stress at the bottom fiber of the steel girder (f_{sb}) decreased from -21.376 ksi to -11.58 (47% reduction). However, it is worth noting that the stress on top of the concrete slab at the intermediate support exceeds the splitting tensile strength of concrete. Thus, concrete may crack at the intermediate supports after temporary shoring is removed. Table 6-23 shows that the stress level remains similar without following the FDOT pouring sequence if the bridge was shored at 1/3 points of center span and ½ points of side spans.

Figure 6-24 shows the time history of creep stresses at the bottom of the steel section, the top of the steel section, and the top of the concrete slab. It shows that the creep stresses are higher for the unshored condition at the early age. However, the difference becomes smaller as the loading duration gets longer.

Table 6-21. Stresses for a 140'-180'-140' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3 (unshored)

#	Load Case	Load carrying element	fsb at support (ksi)	fst at support (ksi)	fct at support (ksi)
1	Self-weight of the girder	steel section	-2.312	2.961	0
2	Pour#1	steel section	1.108	-1.285	0
3	Pour#2	steel section	-3.582	4.846	0
4	Pour#3	steel section	-6.902	7.023	0
5	Pour#4	steel section	-1.104	-3.625	0
6	Pour#5	steel section	-0.387	-6.096	0
7	DC2+WS	composite section	-3.633	1.048	0.19
	Subtotal		-16.812	4.872	0.19
8	Creep(1000 days)	composite section	-0.343	2.195	-0.138
	Subtotal		-17.155 (-17.61)*	7.067 (21.85)*	0.052
9	Shrinkage (1000 days)	composite section	-4.222	-6.334	0.688
	Total		-21.376	0.731	0.739

* In Table 6-21 through Table 6-23, the values in parentheses are design values based on AASHTO 6.10.1.1.1c.

Table 6-22. Stresses for a 140'-180'-140' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3 (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at support (ksi)	fst at support (ksi)	fct at support (ksi)
1	Self-weight of the girder	steel section	-0.392	0.487	0
2	Pour#1	steel section	0.002	0.062	0
3	Pour#2	steel section	-0.135	0.188	0
4	Pour#3	steel section	-0.1	-0.151	0
5	Pour#4	steel section	-1.065	0.963	0
6	Pour#5	steel section	0.028	-1.02	0
7	Removal of Temporary Supports (28 days)	composite section	-6.88	5.5	0.705
	Subtotal		-8.542	6.029	0.705
8	Creep(1000 days)	composite section	-0.264	2.232	-0.159
	Subtotal		-8.806 (-17.66)	8.261 (22.57)	0.546
9	Shrinkage (1000 days)	composite section	-2.776	-4.165	0.452
	Total		-11.582	4.096	0.998

Table 6-23. Stresses for a 140'-180'-140' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3, no pouring sequence (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at support (ksi)	fst at support (ksi)	fct at support (ksi)
1	Self-weight of the girder	steel section	-0.392	0.487	0
2	Wet Concrete	steel section	-1.364	1.118	0
3	Removal of Temporary Supports (28 days)	composite section	-6.88	5.5	0.705
	Subtotal		-8.636	7.105	0.705
4	Creep(1000 days)	composite section	-0.264	2.233	-0.159
	Subtotal		-8.9 (-17.66)	9.338 (22.57)	0.546
5	Shrinkage (1000 days)	composite section	-2.768	-4.153	0.451
	Total		-11.668	5.185	0.997
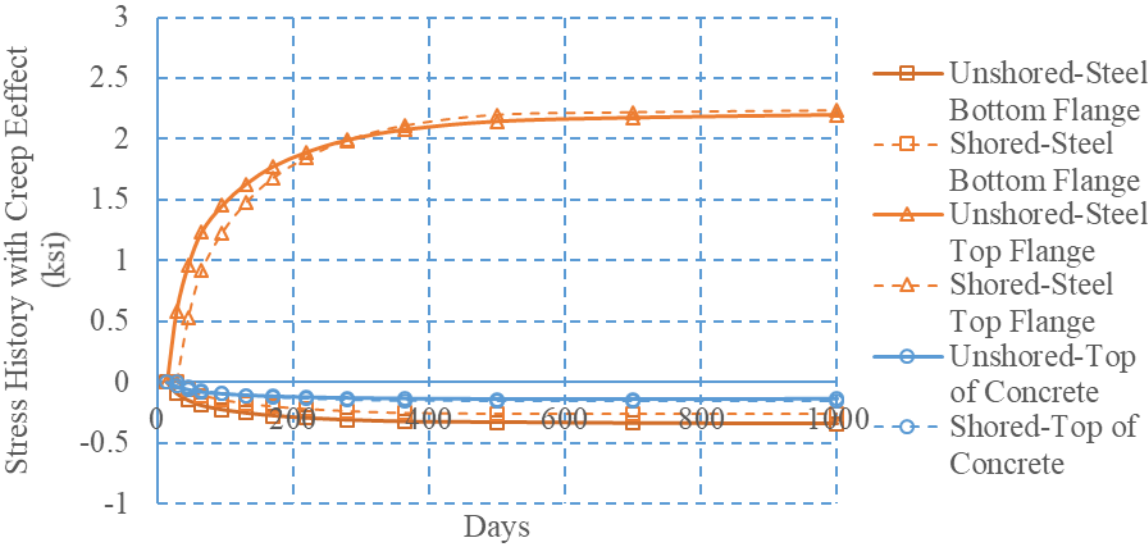
Figure 6-24. Time history for stress due to creep effect (140'-180'-140' three Span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3)

6.2.3.4.2.2 Creep effect

The long-term deflection due to the creep effect was investigated for both unshored and shored conditions. The long-term deflection result at the midspan of span 2 of a 140'-180'-140' 3-span continuous bridge is shown in Figure 6-25. The long-term deflection for the unshored condition is higher than the one with the shored condition. However, the difference becomes smaller as the loading duration gets longer. The reason for this observation is that although the creep coefficient is higher for unshored construction, the applied long-term sustained load is higher for the shored condition since all dead loads are applied to the composite section while only DC2 loads apply to the composite section if the bridge is unshored.
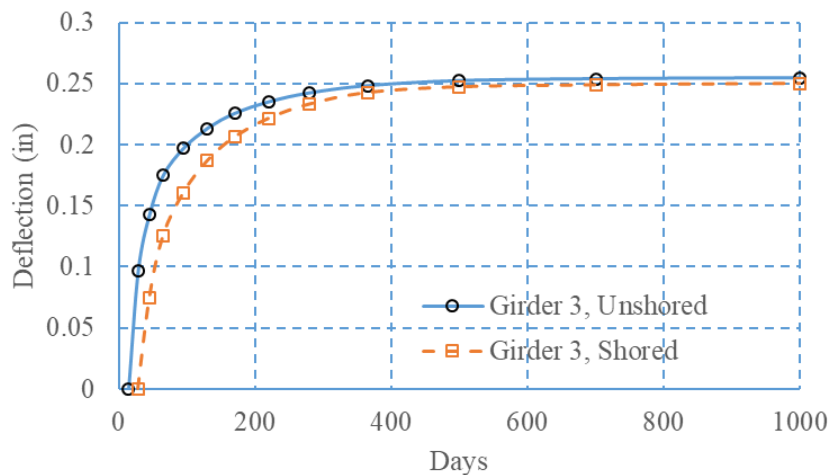
Figure 6-25. Long term deflection due to the creep effect, unshored vs. shored (a 140'-180'-140' three span continuous plate girder bridge with 10.5' spacing, five-girder system at midspan of span 2, girder #3)

6.2.3.4.2.3 Shrinkage effect

The long-term deflection due to the shrinkage effect was also investigated for both unshored and shored conditions. As shown in Figure 6-26, the long-term deflection for the unshored condition is higher than the one with the shored condition. However, similar to the creep effect, the difference becomes smaller as the loading duration gets longer. The reason is that a higher total shrinkage strain was applied for the unshored condition since it was assumed the shrinkage took

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effect at 14 days for the unshored condition while it was assumed as 28 days for the shored condition. However, this difference will get smaller and smaller as the concrete ages.
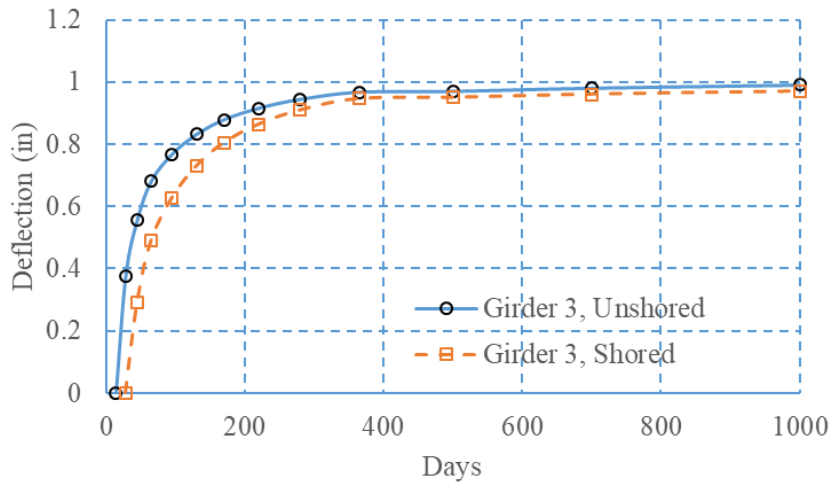
Figure 6-26. Long term deflection due to the shrinkage effect, unshored vs. shored (a 140'-180'-140' three span continuous plate girder bridge with 10.5' spacing, five-girder system at midspan of span 2, girder #3)

6.2.3.4.2.4 Concrete tensile stress at intermediate supports

As shown in Figure 6-27, before the removal of temporary shoring, the maximum tensile strain on top of the concrete slab is about 50 $\mu\varepsilon$, which is less than the crack strain of 131 $\mu\varepsilon$ for concrete with compressive strength of 4.5 ksi. However, after the temporary shoring is removed, the reaction forces applied at the temporary shoring location cause larger strain developed at both positive and negative moment region. Particularly, the tensile strain on top of the concrete slab at intermediate supports exceeds 131 $\mu\varepsilon$, indicating that the concrete cracked at these intermediate supports. Similar behavior was also observed in stress and strain contour maps for the whole bridge and 1st interior support region, as shown in Figure 6-28 through Figure 6-30, respectively. The red area indicates where the possible cracks occurred. As shown in Figure 6-30, the crack didn't propagate through the entire slab depth but rather stopped at the half depth.
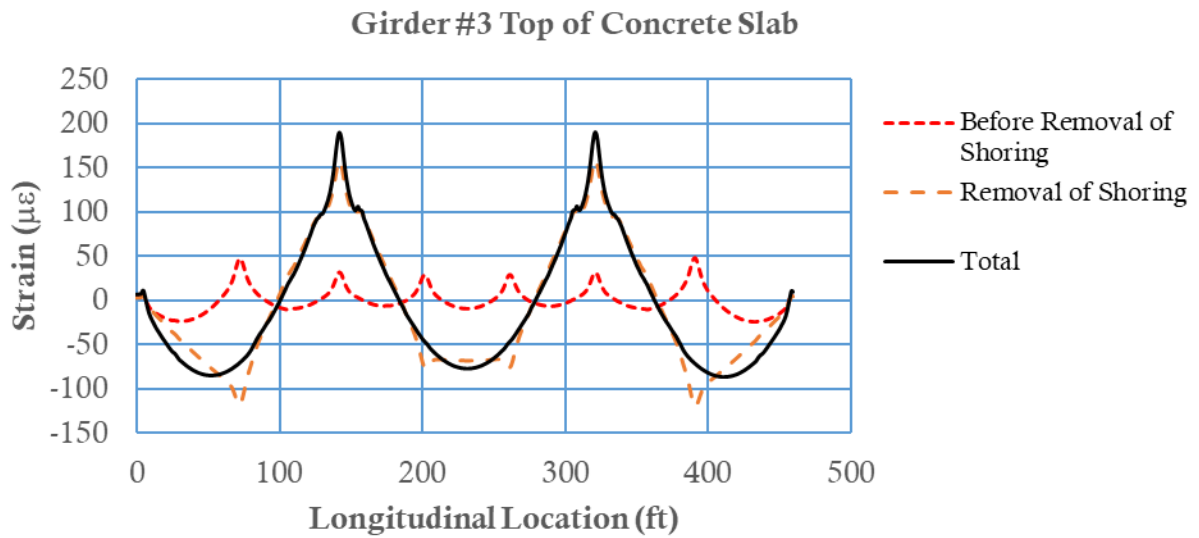
Figure 6-27. Longitudinal strain on top of concrete slab due to DC1 and DC2 loading (shored, a 140'-180'-140' three span continuous plate girder bridge with 10.5' Spacing, five-girder system, girder #3)
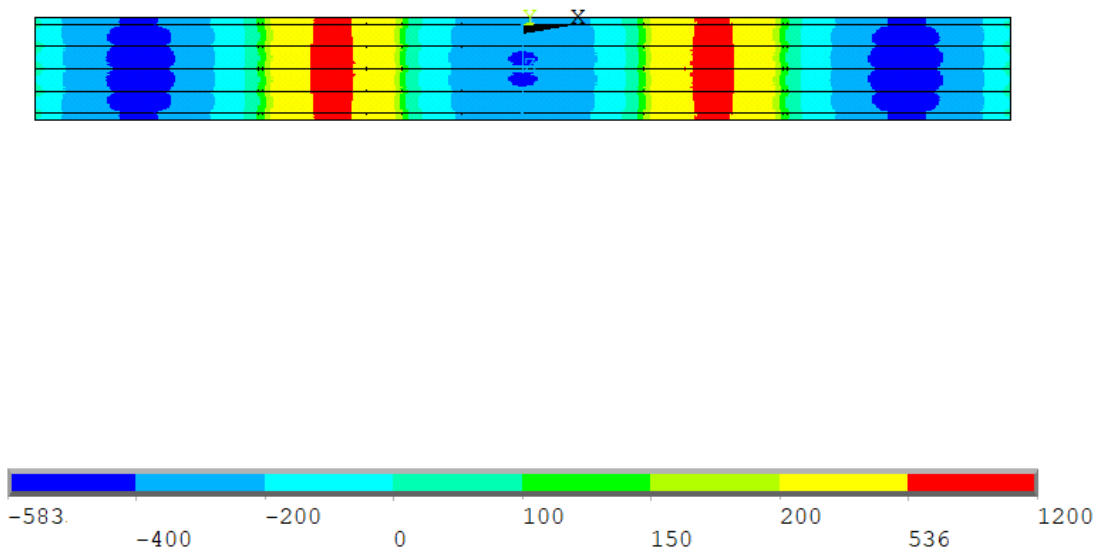
Figure 6-28. Stress contour of concrete slab after temporary supports was removed (shored, a 140'-180'-140' three span continuous plate girder bridge with 10.5' Spacing, five-girder system, in psi)
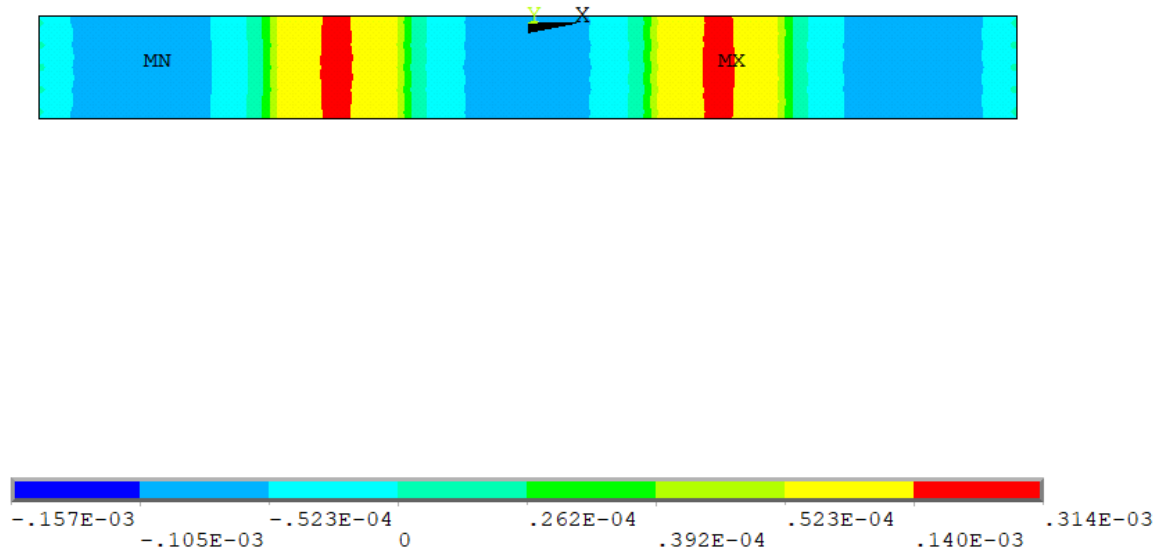
Figure 6-29. Strain contour of concrete slab after temporary supports was removed (shored, a 140'-180'-140' three span continuous plate girder bridge with 10.5' Spacing, five-girder system)

1st Interior Support
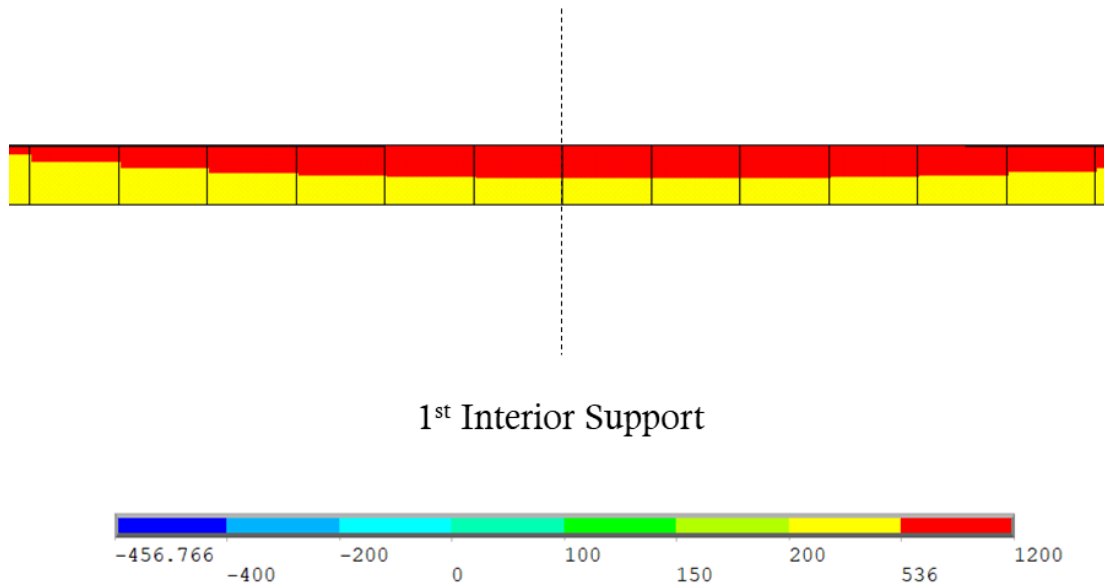
Figure 6-30. Stress contour of concrete slab at 1st interior support after temporary supports was removed (shored, a 140'-180'-140' three span continuous plate girder bridge with 10.5' Spacing, five-girder system, girder #3, length of the section is about 12 ft, in psi)

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6.2.3.4.2.5 Composite action

In order to check the composite action between the concrete slab and steel beam, the longitudinal tensile stresses on top of the steel beam were extracted after the temporary supports were removed. As shown in Figure 6-31, at intermediate supports, the longitudinal stress developed on top of the steel beam is about 5600 psi. According to AISC Steel Construction manual, assuming the deck oriented perpendicular to the steel beam and one shear stud per rid, the shear strength of a typical ¾" shear stud is 39ksi. Thus, the longitudinal stress developed on top of the steel beam is much less than the design strength of the shear studs. Thus, the composite action should hold under DC1 and DC2 loading after temporary supports were removed. However, the composite action need to be examined under live load as well.
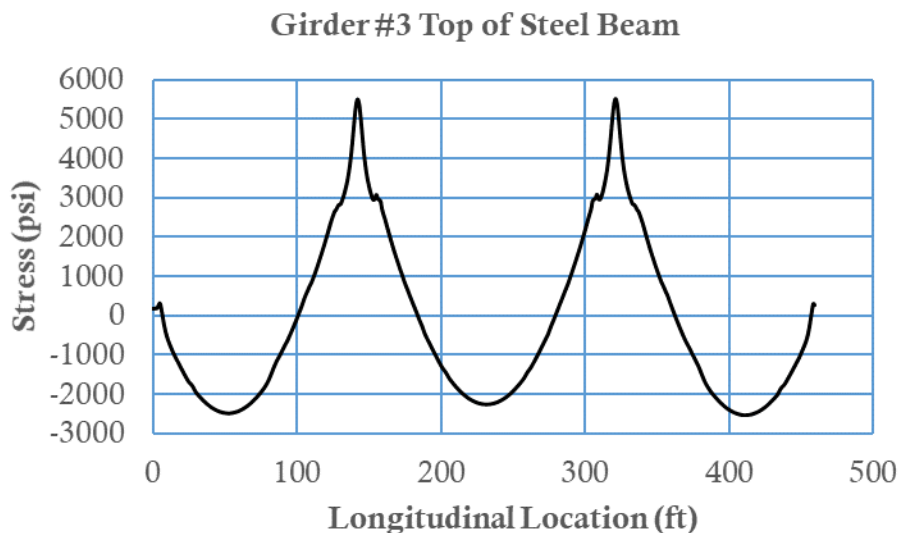
Figure 6-31. Longitudinal stress on top of steel beam due to DC1 and DC2 loading (shored, a 140'-180'-140' three span continuous plate girder bridge with 10.5' Spacing, five-girder system, girder #3)

7 CONCLUSIONS AND RECOMMENDATIONS

Based on the U.S. and international questionnaire survey, the following conclusions were concluded:

1) Most of the States don't have prior experiences in shored construction.
2) The major concerns the agencies have about shored construction are:
    a. Re-decking;
    b. Cost;
    c. Creep and shrinkage.
3) Michigan, Virginia, and West Virginia had successful experiences in shored construction. Shored constructed bridge either demonstrated less deflection or was designed with a smaller section.

The research team also reviewed the as-built construction drawings, specifications, and in-service performance information collected from respective state agencies. In addition, the inspection reports were obtained from Michigan and West Virginia. Based on the information collected, the construction and performance of these shored constructed bridges were reviewed and the findings are:

(1) For shored construction in the field, the shoring methods vary from steel members, timber mat footings, pile foundations, or temporary bent. The shoring method used by VDOT for the ABC bridge is shoring at quarter points until the concrete has attained a strength of 26.25 MPa (3,800 psi) at a nearby casting yard. There is no major construction issue about shored construction.
(2) Shored constructed bridges in Michigan showed comparable performance in comparison with unshored constructed bridges of similar ages.
(3) Shored constructed bridge in West Virginia demonstrated promising results as the shored constructed southbound twin showed less dead load deflection. The inspection report also proved that the shored southbound twin performs better compared to the unshored twin bridge.

The findings of the analytical study are summarized below:

(1) For a single-span steel composite girder bridge, there is a substantial reduction in the size of steel sections if the bridge girder is fully shored. If the steel girder is prefabricated with the fully shored condition, a much lighter section can be used, resulting in possible savings in materials and transportation.

(2) For a three-span continuous bridge, it is observed that the reduction in steel section area is averaging 9% for the positive moment region and is averaging over 17% for the negative moment region. It indicates shoring has a more significant effect on the negative moment region in comparison with the positive moment region.

(3) Based on the cost-benefit analysis, it can be concluded only a fully shored prefabricated bridge unit will be able to provide substantial cost savings considering the relatively higher price tag for shoring.

(4) Based on the FE results, it is observed that the shored construction reduces the stress level significantly. It is also observed without following the FDOT pouring sequence, the stress maintains at a similar level if the bridge was shored at 1/3 points of center span and ½ points of side spans.

(5) For bridges with the unshored condition, design values based on the 3n assumption underestimated the stress level compared with FE results (11% to 16% lower). However, for shored constructed bridges, design values based on the 3n assumption overestimated the stress level compared with FE results with a larger difference. The difference is larger for shored constructed bridges because 3n assumptions are not originally made for shored constructed steel composite girder bridges.

(6) For the three-span continuous bridge, shored construction has a more significant effect on support locations when compared with the midspan location. After the temporary shoring is removed, the reaction forces applied at the temporary shoring location cause larger strain to develop at both the positive and negative moment region. Particularly, the tensile strain on top of the concrete slab at intermediate supports exceeds 131 $\mu\varepsilon$, indicating that the concrete cracked at these intermediate supports.

(7) The long-term deflection for the unshored condition is higher than the one with the shored condition. However, the difference becomes smaller as the loading duration gets longer.

(8) Based on the FE analysis, the longitudinal stress developed on top of the steel beam is much less than the design strength of the shear studs. Thus, the composite action should hold after the temporary supports were removed

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9   APPENDIX A: SURVEY ON SHORED CONSTRUCTION

The survey will be web-based using Microsoft Forms with the survey link included in the body of an email and a pdf list of questions attached. The final form of the survey can be found from the following links as well as attached screenshots:

Part 1: Pre-interview questions

https://forms.office.com/Pages/ResponsePage.aspx?id=Y8kxabcHVkGrDjXR95A1uPizSeN
MCz5Dt7de78L90gtUMlVIMkY1Ukg0TFFXVDIwUzZHTU1NMkRUMC4u

Part 2: Post-survey follow up questions

https://forms.office.com/Pages/ResponsePage.aspx?id=Y8kxabcHVkGrDjXR95A1uPizSeN
MCz5Dt7de78L90gtUQ1FJSFpFQlFSM0dRS1I5UlFXUk9KUzRRVi4u

Survey on Shored Construction

This brief survey questionnaire is part of a research project sponsored by Florida Department of Transportation (FDOT) that seeks to better understand the performance of composite steel girder bridges using shored construction. The answers provided will help determine the design policies, construction specifications and experiences of transportation agencies relevant to this structure type and construction method.

Shored construction for beam-slab bridges uses temporary supports (discretely or fully supported) for the superstructure during the installation of metal forms and the pouring of concrete. Shored construction can also be used in an Accelerated Bridge Construction (ABC) practice where construction occurs at a nearby site and fully-formed elements are transported to the final site using self-propelled modular transporters (SPMTs). A pre-decked steel girder that is shored during the pouring of the deck is another example that offers an advantage for ABC projects.

...

1. Have any slab on steel girder bridges been built in your state/country/region with a fully or discretely shored deck placement process?

   ○ Yes

   ○ No

2. Have any slab on concrete beam bridges been built in your state/country/region with a fully or discretely shored deck placement process?

   ○ Yes

   ○ No

3. Is shoring (fully or discretely) of a slab on steel girder during deck placement permitted in your agency?

   ○ Yes

   ○ No

4. If you answered yes to question 3, what types of construction have shored construction been approved for, e.g., concrete and/or steel beams; pre-decked girders and/or cast-in place deck; simple and/or continuous spans?

   Enter your answer

5. If you answered no to question 3, what is the driving factor in preventing your agency from permitting it (e.g. cost, concerns on composite action, long-term creep and shrinkage effects, redecking)?

   Enter your answer

6. Please provide contact information so we may follow up as necessary (i.e. name, affiliation, phone number and email address).

   Enter your answer

Submit

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Survey on Shored Construction-Follow up Questionnaire

This is a follow-up survey to collect more detailed information on shored construction. Thank you very much for your time and inputs!

...

General

1. What types of construction have shored construction been approved for, e.g., accelerated bridge construction, and/or cast-in place?

> Enter your answer

2. Please provide a list of bridges where shored construction was implemented with a) bridge number, b) girder material (concrete or steel), c) simple or continuous span.

> Enter your answer

3. What was the performance of these bridges?
   a. More/less cracks observed
   b. Loss of composite action
   c. More/less deflection
   d. Other measures?

> Enter your answer

4. Were there any engineering and/or construction issues on these bridges?

> Enter your answer

5. What is your department's major concern with shored construction?

> Enter your answer

Next

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Design

6. Did you make any modifications to the AASHTO Design Specification to accommodate for shored construction? If yes, what are the modifications you made?

   a. Was there a modification in the design of shear connectors to carry additional load from the composite section?
   b. How are time-dependent effects considered? Was there a modification to the modular ratio used?
   c. Others (Please specify your modifications)

   > Enter your answer

7. In your design, were you able to use a lighter steel beam for shored construction comparing to unshored construction?

   > Enter your answer

8. Are there any tensile stress limits for the concrete deck at the intermediate supports?

   > Enter your answer

9. Did you require a tight camber tolerance for shored construction?

   > Enter your answer

10. How were dead load deflections calculated?

   > Enter your answer

11. What computer tools (software) were used in the analysis?

   > Enter your answer

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...

Construction

13. What type of shoring was used on the shored construction project? 📑

Enter your answer

14. When was the shoring removed?

Enter your answer

15. Describe any construction issues that may have occurred.

Enter your answer

16. Do you feel that shored construction could be beneficial for accelerated bridge construction?

Enter your answer

Back   Submit

10 APPENDIX B: SUMMARY OF DESIGNED BRIDGES

Table 10-1 through Table 10-6 show the summary of bridges designed in this project.

Table 10-1. Single span bridges with rolled beam (8 ft spacing with 6 girders)

Span Length (ft)	Girder Spacing (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	8	W30x191	W30x173	W30x173	W30x148	23%
100	8	W36x231	W36x231	W36x231	W36x194	17%
120	8	W40x324	W40x297	W40x297	W40x249	23%

Table 10-2. Single span bridges with rolled beam (10 ft spacing with 5 girders)

Span Length (ft)	Girder Spacing (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	10	W30x235	W30x211	W30x211	W30x191	19%
100	10	W36x302	W36x282	W36x262	W36x247	19%
120	10	W40x372	W40x362	W40x362	W40x297	20%

Table 10-3. Single span bridges with plate girder (8 ft spacing with 6 girders)

Span Length (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	Top : 16x3/4" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x1"	Top: 16x1/2" Web: 32x1/2" Bottom: 16x7/8"	14%
100	Top : 16x3/4" Web: 40x1/2" Bottom: 16x1.25"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1.125"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1.125"	Top : 16x1/2" Web: 40x1/2" Bottom: 16x1"	15.4%
120	Top : 18x3/4" Web: 48x1/2" Bottom: 18x1.25"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 48x1/2" Bottom: 20x1"	10.1%
140	Top : 18x3/4" Web: 54x1/2" Bottom: 18x1.5"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.125"	Top : 18x1/2" Web: 54x1/2" Bottom: 20x1.25"	8.3%

160	Top : 18x3/4" Web: 60x1/2" Bottom: 18x1.75"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.625"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.625"	Top : 18x1/2" Web: 60x1/2" Bottom: 20x1.5"	8%
180	Top : 18x3/4" Web: 68x1/2" Bottom: 18x2"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.875"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.875"	Top : 18x1/2" Web: 68x1/2" Bottom: 18x1.75"	10.7%
200	Top : 20x3/4" Web: 74x1/2" Bottom: 20x2"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.875"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.875"	Top : 20x1/2" Web: 74x1/2" Bottom: 20x1.75"	11%

Table 10-4. Single span bridges with plate girder (10 ft spacing with 5 girders)

Span Length (ft)	Unshored	Shored at 1/3 points	Shored at 1/4 points	Fully Shored	% Diff. if fully shored
80	Top : 16x1" Web: 32x1/2" Bottom: 16x1.5"	Top: 16x3/4" Web: 32x1/2" Bottom: 16x1.25"	Top: 16x3/4" Web: 32x1/2" Bottom: 16x1.125"	Top: 16x3/4" Web: 32x1/2" Bottom: 16x1.125"	18%
100	Top : 18x1" Web: 40x1/2" Bottom: 18x2"	Top: 18x3/4" Web: 40x1/2" Bottom: 18x1.25"	Top: 18x3/4" Web: 40x1/2" Bottom: 18x1.25"	Top: 18x3/4" Web: 40x1/2" Bottom: 18x1.25"	19%
120	Top : 18x1" Web: 48x1/2" Bottom: 20x2"	Top : 18x3/4" Web: 48x1/2" Bottom: 20x1.375"	Top : 18x3/4" Web: 48x1/2" Bottom: 20x1.375"	Top : 18x3/4" Web: 48x1/2" Bottom: 20x1.25"	23%
140	Top : 20x1" Web: 54x1/2" Bottom: 20x2"	Top : 18x3/4" Web: 54x1/2" Bottom: 20x1.625"	Top : 18x3/4" Web: 54x1/2" Bottom: 20x1.625"	Top : 18x3/4" Web: 54x1/2" Bottom: 20x1.5"	19%
160	Top : 20x1" Web: 60x1/2" Bottom: 20x2"	Top : 18x3/4" Web: 60x1/2" Bottom: 20x2"	Top : 18x3/4" Web: 60x1/2" Bottom: 20x1.875"	Top : 18x3/4" Web: 60x1/2" Bottom: 20x1.75"	13%

| 180 | Top : 20x1"
Web: 68x1/2"
Bottom: 20x2.25" | Top : 18x3/4"
Web: 68x1/2"
Bottom: 20x2.25" | Top : 18x3/4"
Web: 68x1/2"
Bottom: 20x2.125" | Top : 18x3/4"
Web: 68x1/2"
Bottom: 20x2" | 11% |
| 200 | Top : 20x1.25"
Web: 74x1/2"
Bottom: 20x2.5" | Top : 18x3/4"
Web: 74x1/2"
Bottom: 20x2.625" | Top : 18x3/4"
Web: 74x1/2"
Bottom: 20x2.375" | Top : 18x3/4"
Web: 74x1/2"
Bottom: 20x2.25" | 15% |

Table 10-5. Three-span continuous bridges with plate girder (10'6" spacing with 5 girders)

Main Span Length (ft)	Region	Unshored	Shored at 1/3 points	Shored at 1/4 points	% Diff. if fully shored
180	M+	Top : 16x1" Web: 66x1/2" Bottom: 18x1.5"	Top : 16x3/4" Web: 66x1/2" Bottom: 18x1.25"	Top : 16x3/4" Web: 66x1/2" Bottom: 18x1.125"	12%
180	M-	Top : 18x1.5" Web: 66x1/2" Bottom: 20x2.5"	Top : 16x1.5" Web: 66x1/2" Bottom: 20x1.75"	Top : 16x1.375" Web: 66x1/2" Bottom: 20x1.75"	18%
225'	M+	Top : 16x1" Web: 76x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 74x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 74x0.5625" Bottom: 16x1.5"	7%
225'	M-	Top : 18x1.5" Web: 76x0.5625" Bottom: 20x2.5"	Top : 16x1.375" Web: 74x0.5625" Bottom: 20x1.75"	Top : 16x1.375" Web: 74x0.5625" Bottom: 20x1.75"	18%
255'	M+	Top : 16x1" Web: 83x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 81x0.5625" Bottom: 16x1.5"	Top : 16x3/4" Web: 81x0.5625" Bottom: 16x1.5"	7%
255'	M-	Top : 18x1.5" Web: 83x0.5625" Bottom: 20x2.5"	Top : 16x1.375" Web: 81x0.5625" Bottom: 20x1.75"	Top : 16x1.375" Web: 81x0.5625" Bottom: 20x1.75"	16%

Table 10-6. Three-span continuous bridges with plate girder (12 ft spacing with 5 girders)

