DESIGN AND CONSTRUCTABILITY OF WIDE-FLANGE UHPC BRIDGE

A Thesis

by

ELDHO SHAJAN

Submitted to the Graduate and Professional School of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

Chair of Committee,	John B. Mander
Committee Members,	Matthew T. Yarnold
	Manish Dixit
Head of Department,	Zachary Grasley

August 2022

Major Subject: Civil Engineering

ABSTRACT

Construction of highway bridge girders using conventional prestressed concrete is a widely accepted norm in the state of Texas. But the limited life and the lengthy on-site construction work is a serious concern to the travelling public. This research looks into the design and constructability of slab-on-girder Ultra High-Performance Concrete (UHPC) bridge spans using different configurations. One associated feature is to minimize the number of field activities in the construction process. Therefore, the use of wide flanges of varying thicknesses are investigated. The thesis evaluates five UHPC design scenarios, which are compared to the standard TxDOT way of constructing highways with Tx54 girders using normal concrete. Because the wide flange topped girders are entirely cast off-site, deflection control is important to ensure the ride surface remains as flat as practicable. The different longitudinal girder-to-girder connection options are investigated. For each design, detailed estimates are made together with construction engineering schedules. Results show that the most economical solution that may be rapidly constructed following the principles of accelerated bridge construction (ABC) used a full-thickness flange. By contrast, the most enduring solution had a half-depth field cast topping which was transversely post-tensioned to actively tie all units together to mitigate the possibility of longitudinal cracking at the girder-to-girder connections.

ACKNOWLEDGEMENTS

I would like to thank my committee chair, Dr. John B. Mander for providing the guidance and recommendations vital to the successful completion of this thesis and for giving me the opportunity to work with him.

I am also thankful to my other committee members, Dr. Manish Dixit and Dr. Matthew T. Yarnold for their guidance and support throughout the course of this research. Dr. Zofia Rybkowski served as a substitute during the defense examination. I am grateful to her expert feedback during that process. I would also like to thank Dr. Pranav Pradeep Kumar for helping me out in organizing the thesis.

Thanks also go to my friends and colleagues and the department faculty and staff for making my time at Texas A&M University a great experience.

Finally, thanks to my parents for their encouragement and support.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Page

CHAPTER 1

INTRODUCTION

1.1 Background

Highway bridge construction is critical to the development of the overall civil infrastructure. Extensive use of prestressed concrete is common in Texas and elsewhere. Prestressing is the process to increase the flexural strength of concrete, to control deflections and to overcome the inherent tensile weakness of concrete by introducing permanent stresses in the concrete members.

The construction of highway bridges has seen a lot of changes over the past decades and the latest popular trend is to explore the viability and use of Ultra High-Performance Concrete (UHPC). Various experimental and analytical investigations have been done in the past exploring the merits and demerits of using UHPC for the construction of highway bridges. Currently there is a critical need for advanced building materials for the US domestic infrastructure, with a particular emphasis on accelerated bridge construction (ABC).

Building materials need to be increasingly more energy-efficient, sustainable, affordable, and cost-effective considering the entire life span of the structure. Comparison of various design considerations and a comprehensive cost analysis is essential to define the benefits of switching from an existing to a new material such as, to Ultra High-Performance Concrete from conventional concrete. The research conducted as part of this thesis addresses multiple questions including the feasibility of using UHPC for a prestressed bridge design and explores the chances of creating an accelerated bridge construction version of the design with minimum on-site deck casting. Adopting new methods of construction along with new materials is essential in the current scenario to complement the benefits of one another. With the adoption of UHPC for the construction of prestressed girders, wider flanges could be cast in the precast plants which will effectively reduce the necessity of using on-site formwork for deck casting.

Accelerated Bridge Construction (ABC) is a new approach promoted by various states and the Federal Highway Administration. Long being the norms for bridge building in the roadway industry, State Departments of Transportation are now able to use ABC principles to replace bridges within 48 to 72 hours. ABC procedures investigated herein follows the 'Prefabricated Bridge Elements and Systems Approach' (PBES), which potentially may save weeks of on-site construction time. This in-turn will result in indirectly saving hundreds of thousands of dollars in lane closure costs and associated expenses. ABC advocates a more effective use of work time. Prefabricated elements are typically constructed in a climate-controlled environment because of which weather only affects the work done on site. Weather delays become less frequent and disruption to traffic will be less. Since there is less disruption to traffic, fewer workers need to be exposed to traffic control and other work-related hazards.

1.2 Research Objectives

The research work conducted as part of this thesis is mainly focused on evaluating multiple prestressed wide flange design options and to conduct a comprehensive cost and time schedule analysis. The primary objective behind this analysis is to investigate several viable design options which adhere to ABC principles to determine the most promising candidates. To accomplish these objectives, the following tasks are completed:

- Review literature on existing models to analyze the strength and deflection calculations for prestressed concrete girders.
- Develop relationship between prestressing force and eccentricity to come up with viable design options for the various scenarios considered.
- Perform cost analyses for the different design options considered and derive the associated construction schedules.
- Explore the options to prevent cracking along longitudinal seams for the wet site-cast connections in girders.

1.3 Organization of the Thesis

The thesis is organized in multiple chapters. Chapter 2 presents a literature review that deals with prestressed concrete design, properties, and design considerations of UHPC and TxDOT guidelines for the construction of girders. Chapter 2 also presents details on existing methods to inhibit longitudinal cracks along girder connections.

Chapter 3 presents the different design options explored for prestressed UHPC girders with varying wide flange thicknesses. For the purpose of analysis, the prototype span considered is 120-ft long, 46-ft wide simply supported span; different design approaches are adopted to come up with multiple design models. The design options considered include the traditional prestressed concrete TX54 girder design using six girders with 4” precast panels and 4.5” normal concrete deck. This standard design is intended to serve as a comparative benchmark for the deflections, costs and schedule associated with the construction. All other design models are compared to the design of the regular TxDOT design with deck cast-on-site.

There are five other designs considered for the purpose of the research which includes topped and un-topped UHPC TX54 girders. The designs considered have different thicknesses of wide flanges cast in the prestress plant prior to transportation to the site. Deflection and stress calculations are made for each of these designs. The chapter presents options on how the girder-to-girder connection should be cast on site. A study on how to transversely post-tension a bridge deck to inhibit the possibility of longitudinal cracking is also presented for one design case.

Chapter 4 investigates the construction schedule and cost estimates involved with each of the design options considered in Chapter 3. One of the significant determinants when it comes to adopting a particular construction method is the indirect costs associated with lane closures and traffic disruptions. While investigating ABC options, it is expected to have a higher construction cost associated with the quickest construction. Additional material costs may be offset by minimizing lane closure delays. Chapter 4 provides a piecewise breakdown cost analysis and construction schedule for completion timing for each of the tasks involved.

Chapter 5 presents the overall findings of the research and suggests the most promising options under different construction circumstances. Comments on the benefits of using such a design for multi-span bridges and scope for future research are also given.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The main objective of this thesis is to evaluate the feasibility of constructing bridges using UHPC and to assess the merits and demerits of doing so. Accordingly, this chapter surveys the existing literature. In the past, there have been multiple studies that investigate the strength and material properties of UHPC (Graybeal 2006, Russel and Graybeal 2013, Deng et.al 2020). These research papers investigate different mechanical properties of Ultra High-Performance Concrete and provide guidelines on the estimation of the strength parameters used for various calculations in this research. The determination of stress blocks and deflection profiles for the different designs were also key for the investigation into the research questions. Previous design models on prestressed concrete design (Hueste and Mander 2012) were analyzed and loads were considered adhering to AASHTO LRFD design considerations. The deflection profiles calculated in this research use long term multipliers specified in existing literature (Martin 1977, Agarwal 2020). One major concern during the review phase of the literature was the absence of adequate material on the cost analysis and scheduling of bridge construction as most of the intricate details of the economics involved are industry know-how which remains inaccessible. However, the FHWA does provide guidelines on Accelerated Bridge Construction philosophy which have been adopted in this research.

2.2 Prestressed Concrete Design

Prestressed concrete design for bridge girder construction has been in use for a long time in the United States. The development of stress blocks to evaluate the strength of a prestressed design is described in detail in 'Prestressed Concrete: Analysis and Design' by Naaman (2004). The evaluation of the structural integrity of the girder is assessed from the stress block analysis for each of the critical sections in a simply supported girder. The load criteria used is AASHTO HL-93 loading, and cross-sectional properties were chosen based on what is generally adopted by TxDOT in accordance with AASHTO LRFD Specifications (AASHTO 2017).

Martin (1977) and Agarwal (2020) evaluated deflection calculations that incorporate material elasticity (EI) along with creep and shrinkage. Martin presents a comprehensive method for evaluating the long-term multipliers that effectively predict the camber and deflections of the prestressed concrete girders at each stage of construction. Although these multipliers take into consideration the material composition of regular concrete, experimental results have shown that the same equations could be used to estimate the deflections in UHPC calculations.

The new multipliers are shown in Table 2-1 and the following equations are used to determine the deflection of the girder at various stages of construction.

1. Deflection due to prestress only at transfer:

$$\Delta_{p,i} = \frac{F_i e_{01} L^2}{8E_{ci} I_0} + \frac{F(e_{c1} - e_{01}) L^2}{6E_{ci} I_0} \left(\frac{3}{4} - \alpha^2\right) \tag{2-1}$$

2. Deflection due to girder self-weight:

$$\Delta_{g,i} = \frac{5W_g L^3}{384E_{ci}I_0} \tag{2-2}$$

where F_i = initial prestressing force; e_{01} = eccentricity of the prestressed strands at endspan from the non-composite centroid; e_{c1} = eccentricity of the prestressed strands at the midspan from the non-composite centroid; L = length of the beam; E_{ci} = elastic modulus of precast concrete at transfer ($E_{ci} = 0.85E_c$); E_c = elastic modulus of concrete at 28 days; α = ratio of harping point to total length; I_0 = moment of inertia of girder; and W_g = total self-weight of girder.

3. Net deflection immediately after the release of prestress:

$$\Delta_i = -0.95\Delta_{p,i} + \Delta_{g,i} \tag{2-3}$$

where $\Delta_{p,i}$ = deflection due to prestress only at transfer as per Equation (2-1); and $\Delta_{g,i}$ = deflection due to girder self-weight as per Equation (2-2).

4. Net deflection during erection at 40-60 days:

$$\Delta_e = -(F_e)\Delta_{p,i}(C_e) + \Delta_{g,i}(C_e) \tag{2-4}$$

where F_e = force at erection (0.875 for 40-60 days curing period) and C_e = multiplier for at erection camber (From Table 2-1(1)).

5. Net deflection immediately after the placement of deck concrete:

$$\Delta_s = -(F_e)\Delta_{p,i}(C_e) + \Delta_{g,i}(C_e) + \Delta_d \tag{2-5}$$

where Δ_d = deflection due the additional deck load given by the following equation

$$\Delta_d = \frac{5W_d L^3}{384 E_c I_0} \qquad (2\text{-}6)$$

where W_d = total weight of the deck concrete.

6. Net long-term service state deflection under the effect of dead load and prestress forces:

$$\Delta_{LT} = -(F_e)\Delta_{p,i}(C_{LT-1}) + \Delta_{g,i}(C_{LT-1}) + \Delta_d(C_{LT-2}) + (\Delta F_e)\Delta_p(C_{LT-2}) + \Delta_{sidl}(C_{LT-3}) \qquad (2\text{-}7)$$

where Δ_p = deflection due to losses in prestress after the deck is cast given in Equation (2-8) and Δ_{sidl} = deflection due to the superimposed dead load given in Equation (2-9).

$$\Delta_p = \frac{F_i e_{02} L^2}{8 E_c I_0} + \frac{F(e_{c2} - e_{02}) L^2}{6 E_c I_c}\left(\frac{3}{4} - \alpha^2\right) \qquad (2\text{-}8)$$

$$\Delta_{sidl} = \frac{5 W_{sidl} L^3}{384 E_c I_c} \qquad (2\text{-}9)$$

where ΔF_e = additional losses in prestress after erection ($\Delta F_e = F_e - 0.8F_i$); C_{LT-1} = long term multiplier (from Table 2-1 (3)); C_{LT-2} = multiplier for long term deflection for composite deck concrete (from Table 2-1 (6)); and C_{LT-2} = multiplier for long term deflection for superimposed dead load (from Table 2-1 (5)).