Main Span Length (ft)	Region	Unshored	Shored at 1/3 points	Shored at 1/4 points	% Diff. if shored at ¼ points
180	M+	Top : 16x3/4" Web: 72x0.5625" Bottom: 20x1.125"	Top : 16x3/4" Web: 68x1/2" Bottom: 20x1.125"	Top : 16x3/4" Web: 68x1/2" Bottom: 20x1.125"	9%
180	M-	Top : 18x1.375" Web: 72x0.5626" Bottom: 20x1.75"	Top : 18x1.25" Web: 68x1/2" Bottom: 20x1.75"	Top : 18x1.25" Web: 68x1/2" Bottom: 20x1.75"	10%
225'	M+	Top : 16x3/4" Web: 74x0.5625" Bottom: 20x1.125"	Top : 16x3/4" Web: 74x1/2" Bottom: 20x1.125"	Top : 16x3/4" Web: 74x1/2" Bottom: 20x1.125"	6%
225'	M-	Top : 18x1.5" Web: 74x0.625" Bottom: 20x1.75"	Top : 18x1.25" Web: 74x0.5625" Bottom: 20x1.75"	Top : 18x1.25" Web: 74x0.5625" Bottom: 20x1.75"	9%
255'	M+	Top : 16x3/4" Web: 87x0.75" Bottom: 20x1.125"	Top : 16x3/4" Web: 87x0.625" Bottom: 20x1.125"	Top : 16x3/4" Web: 87x0.625" Bottom: 20x1.125"	11%
255'	M-	Top : 18x1.5" Web: 87x0.75" Bottom: 22x1.75"	Top : 18x1.25" Web: 87x0.625" Bottom: 22x1.75"	Top : 18x1.25" Web: 87x0.625" Bottom: 22x1.75"	12%

11 APPENDIX C: FE ANALYSIS RESULTS FOR SINGLE SPAN BRIDGES

C-1 80' SPAN WITH UNSHORED CONDITION

This section summarizes the FE analysis results for an 80 ft single span steel composite girder bridge with unshored condition.
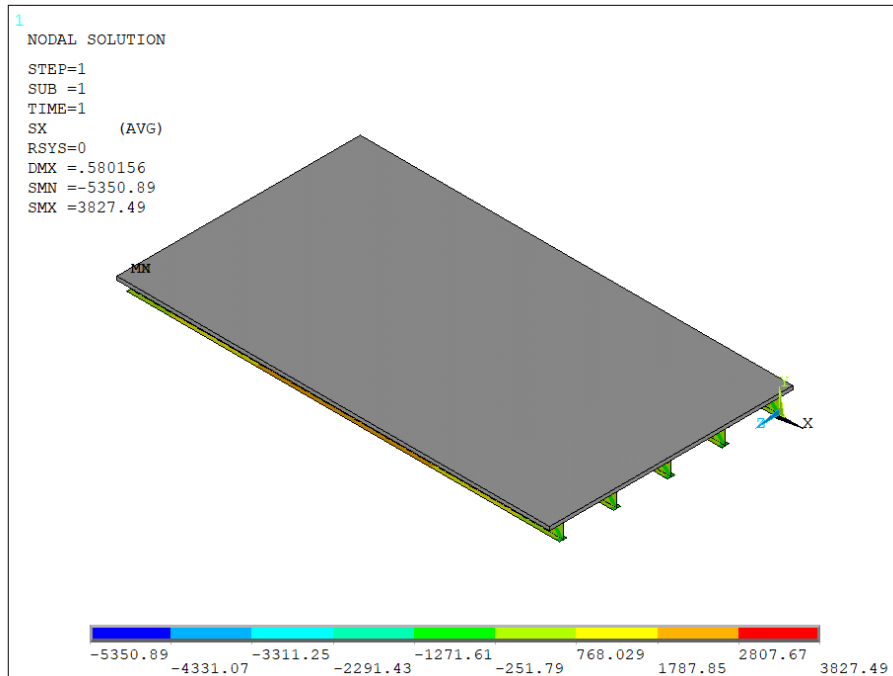
Figure 11-1. Contour map for stress along longitudinal direction (girder selfweight only, an 80 ft single span plate girder bridge with 10' Spacing, five-girder system, unshored condition)
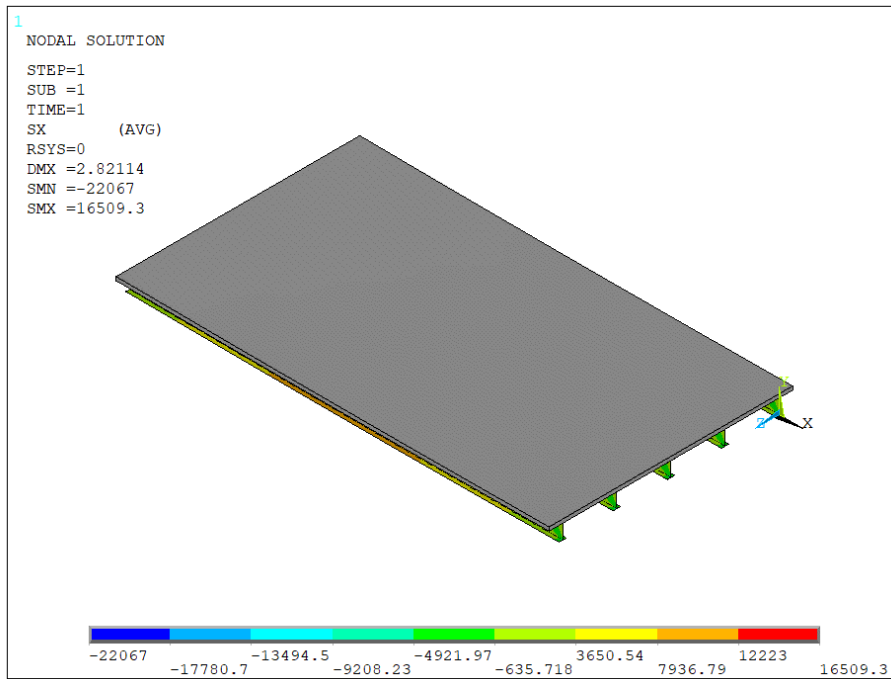
Figure 11-2. Contour map for stress along longitudinal direction (wet concrete loa, an 80 ft single span plate girder bridge with 10' Spacing, five-girder system, unshored condition)
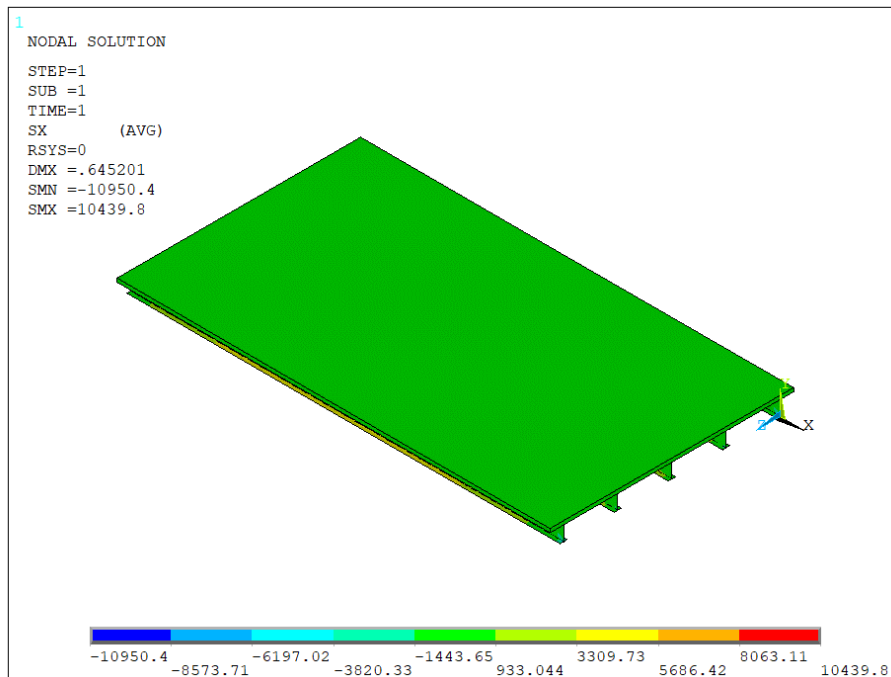
Figure 11-3. Contour map for stress along longitudinal direction (DC2 load, an 80 ft single span plate girder bridge with 10' Spacing, five-girder system, unshored condition)

Table 11-1. Results for girder 1 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.405	-2.953	0.000
2	Wet concrete	steel section	10.264	-12.570	0.000
3	DC2	composite section	5.304	-0.448	-0.206
4	Creep(365 days)	composite section	1.390	-6.268	0.374
5	Shrinkage (365 days)	composite section	1.374	-8.567	0.148
6	Total		20.737	-30.805	0.316

Table 11-2. Results for girder 2 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.448	-3.001	0.000
2	Wet concrete	steel section	11.550	-14.196	0.000
3	DC2	composite section	4.732	-0.451	-0.186
4	Creep(365 days)	composite section	1.327	-5.864	0.374
5	Shrinkage (365 days)	composite section	1.860	-8.625	0.127
6	Total		21.916	-32.138	0.315

Table 11-3. Results for girder 3 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.188	-3.178	0.000
2	Wet concrete	steel section	12.030	-14.756	0.000
3	DC2	composite section	4.473	-0.445	-0.177
4	Creep(365 days)	composite section	1.294	-5.771	0.381
5	Shrinkage (365 days)	composite section	1.868	-8.693	0.111
6	Total		21.853	-32.843	0.315

Table 11-4. Results for girder 4 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.461	-3.019	0.000
2	Wet concrete	steel section	11.396	-13.989	0.000
3	DC2	composite section	4.686	-0.464	-0.187
4	Creep(365 days)	composite section	1.411	-5.820	0.382
5	Shrinkage (365 days)	composite section	1.481	-8.689	0.112
6	Total		21.434	-31.980	0.307

Table 11-5. Results for girder 5 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.398	-2.939	0.000
2	Wet concrete	steel section	10.047	-12.263	0.000
3	DC2	composite section	5.259	-0.466	-0.208
4	Creep(365 days)	composite section	1.565	-6.103	0.380
5	Shrinkage (365 days)	composite section	0.678	-8.699	0.117
6	Total		19.946	-30.470	0.288

Figure 11-4. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-5. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

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Figure 11-6. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-7. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

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Figure 11-8. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-9. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-10. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-11. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-12. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
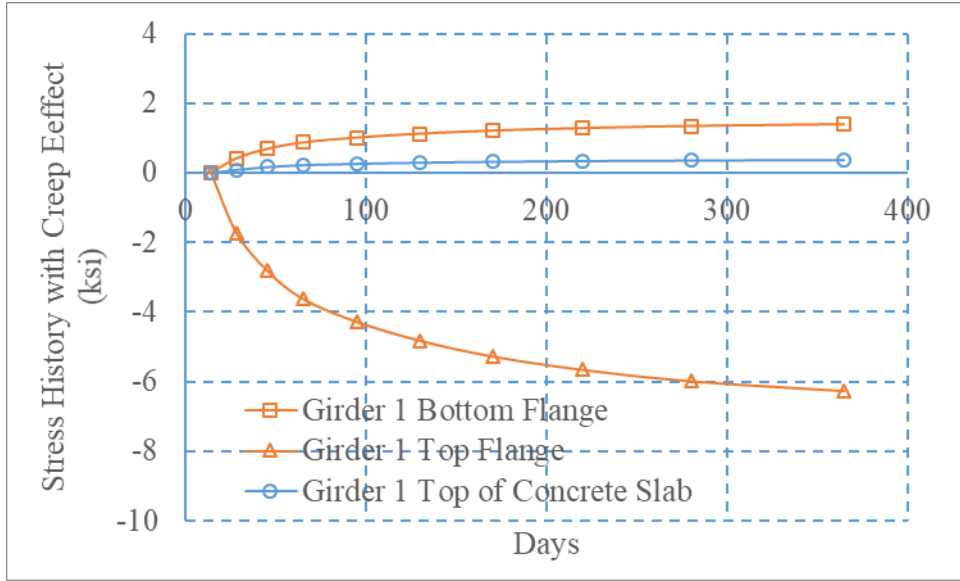

Figure 11-13. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

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C-2 80' SPAN WITH SHORING AT 1/3 POINTS

This section summarizes the FE analysis results for an 80 ft single span steel composite girder bridge with shoring at 1/3 points.

Table 11-6. Results for girder 1 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.060	-0.073	0.000
2	Wet concrete	steel section	0.239	-0.255	0.000
3	DC2	composite section	0.151	-0.016	-0.012
4	Creep(365 days)*	composite section	17.365	-6.123	-0.413
5	Shrinkage (365 days)	composite section	0.723	-4.507	0.078
6	Total		18.539	-10.973	-0.347

*including stress prior to creep taking effect.

Table 11-7. Results for girder 2 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.065	-0.076	0.000
2	Wet concrete	steel section	0.309	-0.340	0.000
3	DC2	composite section	0.096	-0.007	-0.004
4	Creep(365 days)	composite section	17.365	-6.123	-0.399
5	Shrinkage (365 days)	composite section	0.978	-4.538	0.067
6	Total		18.814	-11.083	-0.336

Table 11-8. Results for girder 3 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.054	-0.090	0.000
2	Wet concrete	steel section	0.306	-0.342	0.000
3	DC2	composite section	0.085	-0.002	-0.003
4	Creep(365 days)	composite section	17.536	-5.545	-0.393
5	Shrinkage (365 days)	composite section	0.983	-4.573	0.058
6	Total		18.964	-10.552	-0.338

Table 11-9. Results for girder 4 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.068	-0.076	0.000
2	Wet concrete	steel section	0.298	-0.342	0.000
3	DC2	composite section	0.106	-0.007	-0.004
4	Creep(365 days)	composite section	17.555	-5.621	-0.395
5	Shrinkage (365 days)	composite section	1.481	-8.689	0.112
6	Total		19.507	-14.735	-0.287

Table 11-10. Results for girder 5 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.061	-0.066	0.000
2	Wet concrete	steel section	0.218	-0.258	0.000
3	DC2	composite section	0.177	-0.011	-0.012
4	Creep(365 days)	composite section	17.538	-5.959	-0.408
5	Shrinkage (365 days)	composite section	0.678	-8.699	0.117
6	Total		18.672	-14.992	-0.303

Figure 11-14. Time history for stress along longitudinal lirection for girder #1 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
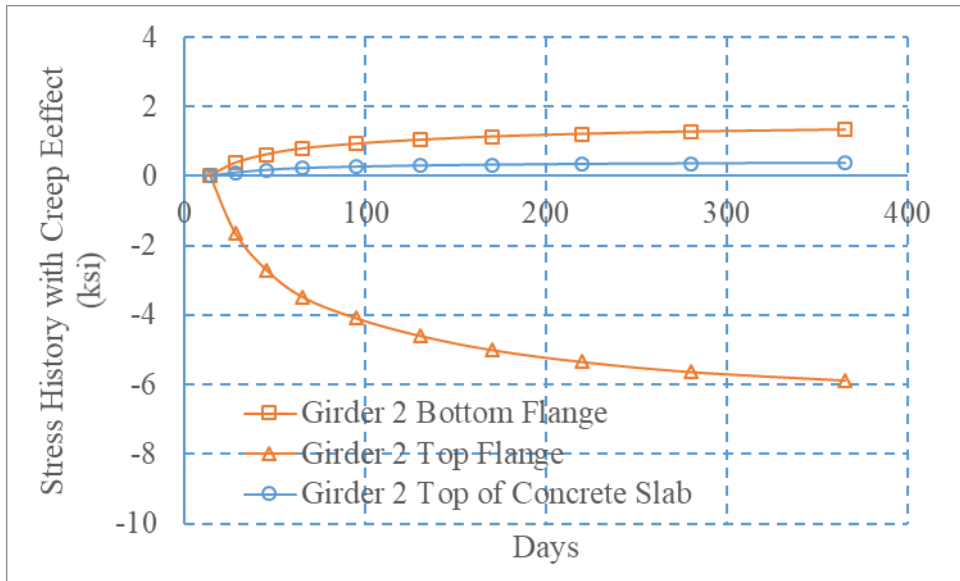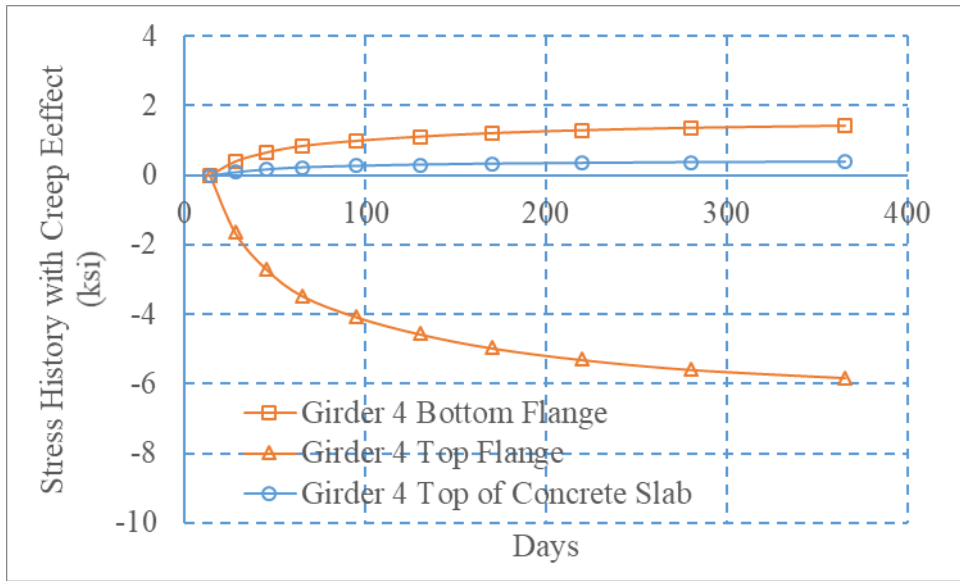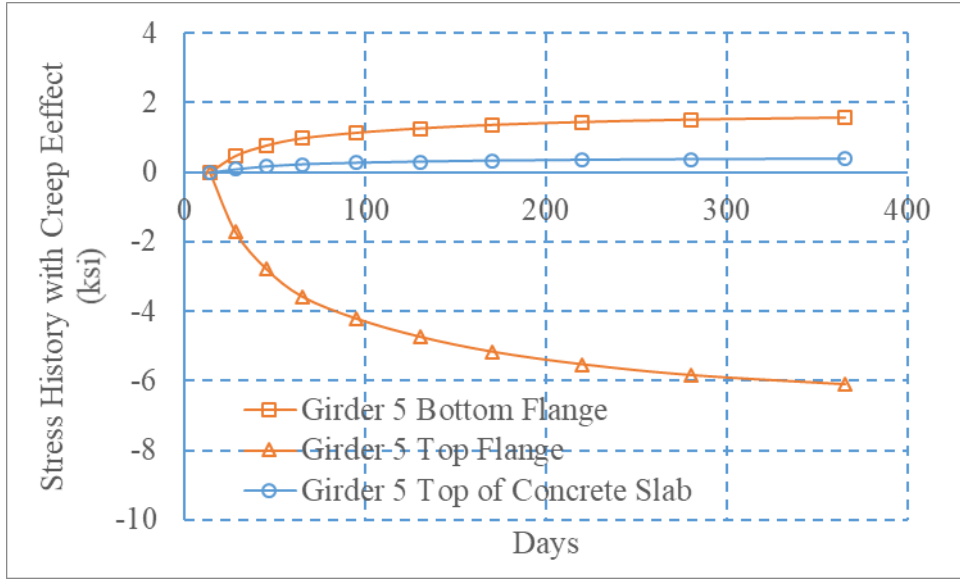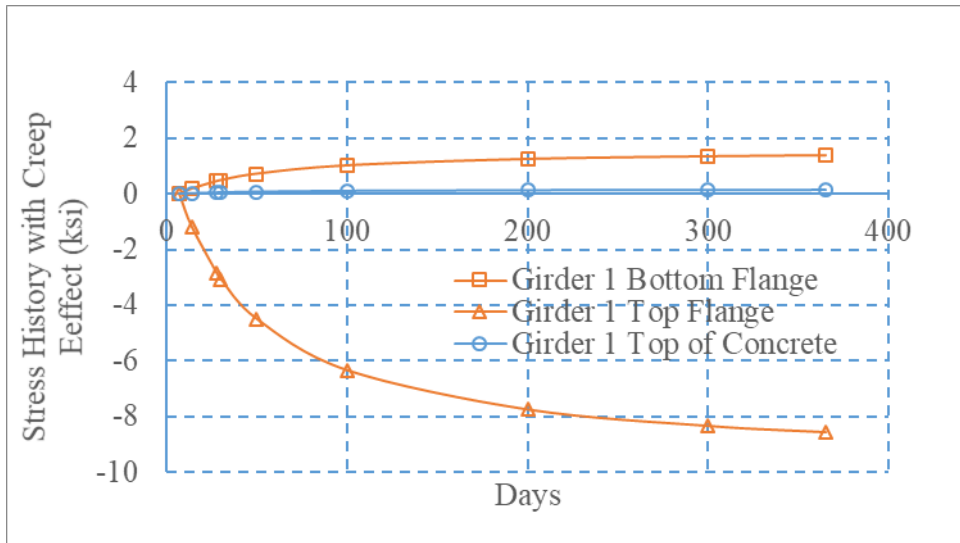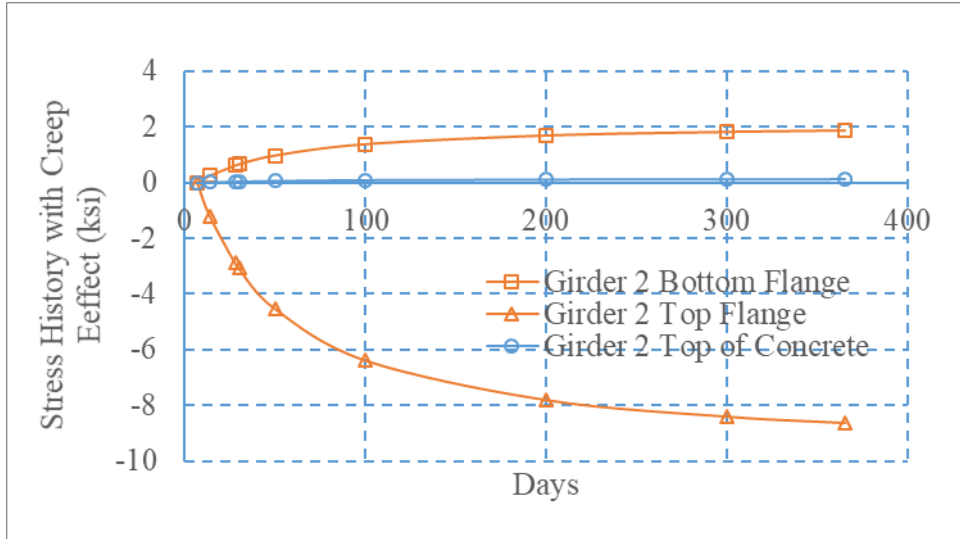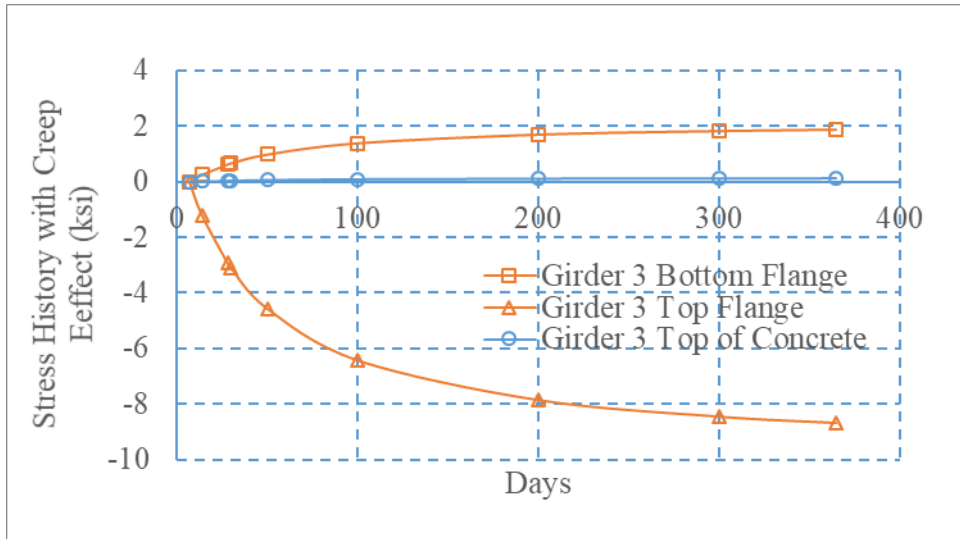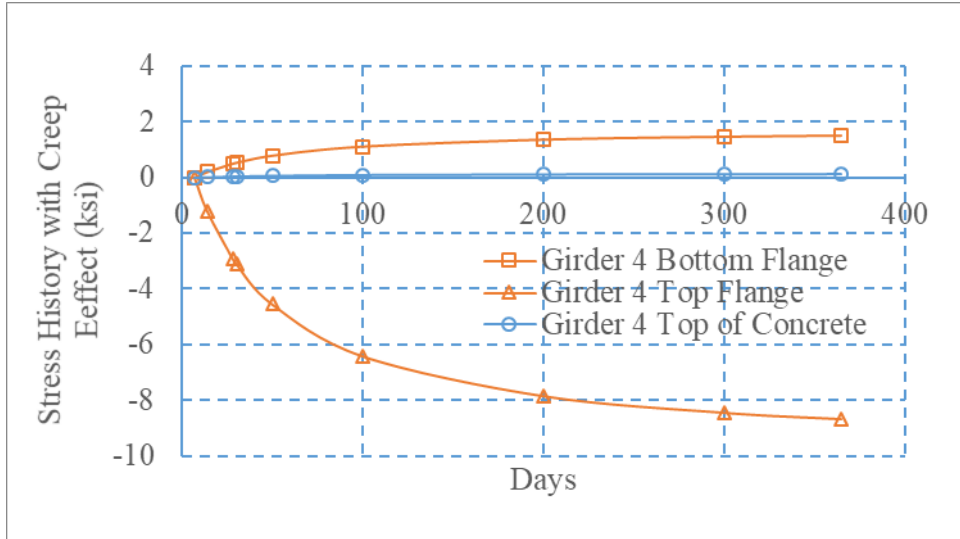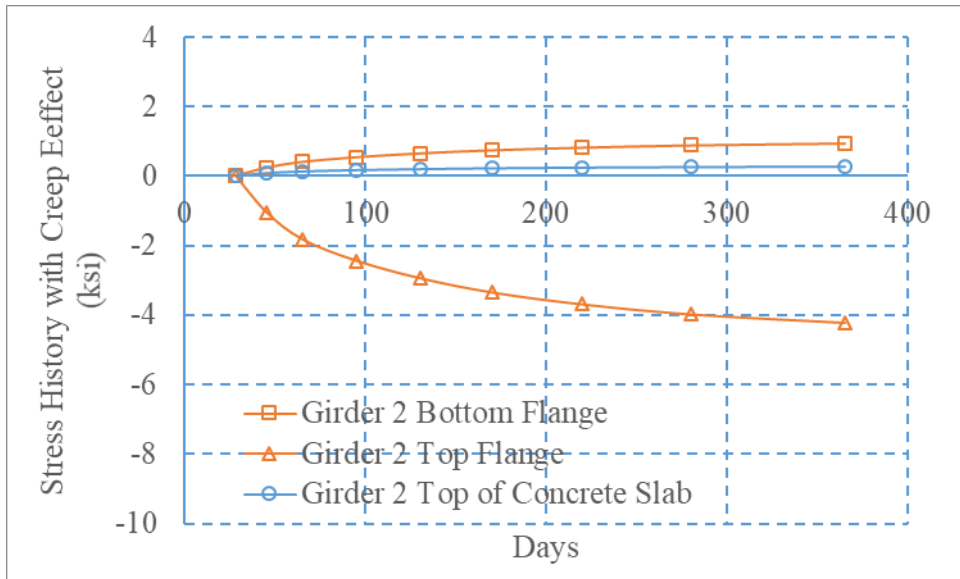

Figure 11-15. Time history for stress along longitudinal lirection for girder #2 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-16. Time history for stress along longitudinal lirection for girder #3 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
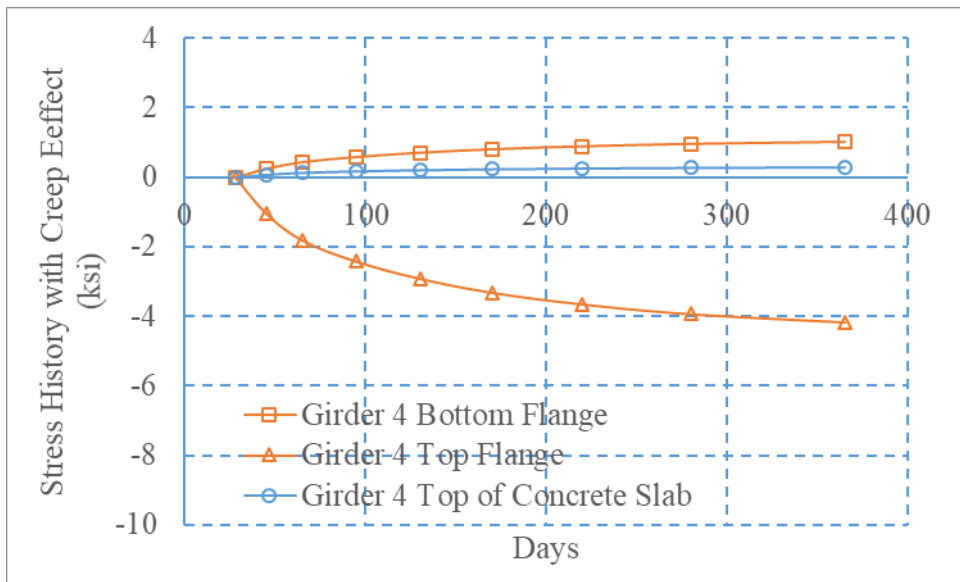
Figure 11-17. Time history for stress along longitudinal lirection for girder #4 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
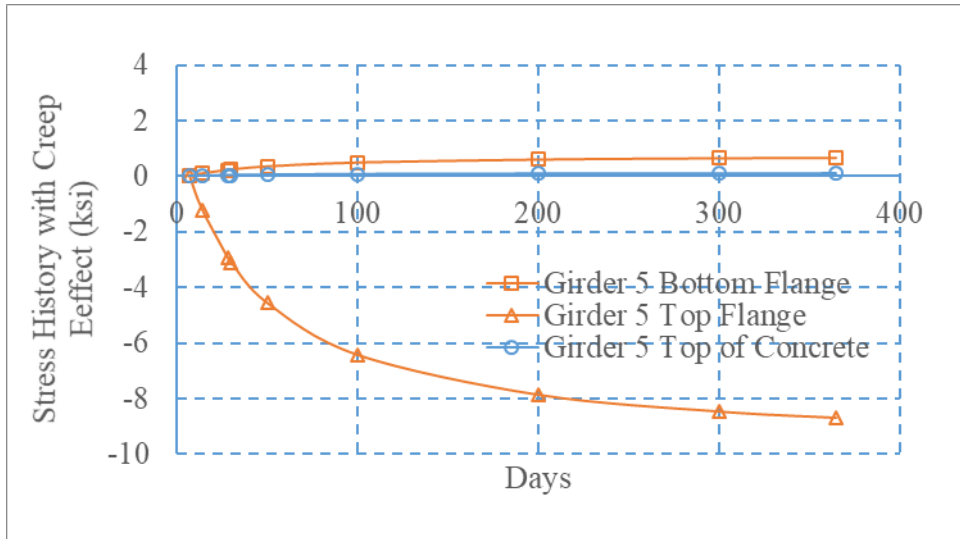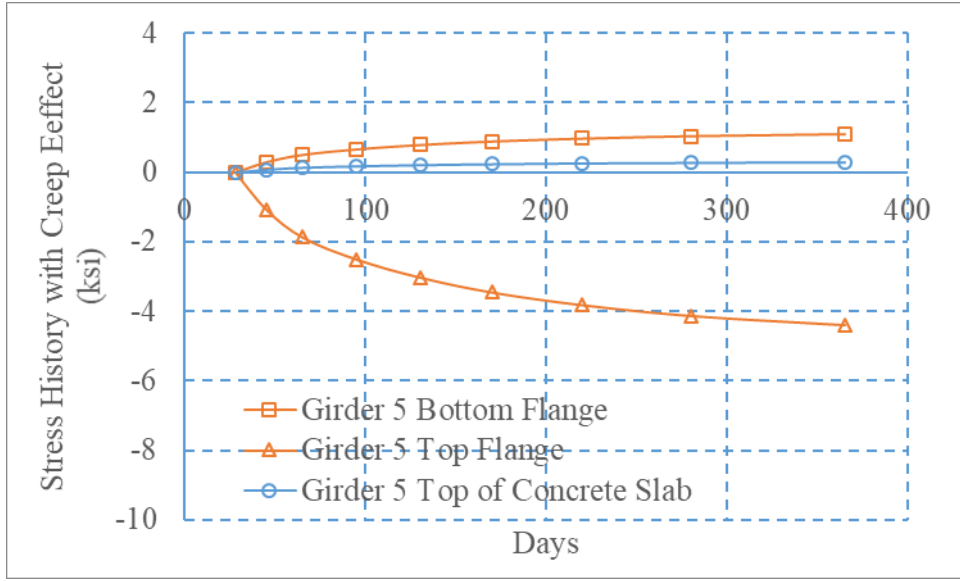
Figure 11-18. Time history for stress along longitudinal lirection for girder #5 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-19. Time history for stress along longitudinal lirection for girder #1 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

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Figure 11-20. Time history for stress along longitudinal lirection for girder #2 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-21. Time history for stress along longitudinal lirection for girder #3 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-22. Time history for stress along longitudinal lirection for girder #4 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-23. Time history for stress along longitudinal lirection for girder #5 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

C-3 80' SPAN WITH FULLY SHORED CONDITION

This section summarizes the FE analysis results for an 80 ft single span steel composite girder bridge with fully shored condition.

Table 11-11. Results for girder 1 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.151	-0.016	-0.012
4	Creep(365 days)	composite section	17.365	-6.123	-0.413
5	Shrinkage (365 days)	composite section	0.723	-4.507	0.078
6	Total		18.239	-10.646	-0.347

Table 11-12. Results for girder 2 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.096	-0.007	-0.004
4	Creep(365 days)	composite section	17.365	-6.123	-0.399
5	Shrinkage (365 days)	composite section	0.978	-4.538	0.067
6	Total		18.439	-10.667	-0.336

Table 11-13. Results for girder 3 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.085	-0.002	-0.003
4	Creep(365 days)	composite section	17.536	-5.545	-0.393
5	Shrinkage (365 days)	composite section	0.983	-4.573	0.058
6	Total		18.604	-10.120	-0.338

Table 11-14. Results for girder 4 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.106	-0.007	-0.004
4	Creep(365 days)	composite section	17.555	-5.621	-0.395
5	Shrinkage (365 days)	composite section	1.481	-8.689	0.112
6	Total		19.141	-14.316	-0.287

Table 11-15. Results for girder 5 (an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.177	-0.011	-0.012
4	Creep(365 days)	composite section	17.538	-5.959	-0.408
5	Shrinkage (365 days)	composite section	0.678	-8.699	0.117
6	Total		18.392	-14.669	-0.303

Figure 11-24. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
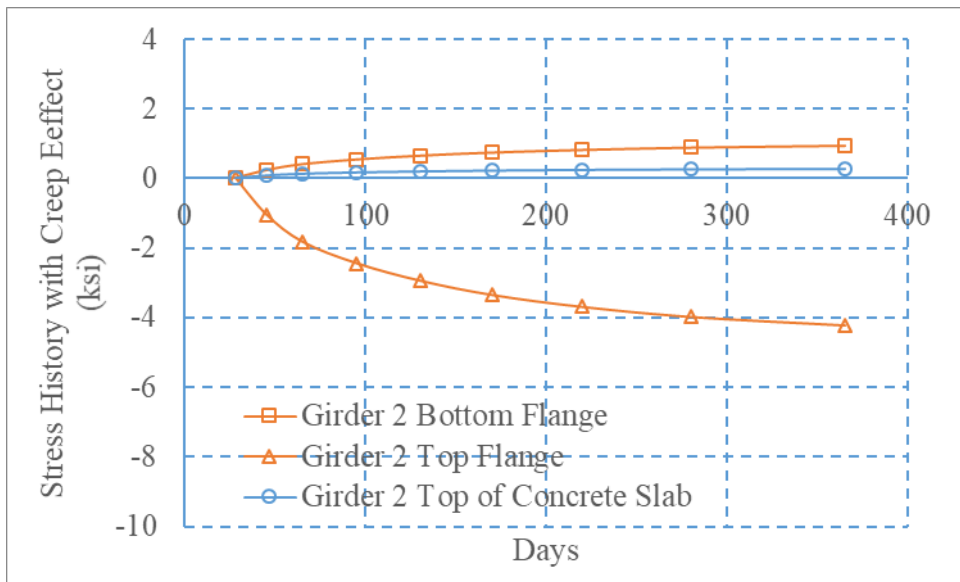
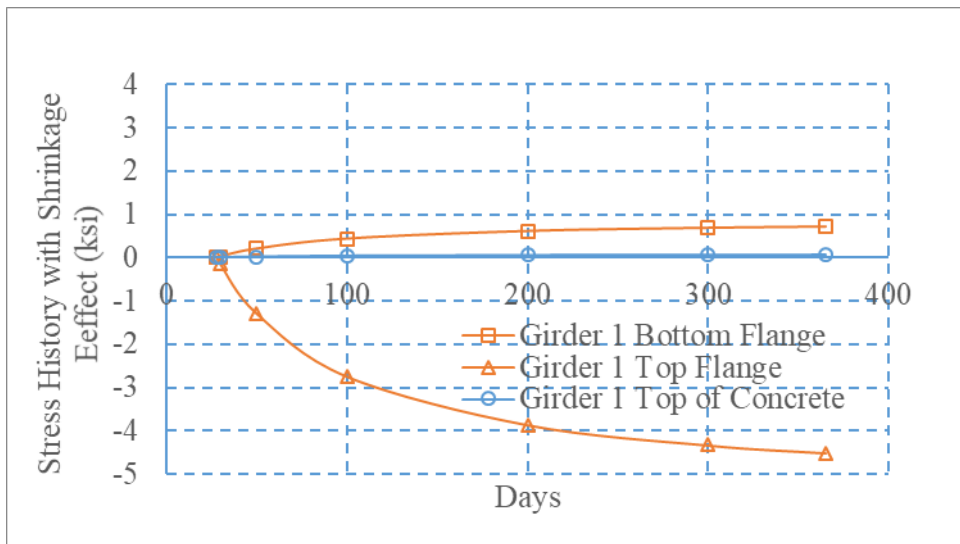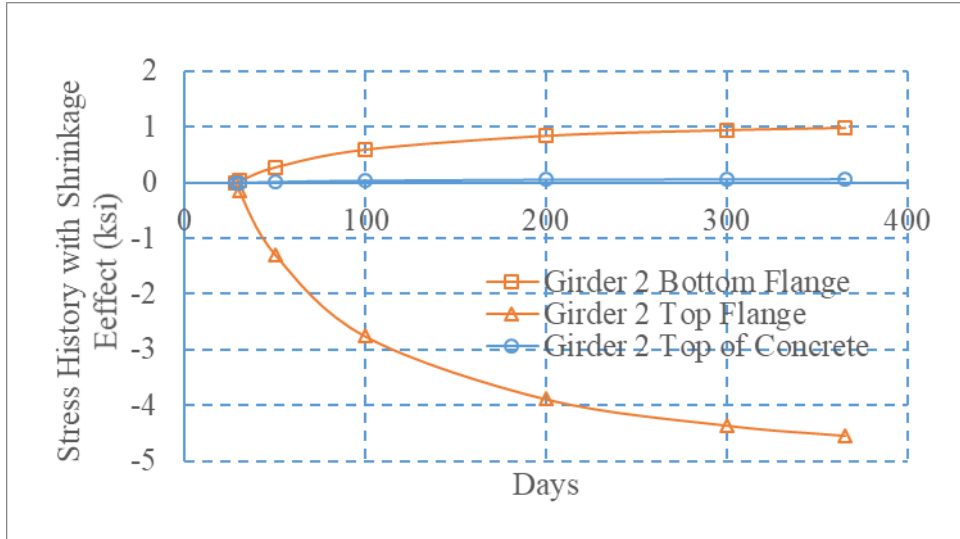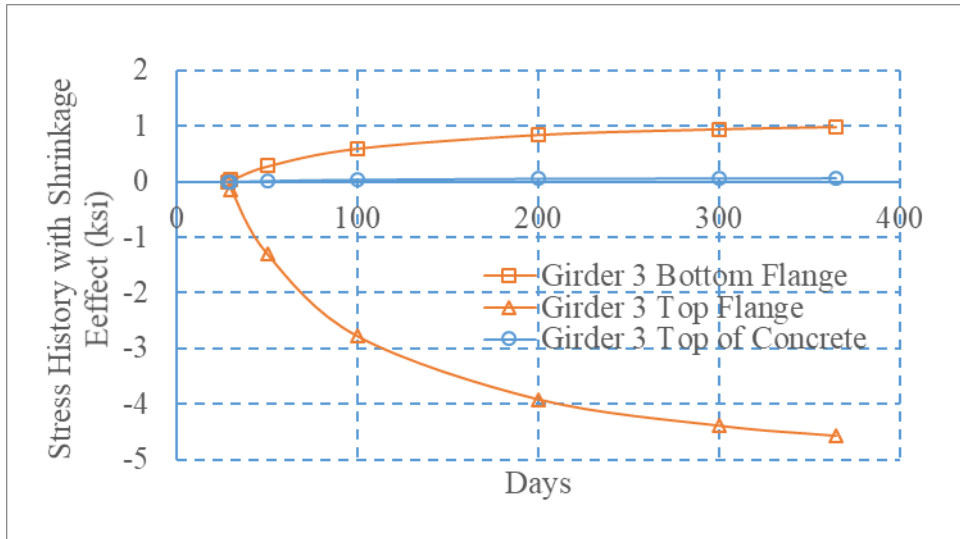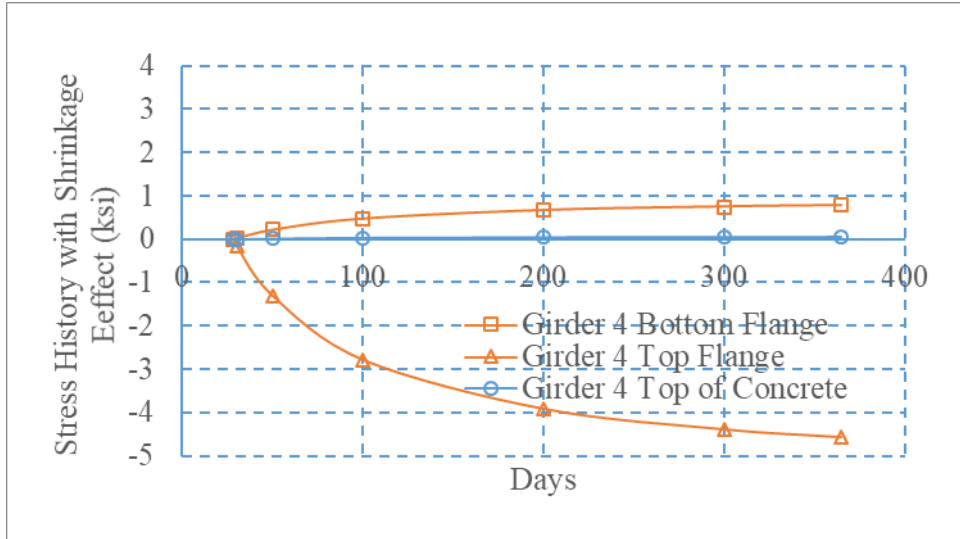Figure 11-25. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

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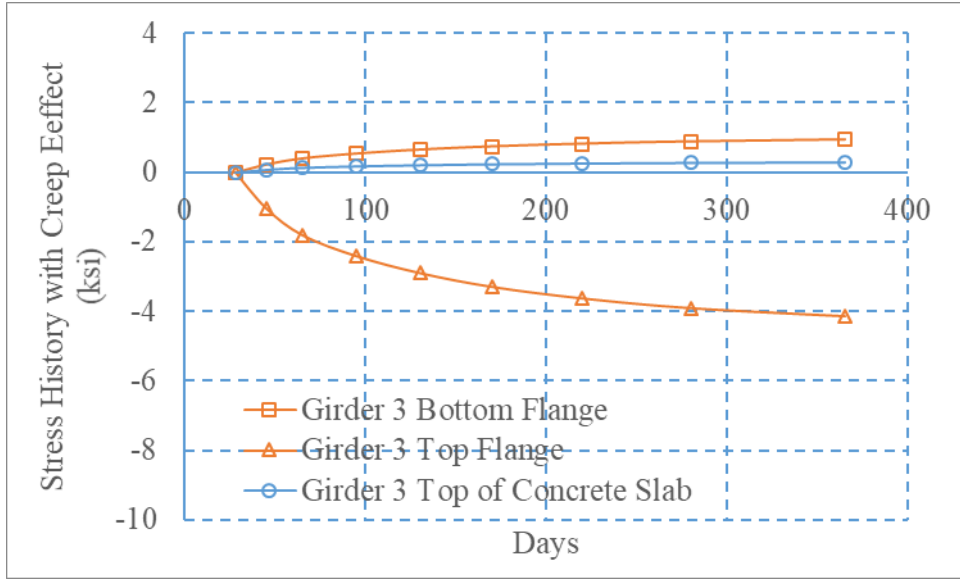
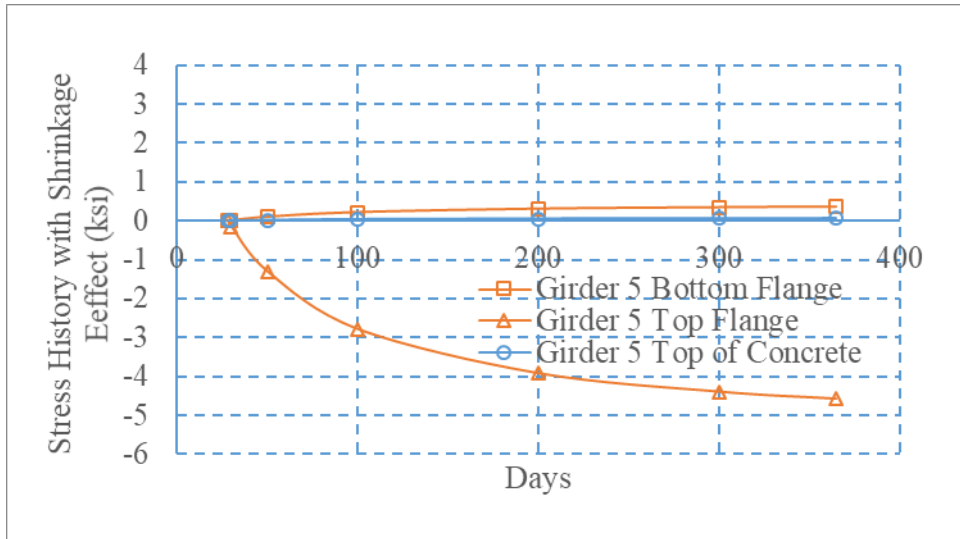Figure 11-26. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
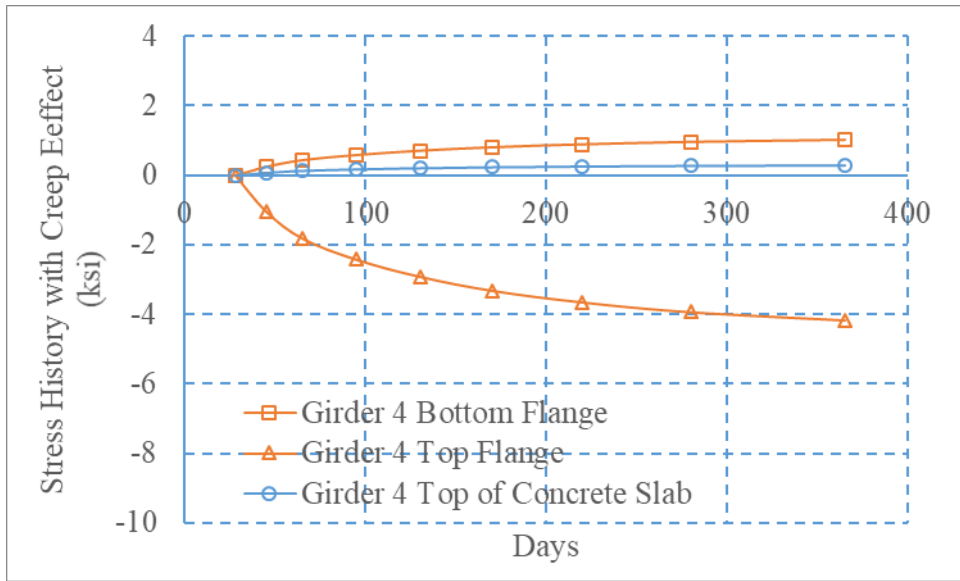
Figure 11-27. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
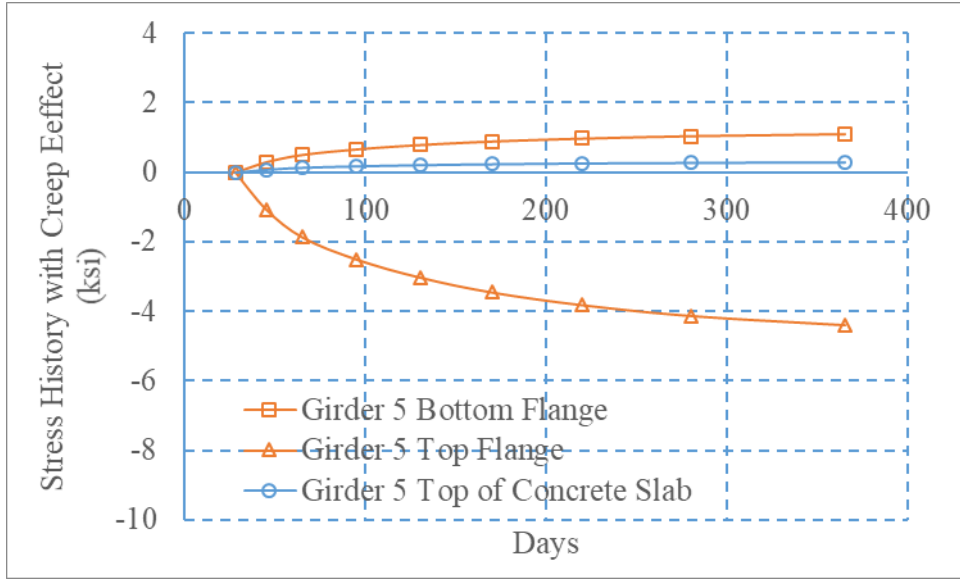
Figure 11-28. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-29. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-30. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-31. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-32. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
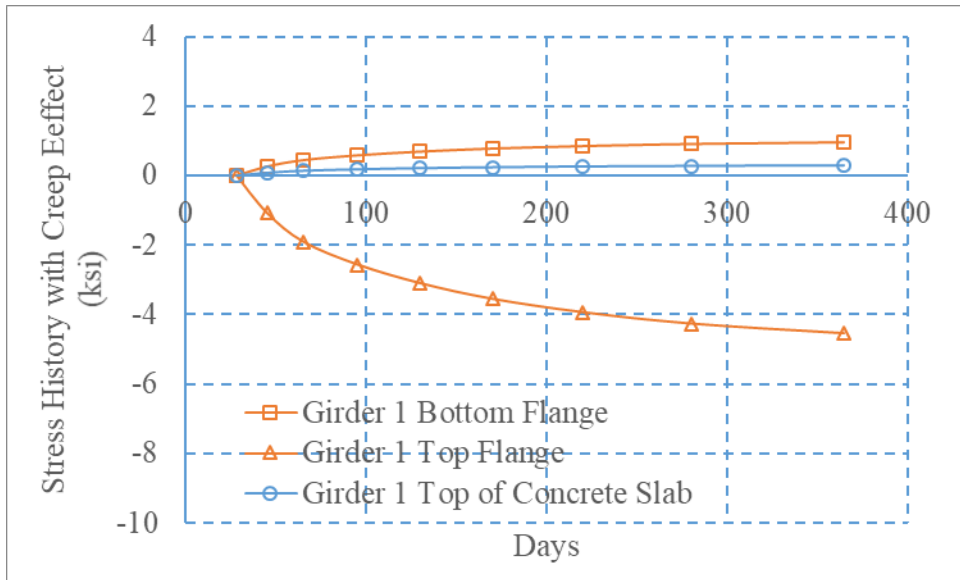

Figure 11-33. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, an 80 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

C-4 100' SPAN WITH UNSHORED CONDITION

This section summarizes the FE analysis results for a 100 ft single span steel composite girder bridge with unshored condition.

Table 11-16. Results for girder 1 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.1957	-4.1329	0
2	Wet concrete	steel section	15.193	-19.711	0
3	DC2	composite section	7.2531	-0.73394	-0.25733
4	Creep(365 days)	composite section	1.873	-7.8648	0.44798
5	Shrinkage (365 days)	composite section	1.2374	-9.2146	0.12066
6	Total		28.7522	-41.65724	0.31131

Table 11-17. Results for girder 2 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.2218	-4.1665	0
2	Wet concrete	steel section	16.176	-20.945	0
3	DC2	composite section	6.6636	-0.77789	-0.24101
4	Creep(365 days)	composite section	1.782	-7.5972	0.44798
5	Shrinkage (365 days)	composite section	1.8416	-9.1871	0.11847
6	Total		29.685	-42.67369	0.32544

Table 11-18. Results for girder 3 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	2.9903	-4.3043	0
2	Wet concrete	steel section	16.587	-21.401	0
3	DC2	composite section	6.4397	-0.786	-0.23389
4	Creep(365 days)	composite section	1.729	-7.5358	0.45468
5	Shrinkage (365 days)	composite section	2.0395	-9.199	0.11462
6	Total		29.7855	-43.2261	0.33541

Table 11-19. Results for girder 4 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.2302	-4.1793	0
2	Wet concrete	steel section	16.123	-20.729	0
3	DC2	composite section	6.6996	-0.7796	-0.23928
4	Creep(365 days)	composite section	1.772	-7.6401	0.45253
5	Shrinkage (365 days)	composite section	1.814	-9.1839	0.11757
6	Total		29.6388	-42.5119	0.33082

Table 11-20. Results for girder 5 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.1918	-4.1278	0
2	Wet concrete	steel section	15.115	-19.436	0
3	DC2	composite section	7.3146	-0.73279	
4	Creep(365 days)	composite section	1.849	-7.9564	0.44373
5	Shrinkage (365 days)	composite section	1.1969	-9.1998	0.12185
6	Total		28.6673	-41.45279	0.56558

Figure 11-34. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
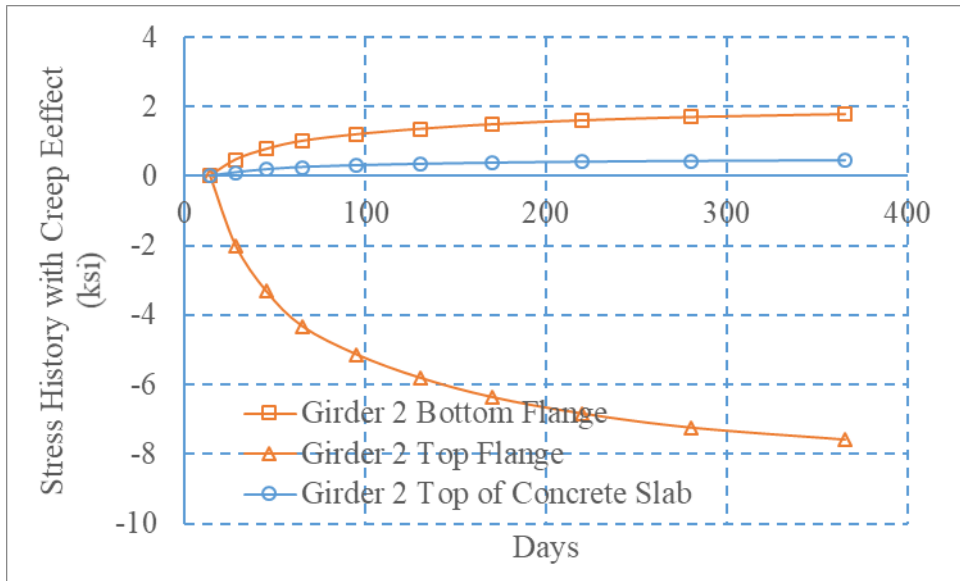
Figure 11-35. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
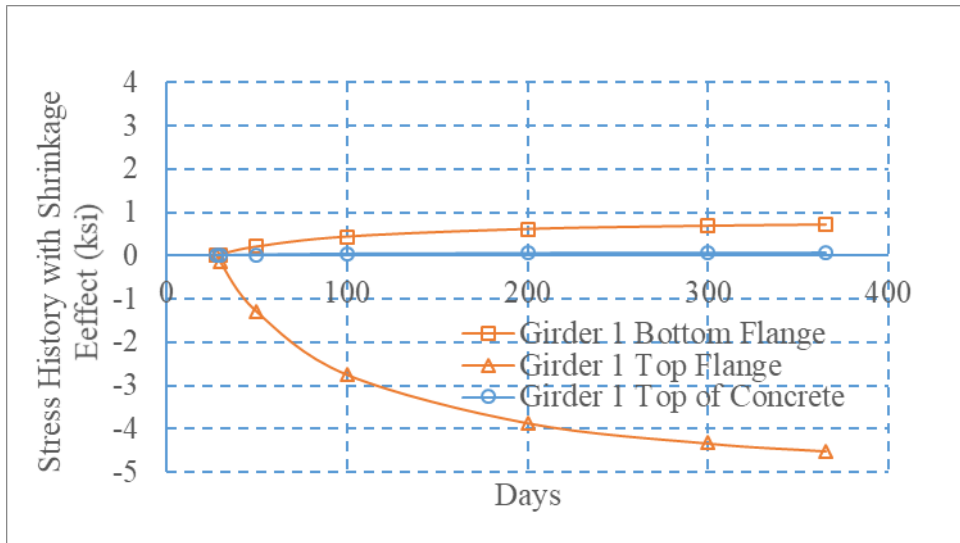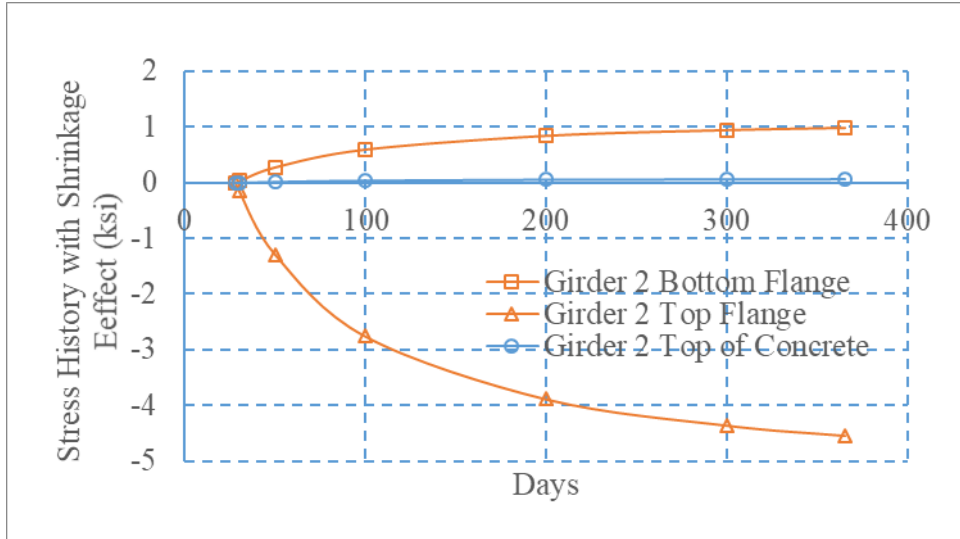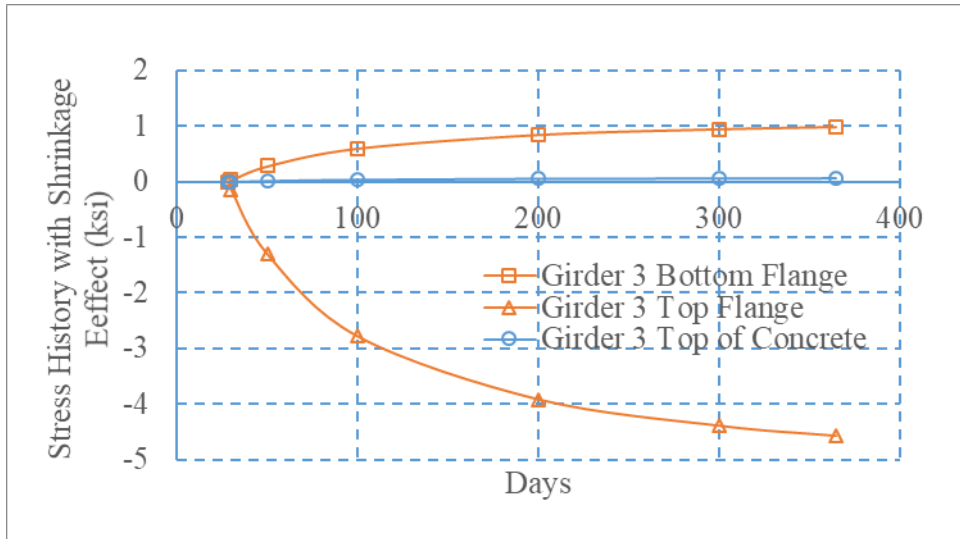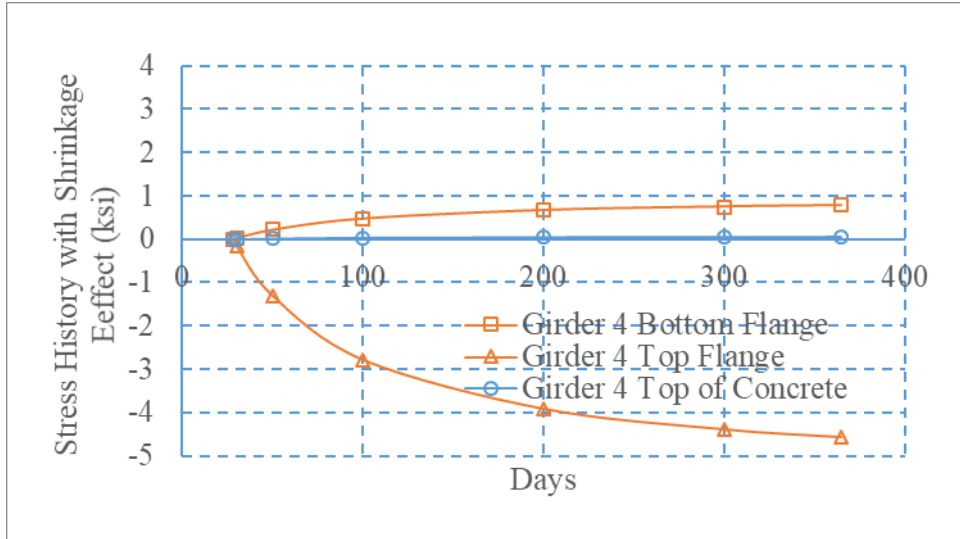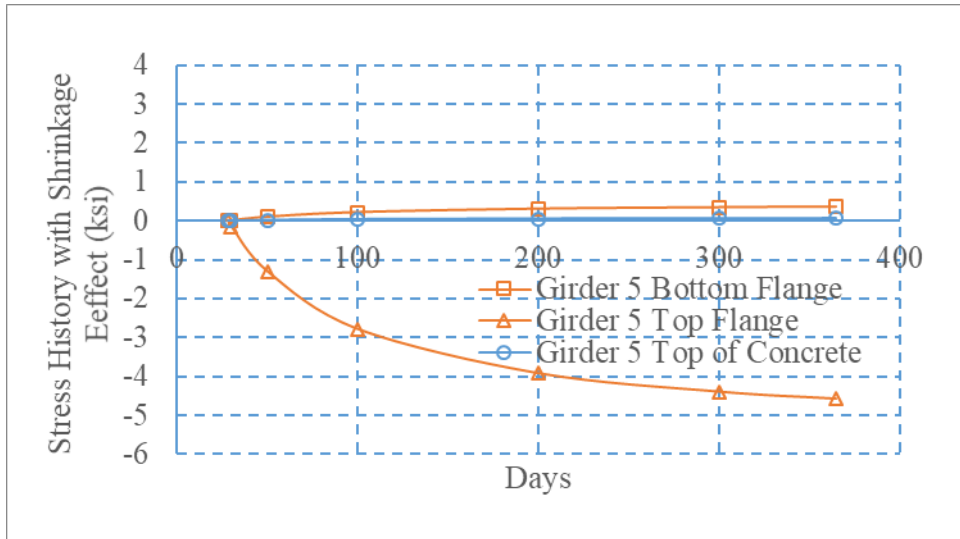
170
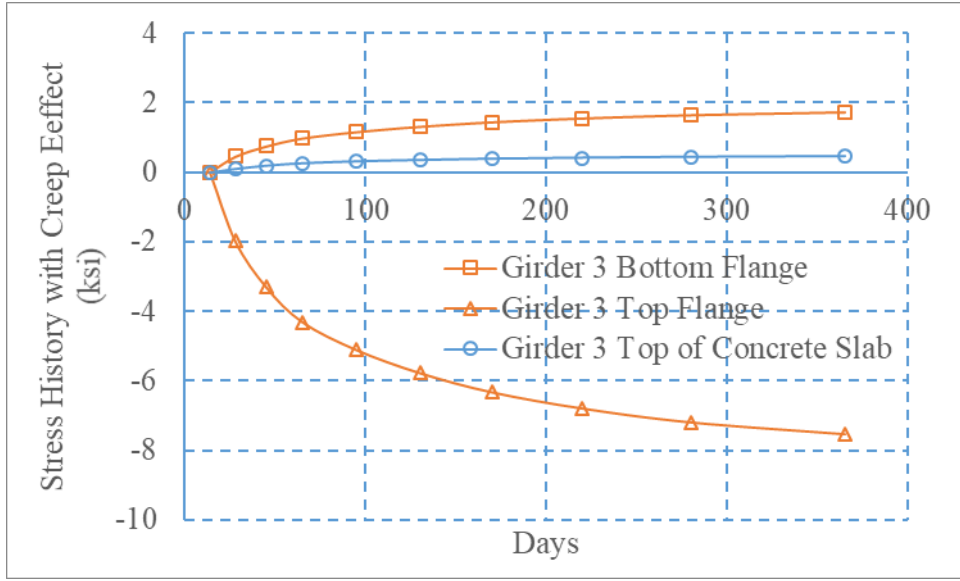
Figure 11-36. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
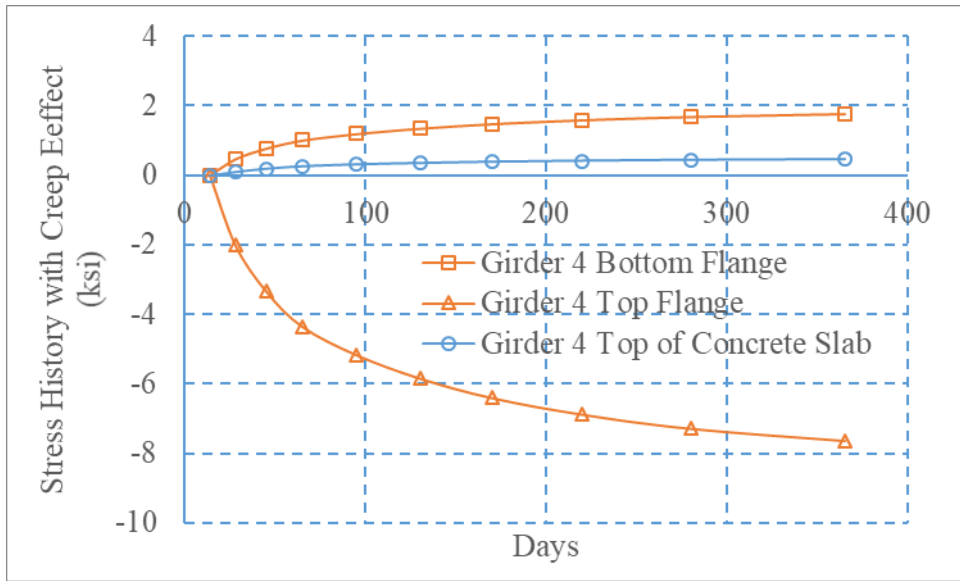
Figure 11-37. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
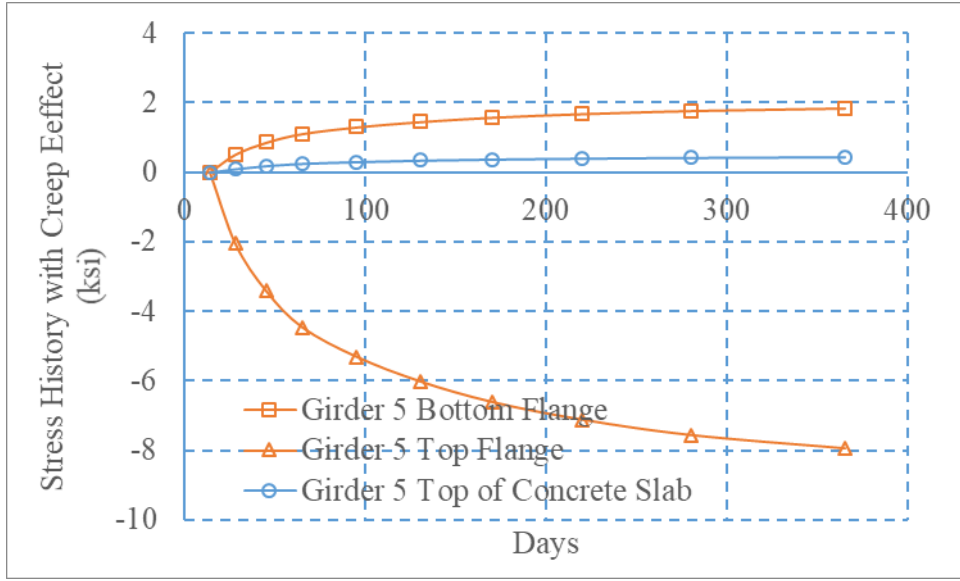
Figure 11-38. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-39. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

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Figure 11-40. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-41. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-42. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
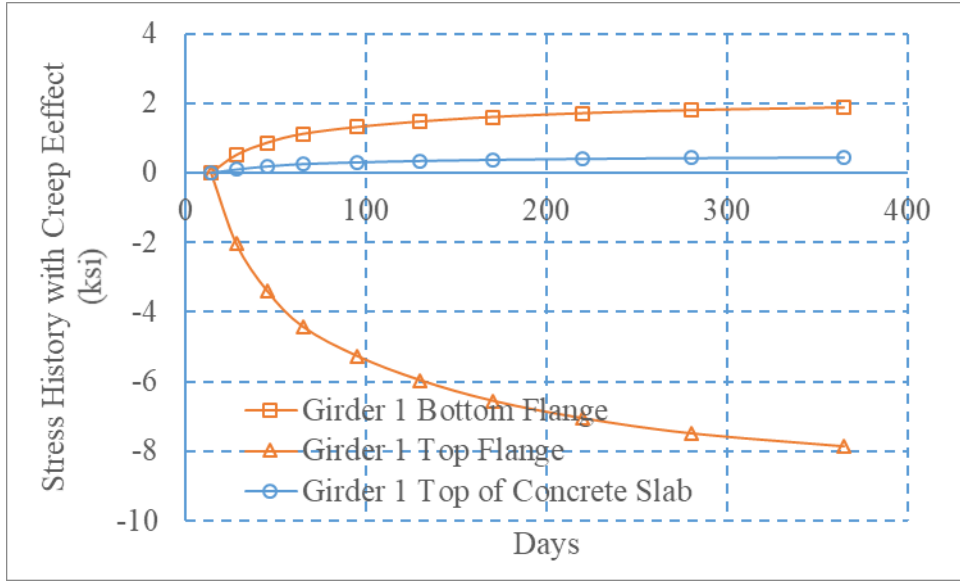

Figure 11-43. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

C-5 100' SPAN WITH SHORING AT 1/3 POINTS

This section summarizes the FE analysis results for a 100 ft single span steel composite girder bridge with shoring at 1/3 points.