The modified long-term multipliers corresponding to 40-60 days of curing period are shown in Table 2-1. This research adopts the above to minimize the deflections in the design of simply supported girders.

Table 2-1: Modified at-erection and long-term deflection multipliers (Agarwal 2020)

Construction Stage		Modified erection at 40-60 days
At Erection		
(1)	Downward deflection component – apply to elastic deflection due to the member weight at release of prestress.	1.85
(2)	Upward camber component – apply to the elastic camber due to prestress at the time of release of prestress.	1.85
Final Stage		
(3)	Downward deflection component – apply to elastic deflection calculated in (1) above	2.23
(4)	Upward camber component – apply to elastic deflection calculated in (2) above	2.23
(5)	Downward deflection component – apply to elastic deflection due to superimposed dead load only	3.00
(6)	Downward deflection component – apply to elastic deflection caused by the composite topping	1.89

2.3 UHPC – Strength and Material Properties

Graybeal (2006) extensively detailed how each of the material properties of UHPC is different from ordinary concrete and how the properties of UHPC changes with the curing method. Even though an optimum curing condition is rarely realized with cast-in place UHPC, high standard temperature curing facilities are available at a few precast plants in the northern United States as well as Canada. These conditions will ensure the proper materialization of the properties that UHPC is known for. The study conducted by Graybeal investigated the early age strength of UHPC for various curing conditions. The experimental observations detailed in the literature mentions the compressive strength of UHPC to be as high as 28 ksi under constrained prestress plant conditions. But for the purpose of this thesis, the strength of UHPC is considered to be 20 ksi in keeping with conditions in Texas.

There are multiple studies and empirical equations that investigates the modulus of elasticity of UHPC. The equation most used for normal strength and normal weight concrete is:

$$E_C = 57000\sqrt{f'_c} \qquad (2\text{-}10)$$

Kakizaki et.al (1992) showed that there is an observable variation in the modulus of elasticity once you surpass a threshold of compressive strength, and modified equation (2-10) as:

$$E_{UHPC} = 43920\sqrt{f'_c} \qquad (2\text{-}11)$$

This equation to determine the modulus of elasticity is valid for concrete specimens having a compression strength ranging from 12 to 20 ksi. In equation (2-10) and (2-11), f'_c is given in ksi.

Park (2015) noted that while UHPC has superior mechanical properties in terms of compressive and tensile strength, ductility and toughness, as well as exceptional durability and flowability, it is very essential to ensure strict quality control during the casting process. This was the motivation behind the investigation into how much of the girder casting can be done in the precast plant rather than on-site. The design proposals investigated in the thesis have been modelled to have a varying percentage of on-site casting. When most of the casting is completed within the factory, the associated site construction cost also comes down drastically.

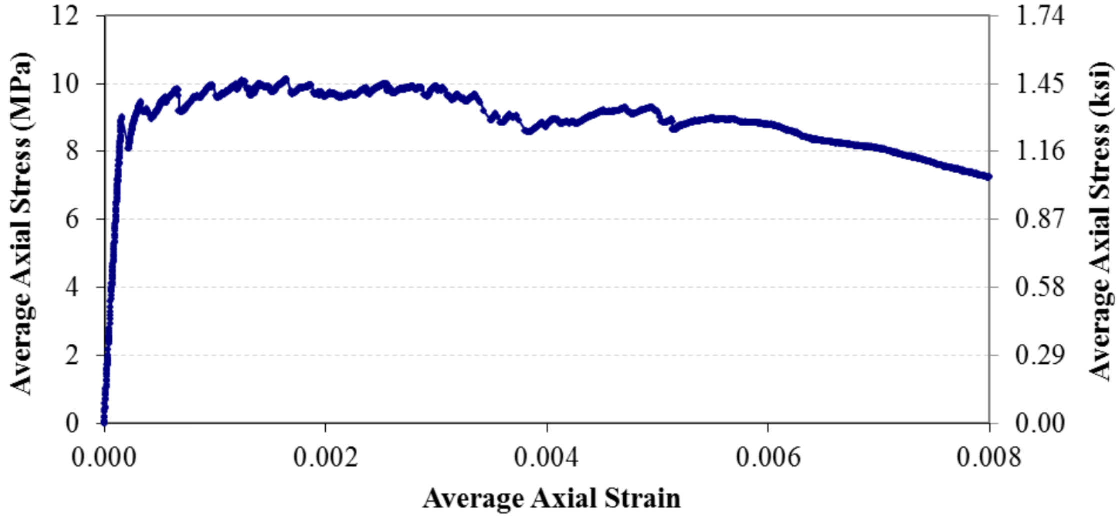
Russel and Graybeal (2013) compared the superior mechanical properties of UHPC with ordinary concrete in their technical report for the FHWA. The report discusses, in detail about the mechanical properties relevant to the structural design of UHPC. One can notice how they stressed on the importance of having the right dispersion and orientation of the fiber reinforcement. This again points to the necessity of bringing more of the casting process inside the prestress plant than on site. Graybeal reported the compressive strength of nearly 1000 specimens subjected to four different curing conditions. The average measured compressive strength at 28 days varied between 18.3 ksi and 28.0 ksi. The density of UHPC ranged from 150 lb/ft^3 to 156 lb/ft^3. Based on a regression analysis of the data he collected, Graybeal determined the compressive strength gain of Ultra High-Performance Concrete cured under laboratory conditions as

$$f'_{ct} = f'_c \left[1 - \exp\left(-\frac{t-0.9}{3}\right)^{0.6}\right] \qquad (2\text{-}12)$$

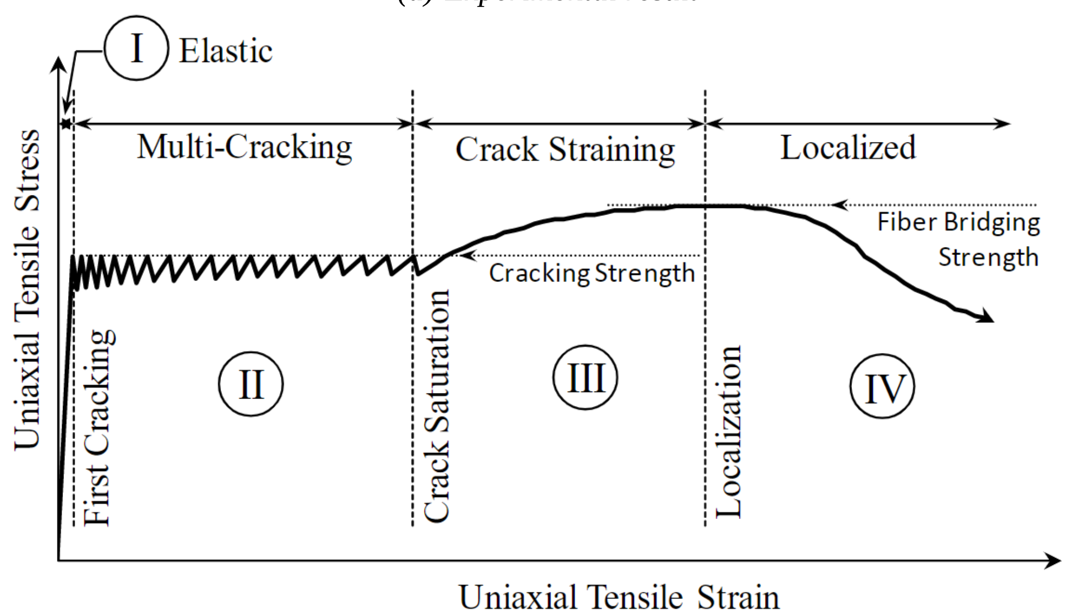
where f'_{ct} = UHPC compressive strength at t days; f'_c= UHPC compressive strength at 28 days; and t = time after casting in days.

Russel and Graybeal (2013) noted from their experimental tests, the superior tensile strength and toughness of UHPC over conventional concrete. The tensile strength of UHPC is somewhat higher (but still a square-root relationship) than that of conventional concrete, and UHPC can exhibit sustained tensile strength after first cracking. Graybeal has proposed an idealized tensile stress-strain response shown in Figure 2.1(a). An example stress-strain response obtained from a UHPC mix containing 2 percent steel fiber reinforcement by volume also gives one an idea about the tensile capacity of Ultra High-Performance Concrete, shown in Figure 2.1(b). The experiments conducted by Graybeal demonstrates an average tensile capacity of 1.3 ksi for a 20 ksi UHPC strength. This additional tensile capacity of UHPC, when used as deck overlay, maybe beneficial for negative moment regions in a multi-span bridge.

The study also demonstrated that UHPC has enhanced resistance for fatigue loading which is ideal for highway bridges. Experimental analysis conducted by Russel and Graybeal shows that UHPC has sufficient fatigue resistance in both tension and compression to resist several million cycles of loading. The report also shows how creep of UHPC is much less than conventional concrete. This in-turn results in reduced prestress losses. Most of the shrinkage in UHPC in autogenous shrinkage and a summary of the material properties of UHPC can be found in Table 2-2.



(a) Experimental result



(b) Idealized tensile stress-strain response

Figure 2-1: Experimental result and observation from the work of Russel and Graybeal (2013)

Table 2-2: Range of UHPC material properties (Russel and Graybeal 2013)

UHPC Material Property	Range
Compressive strength	140 to 200 MPa
Tensile cracking strength	6 to 10 MPa
Modulus of elasticity	40 to 70 GPa
Poisson’s ratio	0.2
Coefficient of thermal expansion	10 to 15 x 10^{-6}/°C
Creep coefficient	0.2 to 0.8
Specific creep	6 to 45 x 10^{-6}
Total shrinkage	Up to 900 microstrain

2.4 Previous Bridge Design Models

Hueste and Mander (2012) discussed about the highway bridge design in the state of Texas. The report details state-of-the art and state-of-the-practice literature and sources about the subject. The design model considers Tx70 and Texas U54 prestressed girders and lays out experiment data to support the material properties used for the design calculations. The report identifies potential key design considerations as the following: deflection; shear demand; moment demand and ultimate strength; flexure-shear interaction at supports; and serviceability stress under live loads.

Almost all these considerations are relevant for a simply supported span as considered in this thesis. The detailed design parameters, assumptions and stepwise procedure to evaluate the allowable stress limits are adopted in the calculations for the design options considered in this research. Hueste and Mander (2012) also describes the losses in prestress, concrete shrinkage, concrete creep, steel relaxation and elastic shortening that should be accounted for while performing calculations.

Gunasekaran (2020), in her thesis, also provided a comprehensive approach with design calculations on the state-of-practice bridge design methodology in Texas. Her work details design examples and points out the steps involved in the design process. The thesis provides comprehensive stress block diagrams illustrating the evolution of critical sections for the girder and lays out the different stages of construction when each of the sections becomes critical. Detailed sections on the shear strength analysis of UHPC is also present in the paper which points out relevant research questions whose answers help develop stress blocks necessary for the calculation of the stress adequacy.

2.5 Accelerated Bridge Construction

Khalegi et.al (2012) presented a study on using accelerated bridge construction in the state of Washington and details the design philosophy adopted by the Washington Department of Transportation aligning with the FHWA guidelines. The Federal Highway Administration has actively promoted accelerated bridge construction as part of their 'Everyday counts' initiative, in an effort to reduce the on-site bridge construction duration. Prefabricated bridge components are in increasing demand in bridge construction. This change in the construction strategy may drastically bring down the site costs and potentially minimize lane closure delays. For conventional bridge construction, traffic delays, rerouting, and traffic congestion for an extended period of time is inevitable. By completing most of the girder casting along with a portion of the deck as a wide flange in the precast plant, the potential for speeding up the construction schedule exists.

Culmo (2011), in his report on Accelerated bridge Construction lays similar claims to the importance of the philosophy in tackling the current traffic congestion issues associated with the construction of highway bridges. The technical report Culmo prepared for the Federal Highway Administration details about the planning and implementation of accelerated bridge construction using prefabricated elements used in bridges (PBES). The document represents the state-of-practice with respect to all aspects of ABC and acts as a manual to fil gaps in previous published versions by the FHWA. Culmo (2011) lists multiple benefits of ABC and PBES, some of which are: reduced road user impacts; improved worker and motorist safety; expedited project planning process; improved quality control; improved constructability; Reduced cost to society.

2.6 UHPC Connection Design

If a bridge is to be constructed with a fully or partially topped girder, when placed side-by-side, longitudinal seams are needed to connect the girders. This section discusses such previous work.

Balakumaran et. al (2018) presented a study on girder-to-girder connections for multi-span bridges and detailed the causes and potential remedies for linear cracking in bridge decks. The research proposed a probabilistic chloride diffusion model to estimate the service life of bridge decks and connections. These cracks could develop despite being structurally sound but will affect the service life of the bridge considerably. Field surveys and laboratory testing of 2 previous studies were used in the research to arrive at the prediction of crack development due to chloride diffusion and other associated parameters.

Dang et.al (2020) discussed the advantages of using UHPC over conventional concrete for the casting of joints on-site. They emphasized that joints are the focus of the precast structure for accelerated bridge construction and presented the benefits of using UHPC in the prevention of longitudinal and transverse crack development due to the dispersion of steel fibers in the concrete mix. They further detailed that there were no visible cracks generated at the UHPC matrix and joint center in comparison to the conventional I-shaped joints evaluated for his experiments.

Graybeal (2014) provided examples of site-cast UHPC connections for precast deck panels 8.5-inch to 9-inch thick and has also specified guidelines for connections with no overlay of UHPC. Panel connections analyzed for this research were taken from New York State Department of Transportation (NYSDOT) design drawings and the research demonstrated that the deformed steel reinforcement in these connections could be developed within comparatively short embedment lengths of about $8d_b$, where d_b stands for the bar diameter. The connections studied by Graybeal are shown in Figure 2.2.

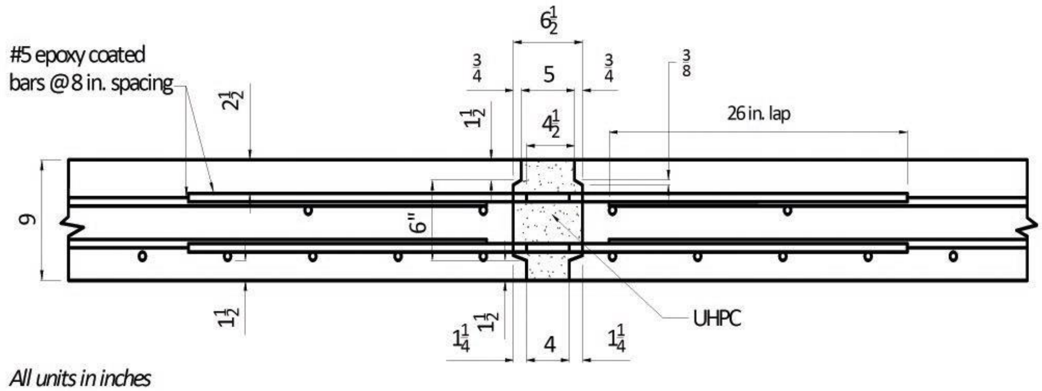
2.7 Summary

Previous investigations have detailed several benefits of using UHPC and ABC for highway bridges. The aim of this research is to combine these concepts and build upon them. While the benefits of using UHPC for girder construction has been explored, the application of wide UHPC flanges/decks has not yet been explored. A proper analysis of the design process using the material properties listed in the literature review and estimating the cost and schedule associated with such a design option will help in determining what design options are feasible for differing site conditions for highway construction in Texas. The main gap in the existing literature is the cost and schedule analysis which will be investigated subsequently.

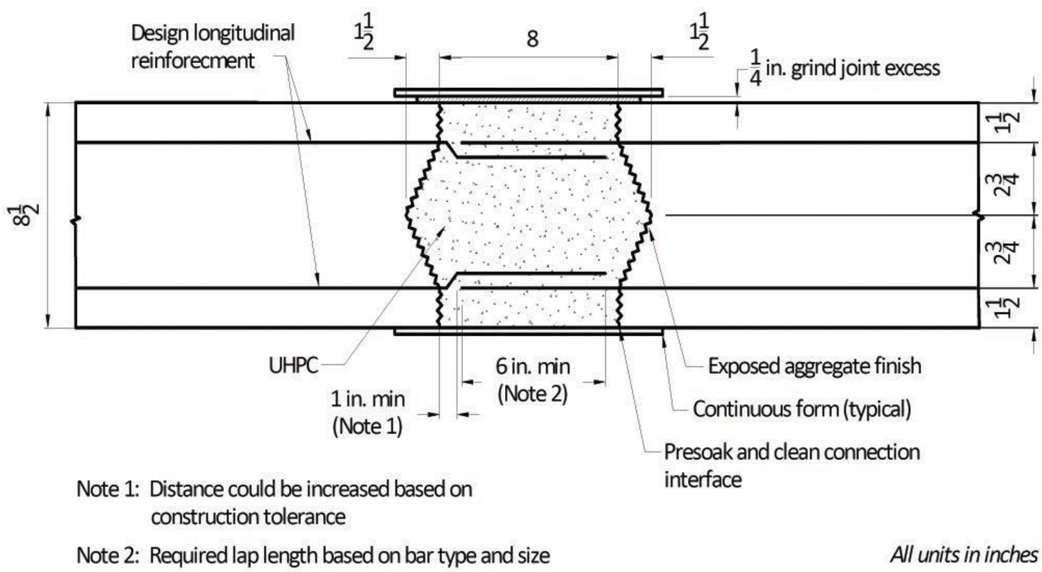
2.8 Research Questions Arising

Based on the foregoing literature review, the main knowledge gaps concern the feasibility of construction while switching to UHPC from normal concrete. The following research questions arise:

1. What impact does a UHPC wide flange cast in the prestress plant have on the overall design strength and long-term deflection associated with the construction of the single span bridge?
2. What thickness of the top wide flange is the most economical solution in comparison with the conventional 4-inch precast panel and 4.5-inch cast-in-place



(a) UHPC connection between precast deck panels as deployed by NYSDOT on I-81 in Syracuse, NY



(b) UHPC connection between precast deck panels as deployed by NYSDOT on CR47 over Trout Brook

Figure 2-2: UHPC deck panel connection examples from NYSDOT (Graybeal, 2014) (dimensions in inches)

topping concrete alternative?

3. Is it possible to eliminate or at least control cracks along longitudinal seams for the girder-to-girder connections?

CHAPTER 3

DESIGN PROCEDURE FOR SIMPLY SUPPORTED PRESTRESSED GIRDERS

3.1 Introduction

Prestressed concrete bridge design has now been in use in the United States for over 60 years. Using Ultra High-performance Concrete (UHPC) for the construction of bridges began gaining popularity only in the past couple of decades, but there is little research leading to guidelines on standardizing the process of using UHPC in bridge design (Graybeal and Zhang 2014). Using UHPC instead of conventional concrete may reduce the necessity for a deeper girder or may potentially reduce the number of girders altogether for the construction of a bridge span.

In this research, several contrasting designs for a 120-ft long simply-supported single span bridge, 46-ft wide with 3-ft overhang on both sides, supporting 2 lanes of traffic were developed. This chapter discusses six different design alternatives for the construction of the above-mentioned bridge span. The benchmark design considers 6-Tx54 girders cast using normal concrete (8.5 ksi) with 4-inch precast prestressed panels, with a 4.5-inch site-cast 4 ksi normal concrete deck. This benchmark design tends to be one of the most widely used approaches for bridge design and construction in Texas.

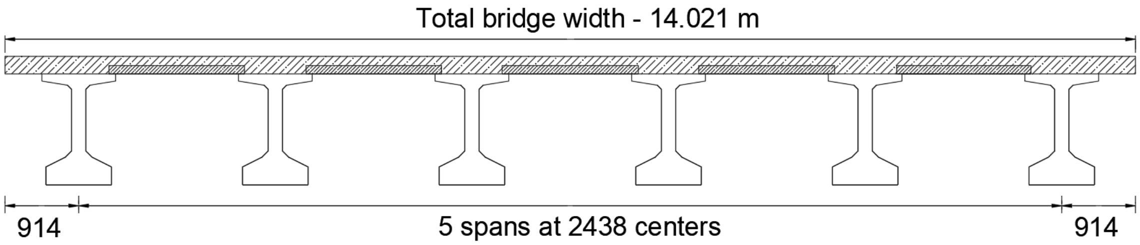
Five additional designs vary different aspects of the benchmark design and are described in detail in the next section. The main objective of this chapter is to answer the first research question posed at the end of the previous chapter. The questions deal with the impact of a UHPC wide flange cast in the precast plant on the overall design strength and long-term deflection of a single span simply supported bridge. The chapter will also explore how different wide flange thicknesses affect the deflection profiles for the optimum number of strands.

Even though a longer span maybe possible when using UHPC, transportation of girders with wide flanges is limited to 200 kips. Existing literature already talks in detail about the performance of UHPC for longer spans (Graybeal and Zhang 2014, El-Helou and Graybeal 2019). The bridges are designed to conform with the AASHTO specifications and deflection analysis is performed based on the long-term multipliers provided by Martin (1977) and Agarwal (2020).

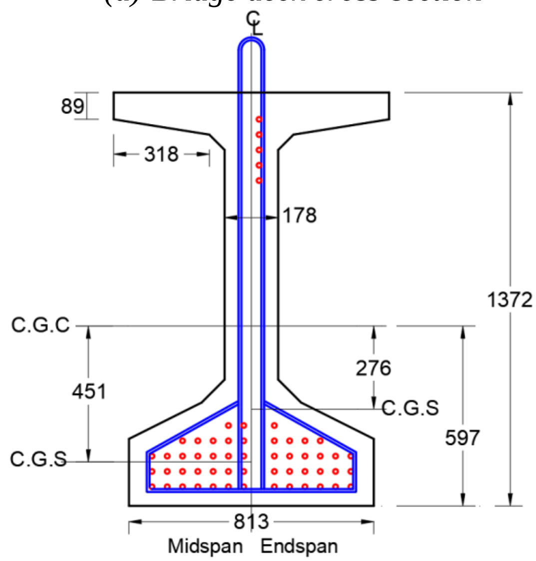
3.2 Tx54 Design Prototypes Analyzed

The design protypes analyzed in this chapter are summarized as shown below:

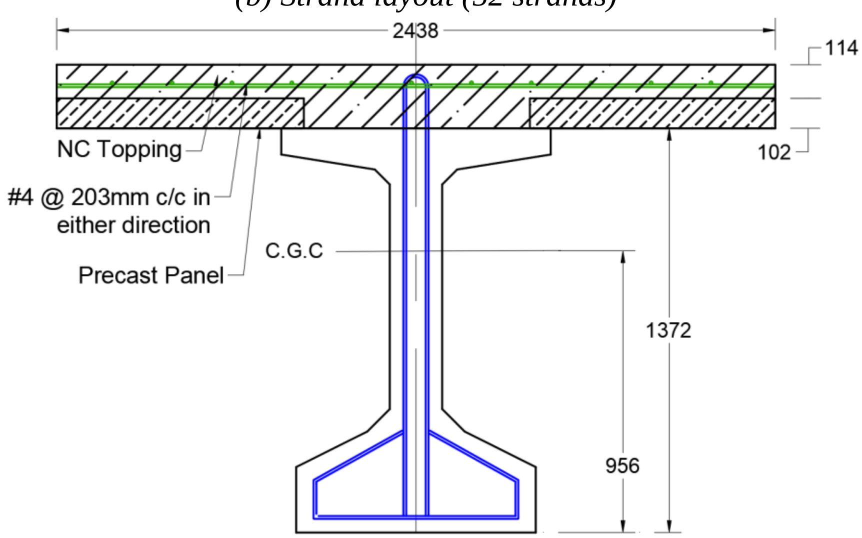
1. Figure 3-1 presents the cross-sections for the benchmark bridge design (Design 1). The 6-NC-girder span consists of 4-inch precast prestressed panels with a 4.5-inch site-cast NC-deck slab. The normal concrete (NC) strengths are 8.5 ksi and 4 ksi for the girder and deck topping respectively. Figure 3-1(a) shows the bridge deck cross section and Figure 3-1(b) shows the detailed strand layout. The cross-section has 52 strands with 8 of them harped at a third of the span (40-ft). Figure 3-1(c) shows the composite section with precast prestressed panels and slab reinforcement. The site-cast deck has #4 rebars spaced at 8-inches as slab reinforcement running in either direction. This Design 1 is herein referred to as: *6-NC girders, 4-inch pcp plus 4.5-inch NC-topping*.