Table 11-21. Results for girder 1 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.075	-0.096	0.000
2	Wet concrete	steel section	0.318	-0.382	0.000
3	DC2	composite section	0.222	-0.033	-0.014
4	Creep(365 days)	composite section	24.285	-8.376	-0.522
5	Shrinkage (365 days)	composite section	0.651	-4.848	0.063
6	Total		25.551	-13.735	-0.472

Table 11-22. Results for girder 2 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.079	-0.103	0.000
2	Wet concrete	steel section	0.424	-0.515	0.000
3	DC2	composite section	0.132	-0.017	-0.004
4	Creep(365 days)	composite section	24.285	-8.376	-0.513
5	Shrinkage (365 days)	composite section	0.969	-4.833	0.062
6	Total		25.889	-13.844	-0.455

Table 11-23. Results for girder 3 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.072	-0.107	0.000
2	Wet concrete	steel section	0.421	-0.515	0.000
3	DC2	composite section	0.112	-0.008	-0.003
4	Creep(365 days)	composite section	24.420	-7.989	-0.508
5	Shrinkage (365 days)	composite section	1.073	-4.840	0.060
6	Total		26.097	-13.458	-0.451

Table 11-24. Results for girder 4 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.079	-0.103	0.000
2	Wet concrete	steel section	0.421	-0.519	0.000
3	DC2	composite section	0.140	-0.016	-0.004
4	Creep(365 days)	composite section	24.360	-8.106	-0.512
5	Shrinkage (365 days)	composite section	1.814	-9.184	0.118
6	Total		26.815	-17.928	-0.398

Table 11-25. Results for girder 5 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.074	-0.095	0.000
2	Wet concrete	steel section	0.311	-0.388	0.000
3	DC2	composite section	0.234	-0.027	-0.013
4	Creep(365 days)	composite section	24.245	-8.450	-0.526
5	Shrinkage (365 days)	composite section	1.197	-9.200	0.122
6	Total		26.061	-18.160	-0.417

Figure 11-44. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
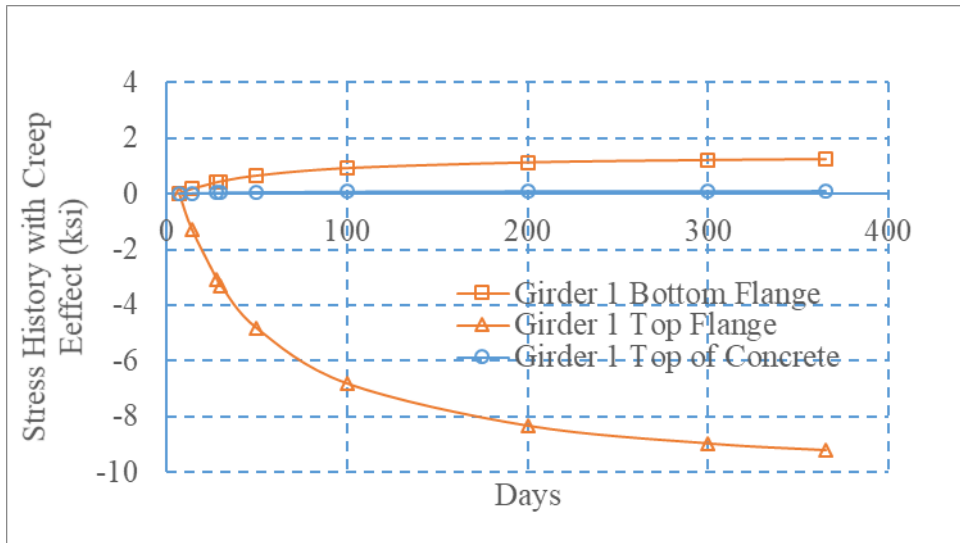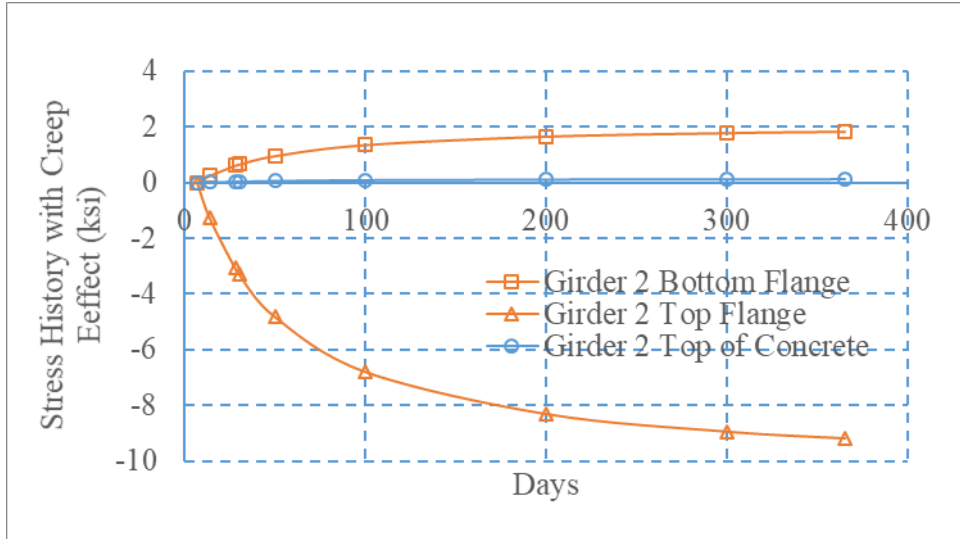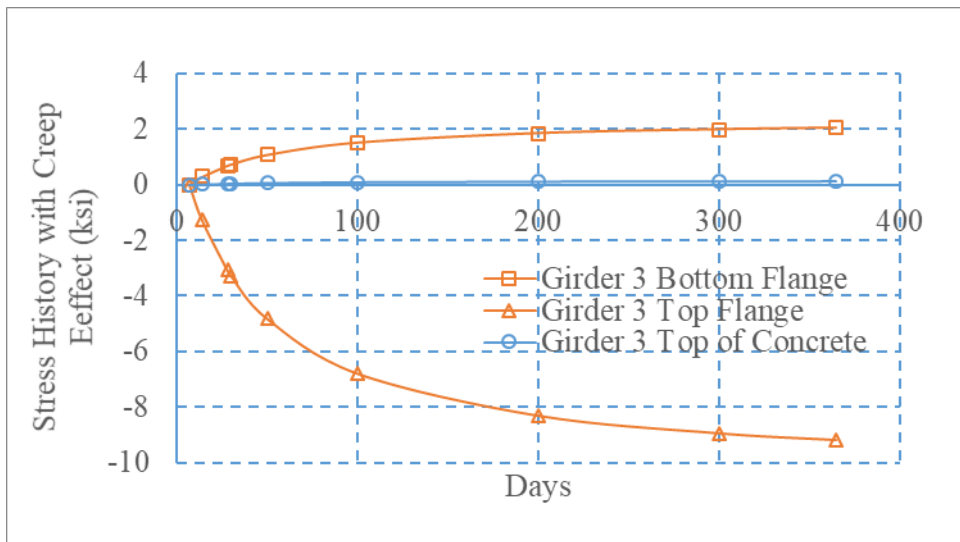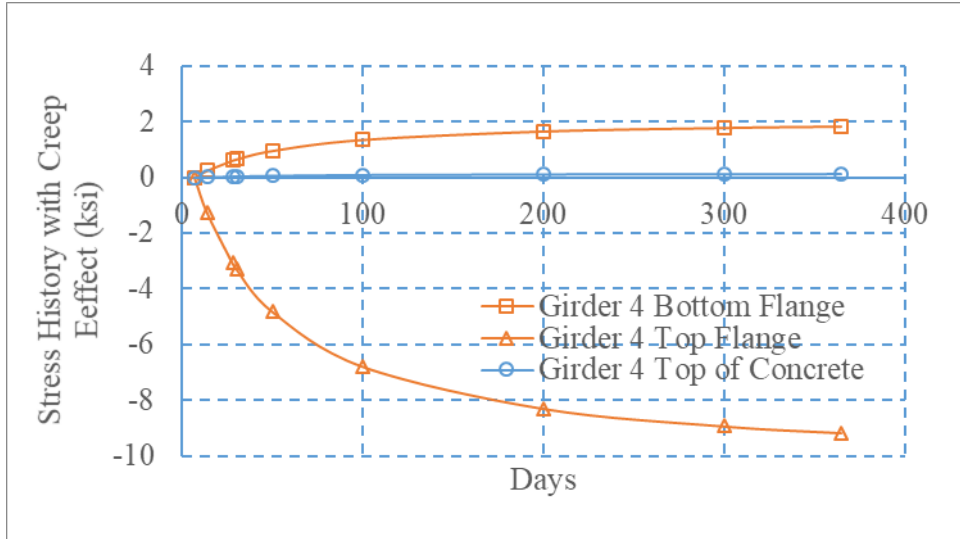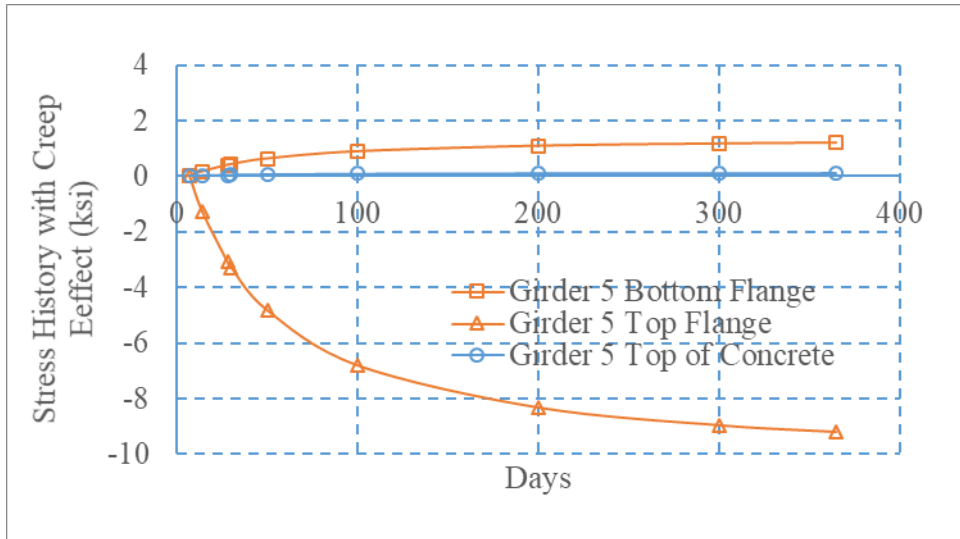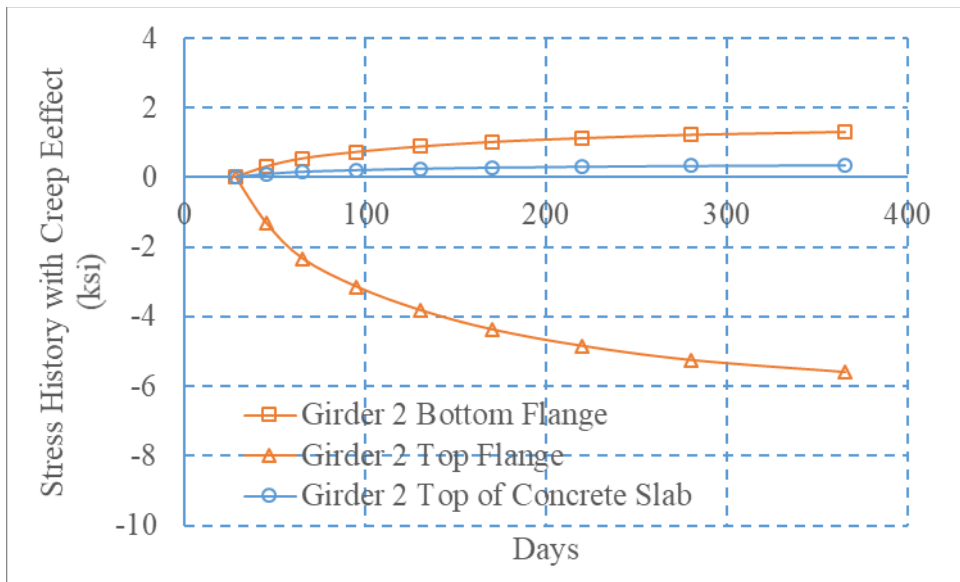

Figure 11-45. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
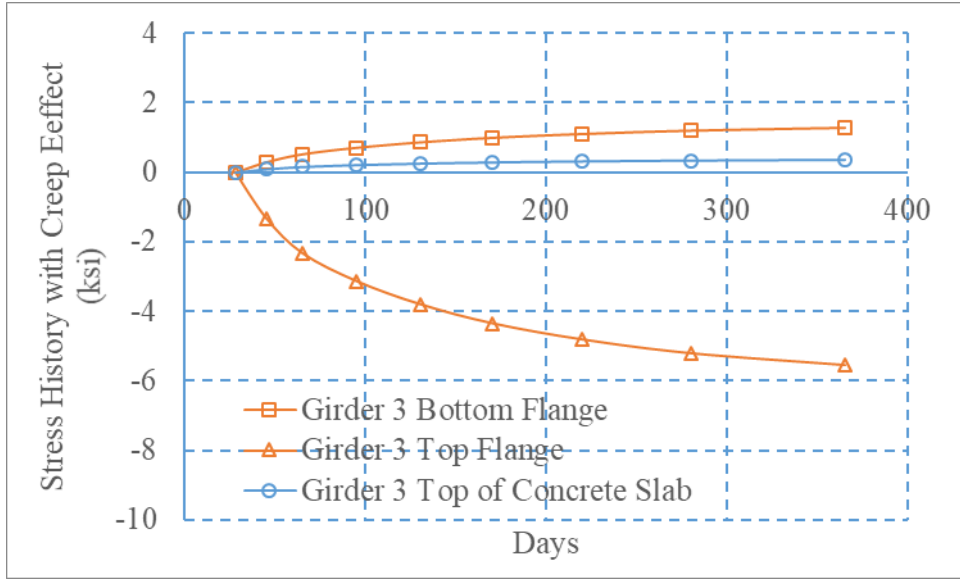
Figure 11-46. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
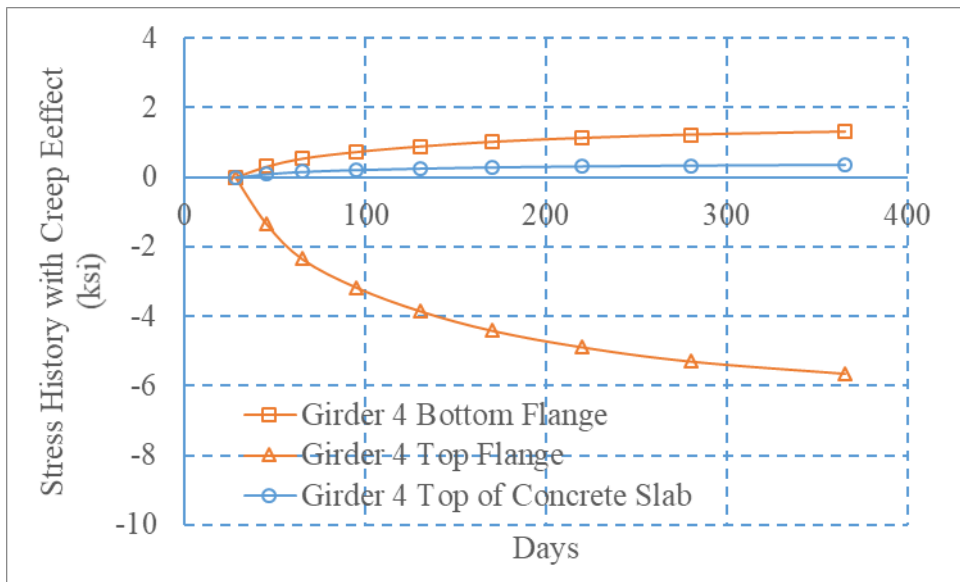
Figure 11-47. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
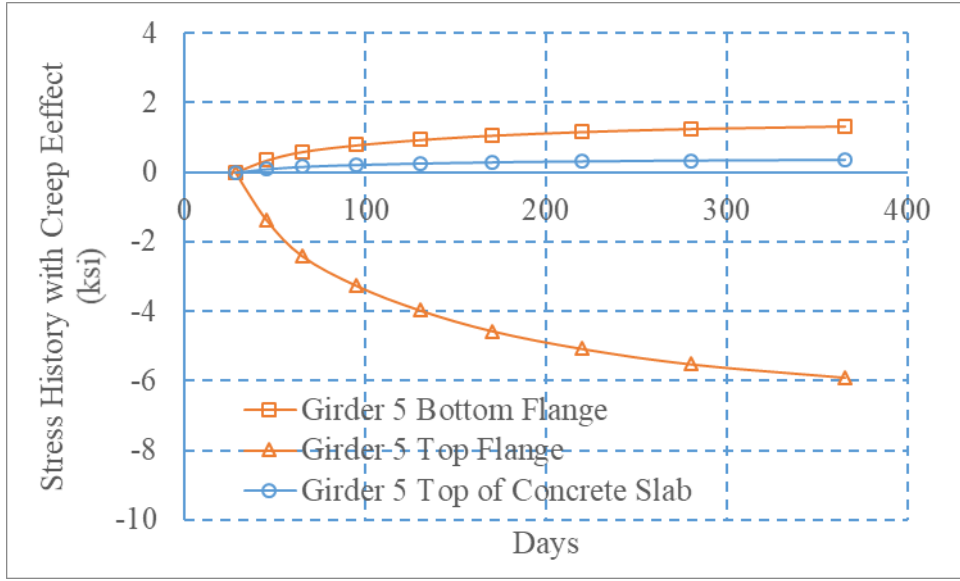
Figure 11-48. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
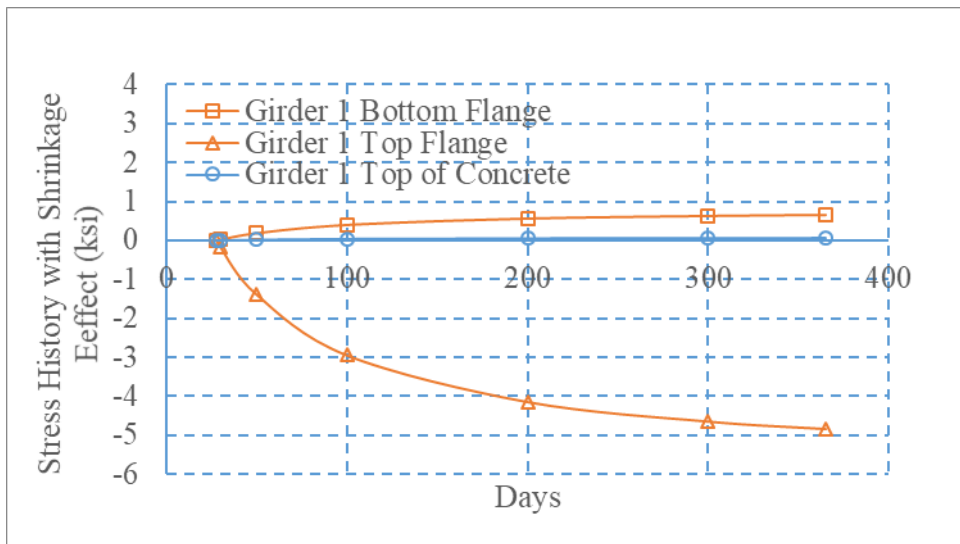
Figure 11-49. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
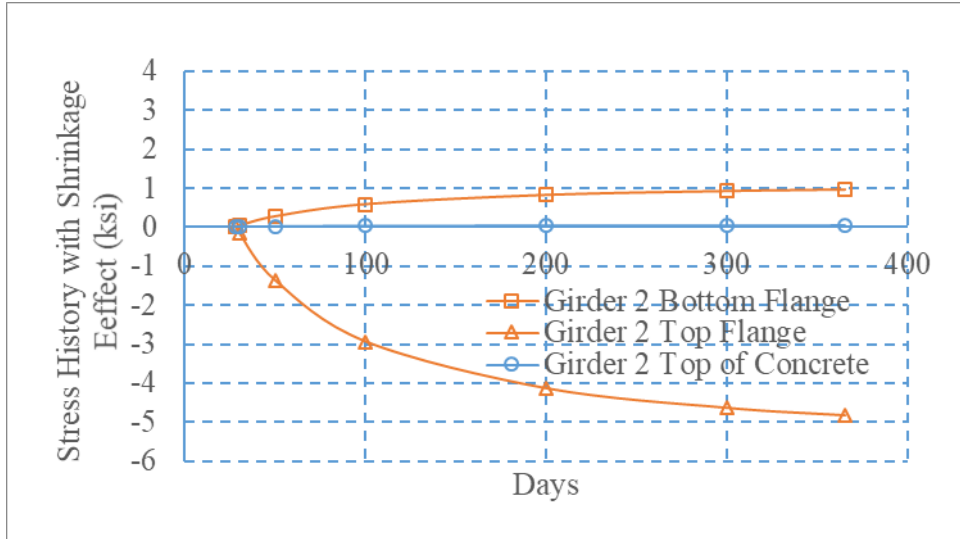
Figure 11-50. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
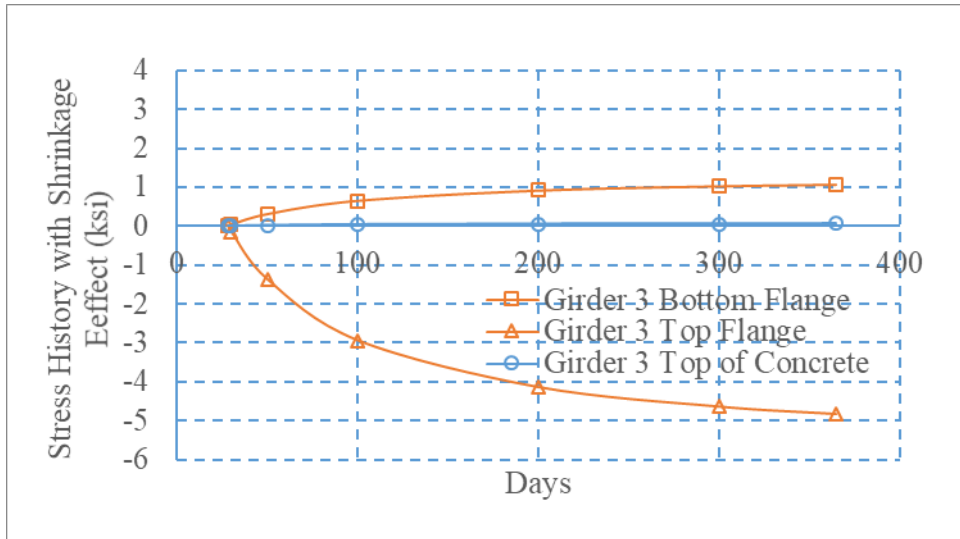
Figure 11-51. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
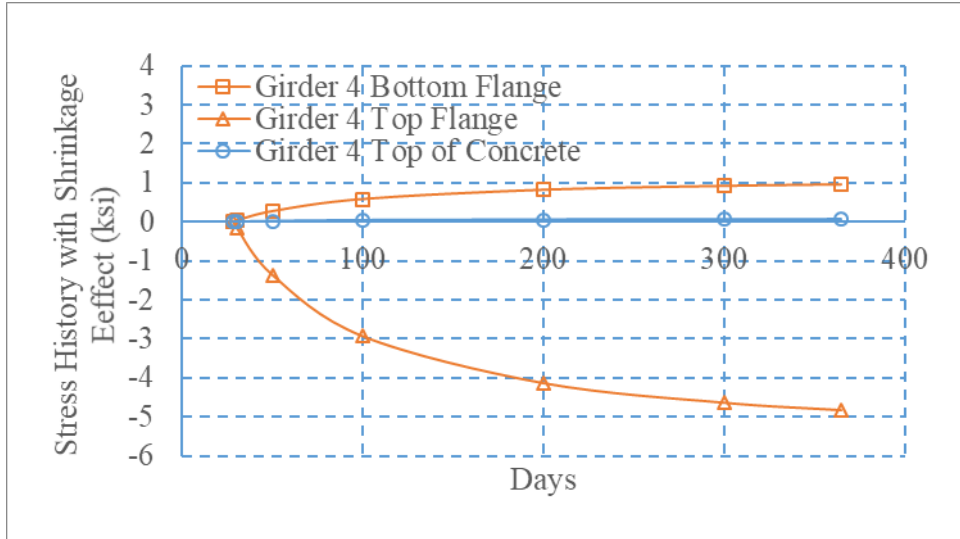
Figure 11-52. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
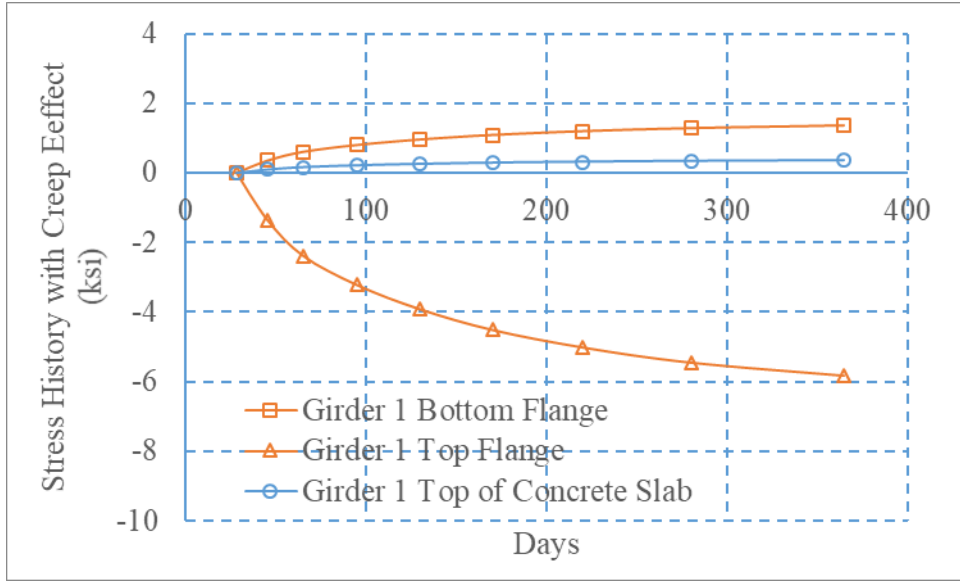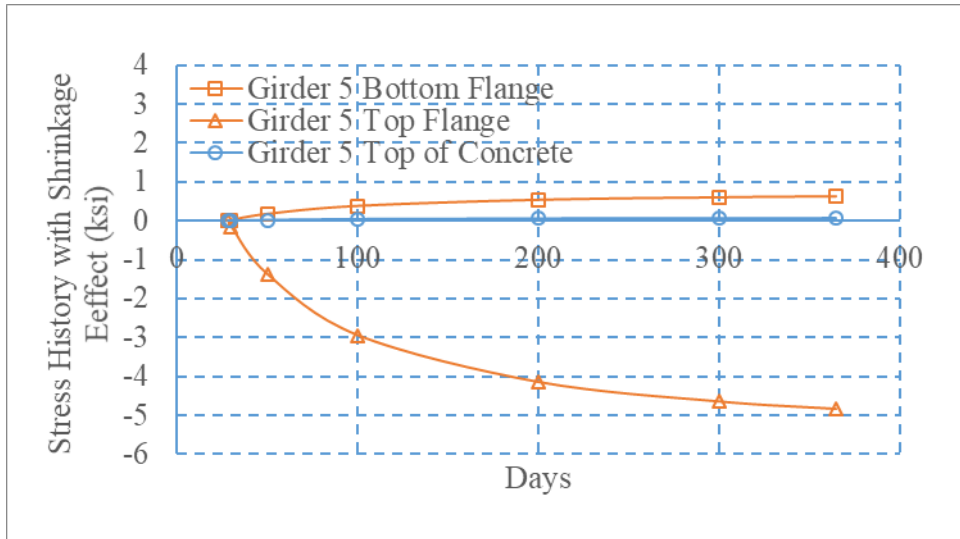

Figure 11-53. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

C-6 100' SPAN WITH FULLY SHORED CONDITION

This section summarizes the FE analysis results for a 100 ft single span steel composite girder bridge with fully shored condition.

Table 11-26. Results for girder 1 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.222	-0.033	-0.014
4	Creep(365 days)	composite section	24.285	-8.376	-0.522
5	Shrinkage (365 days)	composite section	0.651	-4.848	0.063
6	Total		25.158	-13.257	-0.472

Table 11-27. Results for girder 2 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.132	-0.017	-0.004
4	Creep(365 days)	composite section	24.285	-8.376	-0.513
5	Shrinkage (365 days)	composite section	0.969	-4.833	0.062
6	Total		25.386	-13.226	-0.455

Table 11-28. Results for girder 3 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.112	-0.008	-0.003
4	Creep(365 days)	composite section	24.420	-7.989	-0.508
5	Shrinkage (365 days)	composite section	1.073	-4.840	0.060
6	Total		25.605	-12.837	-0.451

Table 11-29. Results for girder 4 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.140	-0.016	-0.004
4	Creep(365 days)	composite section	24.360	-8.106	-0.512
5	Shrinkage (365 days)	composite section	1.814	-9.184	0.118
6	Total		26.314	-17.305	-0.398

Table 11-30. Results for girder 5 (a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.234	-0.027	-0.013
4	Creep(365 days)	composite section	24.245	-8.450	-0.526
5	Shrinkage (365 days)	composite section	1.197	-9.200	0.122
6	Total		25.676	-17.677	-0.417

Figure 11-54. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
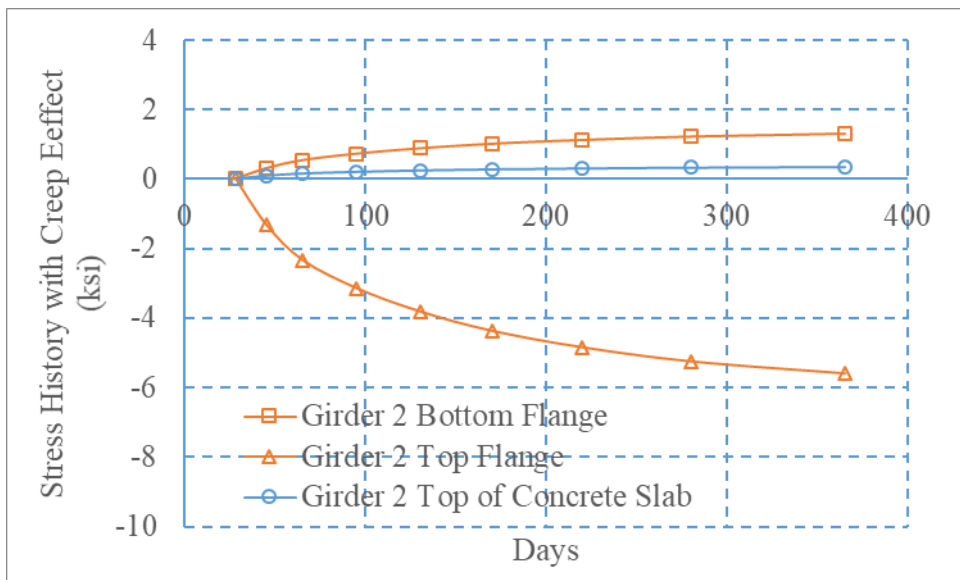
Figure 11-55. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

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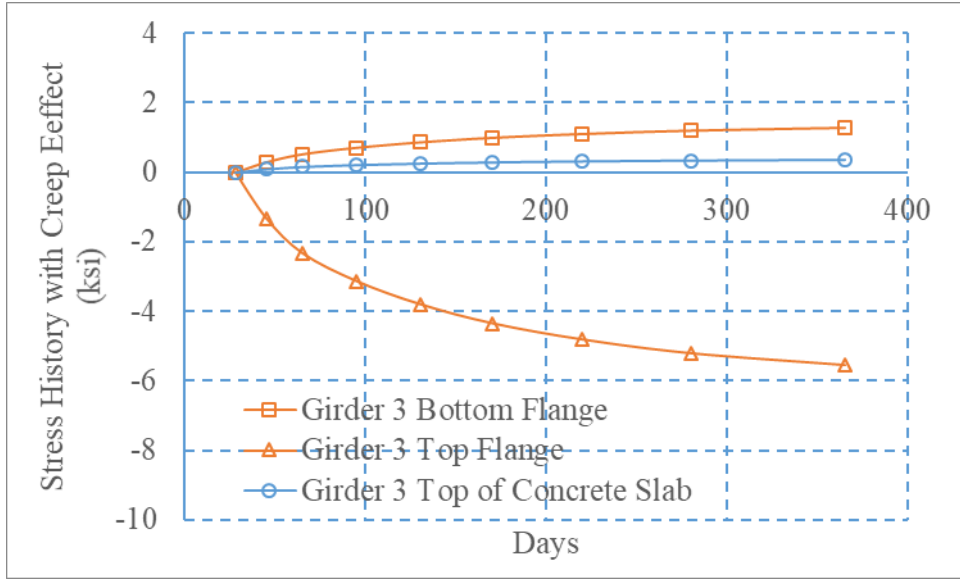
Figure 11-56. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
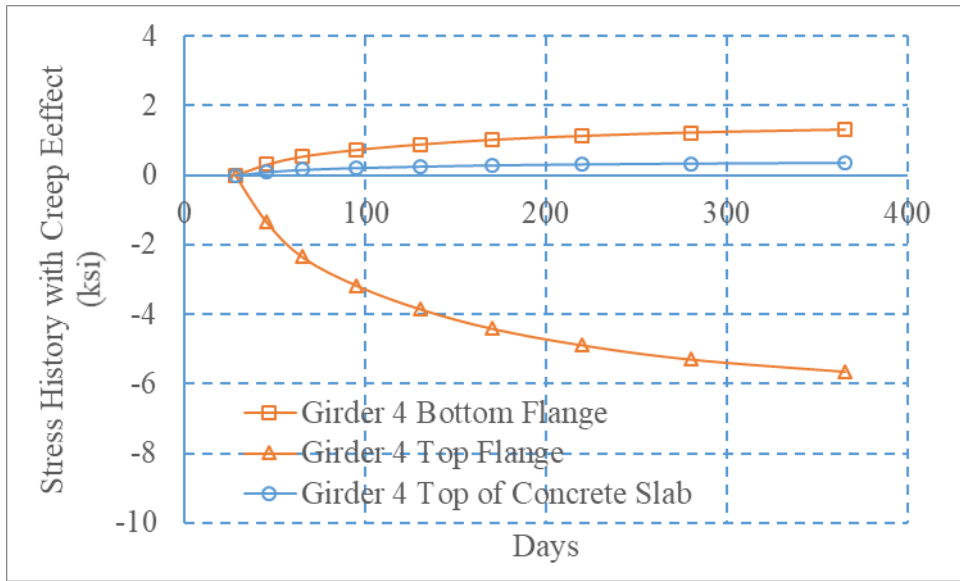
Figure 11-57. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
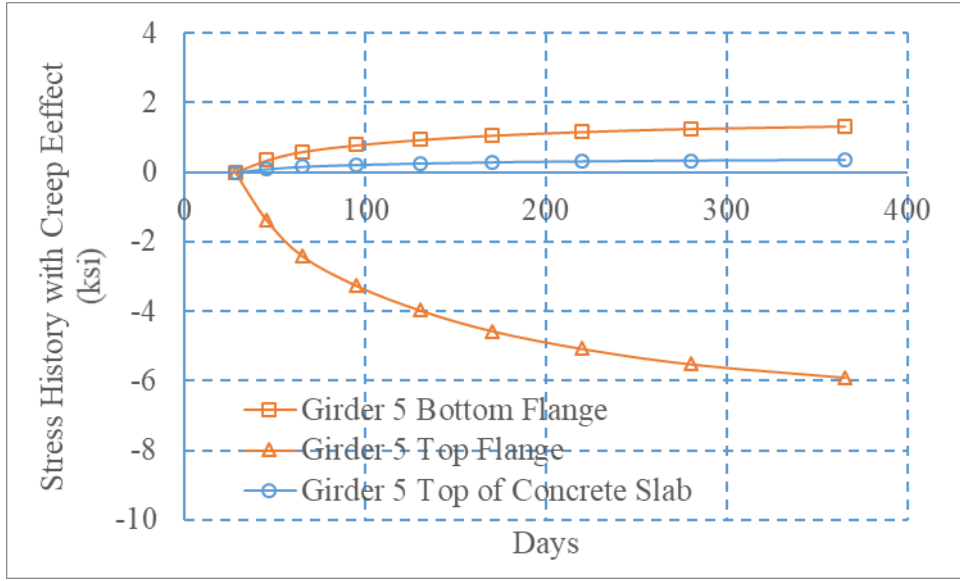
Figure 11-58. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-59. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-60. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-61. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-62. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
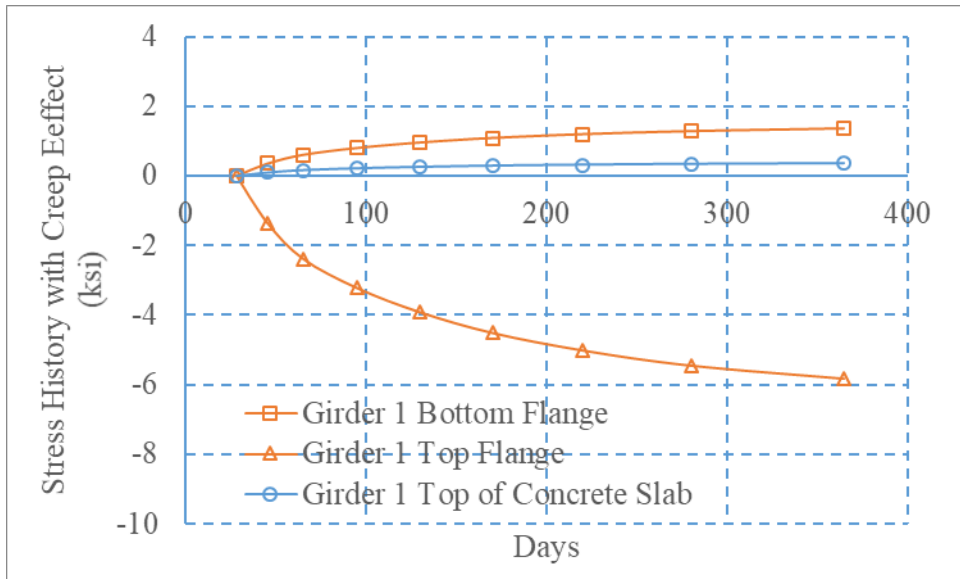

Figure 11-63. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 100 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

C-7 120' SPAN WITH UNSHORED CONDITION

This section summarizes the FE analysis results for a 120 ft single span steel composite girder bridge with unshored condition.

Table 11-31. Results for girder 1 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.7392	-5.3297	0
2	Wet concrete	steel section	15.943	-22.723	0
3	DC2	composite section	7.4584	-1.1238	-0.27902
4	Creep(365 days)	composite section	1.796	-9.2826	0.50192
5	Shrinkage (365 days)	composite section	1.4812	-9.225	0.15556
6	Total		30.4178	-47.6841	0.37846

Table 11-32. Results for girder 2 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.7538	-5.3441	0
2	Wet concrete	steel section	16.331	-23.322	0
3	DC2	composite section	6.9889	-1.1639	-0.27567
4	Creep(365 days)	composite section	1.791	-8.9177	0.50192
5	Shrinkage (365 days)	composite section	1.8823	-9.2609	0.14952
6	Total		30.747	-48.0086	0.37577

Table 11-33. Results for girder 3 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.4674	-5.4902	0
2	Wet concrete	steel section	16.687	-23.73	0
3	DC2	composite section	6.7364	-1.1878	-0.27374
4	Creep(365 days)	composite section	1.791	-8.9177	0.50656
5	Shrinkage (365 days)	composite section	1.935	-9.2695	0.13833
6	Total		30.6168	-48.5952	0.37115

Table 11-34. Results for girder 4 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.7585	-5.3531	0
2	Wet concrete	steel section	16.253	-23.047	0
3	DC2	composite section	6.9012	-1.1857	-0.27991
4	Creep(365 days)	composite section	1.854	-8.7737	0.51281
5	Shrinkage (365 days)	composite section	1.5486	-9.3014	0.1374
6	Total		30.3153	-47.6609	0.3703

Table 11-35. Results for girder 5 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.736	-5.3147	0
2	Wet concrete	steel section	15.813	-22.382	0
3	DC2	composite section	7.3581	-1.1517	
4	Creep(365 days)	composite section	1.964	-8.9184	0.51358
5	Shrinkage (365 days)	composite section	0.92191	-9.3188	0.12748
6	Total		29.79301	-47.0856	0.64106

Figure 11-64. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
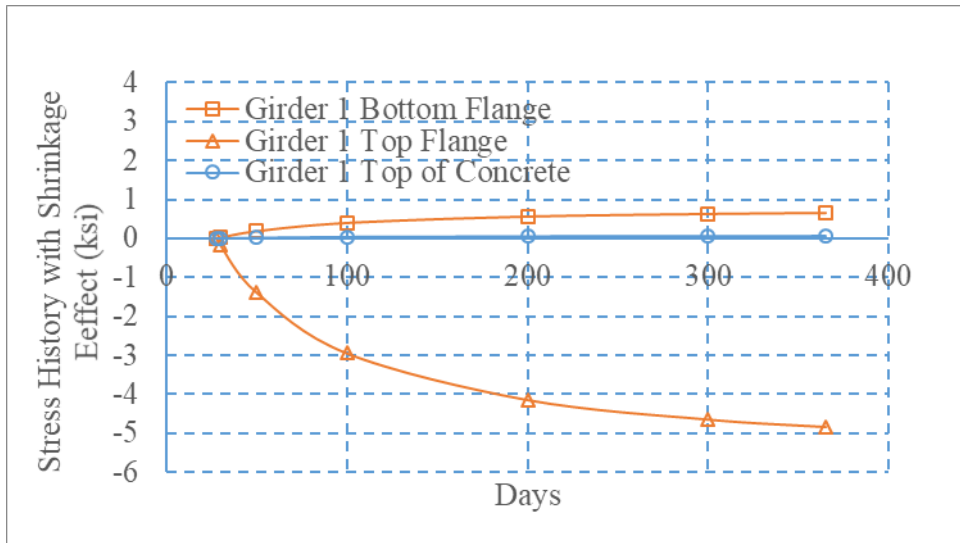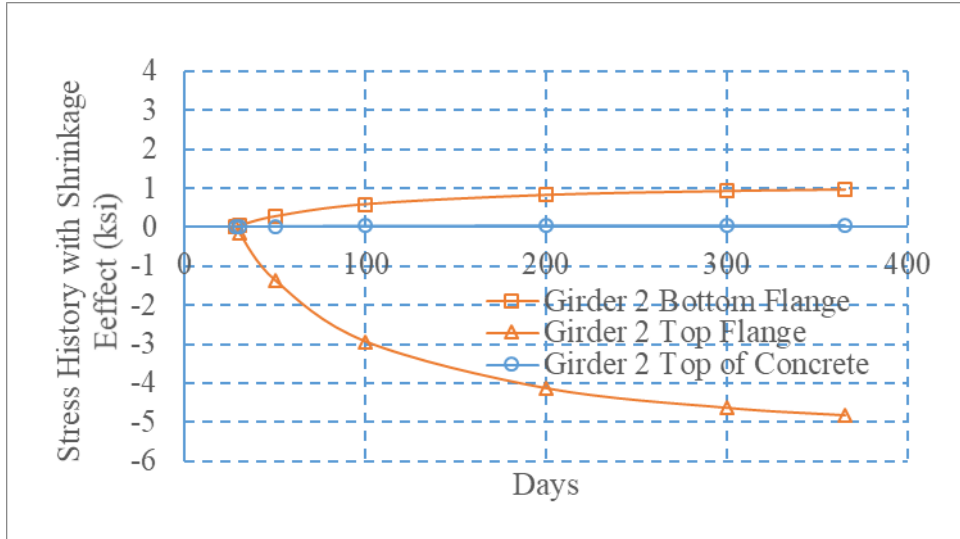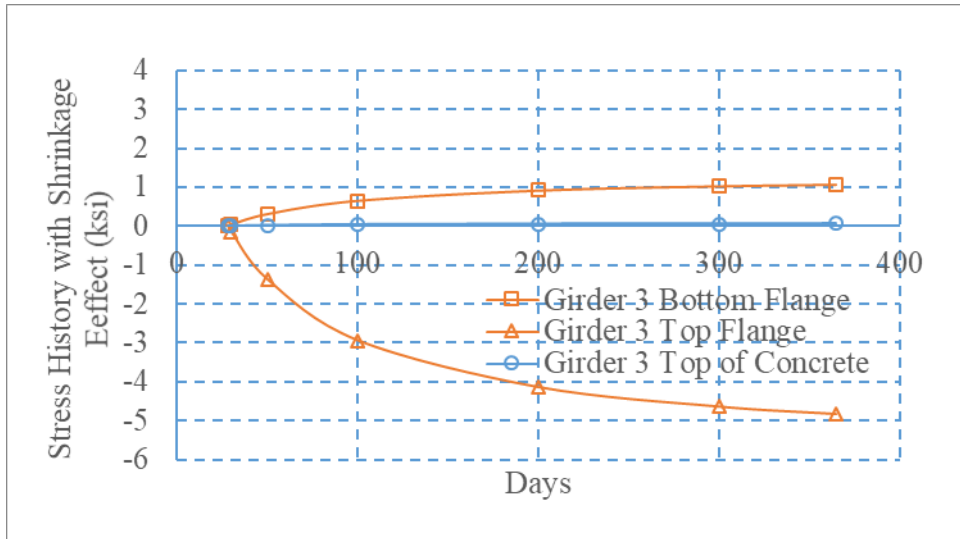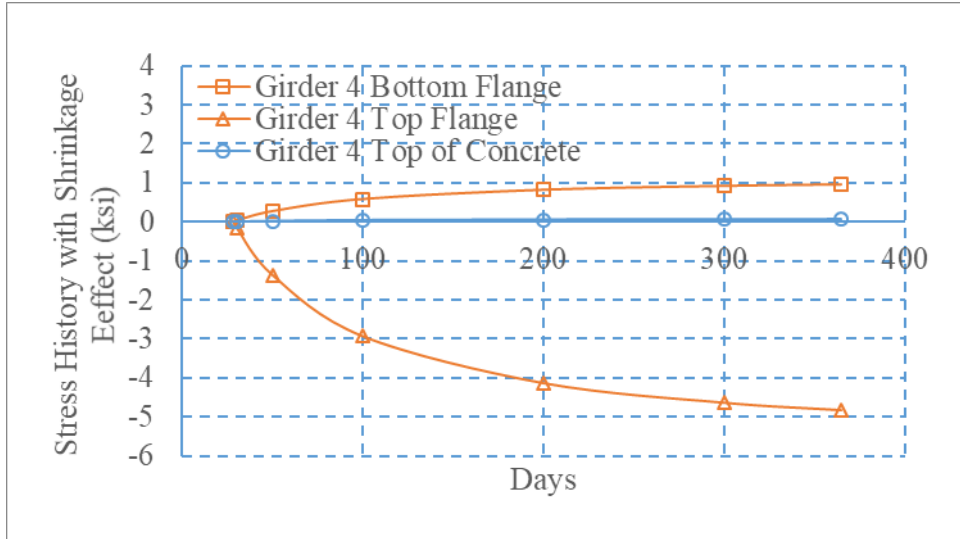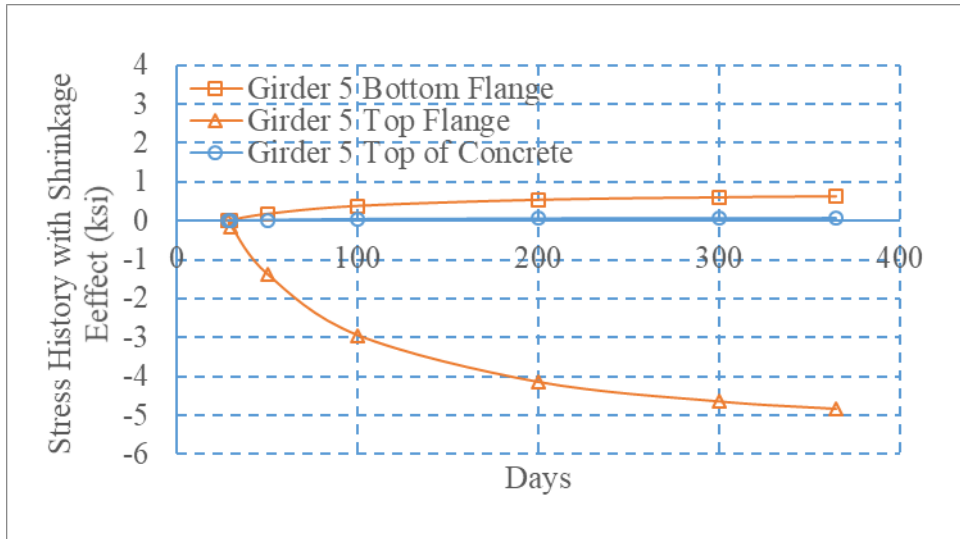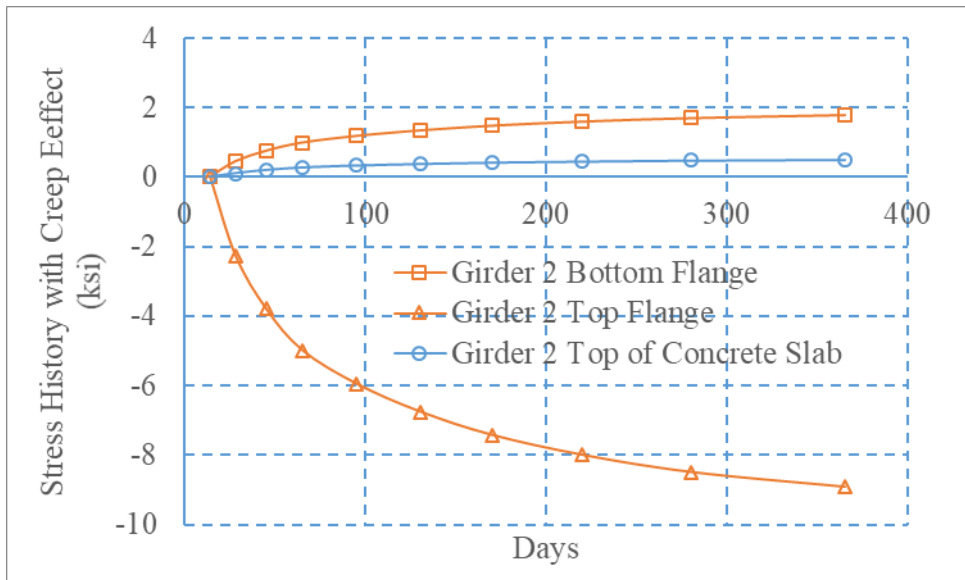

Figure 11-65. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
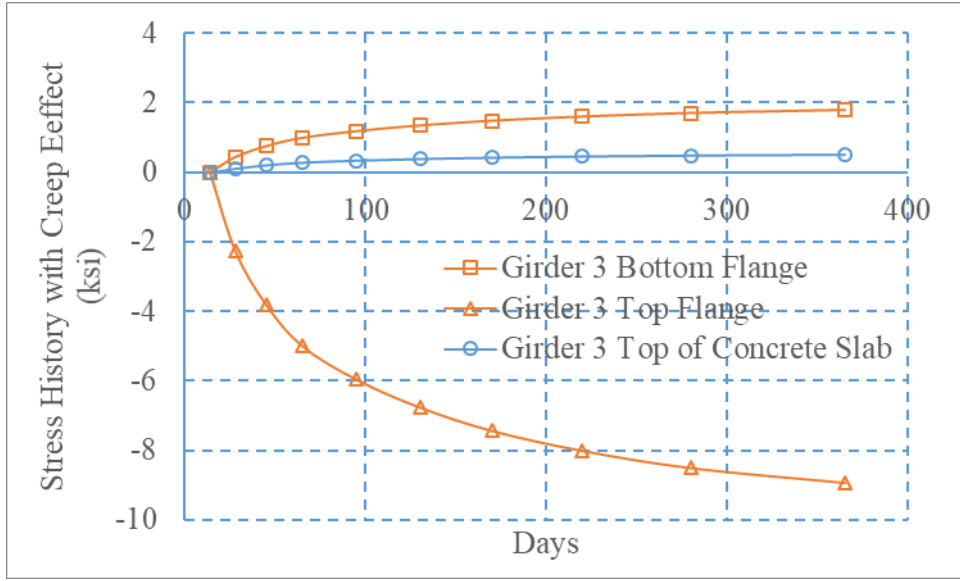
Figure 11-66. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
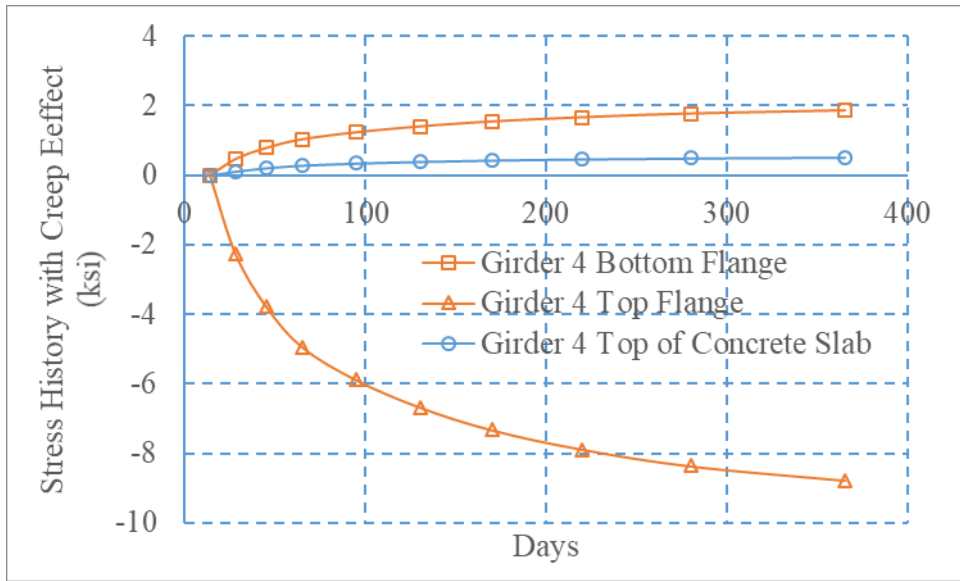
Figure 11-67. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
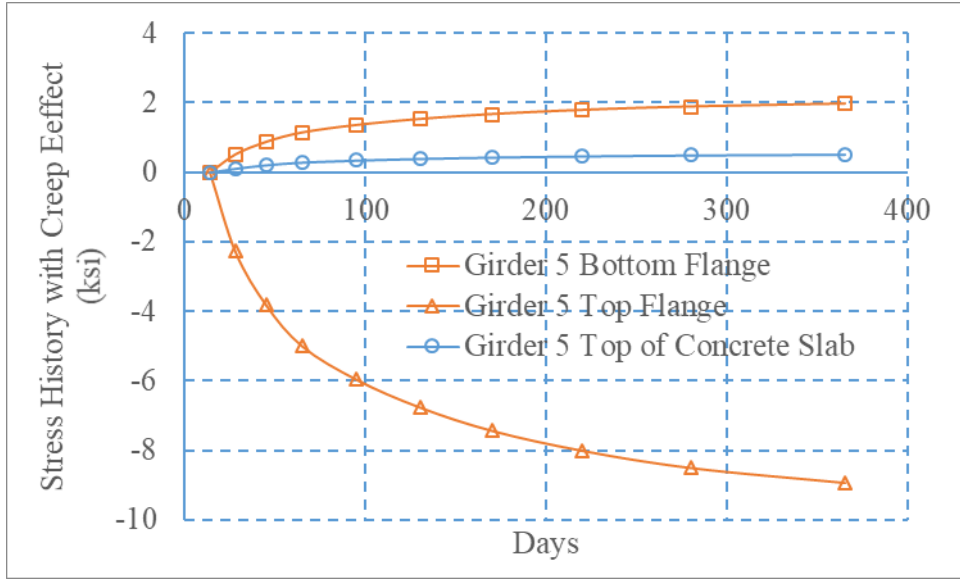
Figure 11-68. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
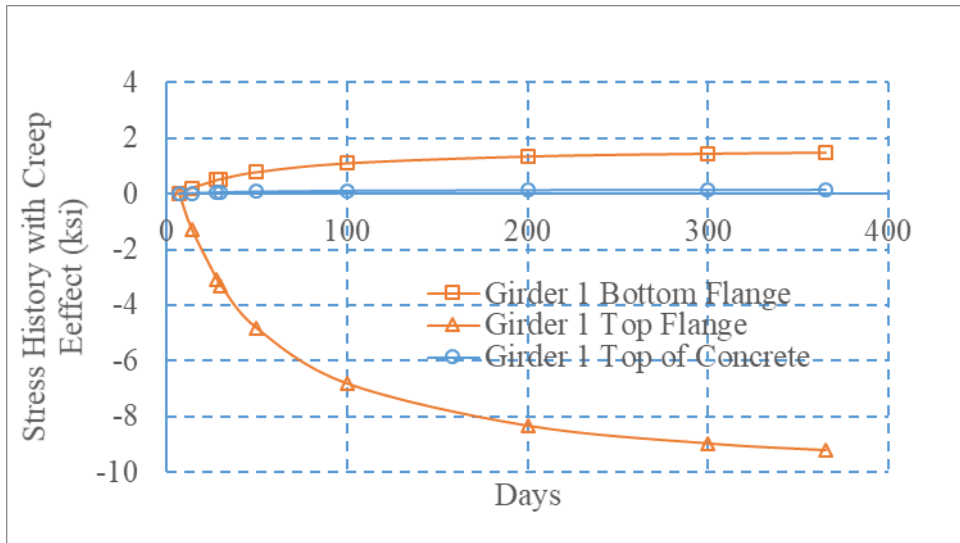
Figure 11-69. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
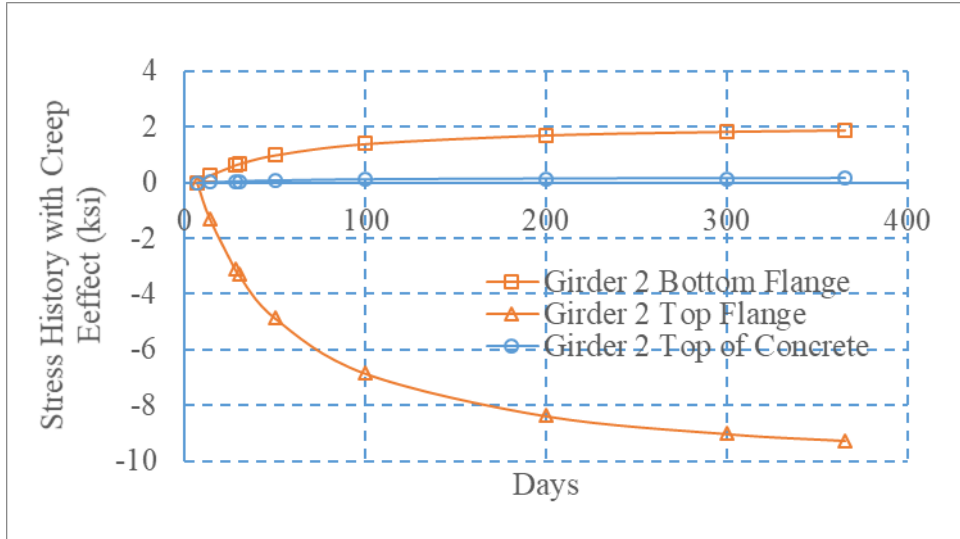
Figure 11-70. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
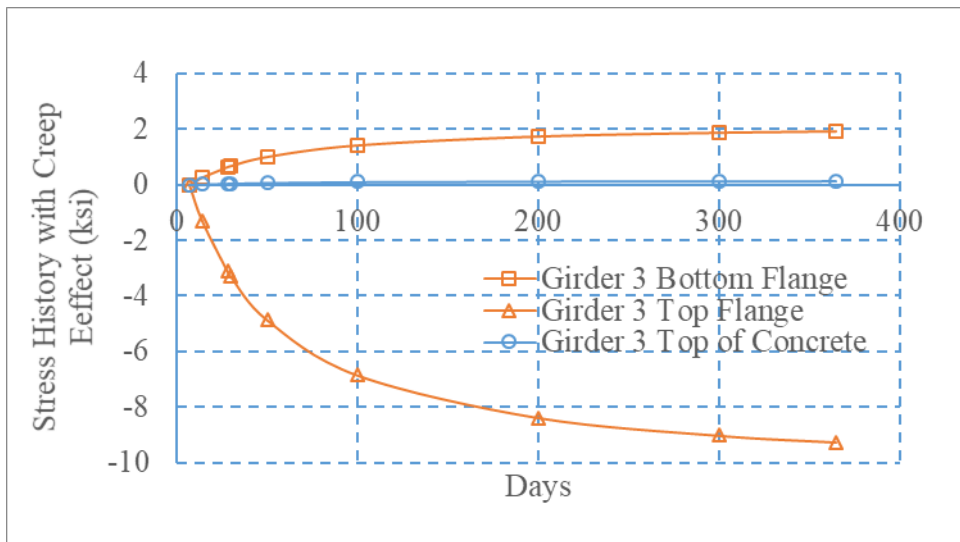
Figure 11-71. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
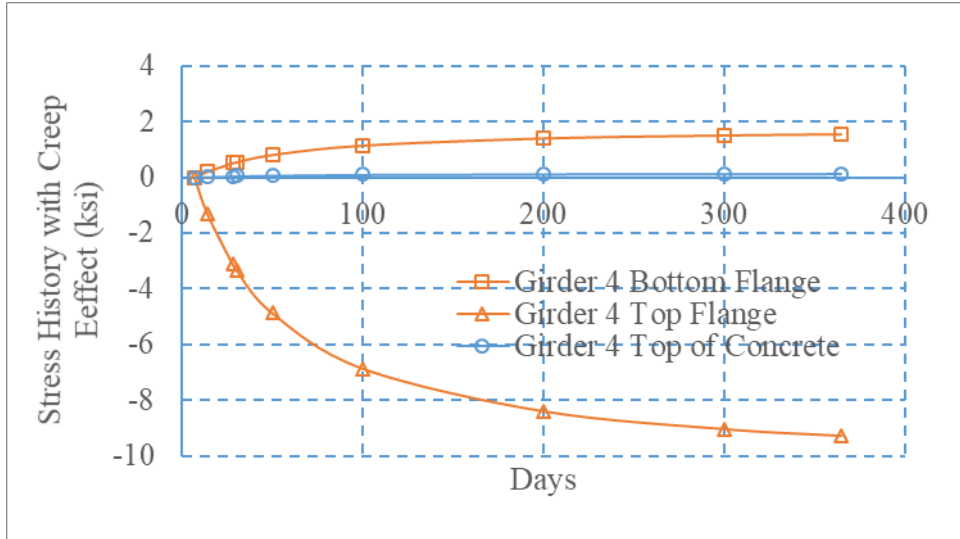
Figure 11-72. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
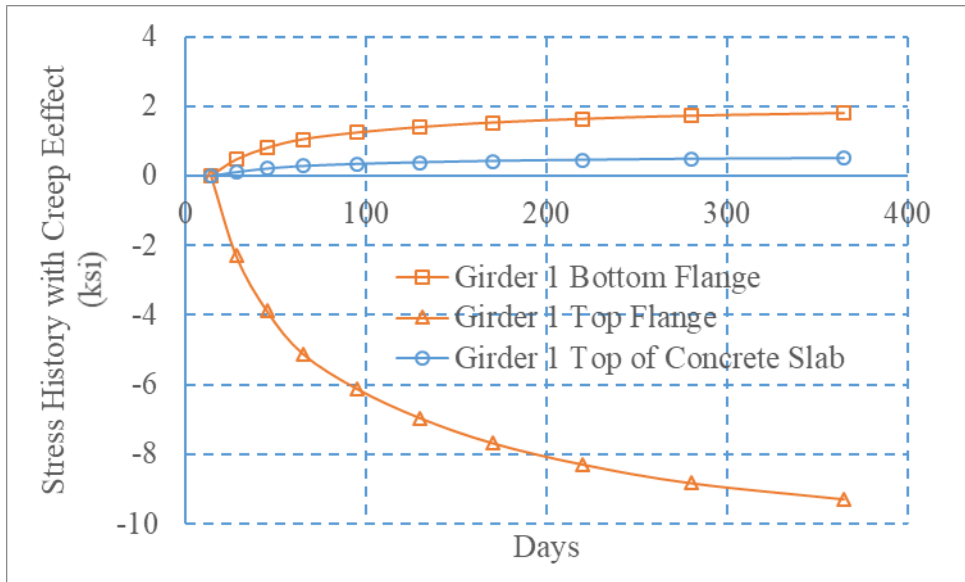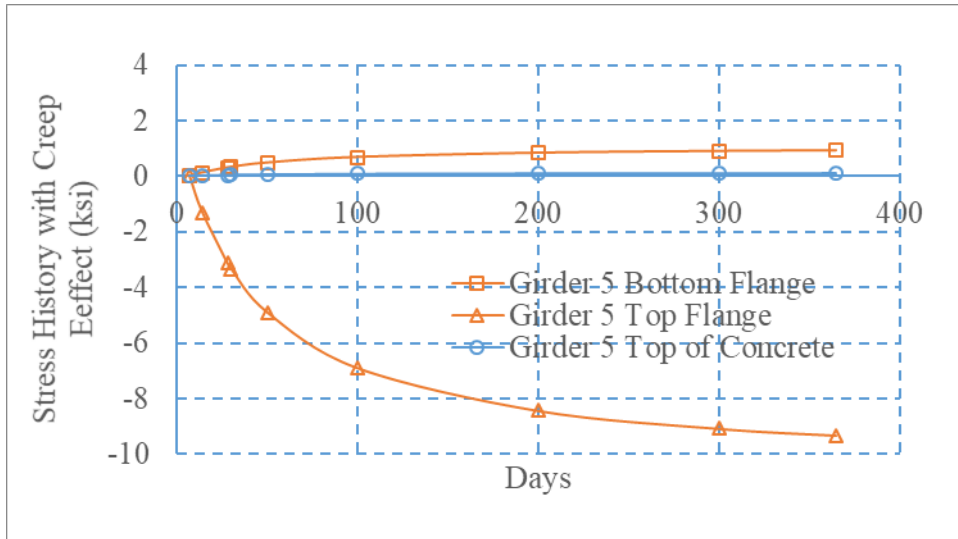

Figure 11-73. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

C-8 120' SPAN WITH SHORING AT 1/3 POINTS

This section summarizes the FE analysis results for a 120 ft single span steel composite girder bridge with shoring at 1/3 points.