(a) Bridge deck cross-section



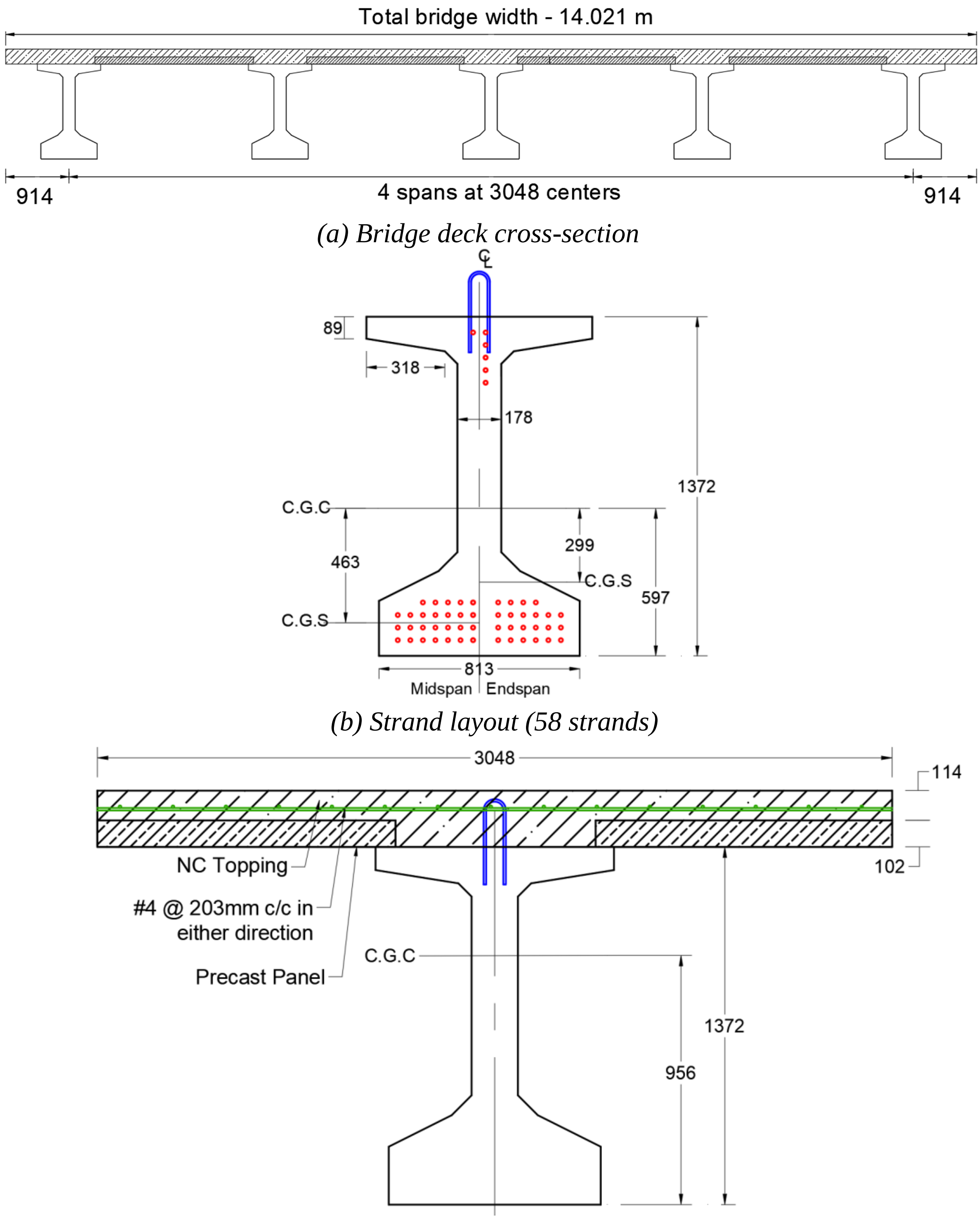
(b) Strand layout (52 strands)



(c) Composite section with PCP

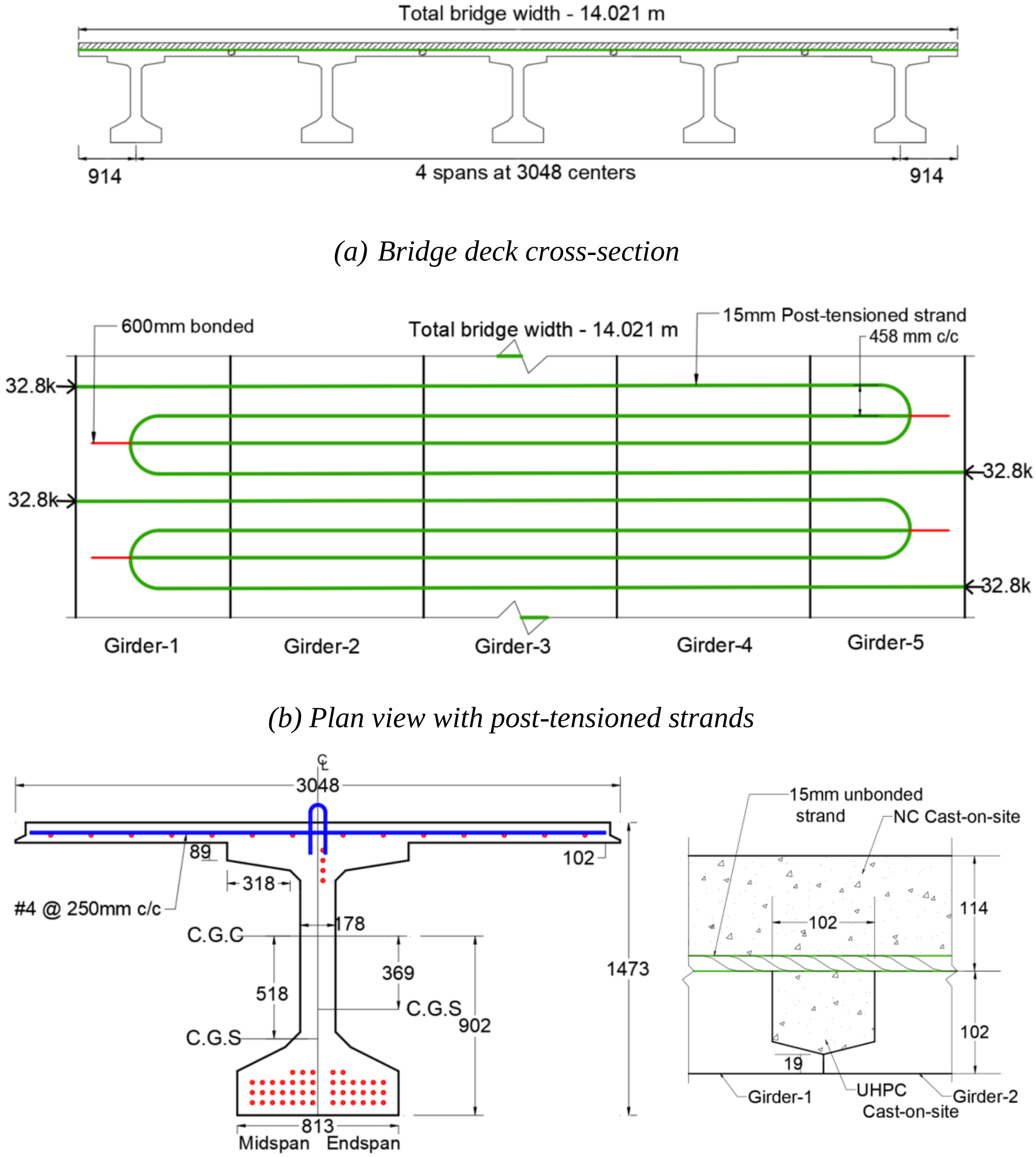
Figure 3-1: Bridge cross-section for Design 1: 6-NC-Girders, 4-inch precast panels + 4.5-inch site-cast NC-deck

2. Figure 3-2 presents the cross-sections for Design 2. The 5-UHPC-girder span consists of 4-inch precast prestressed panels with a 4.5-inch site-cast NC-deck slab. The strengths of UHPC used for casting the girders and NC used for the deck slab are 20 ksi and 4 ksi respectively. Figure 3-2(a) shows the bridge deck cross section and Figure 3-2(b) shows the detailed strand layout. The cross-section has 58 strands with 10 harped at a third of the span (40-ft). Figure 3-2(c) shows the composite section with precast prestressed panels and slab reinforcement. The site-cast deck has #4 rebars spaced at 8-inches as slab reinforcement running in either direction. As the material used for casting the girder changed from normal concrete to UHPC, the number of girders reduced from six to five. This Design 2 is herein referred to as: *5-UHPC girders, 4-inch pcp plus 4.5-inch NC-topping*.
3. Figure 3-3 presents the cross section for Design 3. Being the first design analyzed with a wide flange, the girder in this design has a 4-inch thick ,120-inch wide flange attached to the standard Tx54 shape. This 5-UHPC-girders with 4-inch wide flange has a 4.5-inch site-cast NC-deck. The strengths of UHPC used for casting the girders and NC used for the deck slab are 20 ksi and 4 ksi respectively. Figure 3-3(a) shows the cross-section of the bridge deck. Adjacent girders touch each other at the girder interfaces and has transverse post-tensioned strands placed on top at 18-inch spacing to introduce transverse prestress throughout the deck. Strands used are 0.6-inch diameter-270 ksi strands, post-tensioned to an effective stress of 32.8 kips. Figure 3-3(b) shows the top view of the bridge deck with the transverse strand layout. Duct cover is removed from the first 24-inches of the



(c) Composite section with PCP

Figure 3-2: Bridge cross-section for Design 2: 5-UHPC-Girders, 4-inch precast panels + 4.5-inch site-cast NC-deck