Table 11-36 Results for girder 1 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	-0.019	-0.014	0.000
2	Wet concrete	steel section	-0.069	-0.050	0.000
3	DC2	composite section	-0.021	-0.013	-0.002
4	Creep(365 days)	composite section	25.274	-10.920	-0.605
5	Shrinkage (365 days)	composite section	0.779	-4.853	0.082
6	Total		25.945	-15.850	-0.526

Table 11-37. Results for girder 2 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.066	0.028	0.000
2	Wet concrete	steel section	0.335	0.117	0.000
3	DC2	composite section	0.125	0.007	-0.007
4	Creep(365 days)	composite section	25.274	-10.920	-0.594
5	Shrinkage (365 days)	composite section	0.990	-4.872	0.079
6	Total		26.790	-15.640	-0.522

Table 11-38. Results for girder 3 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.059	0.027	0.000
2	Wet concrete	steel section	1.117	-0.440	0.000
3	DC2	composite section	0.405	-0.022	0.021
4	Creep(365 days)	composite section	25.488	-10.533	-0.594
5	Shrinkage (365 days)	composite section	1.018	-4.877	0.073
6	Total		28.087	-15.844	-0.499

Table 11-39. Results for girder 4 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.230	-0.094	0.000
2	Wet concrete	steel section	1.117	-1.561	0.000
3	DC2	composite section	0.316	-0.041	0.021
4	Creep(365 days)	composite section	25.518	-10.399	-0.587
5	Shrinkage (365 days)	composite section	1.549	-9.301	0.137
6	Total		28.730	-21.396	-0.428

Table 11-40. Results for girder 5 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.056	-0.315	0.000
2	Wet concrete	steel section	0.316	-1.561	0.000
3	DC2	composite section	0.108	-0.050	0.016
4	Creep(365 days)	composite section	25.473	-10.565	-0.593
5	Shrinkage (365 days)	composite section	0.922	-9.319	0.127
6	Total		26.876	-21.810	-0.450

Figure 11-74. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-75. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-76. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-77. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-78. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-79. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-80. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-81. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-82. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
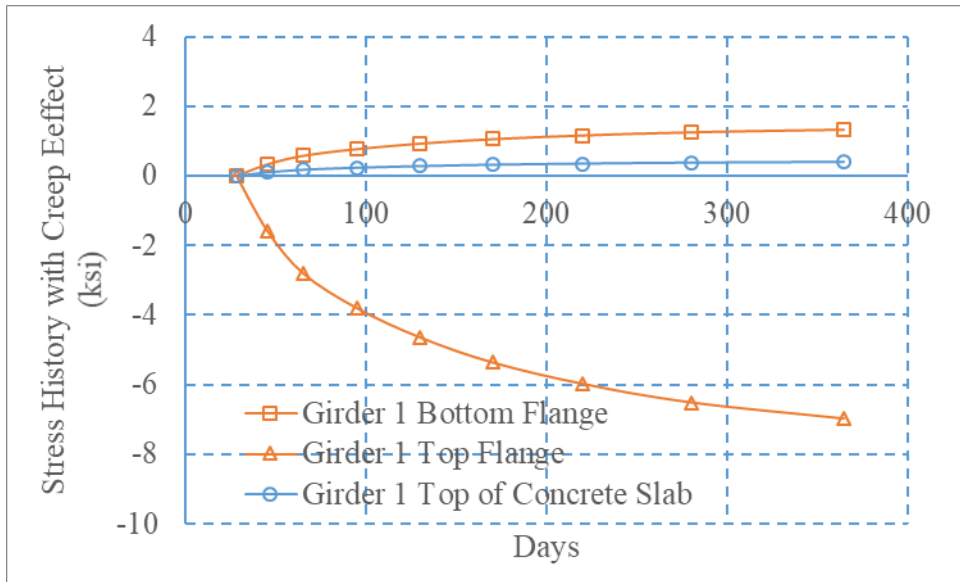

Figure 11-83. Time history for stress along longitudinal direction for girder #5 at midspan

(shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system,
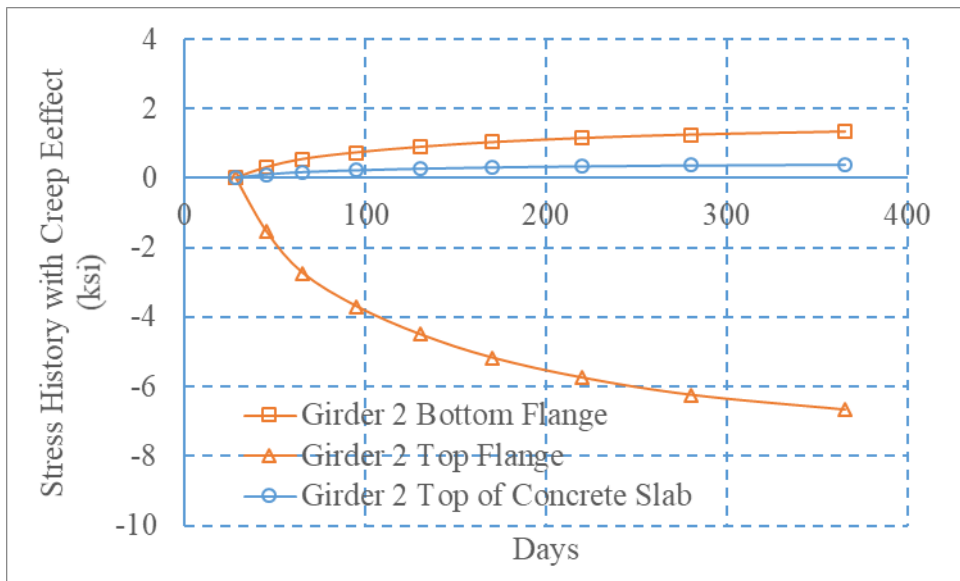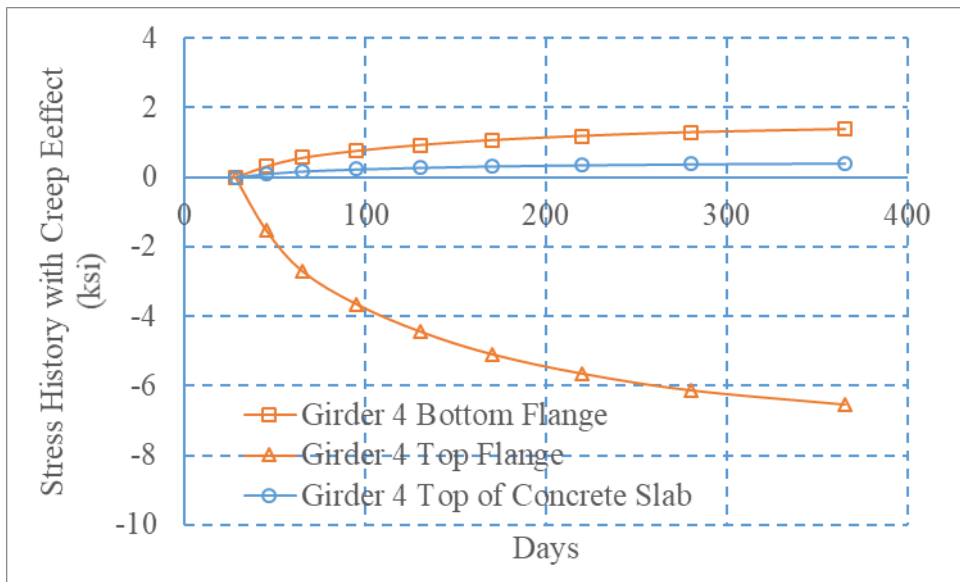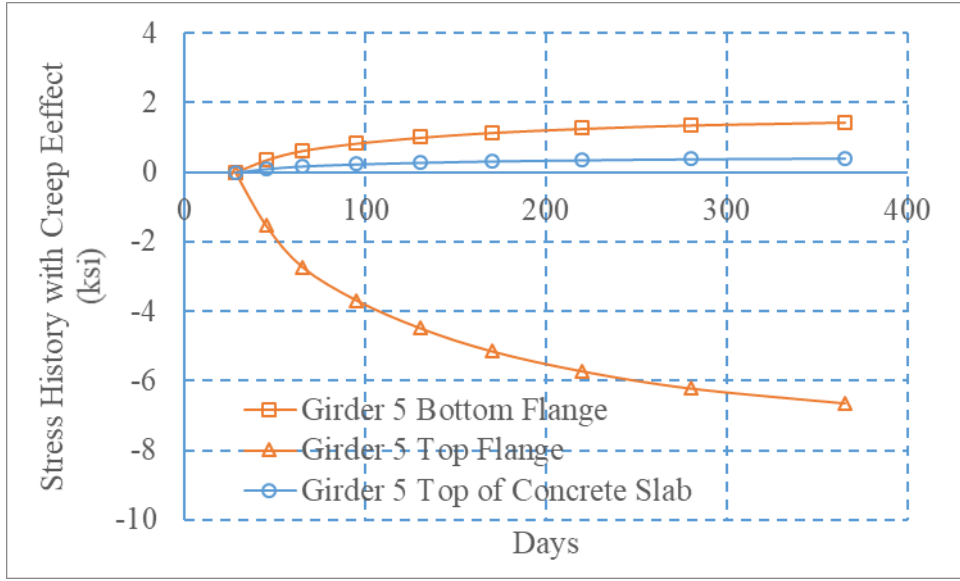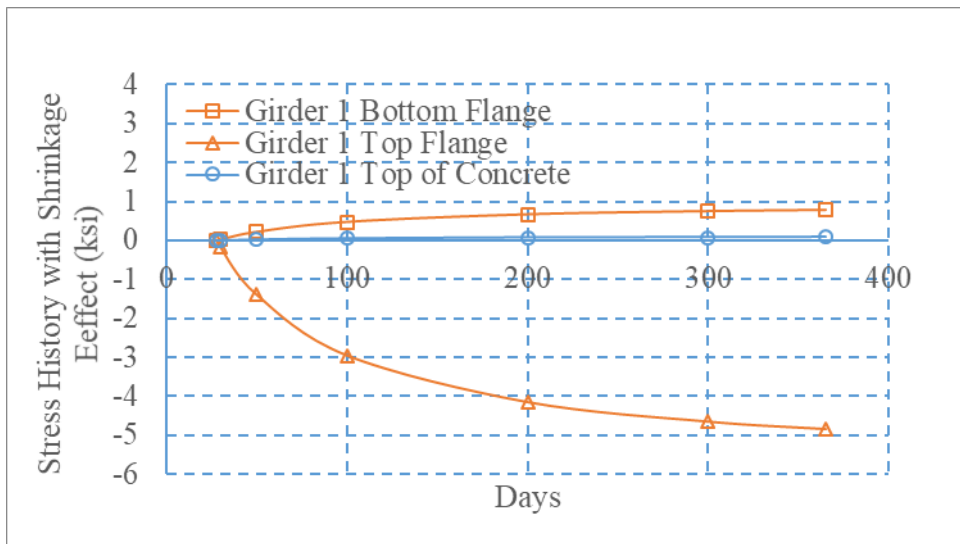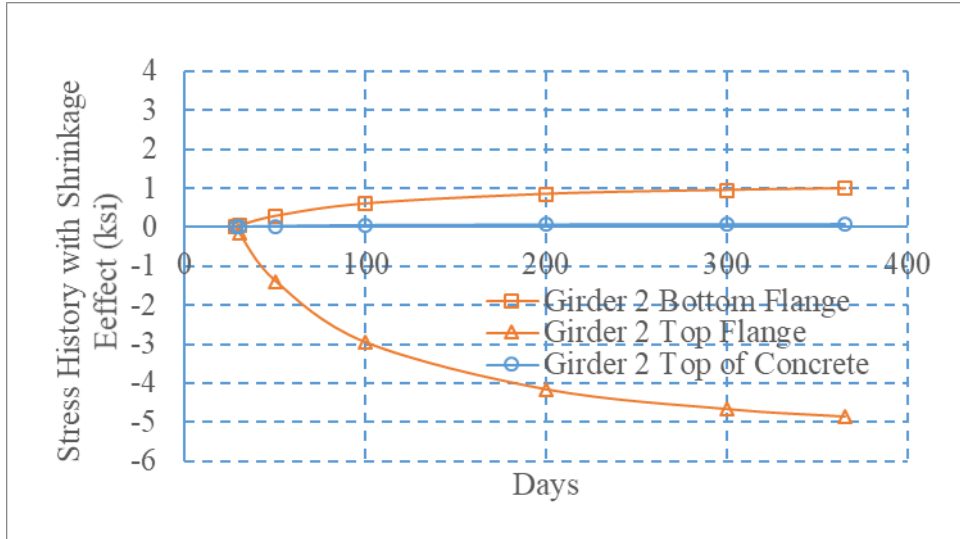
shoring at 1/3 points)

C-9 120' SPAN WITH FULLY SHORED CONDITION

This section summarizes the FE analysis results for a 120 ft single span steel composite girder bridge with fully shored condition.

Table 11-41. Results for girder 1 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	-0.021	-0.013	-0.002
4	Creep(365 days)	composite section	25.274	-10.920	-0.605
5	Shrinkage (365 days)	composite section	0.779	-4.853	0.082
6	Total		26.033	-15.786	-0.526

Table 11-42. Results for girder 2 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.125	0.007	-0.007
4	Creep(365 days)	composite section	25.274	-10.920	-0.594
5	Shrinkage (365 days)	composite section	0.990	-4.872	0.079
6	Total		26.389	-15.785	-0.522

Table 11-43. Results for girder 3 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.405	-0.022	0.021
4	Creep(365 days)	composite section	25.488	-10.533	-0.594
5	Shrinkage (365 days)	composite section	1.018	-4.877	0.073
6	Total		26.911	-15.432	-0.499

Table 11-44. Results for girder 4 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.316	-0.041	0.021
4	Creep(365 days)	composite section	25.518	-10.399	-0.587
5	Shrinkage (365 days)	composite section	1.549	-9.301	0.137
6	Total		27.383	-19.741	-0.428

Table 11-45. Results for girder 5 (a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.108	-0.050	0.016
4	Creep(365 days)	composite section	25.473	-10.565	-0.593
5	Shrinkage (365 days)	composite section	0.922	-9.319	0.127
6	Total		26.503	-19.933	-0.450

Figure 11-84. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
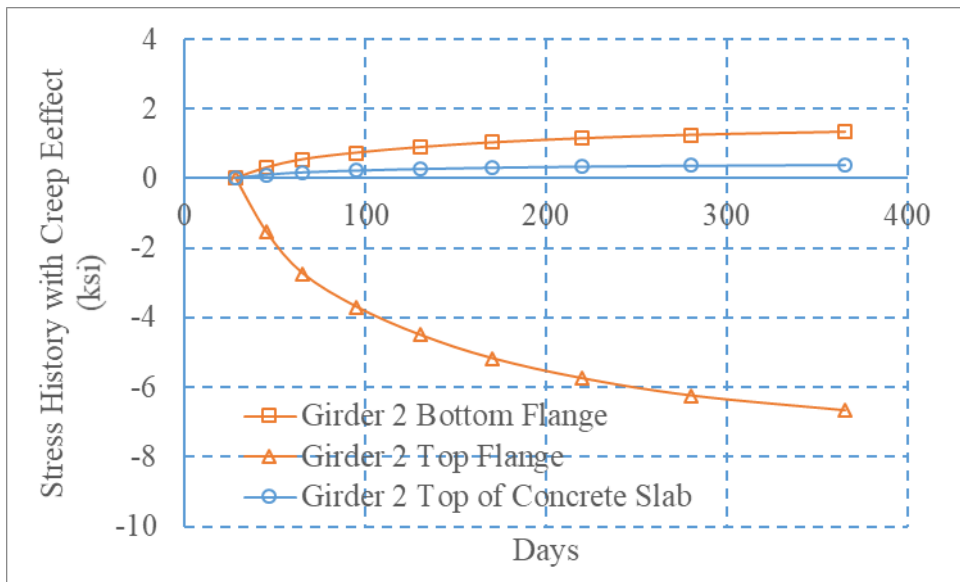
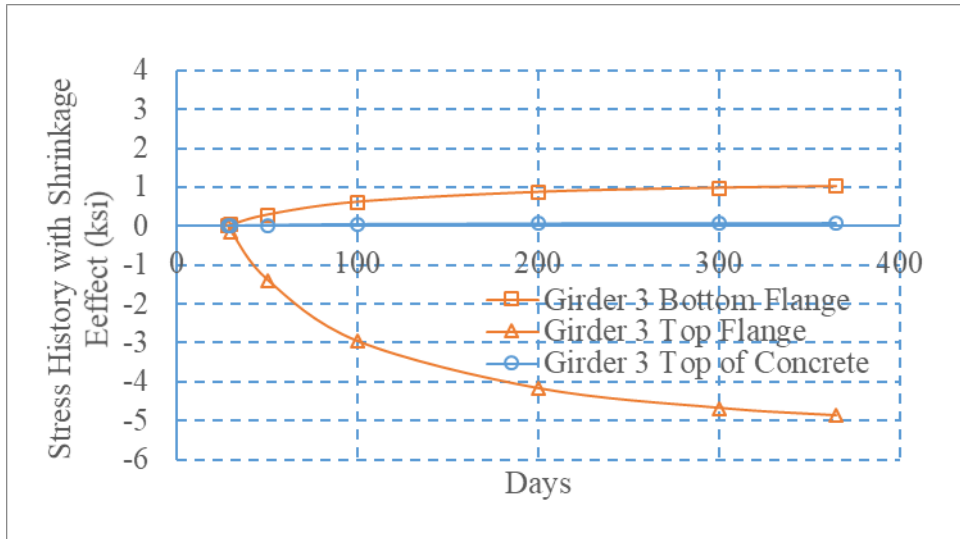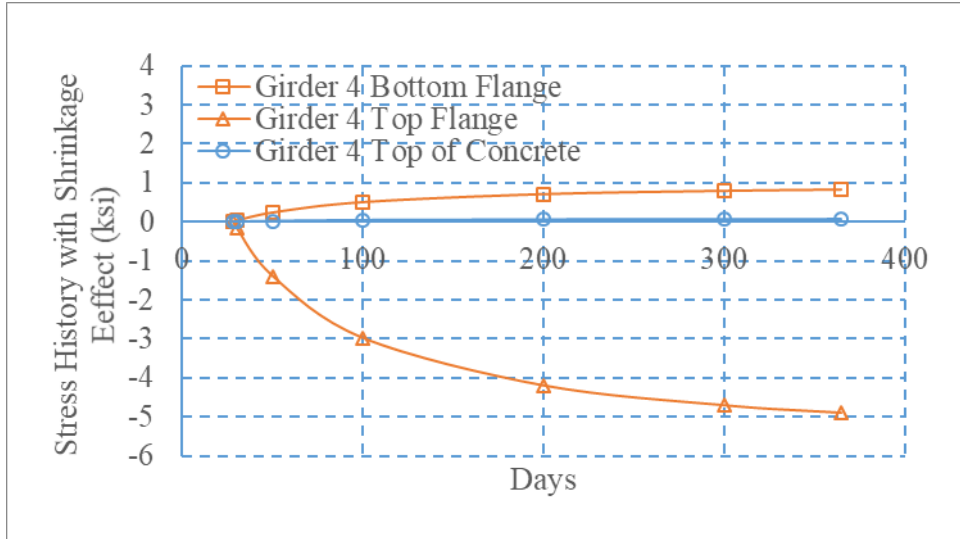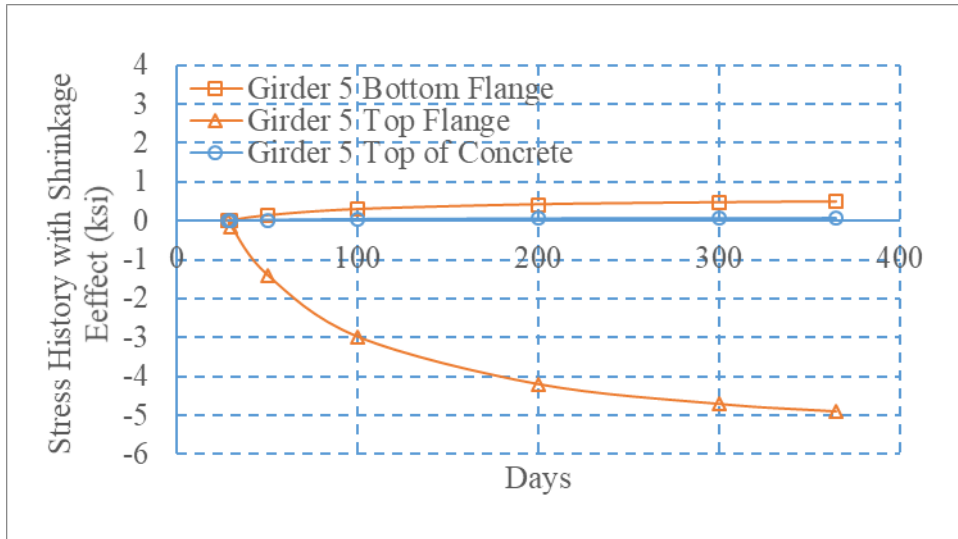Figure 11-85. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-86. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
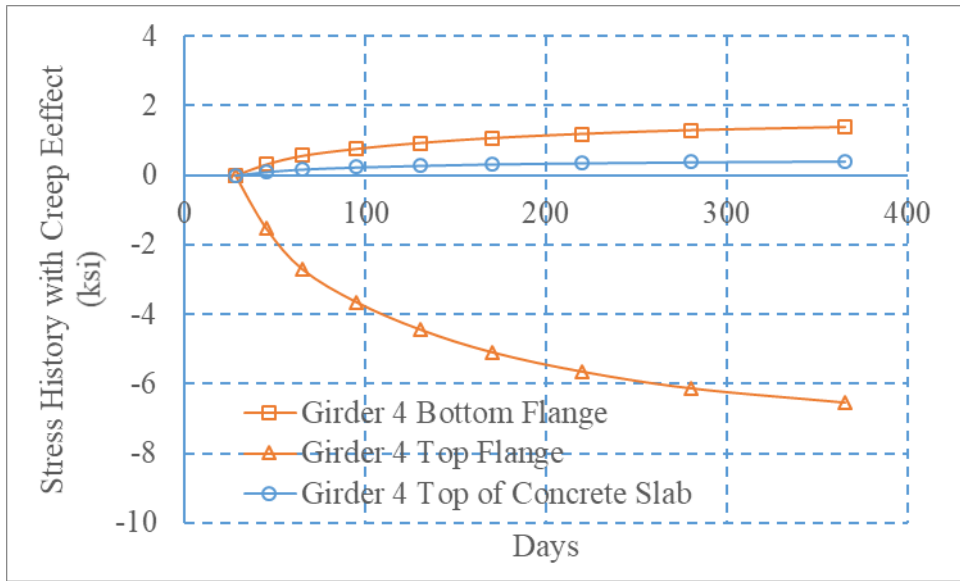
Figure 11-87. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
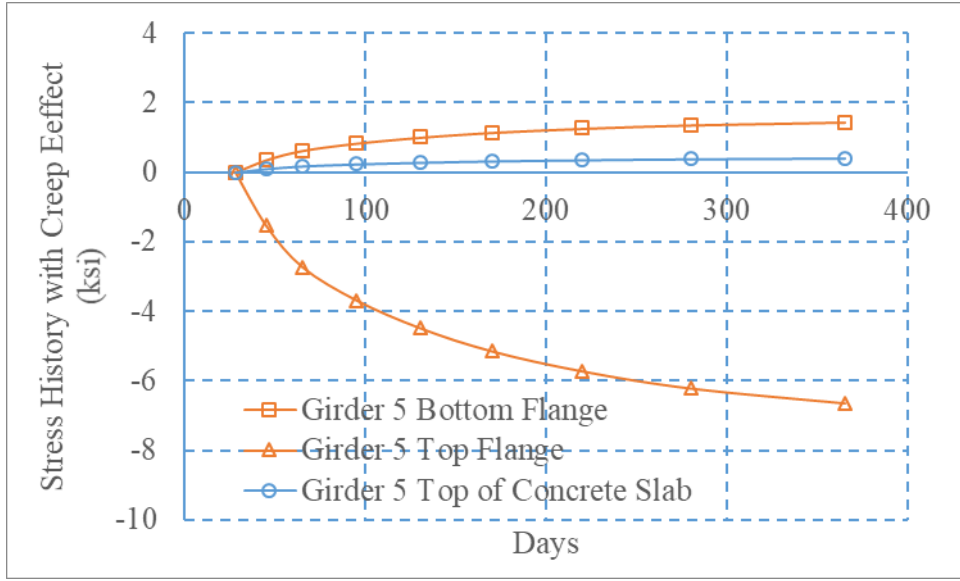
Figure 11-88. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
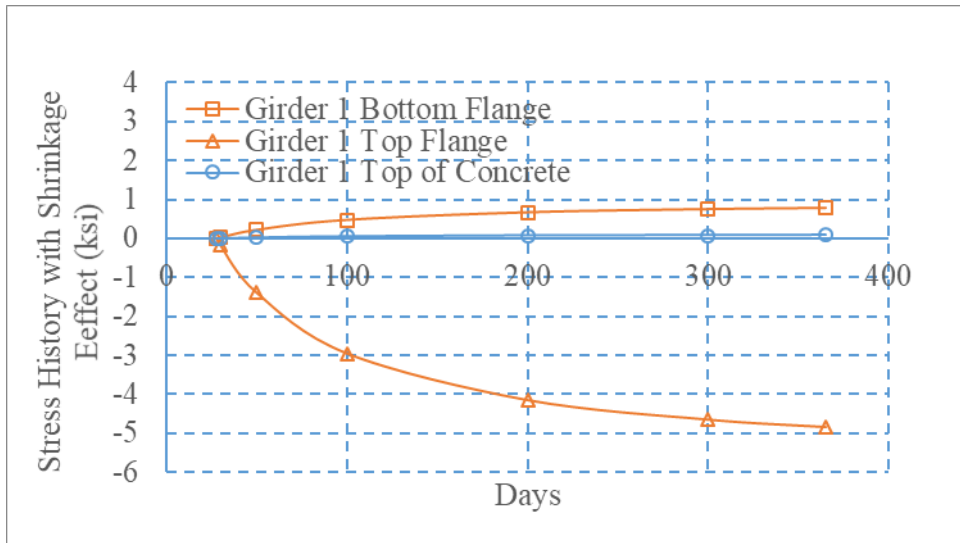
Figure 11-89. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
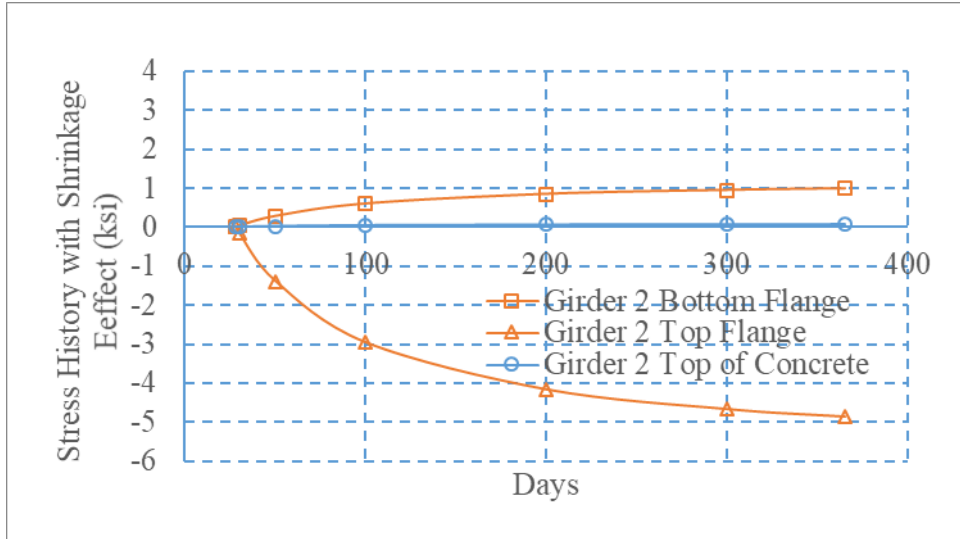
Figure 11-90. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-91. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
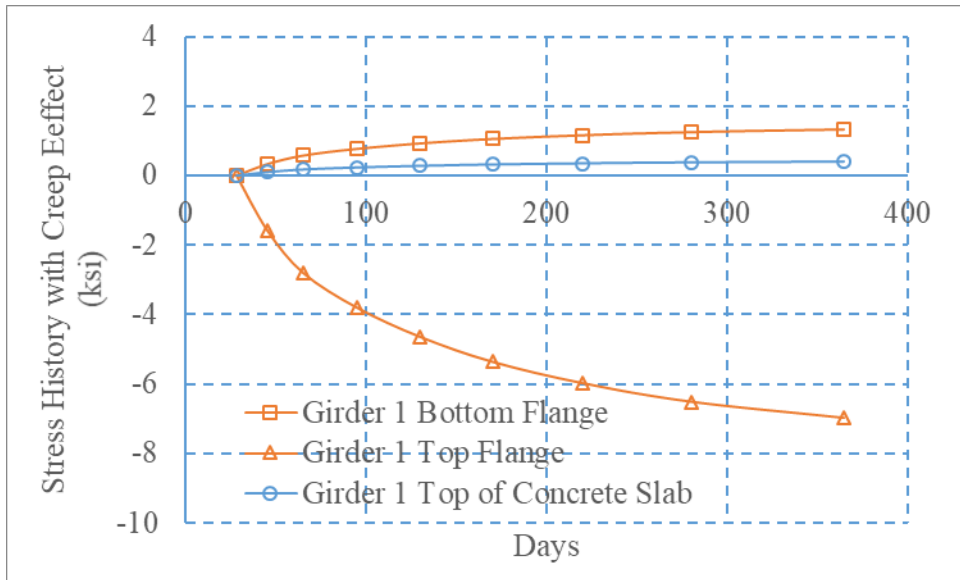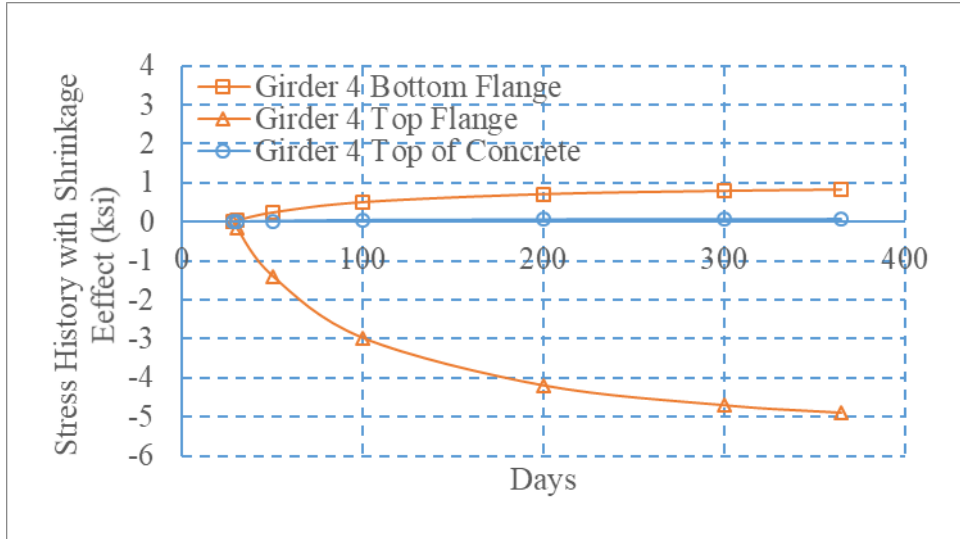
Figure 11-92. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
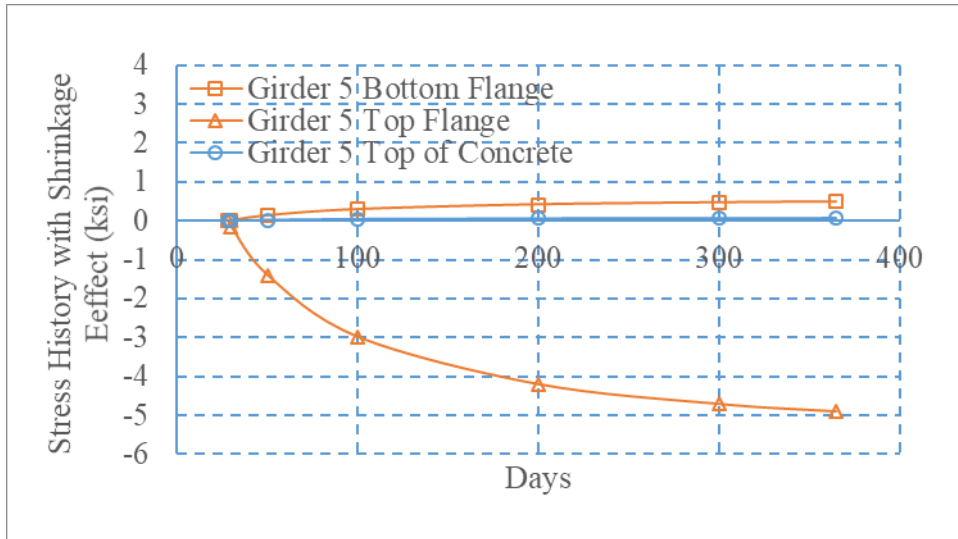
Figure 11-93. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 120 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
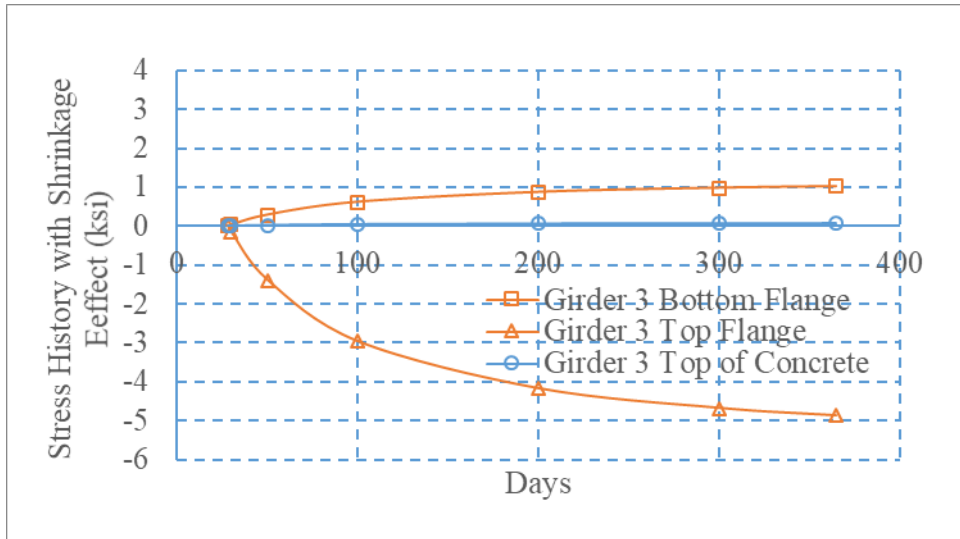
C-10 160' SPAN WITH UNSHORED CONDITION

This section summarizes the FE analysis results for a 160 ft single span steel composite girder bridge with unshored condition.

Table 11-46. Results for girder 1 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	4.8488	-7.205	0
2	Wet concrete	steel section	13.752	-20.509	0
3	DC2	composite section	7.6363	-1.991	-0.38507
4	Creep(365 days)	composite section	1.838	-10.7073	0.70693
5	Shrinkage (365 days)	composite section	0.90681	-8.7116	0.23151
6	Total		28.98191	-49.1239	0.55337

Table 11-47. Results for girder 2 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	4.8546	-7.213	0
2	Wet concrete	steel section	14.572	-21.686	0
3	DC2	composite section	7.0113	-2.0527	-0.36929
4	Creep(365 days)	composite section	1.7	-10.5914	0.70693
5	Shrinkage (365 days)	composite section	1.4302	-8.6891	0.23432
6	Total		29.5681	-50.2322	0.57196

Table 11-48. Results for girder 3 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	4.4516	-7.4138	0
2	Wet concrete	steel section	14.805	-21.94	0
3	DC2	composite section	6.7822	-2.0732	-0.36307
4	Creep(365 days)	composite section	1.655	-10.5531	0.71208
5	Shrinkage (365 days)	composite section	1.6073	-8.6881	0.2349
6	Total		29.3011	-50.6682	0.58391

Table 11-49. Results for girder 4 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	4.8565	-7.2171	0
2	Wet concrete	steel section	14.537	-21.452	0
3	DC2	composite section	7.0606	-2.0573	-0.36736
4	Creep(365 days)	composite section	1.689	-10.6161	0.70932
5	Shrinkage (365 days)	composite section	1.4132	-8.6886	0.23422
6	Total		29.5563	-50.0311	0.57618

Table 11-50. Results for girder 5 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	4.8471	-7.2008	0
2	Wet concrete	steel section	13.69	-20.219	0
3	DC2	composite section	7.7108	-1.9929	
4	Creep(365 days)	composite section	1.811	-10.7564	0.70176
5	Shrinkage (365 days)	composite section	0.87159	-8.7083	0.23168
6	Total		28.93049	-48.8774	0.93344

Figure 11-94. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-95. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
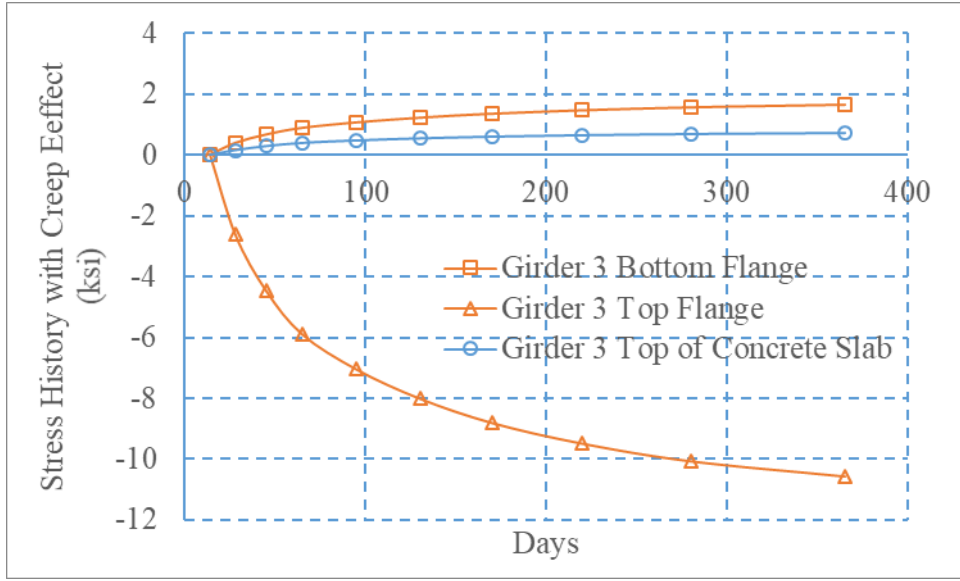
Figure 11-96. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
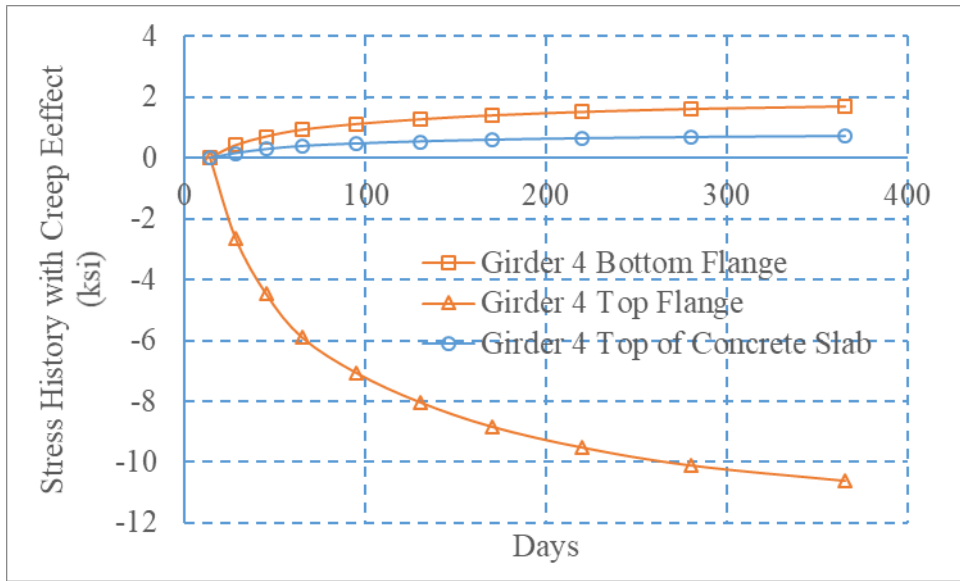
Figure 11-97. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
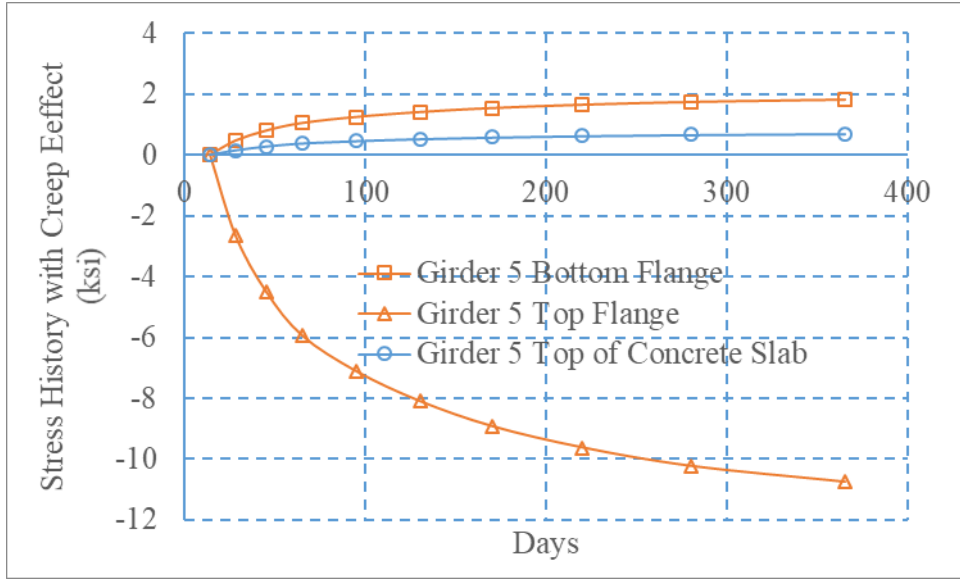
Figure 11-98. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
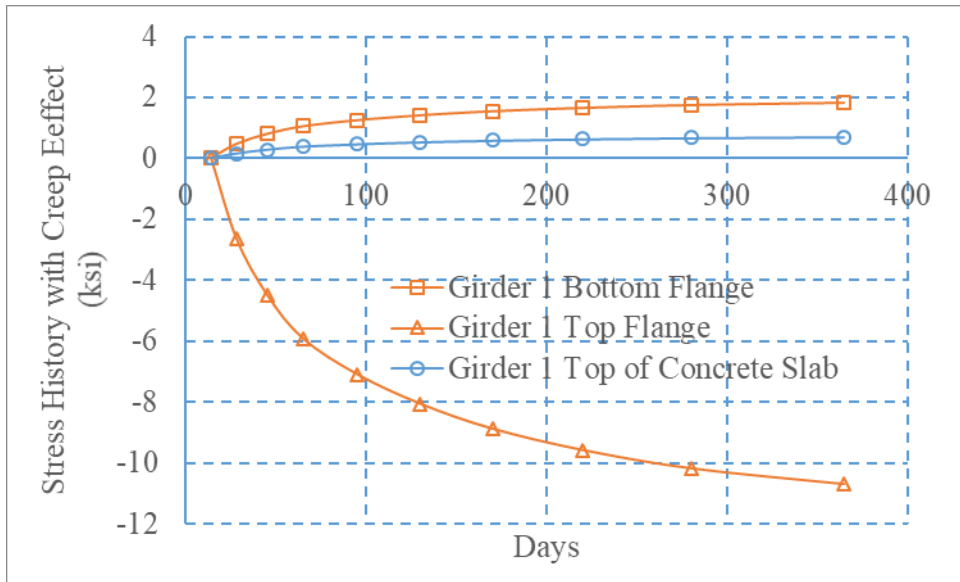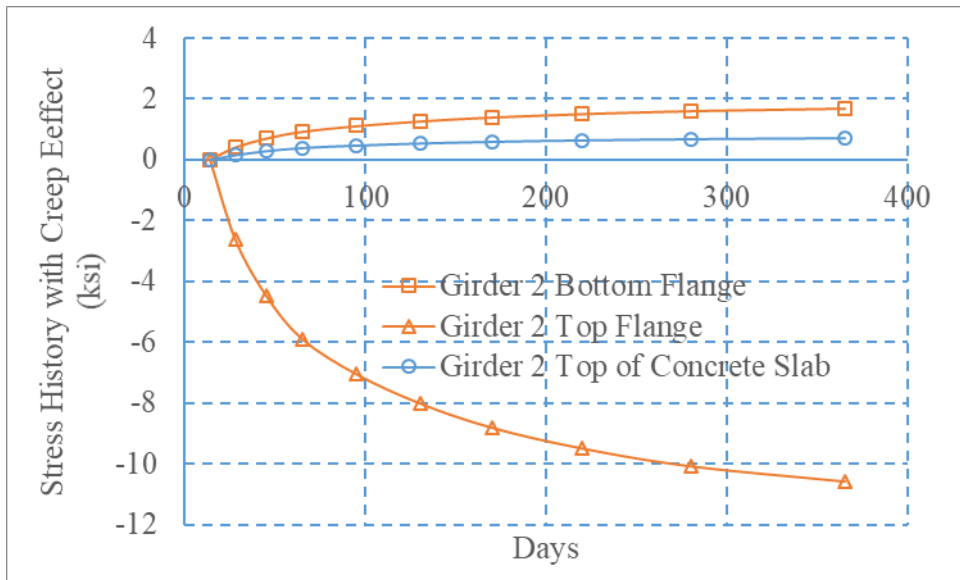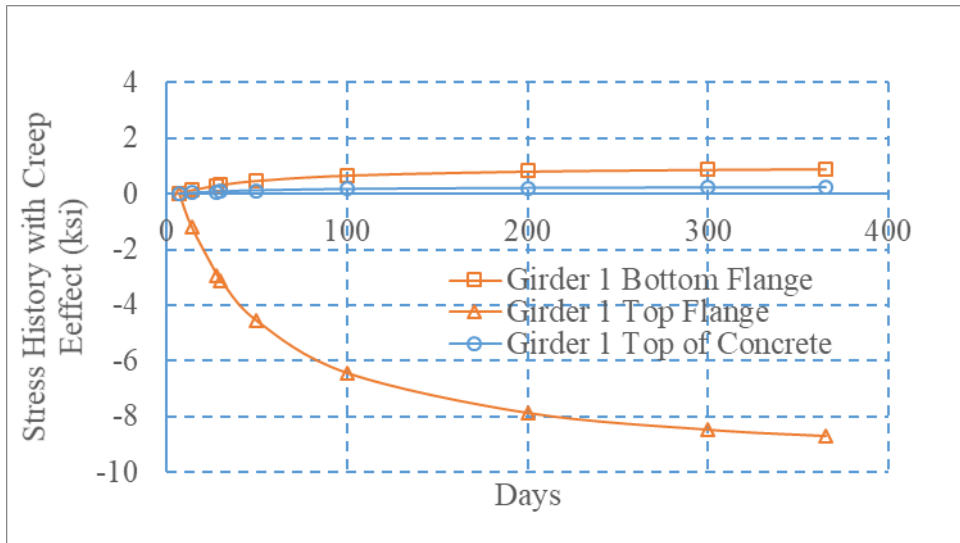

Figure 11-99. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
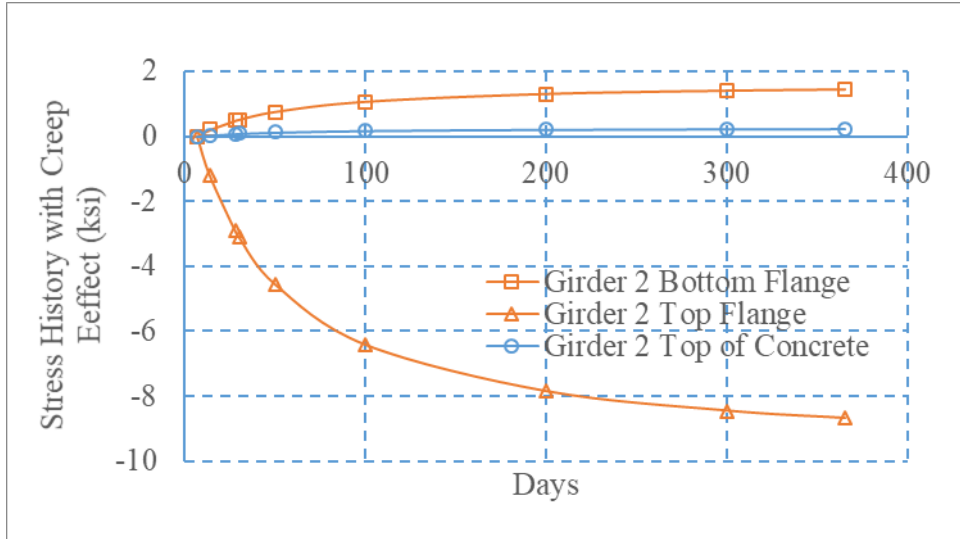
Figure 11-100. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

Figure 11-101. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)

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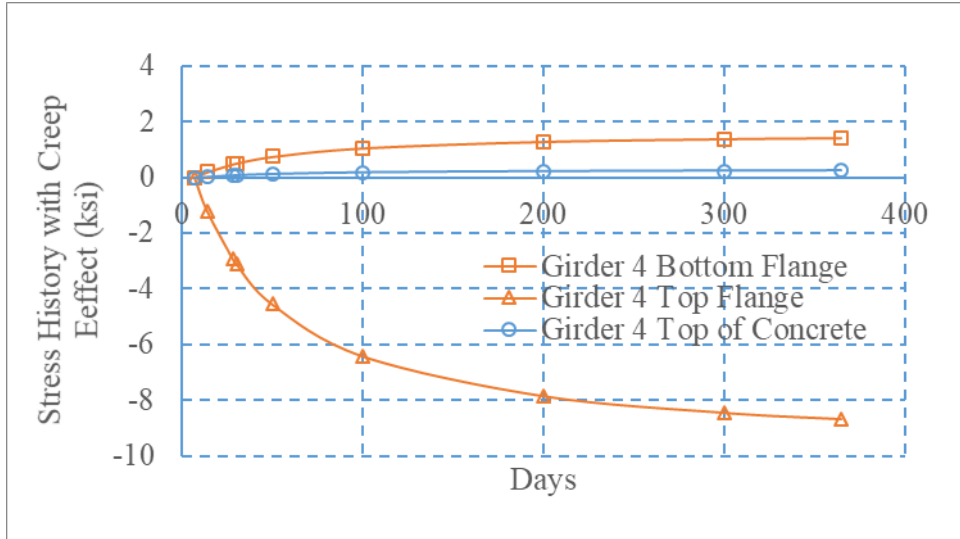
Figure 11-102. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
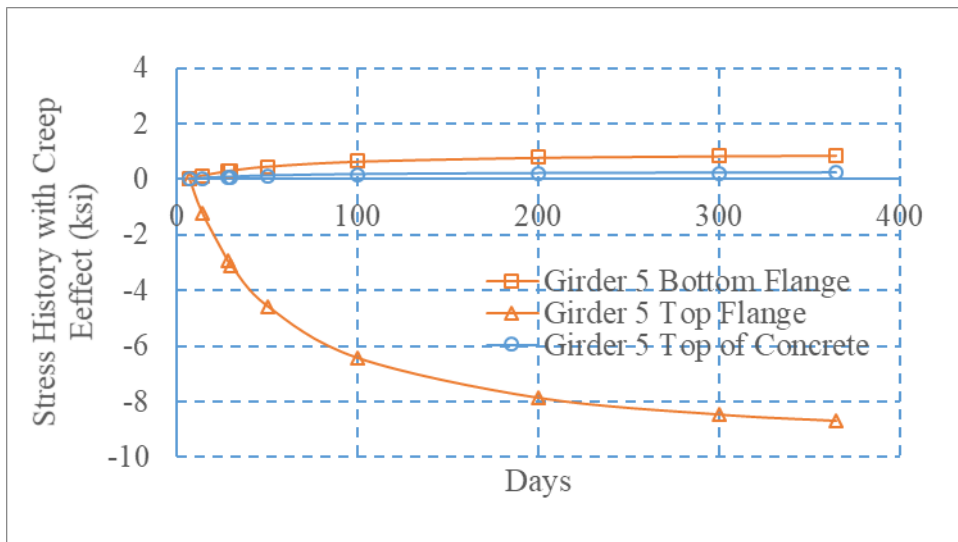
Figure 11-103. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, unshored)
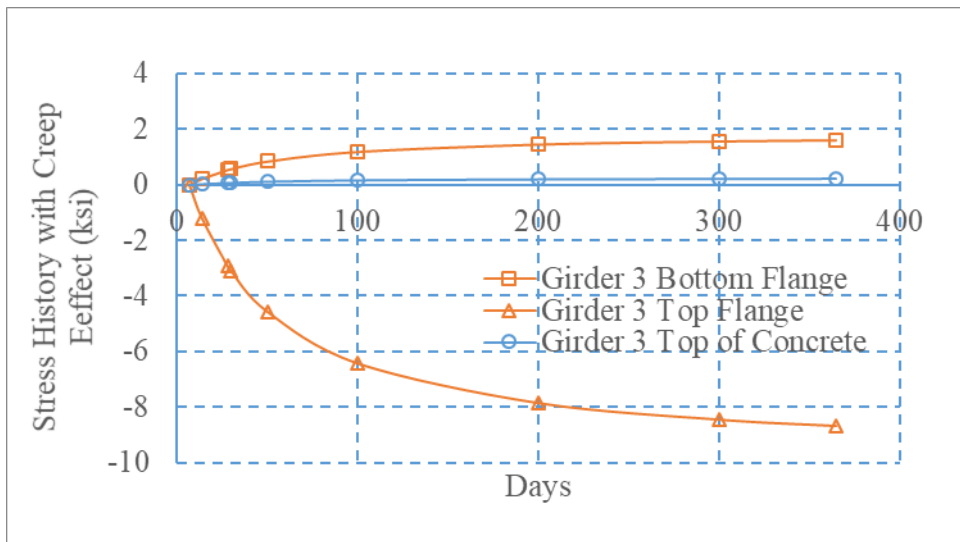
C-11 160' SPAN WITH SHORING AT 1/3 POINTS

This section summarizes the FE analysis results for a 160 ft single span steel composite girder bridge with shoring at 1/3 points.

Table 11-51. Results for girder 1 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.111	-0.163	0.000
2	Wet concrete	steel section	0.281	-0.393	0.000
3	DC2	composite section	0.260	-0.078	-0.018
4	Creep(365 days)	composite section	25.911	-14.907	-0.765
5	Shrinkage (365 days)	composite section	0.477	-4.583	0.122
6	Total		27.040	-20.123	-0.662

Table 11-52. Results for girder 2 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.110	-0.162	0.000
2	Wet concrete	steel section	0.389	-0.547	0.000
3	DC2	composite section	0.133	-0.040	-0.006
4	Creep(365 days)	composite section	25.911	-14.907	-0.759
5	Shrinkage (365 days)	composite section	0.752	-4.571	0.123
6	Total		27.296	-20.226	-0.641

Table 11-53. Results for girder 3 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.097	-0.168	0.000
2	Wet concrete	steel section	0.381	-0.535	0.000
3	DC2	composite section	0.101	-0.018	-0.004
4	Creep(365 days)	composite section	25.854	-14.736	-0.754
5	Shrinkage (365 days)	composite section	0.846	-4.571	0.124
6	Total		27.278	-20.029	-0.634

Table 11-54. Results for girder 4 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.110	-0.162	0.000
2	Wet concrete	steel section	0.390	-0.551	0.000
3	DC2	composite section	0.139	-0.037	-0.005
4	Creep(365 days)	composite section	25.848	-14.798	-0.758
5	Shrinkage (365 days)	composite section	1.413	-8.689	0.234
6	Total		27.900	-24.237	-0.528

Table 11-55. Results for girder 5 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

| #	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.111	-0.162	0.000
2	Wet concrete	steel section	0.281	-0.397	0.000
3	DC2	composite section	0.268	-0.072	-0.017
4	Creep(365 days)	composite section	25.862	-14.940	-0.770
5	Shrinkage (365 days)	composite section	0.872	-8.708	0.232
6	Total		27.393	-24.279	-0.556

Figure 11-104. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-105. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-106. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
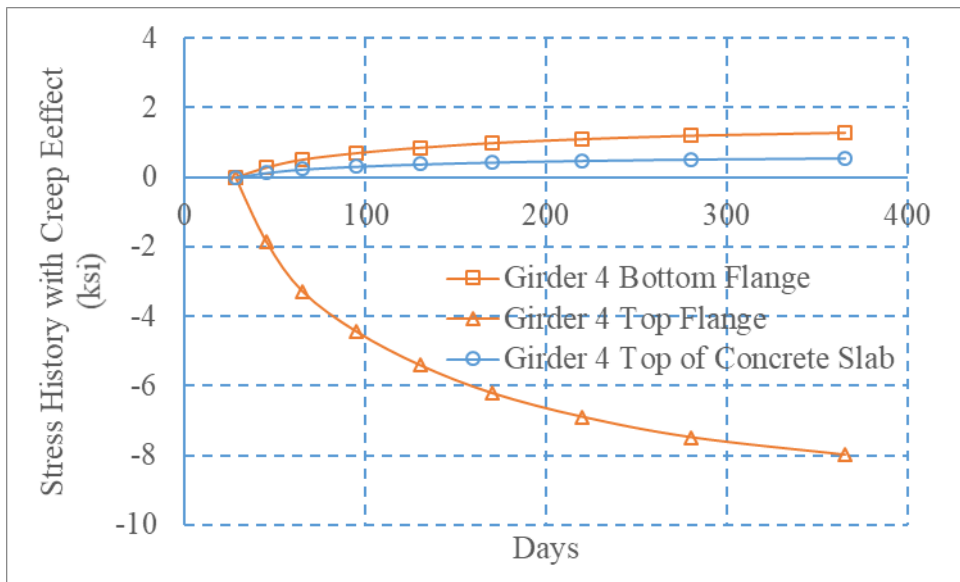
Figure 11-107. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
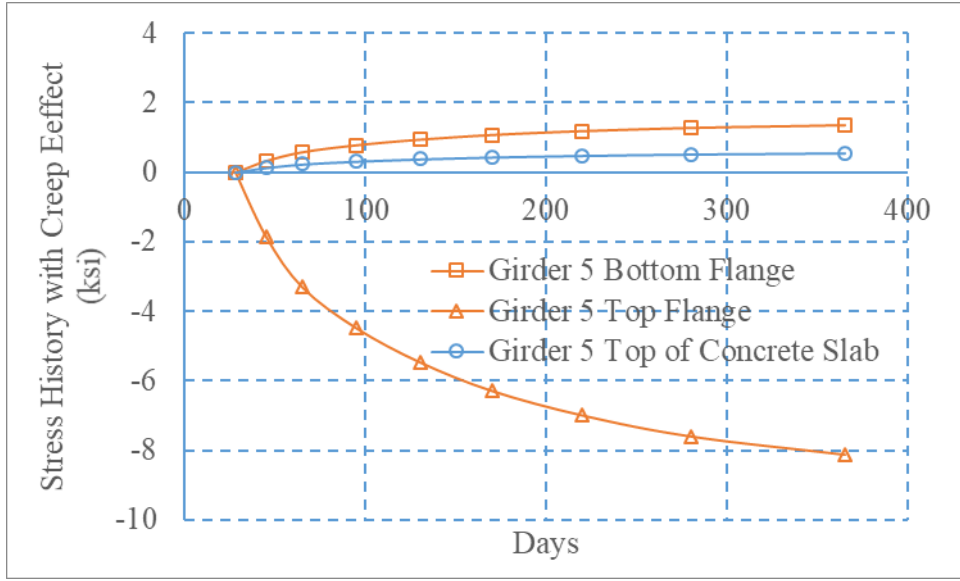
Figure 11-108. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
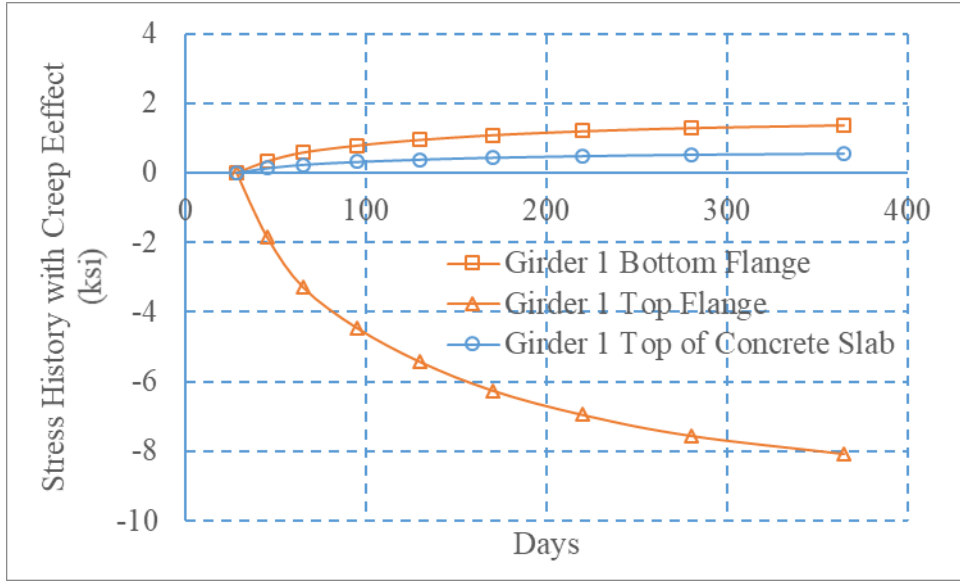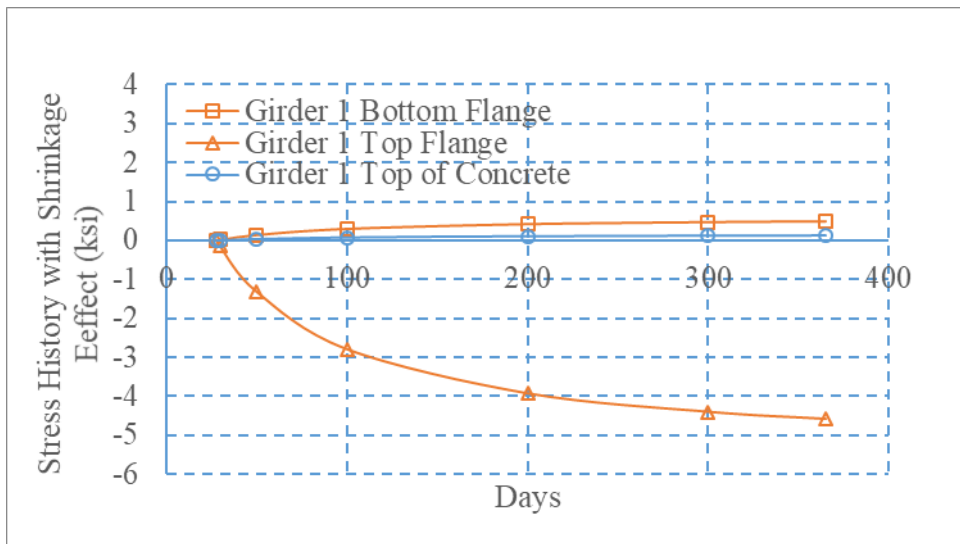

Figure 11-109. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
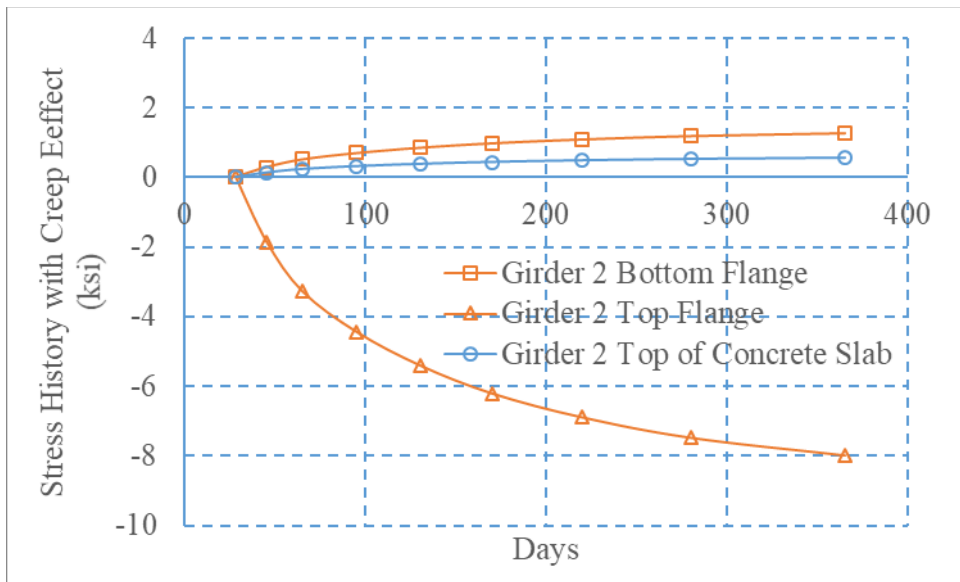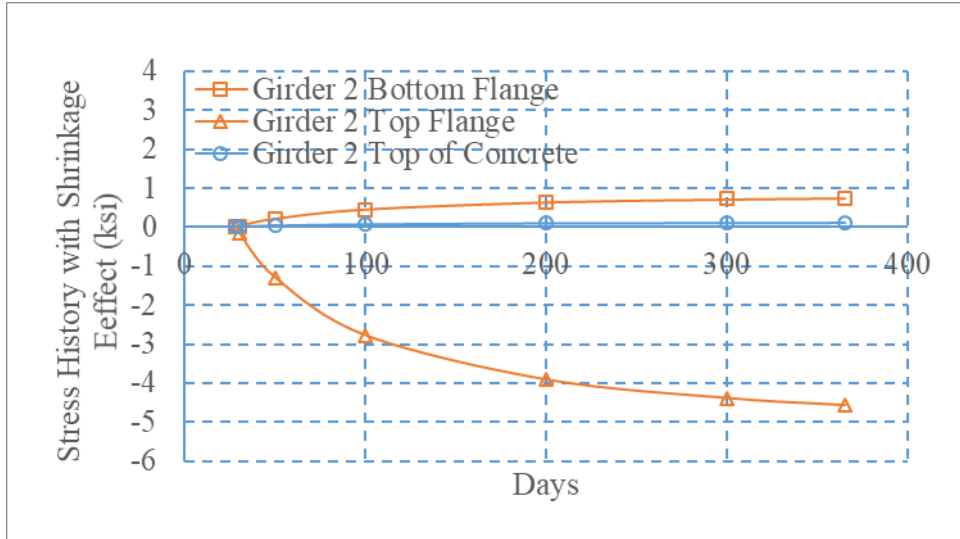
Figure 11-110. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
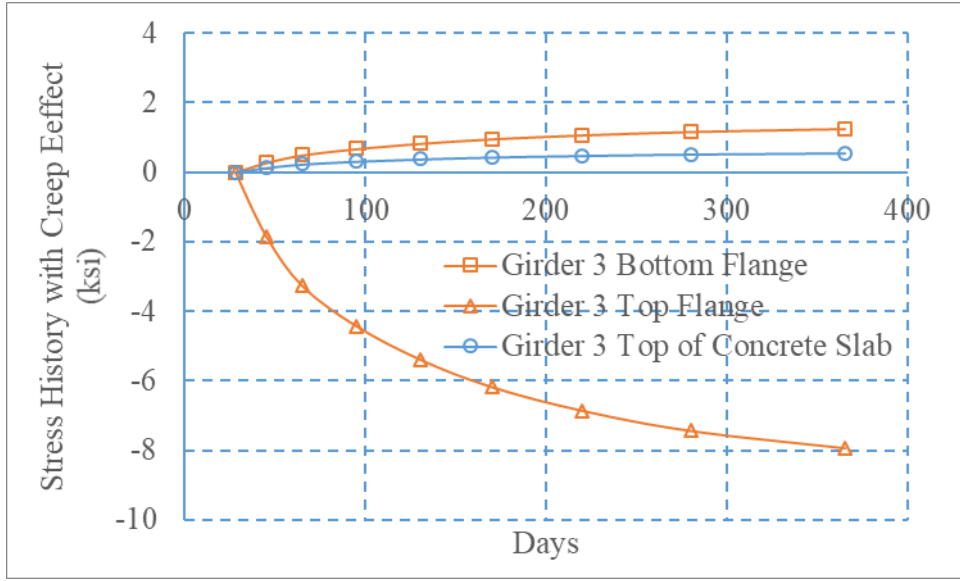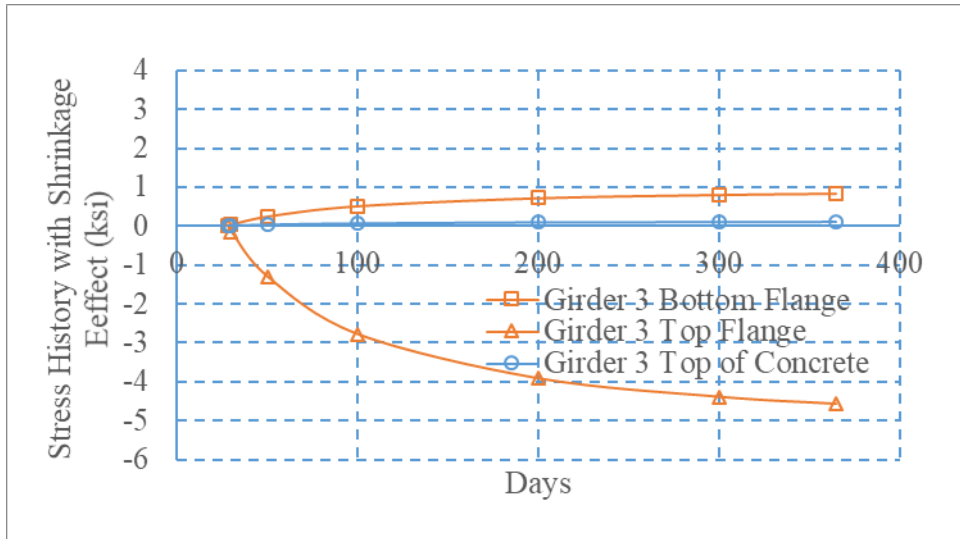

Figure 11-111. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-112. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)

Figure 11-113. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, shoring at 1/3 points)
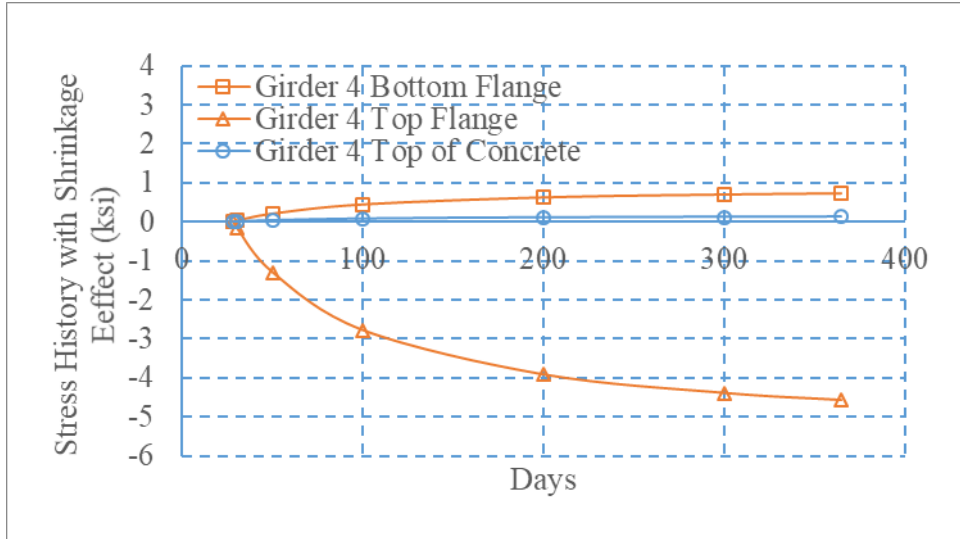
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C-12 160' SPAN WITH FULLY SHORED CONDITION

This section summarizes the FE analysis results for a 160 ft single span steel composite girder bridge with fully shored condition.