(a) Bridge deck cross-section

(b) Plan view with post-tensioned strands

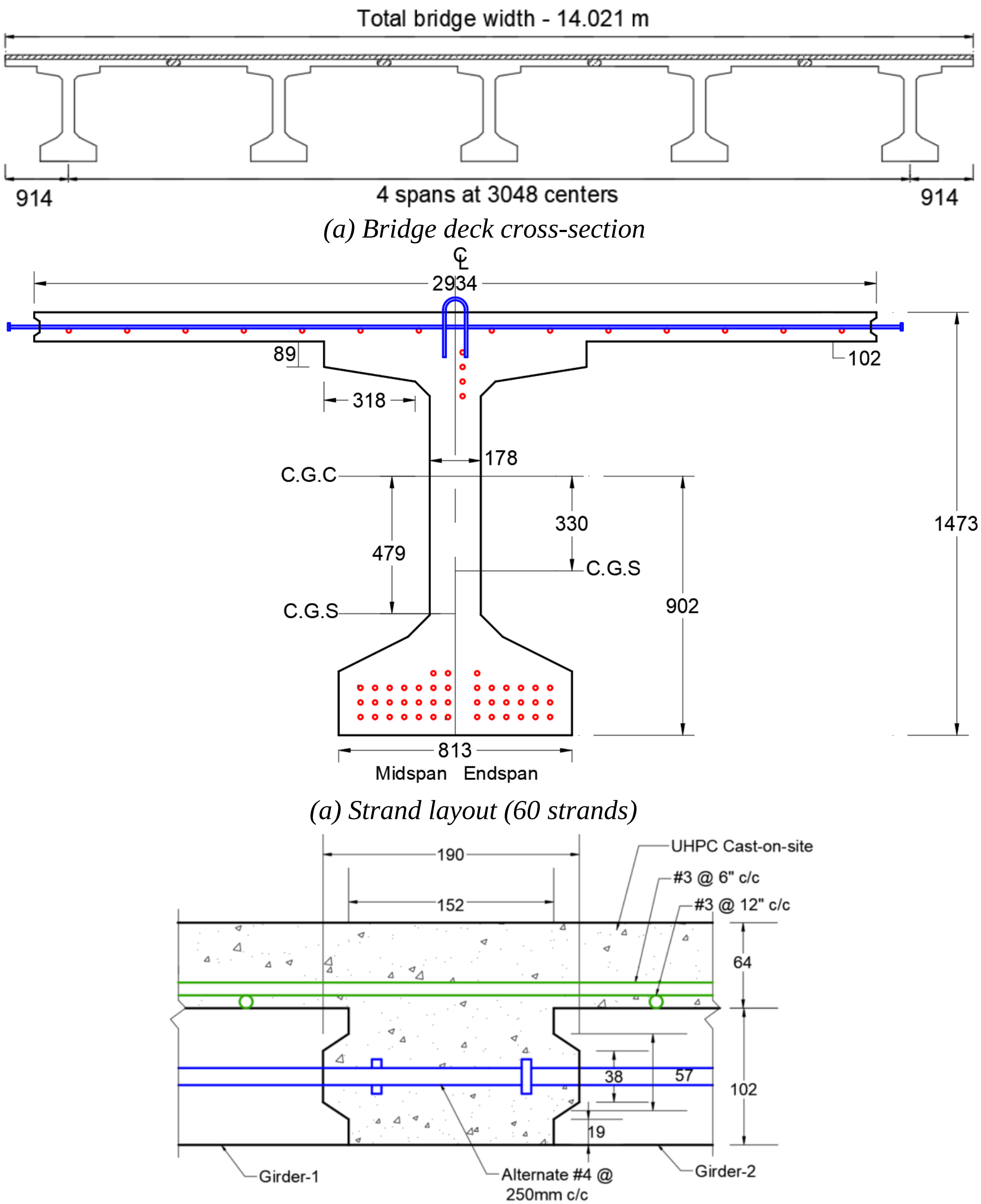
(c) Strand layout (60 strands) *(d) Connection detail*

Figure 3-3: Bridge cross-section and plan view for Design 3: 5-UHPC girders with 4-inch wide flange, 4.5-inch NC-topping with transverse PT

strand so that the uncovered portion acts as a bonded strand and anchors effectively to the deck concrete. Figure 3-3(c) shows the detailed strand layout and Figure 3-3(d) shows the detailed view of the girder-to-girder connection with the transverse post-tensioned strand. The cross-section has 60 strands with 8 of them harped at a third of the span (40-ft). The top flange has #4 rebar placed at 10-inch spacing as flange steel. This Design 3 is herein referred to as: *5-UHPC girders with 4-inch wide flange, 4.5-inch NC-topping with transverse PT.*

4. Figure 3-4 shows the cross-sections for Design 4. The girder design is similar to Design 3 and considers 5-UHPC-girders with 4-inch wide flange. This design consists of a 2.5-inch site-cast UHPC deck with slab reinforcement running in either direction. Figure 3-4(a) shows the bridge cross-section and Figure 3-4(b) shows the detailed strand layout. The cross-section has 60 strands with 8 of them harped at a third of the span (40-ft). The top flange has #4 rebar placed at 20-inch spacing as flange steel. Figure 3-4(c) shows the detailed view of the connection used and slab reinforcement. The site-cast-UHPC deck has #3 rebar placed at 6-inch spacing in the transverse direction and 12-inch spacing in the longitudinal direction. UHPC used has a strength of 20 ksi. This Design 4 is herein referred to as: *5-UHPC girders with 4-inch wide flange, 2.5-inch UHPC-topping with slab reinforcement.*

5. Figure 3-5 presents the cross-sections for Design 5. This 5-UHPC-girder system has 5-inch thick wide flanges attached to the girders. The design consists of a 1.5-inch site-cast UHPC deck. Figure 3-5(a) shows the bridge cross-section and

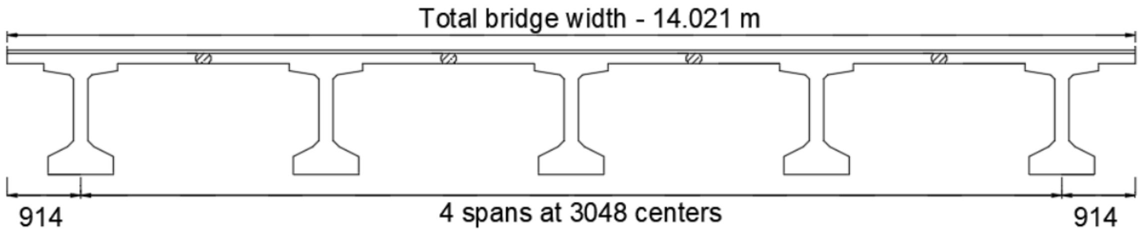


(a) Bridge deck cross-section

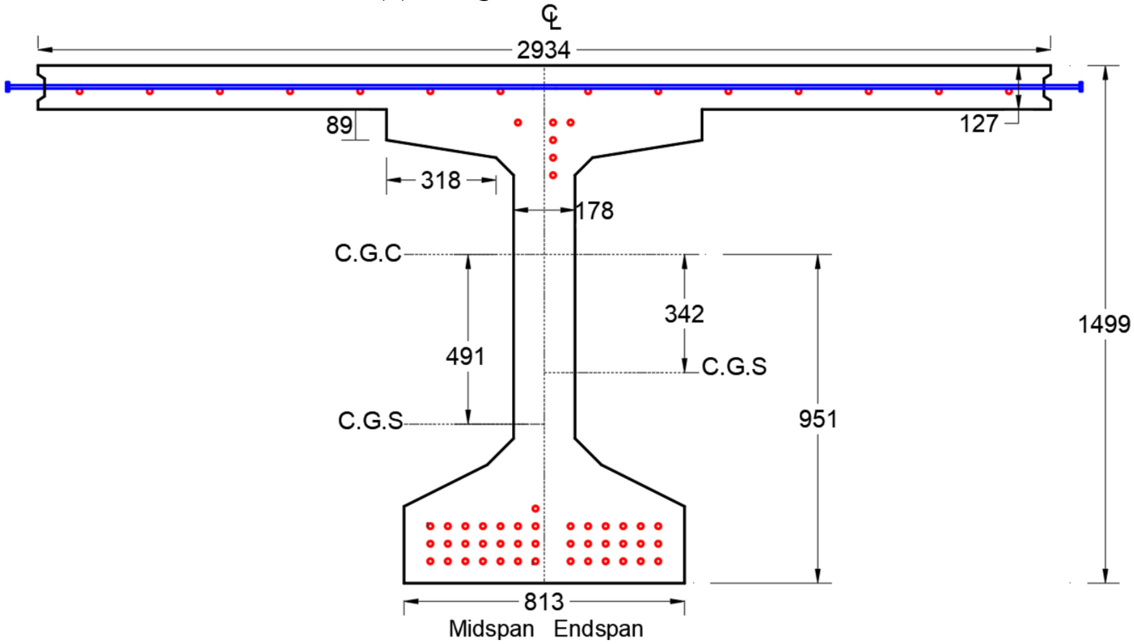
(a) Strand layout (60 strands)

(b) Connection Detail

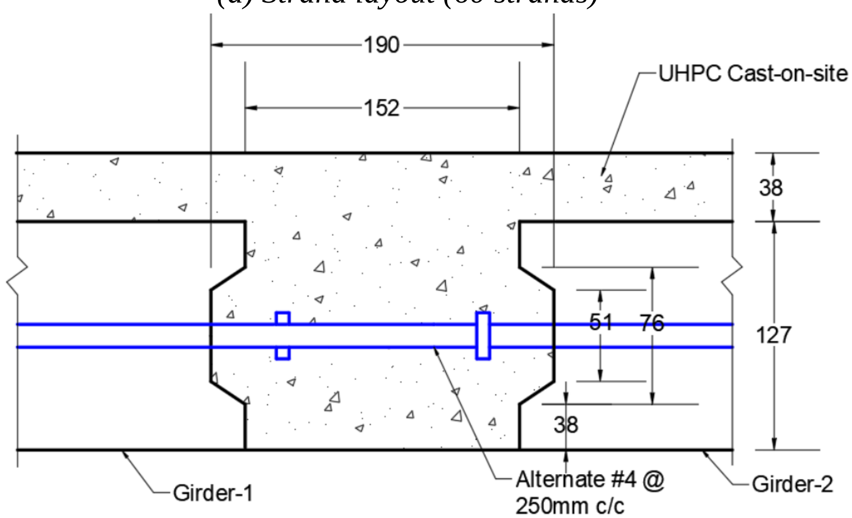
Figure 3-4: Strand layout and connection detail for Design 4: 5-Tx54-UHPC-Girders with 4-inch wide flange, 2.5-inch site-cast UHPC-topping with slab reinforcement



(a) Bridge deck cross-section



(a) Strand layout (60 strands)



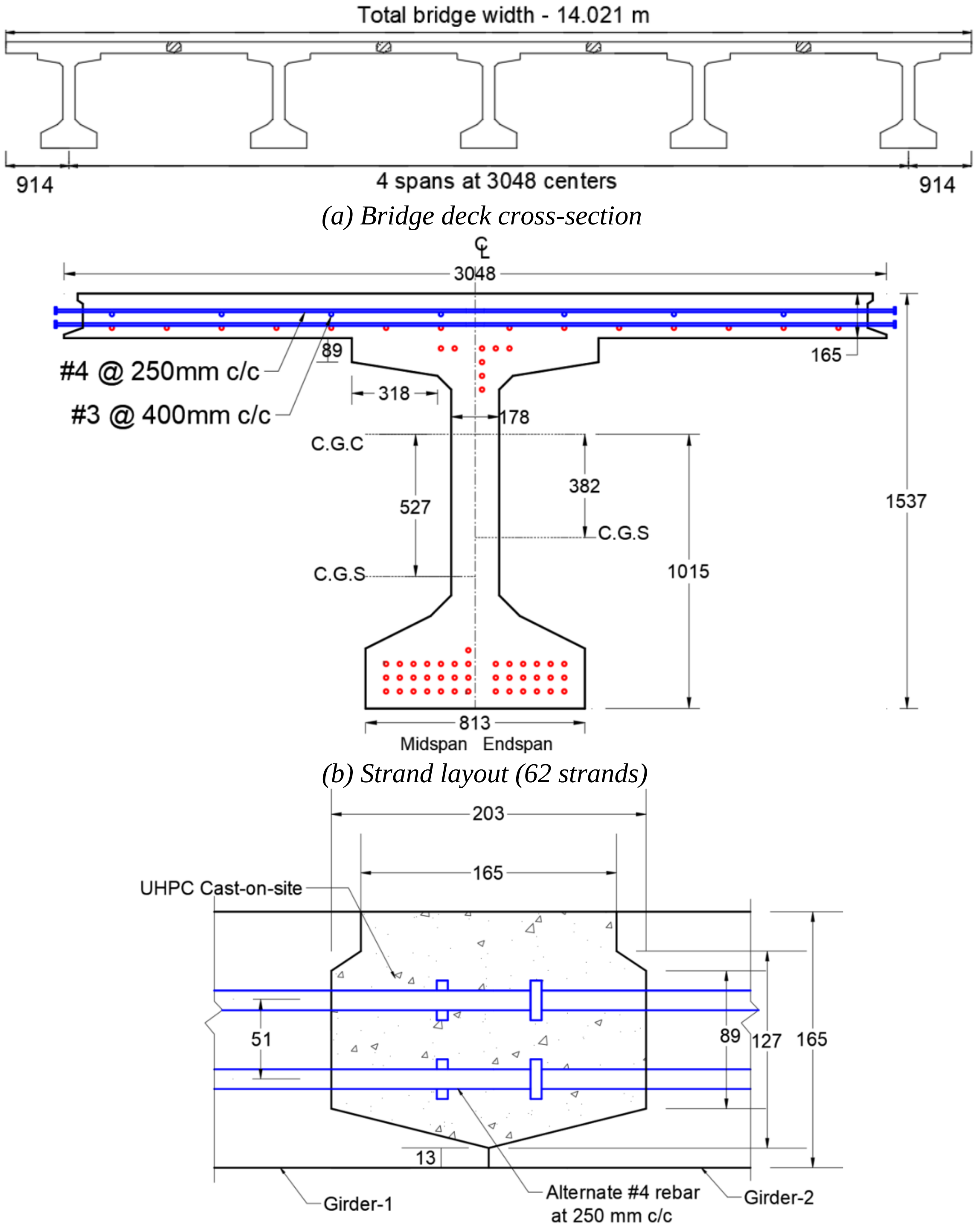
(c) Connection Detail

Figure 3-5: Strand layout and connection detail for Design 5: 5-Tx54-UHPC-Girders with 5-inch wide flange, 1.5-inch site-cast UHPC topping