Table 11-56. Results for girder 1 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.260	-0.078	-0.018
4	Creep(365 days)	composite section	25.911	-14.907	-0.765
5	Shrinkage (365 days)	composite section	0.477	-4.583	0.122
6	Total		26.648	-19.567	-0.662

Table 11-57. Results for girder 2 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.133	-0.040	-0.006
4	Creep(365 days)	composite section	25.911	-14.907	-0.759
5	Shrinkage (365 days)	composite section	0.752	-4.571	0.123
6	Total		26.797	-19.518	-0.641

Table 11-58. Results for girder 3 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.101	-0.018	-0.004
4	Creep(365 days)	composite section	25.854	-14.736	-0.754
5	Shrinkage (365 days)	composite section	0.846	-4.571	0.124
6	Total		26.800	-19.325	-0.634

Table 11-59. Results for girder 4 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.139	-0.037	-0.005
4	Creep(365 days)	composite section	25.848	-14.798	-0.758
5	Shrinkage (365 days)	composite section	1.413	-8.689	0.234
6	Total		27.400	-23.524	-0.528

Table 11-60. Results for girder 5 (a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.000	0.000	0.000
2	Wet concrete	steel section	0.000	0.000	0.000
3	DC2	composite section	0.268	-0.072	-0.017
4	Creep(365 days)	composite section	25.862	-14.940	-0.770
5	Shrinkage (365 days)	composite section	0.872	-8.708	0.232
6	Total		27.002	-23.720	-0.556

Figure 11-114. Time history for stress along longitudinal direction for girder #1 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
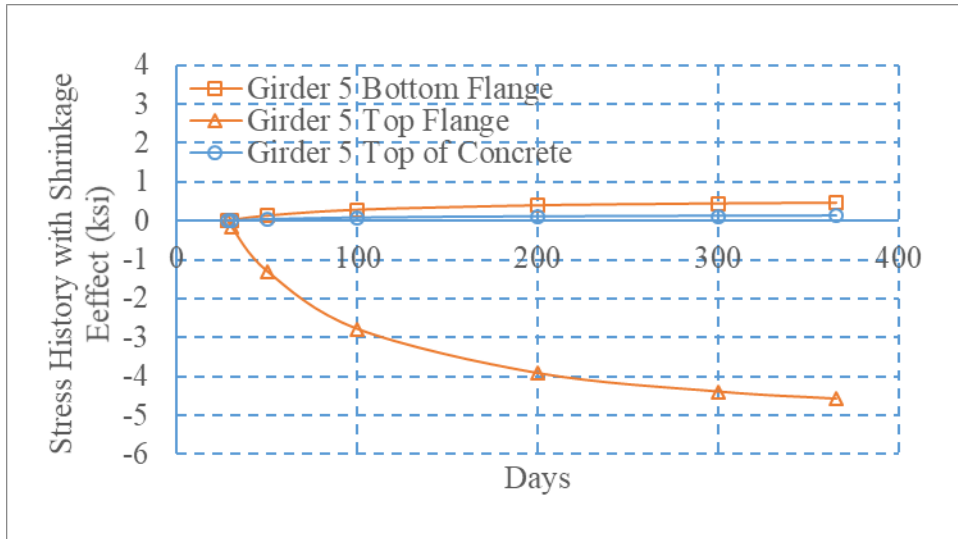

Figure 11-115. Time history for stress along longitudinal direction for girder #2 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-116. Time history for stress along longitudinal direction for girder #3 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-117. Time history for stress along longitudinal direction for girder #4 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
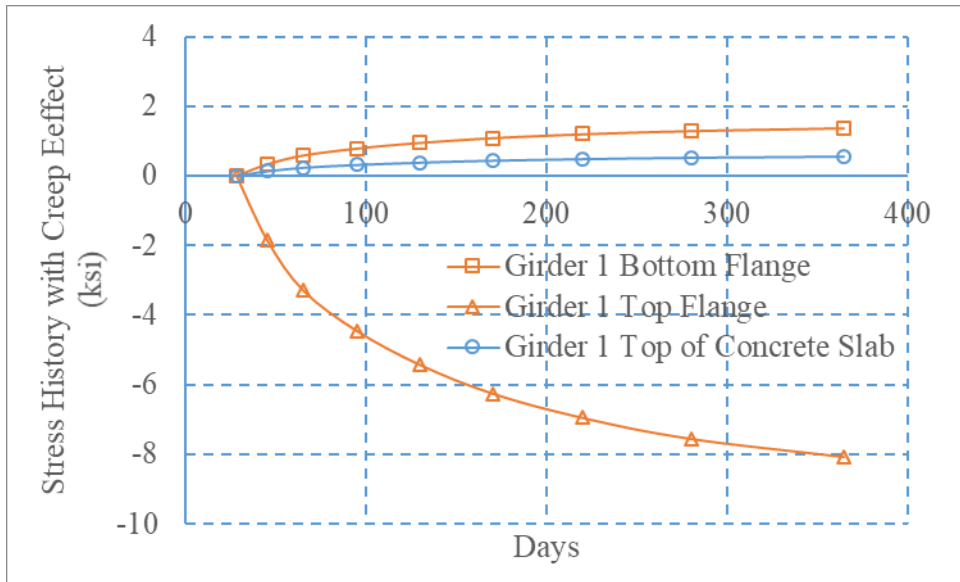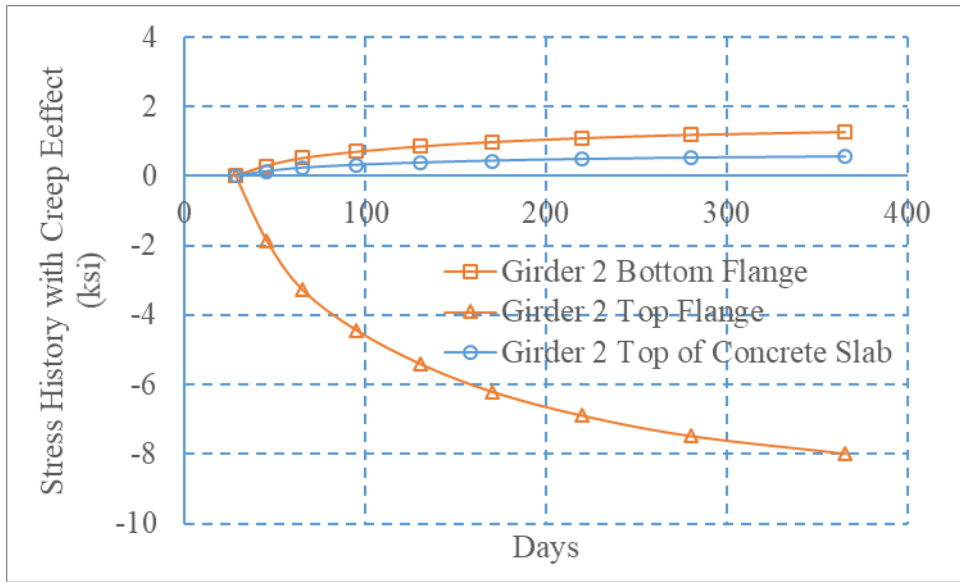
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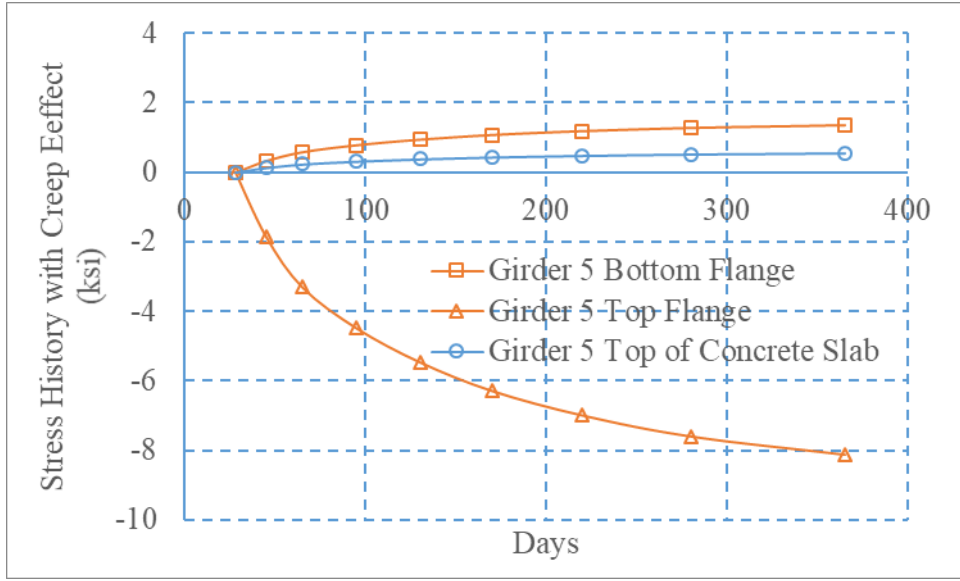
Figure 11-118. Time history for stress along longitudinal direction for girder #5 at midspan (creep effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
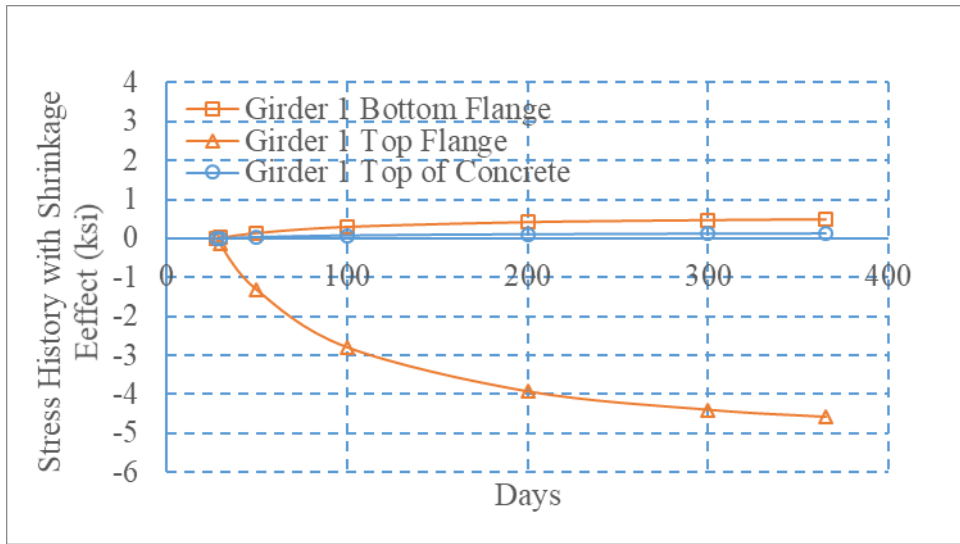
Figure 11-119. Time history for stress along longitudinal direction for girder #1 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
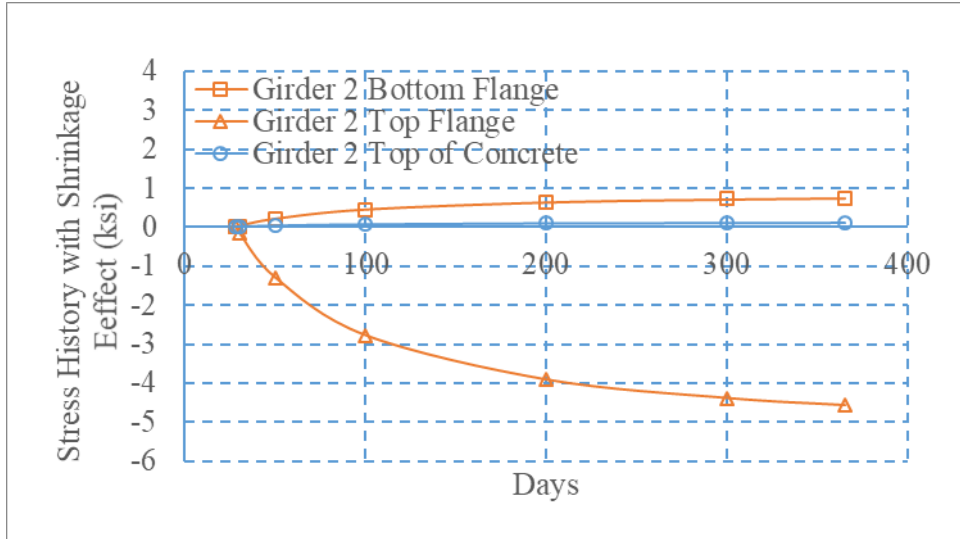
Figure 11-120. Time history for stress along longitudinal direction for girder #2 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
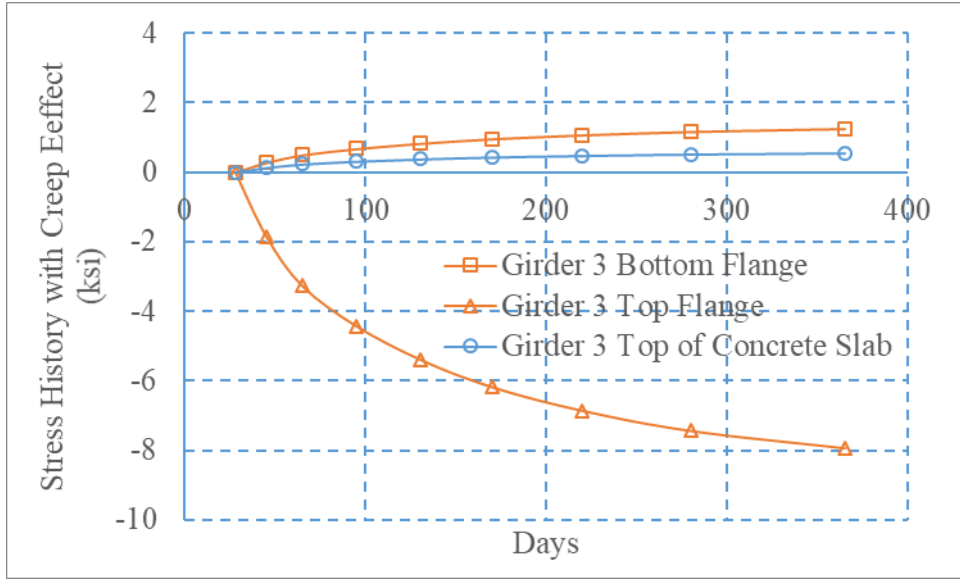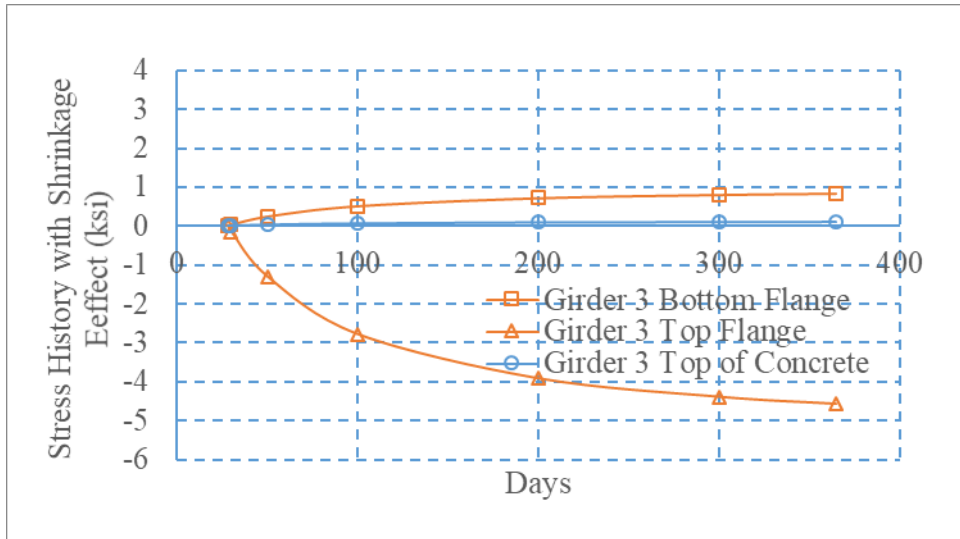

Figure 11-121. Time history for stress along longitudinal direction for girder #3 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)

Figure 11-122. Time history for stress along longitudinal direction for girder #4 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
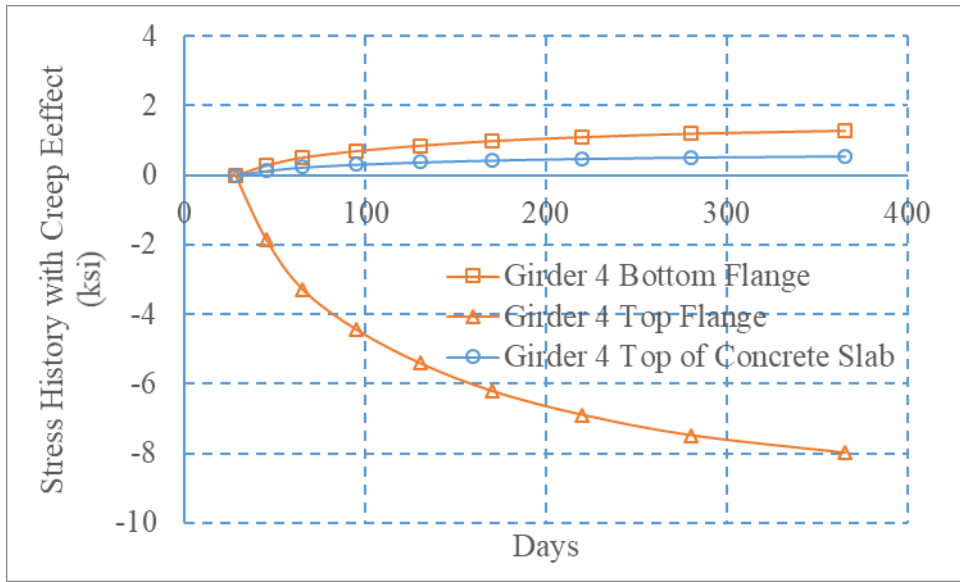

Figure 11-123. Time history for stress along longitudinal direction for girder #5 at midspan (shrinkage effect, a 160 ft single span plate girder bridge with 10' spacing, five-girder system, fully shored)
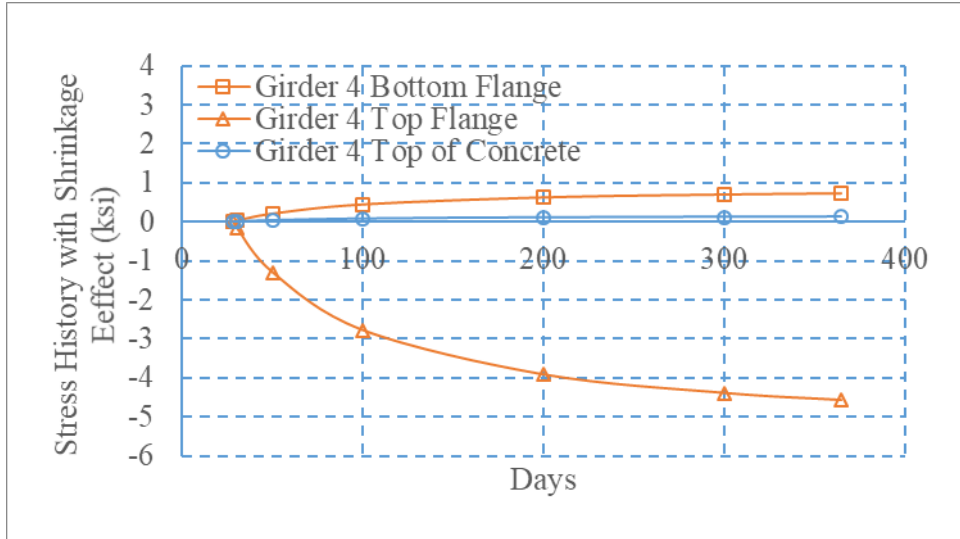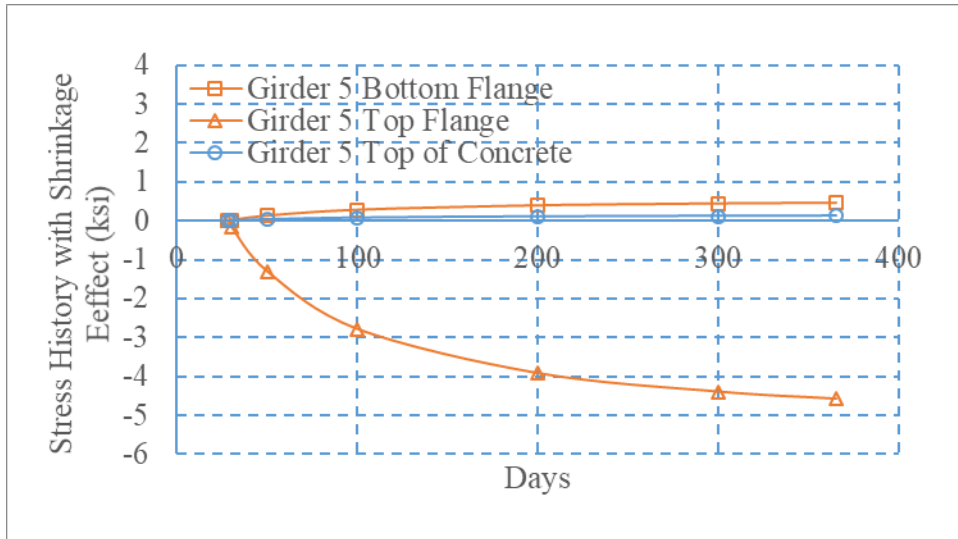
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12 APPENDIX D: FE ANALYSIS RESULTS FOR THREE-SPAN-CONTINUOUS BRIDGES (199'-255'-199')

Table 12-1 through Table 12-3 show the stresses at midspan of span 2 of interior girder #3 for a 199'-255'-199' three-span continuous bridge with three shoring conditions: (1) unshored, (2) shored at 1/3 points of center span and ½ points of side spans with FDOT specified pouring sequence, and (3) shored at 1/3 points of center span and ½ points of side spans without FDOT specified pouring sequence.

Table 12-1. Stresses for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at midspan of span 2, girder #3 (unshored)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	3.80	-3.94	0.00
2	Pour#1	steel section	-1.34	1.46	0.00
3	Pour#2	steel section	-5.34	5.37	0.00
4	Pour#3	steel section	28.71	-14.93	0.00
5	Pour#4	steel section	-0.99	5.96	-0.34
6	Pour#5	steel section	-1.84	0.35	0.04
7	DC2+WS	composite section	5.35	-0.70	-0.13
	Subtotal		28.35	-6.43	-0.44
8	Creep(1000 days)	composite section	0.76	-2.07	0.04
	Subtotal		29.10 (23.36)*	-8.51 (-12.35)	-0.40
9	Shrinkage (1000 days)	composite section	-11.86	-7.55	0.64
	Total		17.24	-16.05	0.25

* In Table 12-1 through Table 12-3, the values in parentheses are design values based on 3n calculations.

Table 12-2. Stresses for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at midspan of span 2, girder #3 (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.502	-0.510	0.000
2	Pour#1	steel section	0.538	-0.578	0.000
3	Pour#2	steel section	0.581	-0.637	0.000
4	Pour#3	steel section	3.917	-1.431	0.000
5	Pour#4	steel section	-0.995	1.460	-0.072
6	Pour#5	steel section	-0.644	0.097	0.010
7	Removal of Temporary Supports (28 days)	composite section	13.485	-3.740	-0.420
	Subtotal		17.38	-5.34	-0.48
8	Creep(1000 days)	composite section	0.745	-2.036	0.063
	Subtotal		18.13 (15.28)	-7.37 (-19.1)	-0.42
9	Shrinkage (1000 days)	composite section	-7.421	-5.057	0.419
	Total		10.71	-12.43	0.00

Table 12-3. Stresses for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at midspan of span 2, girder #3, no pouring sequence (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at midspan (ksi)	fst at midspan (ksi)	fct at midspan (ksi)
1	Self-weight of the girder	steel section	0.500	-0.510	0.000
2	Wet Concrete	steel section	2.570	-1.005	0.000
3	Removal of Temporary Supports (28 days)	composite section	13.187	-3.652	-0.425
	Subtotal		16.256	-5.167	-0.425
4	Creep(1000 days)	composite section	0.739	-2.063	0.061
	Subtotal		16.995 (15.28)	-7.230 (-19.1)	-0.364
5	Shrinkage (1000 days)	composite section	-8.043	-5.120	0.424
	Total		8.952	-12.350	0.060

Table 12-4 through Table 12-6 show the results at intermediate support of girder #3 for unshored, shored at 1/3 points of center span and ½ points of side spans following FDOT specified pouring sequence, and shored at 1/3 points of center span and ½ points of side spans without following FDOT specified pouring sequence, respectively.

Table 12-4. Stresses for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3 (unshored)

#	Load Case	Load carrying element	fsb at support (ksi)	fst at support (ksi)	fct at support (ksi)
1	Self-weight of the girder	steel section	-4.162	4.886	0.000
2	Pour#1	steel section	1.950	-2.159	0.000
3	Pour#2	steel section	-6.340	8.044	0.000
4	Pour#3	steel section	-12.493	11.939	0.000
5	Pour#4	steel section	-1.998	-5.981	0.000
6	Pour#5	steel section	-0.681	-10.058	0.000
7	DC2+WS	composite section	-6.467	1.782	0.323
	Subtotal		-30.190	8.452	0.323
8	Creep(1000 days)	composite section	-0.607	3.644	-0.232
	Subtotal		-30.797 (-27.82)*	12.096 (34.02)	0.091
9	Shrinkage (1000 days)	composite section	-4.518	-6.841	0.743
	Total		-35.315	5.255	0.834

* In Table 12-4 through Table 12-6, the values in parentheses are design values based on AASHTO 6.10.1.1.1c.

Table 12-5. Stresses for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3 (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at support (ksi)	fst at support (ksi)	fct at support (ksi)
1	Self-weight of the girder	steel section	-0.717	0.804	0.000
2	Pour#1	steel section	0.004	0.105	0.000
3	Pour#2	steel section	-0.243	0.312	0.000
4	Pour#3	steel section	-0.182	-0.249	0.000
5	Pour#4	steel section	-1.917	1.637	0.000
6	Pour#5	steel section	0.050	-1.703	0.000
7	Removal of Temporary Supports (28 days)	composite section	-12.384	9.130	1.177
	Subtotal		-15.39	10.04	1.18
8	Creep(1000 days)	composite section	-0.480	3.750	-0.264
	Subtotal		-15.87 (-28.05)	13.79 (35.17)	0.91
9	Shrinkage (1000 days)	composite section	-2.970	-4.457	0.488
	Total		-18.84	9.33	1.40

Table 12-6. Stresses for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3, no pouring sequence (shored at 1/3 points of center span and ½ points of side spans)

#	Load Case	Load carrying element	fsb at support (ksi)	fst at support (ksi)	fct at support (ksi)
1	Self-weight of the girder	steel section	-0.706	0.818	0.000
2	Wet Concrete	steel section	-2.414	1.889	0.000
3	Removal of Temporary Supports (28 days)	composite section	-12.384	9.295	1.199
	Subtotal		-15.504	12.003	1.199
4	Creep(1000 days)	composite section	-0.465	3.707	-0.269
	Subtotal		-15.969 (-28.05)	15.709 (35.17)	0.930
5	Shrinkage (1000 days)	composite section	-3.017	-4.361	0.474
	Total		-18.986	11.349	1.403

Figure 12-1 shows the time history for stress at 1st Interior Support of girder 3 due to creep effect for a 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five girder system.
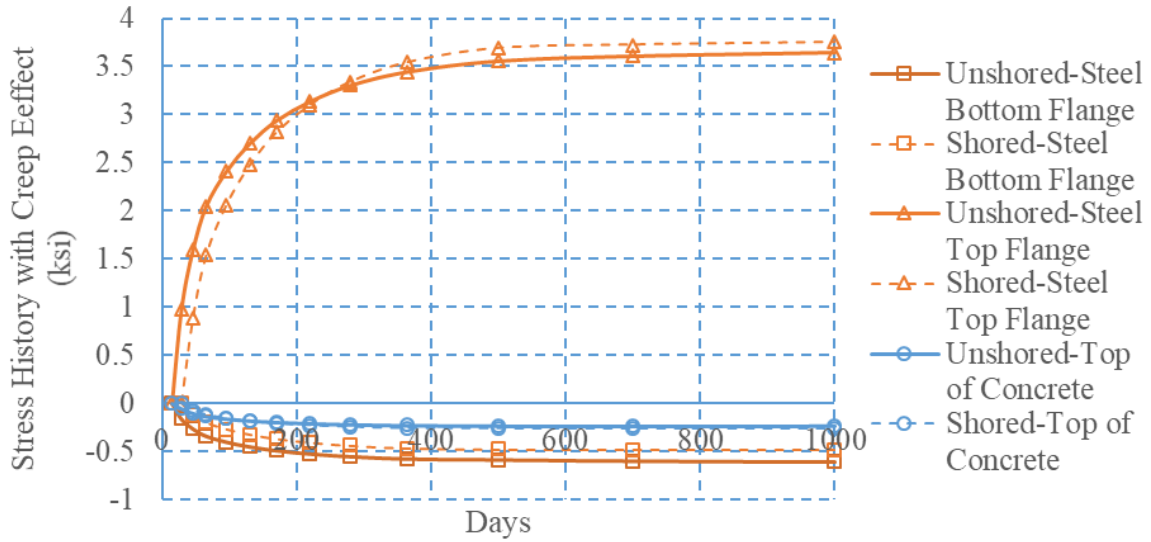

Figure 12-1. Time history for stress due to creep effect (199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system at 1st interior support, girder #3)

Figure 12-2 shows the longitudinal strain on the top of the concrete slab due to DC1 and DC2 loading. Figure 12-3 through Figure 12-5 show the stress and strain contour of the concrete slab after temporary supports was removed.
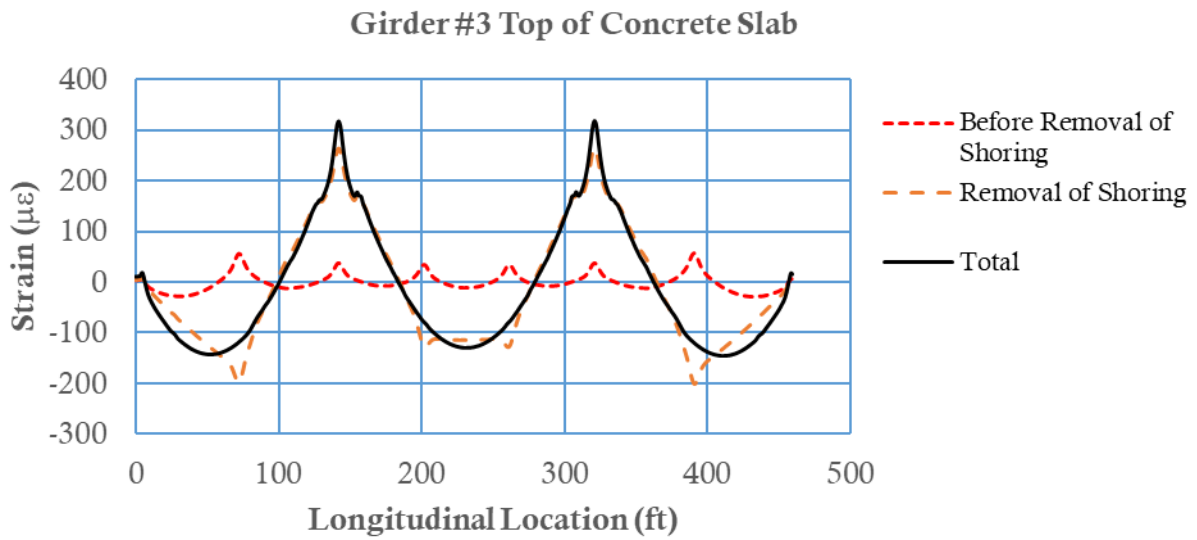
Figure 12-2. Longitudinal strain on top of concrete slab due to DC1 and DC2 loading (shored, 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system, girder #3)
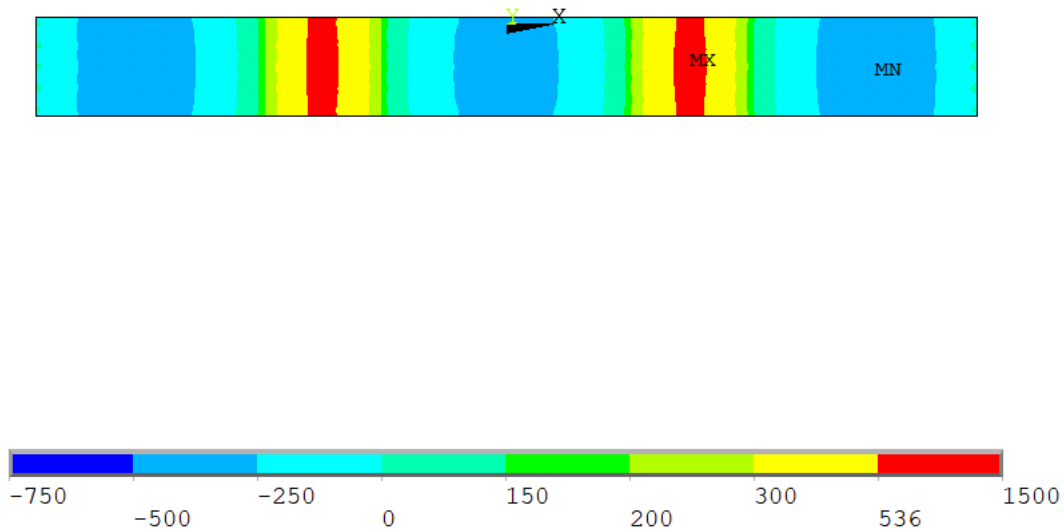

Figure 12-3. Stress contour of concrete slab after temporary supports was removed (shored, 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system, in psi)
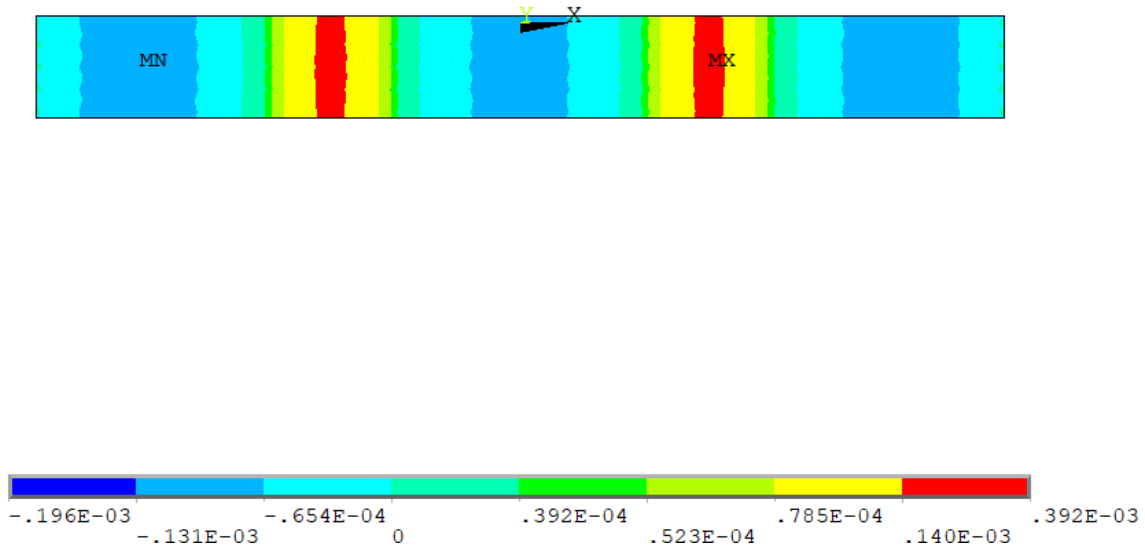
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Figure 12-4. Strain contour of concrete slab after temporary supports was removed (shored, 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system)
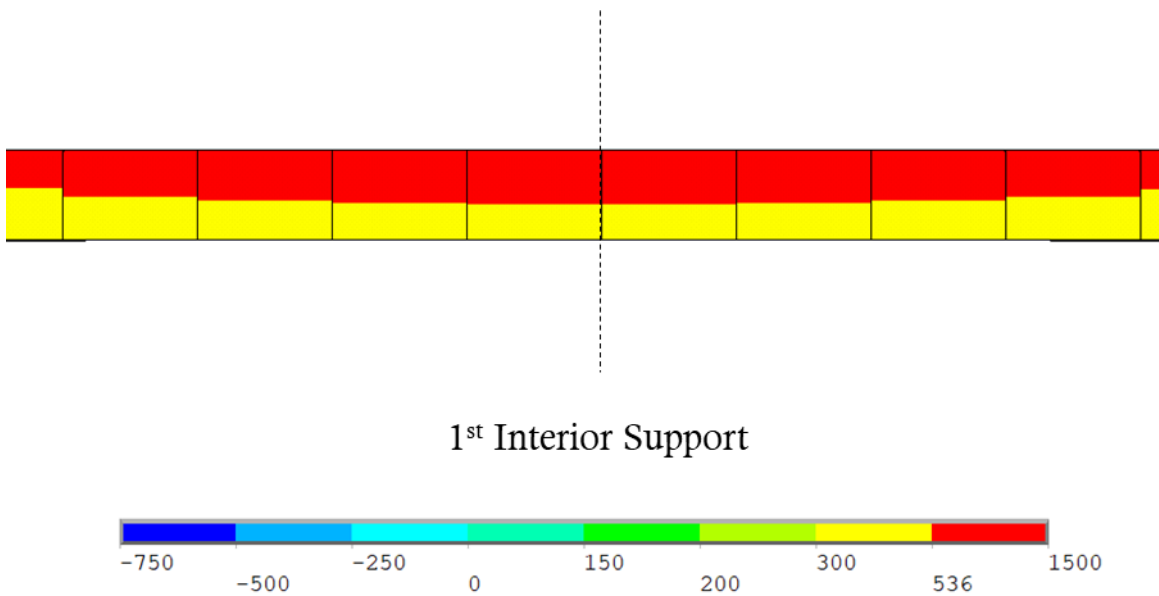

1st Interior Support

Figure 12-5. Stress contour of concrete slab at first interior support after temporary supports was removed (shored, 199'-255'-199' three span continuous plate girder bridge with 10.5' spacing, five-girder system, girder #3, length of the section is about 9ft, in psi)