Figure3-5(b) shows the detailed strand layout. The cross-section has 60 strands with 8 of them harped at a third of the span (40m). The top flange has #4 rebar placed at 20-inch spacing as flange steel. Figure 3-5(c) shows a detailed view of the girder-to-girder connection. UHPC used has a strength of 20 ksi. This Design 5 is herein referred to as: *5-UHPC girders with 5-inch wide flange, 1.5-inch UHPC-topping.*

6. Figure 3-6 presents the cross-sections for Design 6. The 5-UHPC-girder span has full-thickness 6.5-inch thick wide flanges and no site-cast deck. Figure 3-6(a) shows the bridge cross-section. Figure 3-6(b) shows the detailed strand layout. The cross-section has 62 strands with 8 of them harped at a third of the span (40-ft). Figure 3-5(c) shows a detailed view of the girder-to-girder connection. The full-thickness flange has a double layer of rebar to strengthen the longitudinal seam. The only site-casting involved for this design is the casting of the longitudinal seams. UHPC used has a strength of 20 ksi. This Design 6 is herein referred to as: *5-UHPC girders with 6.5-inch wide flange.*

The six different designs and key differences can be summarized as follows: Design 1 is the benchmark TxDOT design using 6-8.5 ksi-NC-girders with 4-inch precast prestressed panels and 4.5-inch-4 ksi-site-cast NC-deck. For design 2, the material used for casting the girder changed from normal concrete to 20 ksi-Ultra High-Performance Concrete. As a result of this change in material, the number of girders reduced from six to five. In Design 3, the UHPC girder changed to a wide flange girder cast using the same material. The wide flange is 4-inch thick and uses transverse post-tensioned strands on-



(c) Connection Detail

Figure 3-6: Strand layout and connection detail for Design 6: 5-Tx54-UHPC-Girders with 6.5-inch wide flange

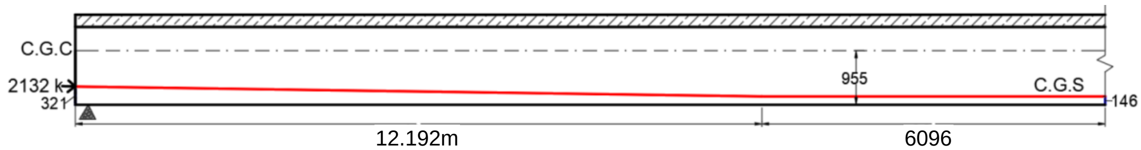
site while casting a 4.5-inch-4 ksi deck topping. Design 4 removes the transverse post-tensioning in Design 3 and introduces slab reinforcement to a 2.5-inch site-cast-UHPC-deck. In Design 5, the thickness of the wide flange is increased to 5-inch and the deck steel is removed. Design 6 takes away the casting of on-site deck topping altogether and has a double layer of steel in the 6.5-inch thick top flange.

Figure 3-7 shows the side-elevation of half of the bridge span for all the six different design scenarios considered. The figure shows the harping point at 40m from the girder end and traces the position of the center of gravity of the steel. Eccentricity is calculated as the difference between the center of gravity of concrete (C.G.C) and center of gravity of steel (C.G.S).

Table 3-1 shows the section properties of the six designs considered and also calculates important parameters necessary for the stress and deflection analysis. The section properties are again subdivided into girder properties, deck properties and composite properties. The concept of modular ratio is used here compare and integrate cross sections that use different strengths of concrete for the girder and deck as is the case in Designs 1, 2 and 3. The modular ratio between two materials with different moduli of elasticity is defined as the ratio of the modulus of elasticity of one material to the modulus of elasticity of another material.

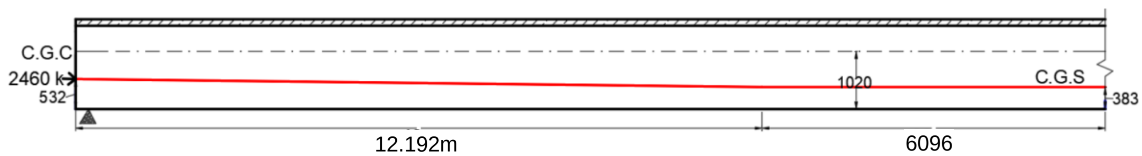
$$n_{12} = E_1/E_2 \tag{3-1}$$

where n_{12} = modular ratio between materials 1 and 2; E_1 = modulus of elasticity of material 1; and E_2 = modulus of elasticity of material 2.

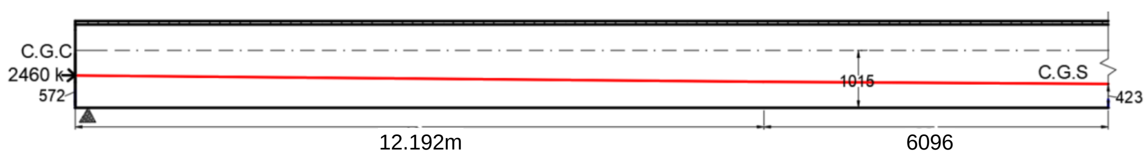


(a) Design 1(52 strands with 10 harped)

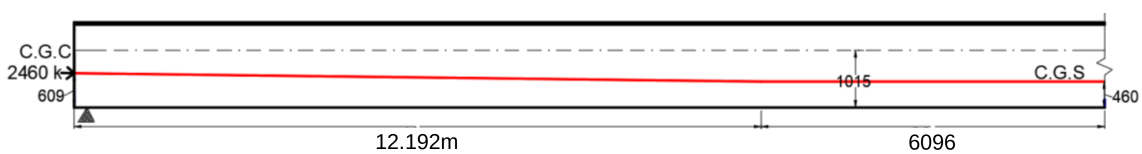
(b) Design 2 (58 strands with 10 harped)



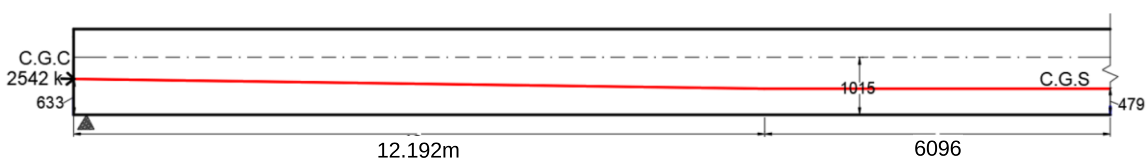
(c) Design 3 (60 strands with 8 harped)



(d) Design 4 (60 strands with 8 harped)



(e) Design 5 (60 strands with 8 harped)



(f) Design 6 (62 strands with 8 harped)

Figure 3-7: Side elevation of half of the composite bridge span for the six designs considered (the span is symmetric).

Table 3-1: Section Properties of the six different designs

Section Properties		Design 1	2	3	4	5	6
Girder	E (ksi)	5225	6449	6449	6449	6449	6449
	Depth, d(in)	54	54	58	58	59	60.5
	A (in^2)	817	817	1297	1297	1417	1597
	y_t (in)	30.51	30.51	22.49	22.49	21.54	20.53
	y_b (in)	23.49	23.49	35.51	35.51	37.46	39.97
	I_{xx} (in^4)	300043	300043	620336	620336	678380	757772
	S_{xt} (in^3)	9834	9834	27583	27583	31494	36910
	S_{xb} (in^3)	12773	12773	17469	17469	18109	18959
	K_t (in)	12.04	12.04	21.27	21.27	22.23	23.11
	K_b (in)	15.63	15.63	13.47	13.47	12.78	11.87
Deck	b_d (in)	96	120	120	120	120	120
	t_d (in)	8.50	8.50	4.50	2.50	1.50	0
	n	1.46	1.79	1.79	1.00	1.00	1.00
Composite	Depth, d(in)	62.5	62.5	62.5	60.5	60.5	60.5
	A_c (in^2)	1377	1387	1599	1597	1597	1597
	y_{tc} (in)	24.89	24.75	22.33	20.53	20.53	20.53
	y_{bc} (in)	37.61	37.75	40.17	39.97	39.97	39.97
	I_{xxc} (in^4)	704845	709063	770541	757772	757772	757772
	S_{xtc} (in^3)	28318	28649	34507	36910	36910	36910
	S_{xbc} (in^3)	18741	18783	19182	18959	18959	18959
	K_{tc} (in)	20.57	20.66	21.58	23.11	23.11	23.11
	K_bc (in)	13.61	13.54	12.00	11.87	11.87	11.87
$EI_{Composite}$ (per girder)		3.68 x10^9	4.57 x10^9	4.97 x10^9	4.89 x10^9	4.89 x10^9	4.89 x10^9
Girders		6	5	5	5	5	5
$EI_{Deck-span}$		22.1 x10^9	22.8 x10^9	24.8 x10^9	24.4 x10^9	24.4 x10^9	24.4 x10^9

3.3 Design Assumptions

The prototype bridge designs developed herein are based on the following design assumptions. Material properties of concrete and steel were chosen according to general design considerations of TxDOT.

- The total prestress loss in the strands is assumed to be 20% of the initial prestressing force. 5% in this loss occurs immediately as the strands are cut at the precast plant. It is also assumed that 7.5% of the losses occur during the storage of the girders and the remaining 7.5% loss is considered to be long term loss that occurs during the service life of the bridge (Agarwal 2020, Martin 1977).
- Tensile stresses are considered positive and compressive stresses are negative.
- Upward deflection (hogging) is considered negative and downward deflection (sagging) is considered positive.
- Wherever dry connections are considered, it is assumed that the misalignments get corrected with the deck casting on-site.

3.4 Preliminary Design Considerations

The maximum span of the bridge was evaluated based the maximum deflection limit as per AASHTO LRFD specifications, (L/800). HL-93 loading was considered as live loads as per AASHTO 2017. The live load moments were transferred to the individual girders using Lane Load Distribution Factors. Although the design truck load is given as 8 kips, 32 kips and 32 kips spaced 14-ft apart, for the purpose of this analysis a symmetric 20 kips, 32 kips, 20 kips load was considered spaced 14-ft apart, placed at the center of the span for the worst-case live load moments. The point loads were magnified by 33% to account for the impact load factor. Along with that, a design lane load of 0.64 kip/feet distributed uniformly along the longitudinal span was also considered.

The primary objective of the design is to obtain minimum long-term deflection for each of the designs while accounting for the stress checks at the different stages of construction. The allowable stress limits in ksi are tabulated for each of the concrete types considered based on the following equations.

$$f'_{ci} = 0.9f'_c \tag{3-2}$$

$$f_{ci} = -0.6f'_{ci} \tag{3-3}$$

$$f_{ti} = 0.19\sqrt{f'_{ci}} \tag{3-4}$$

$$f_c = -0.6f'_c \tag{3-5}$$

$$f_t = 0.19\sqrt{f'_c} \tag{3-6}$$

where f'_c = design compressive strength at 28 days in ksi units; f'_{ci} = assumed compressive strength; f_{ci} = compressive strength at transfer; f_{ti} = tensile strength at transfer; f_c = compressive strength at service; and f_t = tensile strength at service.

The strands used are 270 ksi 0.6-inch strands stressed in the precast plant such that the effective forces on each of the strands ends up being 32.8 kips after all the losses according to above mentioned assumptions.

3.5 Design Example

As mentioned above, the single span bridge under consideration is 120-ft long, 46-ft wide with 3-ft overhang on either side and supports two lanes of traffic. All solutions consider strands harped at one third of the total span or at 40-ft from the ends. Table 3-1 shows the section properties of the six designs.

Table 3-2 presents the value of stresses at the endspan and midspan at the top and bottom of the girder cross section. Stresses were calculated for the different stages of construction and were compared with the allowable limits (Agarwal 2020). The different stages considered are at the transfer of loads when the strands are cut in the precast plant, after deck casting on-site, and the long-term service condition.

Figure 3-8 shows the equations used and stress blocks obtained for Design 4. The figure is divided into 2 parts: (a) evolution of stresses before the election of the girders on-site and (b) after the erection of girders on-site. The values of stresses are given in ksi.

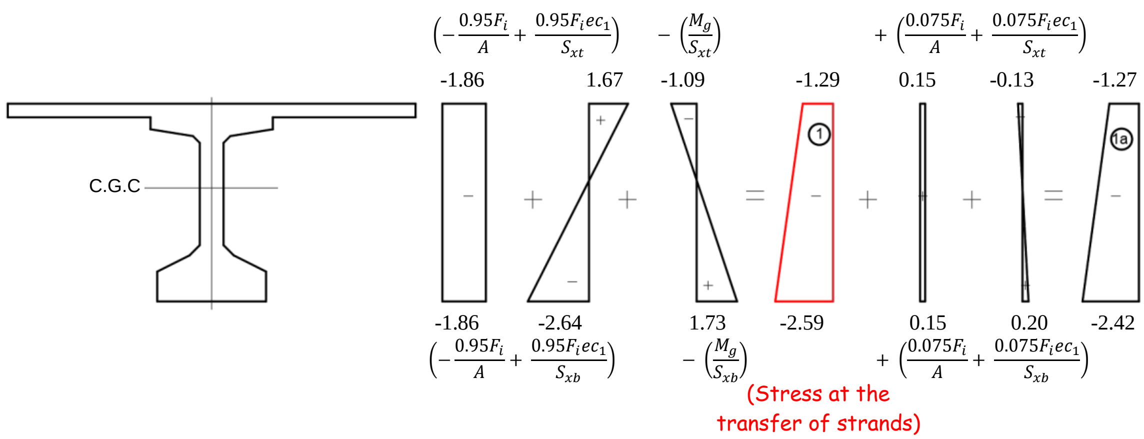
3.6 Deflection Analysis

Deflection profiles for the six different designs were calculated based on the long-term deflection multiplier method proposed by Martin (1977) and Agarwal (2020). Martin (1997) makes some general assumptions on the age at which the girders are erected and has derived the multipliers based on this consideration. However, Agarwal (2020) suggests altering these constants considering a storage length of 40-60 days for these girders.

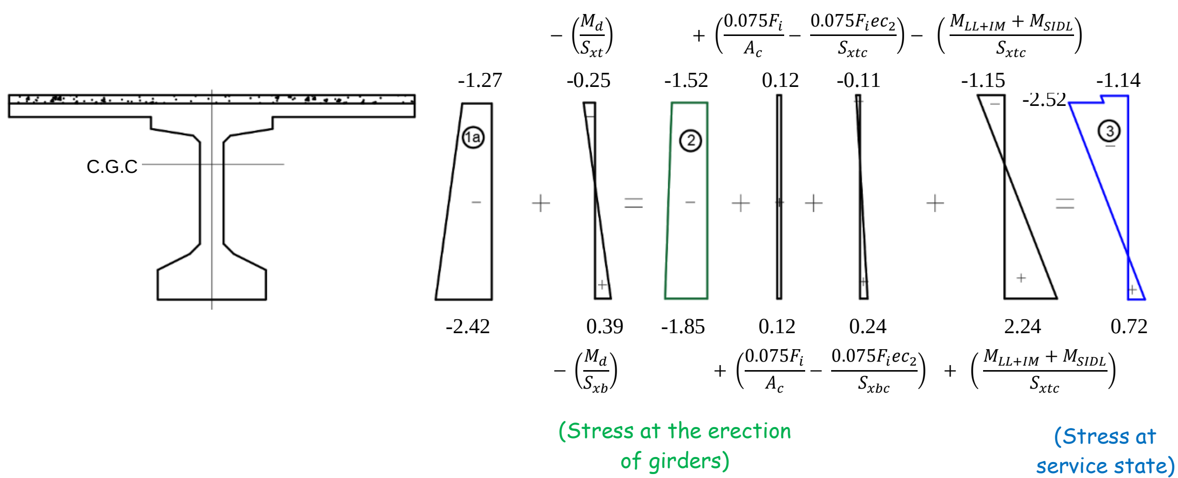
Figure 3-9 shows the deflection profiles obtained for the six designs considered. The deflection equations described in chapter 2 corresponds to the maximum deflection at the midspan for a simply supported bridge span and the generalized version of these equation were used to arrive at the deflection profiles.

Table 3-2: Stresses at various stages of construction for the six designs

Stage	Location	Position	Design Stress (ksi)					
			1	2	3	4	5	6
(1) At Transfer	Midspan	Top	-0.66	-0.62	-1.16	-1.29	-1.26	-1.27
		Bottom	-3.88	-4.41	-2.80	-2.59	-2.32	-2.29
	Endspan	Top	-0.06	-0.27	-0.57	-0.70	-0.65	-0.63
		Bottom	-4.34	-4.68	-3.74	-3.54	-3.38	-3.39
(2) At Construction	Midspan	Top	-2.69	-3.13	-1.61	-1.53	-1.38	-1.29
		Bottom	-1.97	-2.09	-1.73	-1.85	-1.76	-1.90
	Endspan	Top	-0.06	-0.25	-0.52	-0.64	-0.60	-0.58
		Bottom	-4.00	-4.31	-3.45	-3.26	-3.12	-3.13
(3) At Service State	Midspan	Top	-3.97	-4.72	-2.88	-2.68	-2.52	-2.42
		Bottom	+0.21	+0.57	+0.82	+0.72	+0.80	+0.67
	Endspan	Top	-0.10	-0.31	-0.52	-0.62	-0.56	-0.53
		Bottom	-3.68	-3.98	-3.16	-2.97	-2.85	-2.85



(a) Evolution of stresses before erection on-site



(b) Evolution of stresses after erection on-site

Figure 3-8: Stresses at midspan of the girder at various stages of construction for Design 4 – 5-Tx54-UHPC-Girders with 4-inch wide flange, 2.5-inch site-cast UHPC topping with slab reinforcement

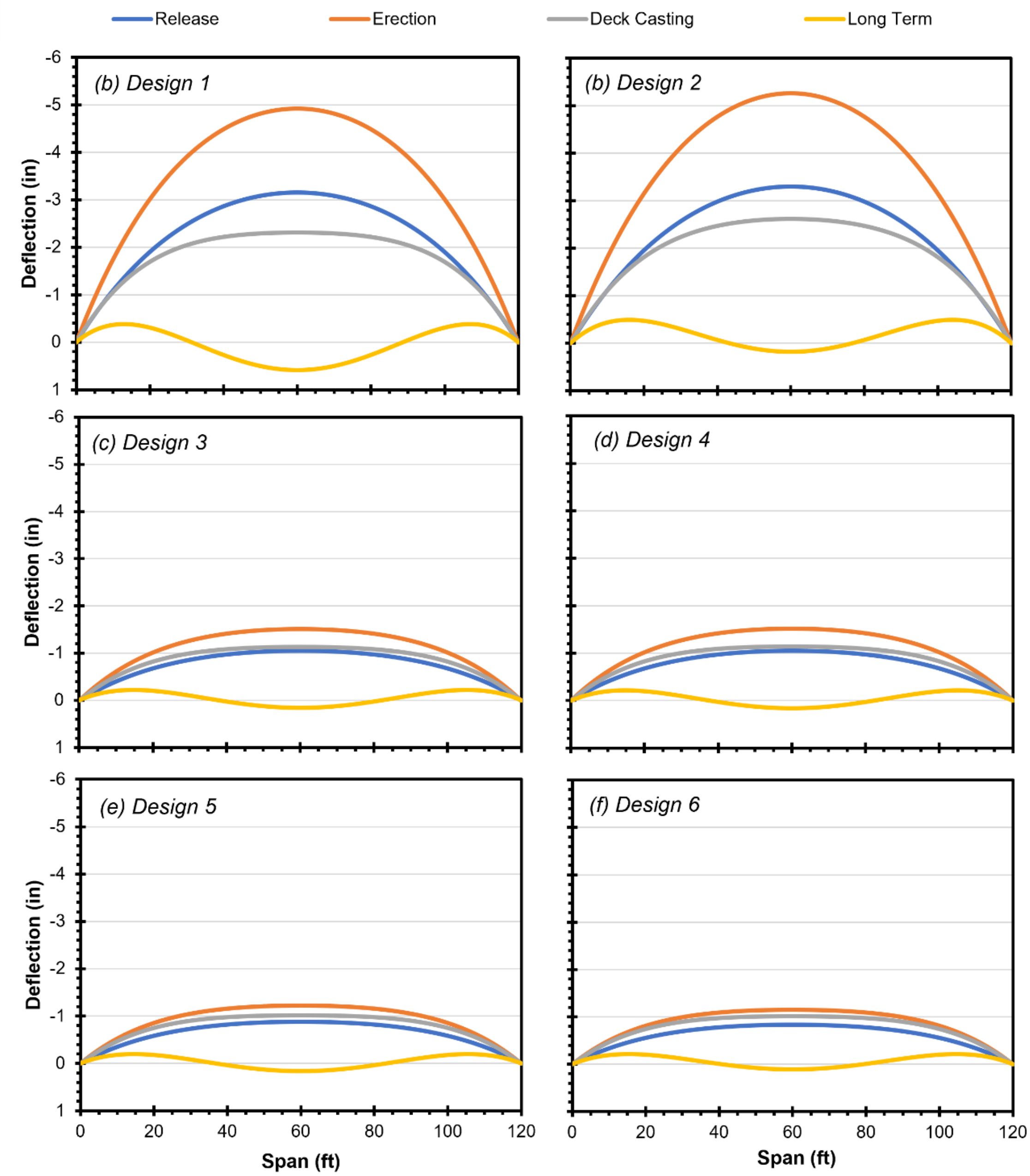


Figure 3-9: Deflection profile for the six designs considered

The initial camber at the time of release of the prestress force, the maximum camber that evolves during the storage of the girder, the reduction in camber due to the casting of the deck on site, and the final long-term deflection are analyzed and compared for each of the designs. The ideal deflection profile should have a minimum camber at the time of erection and a near-flat long-term deflection profile. The strand layout was designed for each of the designs to have a near-flat long-term deflection as observable in the deflection profiles.

Table 3-3 presents a summary of strand layout for the six designs and also presents the maximum long-term deflections. The long-term deflection is found to be maximum for Design 1 and minimum for Design 6.

3.7 Ultimate Strength Check

Bending moments at the ultimate limit states are required to verify that the reduced nominal capacity of the girder cross section is sufficient to counter the factored moment demand on the girder. For this purpose, the lane load distribution factor is used, and the bending moment strength is checked using the following equations as per the AASHTO LRFD Specifications.

$$\phi M_n \geq M_u \tag{3-7}$$

$$M_u = 1.25(M_{DC}) + 1.5(M_{DW}) + 1.75(M_{LL+IM}) \tag{3-8}$$

$$LLDF = S/12 \tag{3-9}$$

where M_u = ultimate flexural demand; M_{DC} = moment due to dead loads except wearing surface; M_{DW} = moment due to wearing surface or superimposed dead load; M_{LL+IM} = moment due to live load and impact load; M_n = flexural capacity of the cross-section;

Table 3-3: Summary of strand layout and maximum long-term deflection for the different designs considered

Design Number	Number of Strands			Maximum Long-Term Deflection (in)
	Number of Girders	Number of Strands per Girder	Total Number of Strands	
Design 1	6	52	312	0.58
Design 2	5	58	290	0.18
Design 3	5	60	300	0.16
Design 4	5	60	300	0.16
Design 5	5	60	300	0.16
Design 6	5	62	310	0.11

ϕ = strength reduction factor (1.0 for tension-controlled sections and 0.75 for compression-controlled sections) (AASHTO 2017); $LLDF$ = lane load distribution factor; and S = spacing between the girders in feet. Methods of obtaining lane load distribution factors are quite complex and a simplified approach proposed by Jiang (2015) is adopted in Eq. (3-9).

3.8 Shear Analysis

One of the main motivations of using Ultra High-Performance Concrete over conventional concrete for the girder and deck casting is the special ability of UHPC to prevent shear cracks due to the presence of the steel fiber reinforcement. For the designs considered, a fiber reinforcement equivalent to 1.5% by volume is considered so that on-site deck casting can avoid the use of transverse reinforcement. The fiber mat created by the embedded steel fibers provide sufficient strength to counter the shear demand on the structure. Contrary to this, the deck cast using conventional concrete for design options 1 and 2 will require longitudinal and transverse reinforcement as it is required to prevent cracks.

The shear capacity of the girder itself can be estimated using the equations provided by Gunasekaran (2020) and can be compared to the shear demand on the structure as per AASHTO LRFD Specifications. The following equations are used in the estimation of the shear capacity of the sections.

$$\phi V_n \geq V_u \tag{3-10}$$

$$V_u = 1.25(V_{DC}) + 1.5(V_{DW}) + 1.75(V_{LL+IM}) \tag{3-11}$$

where V_{DC} = shear force due to dead loads except wearing surface; V_{DW} = shear force due to wearing surface; V_{LL+IM} = shear force due to live and impact load; V_n = shear resistance of the cross section; ϕ = strength reduction factor = 0.9.

Shear strength of UHPC due to the distribution of steel fibers can be estimated using the equation provided by Gunasekaran (2020):

$$V_{UHPC} = f'_t b_w d_v \sqrt{1 + \left|\frac{F/A}{f'_t}\right|} \tag{3-12}$$

where f'_t = sustained tensile strength of UHPC; b_w = web depth; d_v = effective shear depth; F = prestressing force after losses; and A = area of cross section. In Eq. (3-12),

$$cot\theta = \sqrt{1 + \left|\frac{F/A}{f'_t}\right|} \tag{3-13}$$

where θ = orientation of the principal compression stress to the horizontal. This angle is also the expected orientation of the initial crack angle.

3.9 Discussion on Connections

A dominant concern for highway bridge girders is the possible evolution of cracks along longitudinal seams. The cracks along the girder-to-girder connections are quite common among United States highways and in accelerated bridge construction using precast members, joint connections are a huge focus (Deng.et.al 2020).

Existing methods proposed by multiple researchers (Graybeal 2014, Dang.et.al 2020, Balakumaran.et.al 2018) discuss deck panel-to-panel connections, girder-to-girder connections for multi-span bridges and concrete deck-to-steel girder connections. However, cracking potential along the length of the girder with dry connections and thin deck overlays have not been addressed. Multiple parameters are involved in the determination of crack evolution and crack width. These include annual average daily truck traffic, concrete pore volume, geographical zone, soil settlement, moisture/saturation percentage, diffusion co-efficient/ air chloride concentration etc.

As the thickness of site-cast deck decreases, the possibility of having a longitudinal crack running along the length of the bridge increases. To counter this in Design 3, 270 ksi 0.6-inch diameter post-tensioned strands stressed to 32.8 kips after losses are proposed for the transverse direction of the bridge (46-ft) as shown in Figure 3-3(b) to provide prestress throughout the length of the bridge. Spacing them at 18-inches center-to-center creates a compression stress of 0.20 ksi across the connections which may inhibit the potential evolution of such longitudinal cracks.

For the other wide flange design options considered, full-thickness UHPC connections are proposed with Design 4 having an open-base connection with alternate #4 rebars as shown in Figure 3-4(b). This design also considers deck reinforcement to provide additional strength to the connection. Design 5 has a thinner deck overlay and only has the alternate rebars to counter potential inter-girder shear as shown in Figure 3-5(b). The open base connections used in Designs 4 and 5 require additional formwork assembly to cast the UHPC. However, Designs 3 and 6 considers a closed base connection as shown in Figure 3-3(d) and 3-6(c) respectively.

3.10 Summary and Findings

Chapter three describes the different designs considered for analysis and the design procedure adopted to perform the calculations. The different designs considers both UHPC and conventional concrete and considers different thicknesses for the wide flange attached to the girder. The key finding obtained from this chapter are:

1. The stress block analysis for the optimal deflection profile yielded the strand layout for each of the designs and it can be observed that the number of strands used does not vary significantly in comparison with the benchmark design, which is the standard construction approach adopted by TxDOT.
2. On examination of the deflection profile, it is evident that the deflection reduces as the thickness of the top wide flange increases. A tabulated summary of the number of strands used and maximum midspan deflection for the different designs was shown in Table 3-3. This is because the initial strand layout already balances out the deck moments which are applied at a later stage for the other designs. The only concern with Design 6 would be transporting the heavier girders on Texas highways. But according to Hueste and Mander (2012), a maximum load of 220 kips is permitted on the highways and Design 6 with a linear weight of 1.6 kip/ft has a total weight of 190 kips and is within the shipping limitation.
4. This chapter addresses the first research question of the thesis – What impact does a UHPC wide flange cast in the prestress plant have on the overall design strength and long-term deflection associated with the construction of the single span bridge? From the analysis, it is found that having a wide flange cast in the plant helps the moment capacity and the long-term deflection of the girder in a beneficial way. Increasing the thickness of the wide flange and reducing the amount of on-site deck casting is potentially advantageous in comparison with the conventional

concrete Tx54 design regularly adopted by TxDOT. Figure 3-9 addresses the second part of question 1 and depicts how the deflection profile changes with an increase in the thickness of the wide flange for an average number of strands. The minimum long-term deflection was found for Design 6 with 6.5-inch-wide flange and 62 strands per girder. This aligns with our assumption of balancing out most of the deck weight during the initial prestressing of the girder.

5. Chapter 3 also partially answers question three – Is it possible to eliminate or at least control cracks along longitudinal seams for the girder-to-girder connections? Different connection types used for the designs were analyzed including transverse post-tensioning (Design-3), UHPC-reinforced deck connection (Design-4) and full-thickness UHPC-reinforced connection (Design-6). In order to prevent the evolution of longitudinal cracks along the seams, the best alternative will be transverse post-tensioning, followed by the UHPC-reinforced deck. The reinforced deck has the potential ability to inhibit the evolution of longitudinal and temperature cracks but cannot prevent the possibility as effectively as transverse post-tensioning. The double layer strong reinforced UHPC-connection used in Design-6 should also limit potential cracking.

CHAPTER 4

ESTIMATES AND CONSTRUCTION SCHEDULING

4.1 Introduction

To investigate the feasibility of the six contrasting designs, cost estimates and time schedules are first developed, then compared and contrasted. Where appropriate, ABC principles have been applied. The estimates are considered in two parts: girder fabrication work done in the precast plant; and site work performed to complete one span.

The entire casting process and assembly on-site are broken down into several steps and assumptions common to all the design options are considered to estimate procurement costs for each design. Construction on site is then evaluated in terms of costs and site occupation time. A particular focus which distinguishes the six designs is the longitudinal seams for the girder-to-girder connections. Results show that as each design solution becomes more sophisticated, the fabrication cost of the girders progressively increases. However, the site costs and construction time are more than offset by this extra effort at the front end.

4.2 Assumptions and Industry Standards

The cost involved in the construction of bridges and the time it takes to implement designs on site and in the precasting plant are generally region and company-specific and highly variable depending on the time of construction, climatic conditions, traffic situations etc. The variability in these factors clearly affect the estimations of the cost and schedule prepared in this section. However, a general set of assumptions are considered which apply to all the design options considered and thus provide a justifiable comparison between the models. Some of the assumptions considered for the estimation of the cost and schedule associated with the implementation of the designs are listed below:

1. All tasks are assumed to be executed by crews of six members in the precast plant. Tasks which require a greater work force will hire people in multiples of six as it is considered as a representative crew size. Tasks executed on-site considers different number of laborers for different tasks.
2. Labor charges are assumed to consider 100% overhead cost for supervision and contractor charges. Davis-Bacon regulations for non-exempt construction laborers are considered while estimating on-site labor charges. The qualifying wage for on-site construction is stated as $17.10 per hour. Average wages for the different activities involved in the construction process takes this into account.
3. Materials are assumed to be readily available at the precast plant and at the site for construction. No specific transportation cost is considered to source the materials at the plant or at the site.
4. Plant establishment cost is factored in as an additional item called ‘site charges’ which are applied to each girder cast in the precast plant.
5. The site for construction is assumed to be at a distance of 100 miles from the plant and separate expenses are itemized for the transportation workers and for the truck transporting the girders.
6. Cost of UHPC is a highly contested input parameter whose price has been rapidly decreasing over the decades. The UHPC considered has 2% fibers. Perry and

Siebert (2011) estimated the price of UHPC with steel fibers to be ranging from \$3270/yd^3 to \$5886/yd^3. However, in the report prepared for the Colorado Department of Transportation, Kim (2018) puts the cost of Ultra High-Performance Concrete without steel fibers at \$1535/yd^3 and with steel fibers at \$2573/yd^3. This shows a reduction of up to 56% in the cost of UHPC. More recent studies suggest a much cheaper price for UHPC at \$600/yd^3 to \$700/yd^3. As the price of UHPC is reducing at a quick pace, the calculations assume the price of UHPC cast in the factory to be \$650/yd^3 and the price of UHPC cast on site to be equal to \$2000/yd^3, the latter being a proprietary mix procured for consistency purposes.

7. Site mobilization and demobilization charges are considered the same for all types of girder construction as the construction is assumed to take place at the same time of the year with similar terrain and climatic conditions for all the design options for uniformity.
8. The cost of precast panels and barriers are assumed to be twice the cost of concrete used in the production of these pe-made members. The unit costs considered for different strengths of concrete and steel are presented in tables and conform to current practice.
9. The time taken for each of the activities inside the precast plant and on-site and judicious estimates and are assumed to be valid for deductions as uniformity is maintained for all the designs.

10. Contingency costs are assumed to be 20% of total cost for the girder manufacturing in the precast plant and for on-site activities. Although activities on site are subject to a higher degree of change and are unpredictable to a larger extent, the contingency percentage is assumed to be the same for the activities inside the precast plant and on-site.
11. Rental expenses for smaller equipment required for performing the various tasks are factored into the labor and material charges.

The subsequent sections discuss an example cost analysis and schedule for Design 4: 5-Tx54-UHPC-girders with 4-inch wide flange, 2.5-inch site-cast UHPC deck with slab reinforcement; in detail and provides summarized tables for all the six different designs considered. As the assumptions and unit costs considered are similar for all the design options, the changes in the cost and schedule will be reflective of the changes in girder geometry and connection designs.

4.3 Prestress Plant Cost Analysis

Table 4-1 shows the summary of the cost analysis for the different activities that take place in the prestress plant for Design 4.

The entire process of casting the girder is divided into sub-tasks and the cost associated with each task is divided into material cost and labor charges. The unit costs for the materials, hourly wages including overhead charges for the labor, time spend for each of the processes etc. are detailed in the table with units.

Table 4-1: Design 4 – Precast plant cost analysis per girder

Activity	Labor Cost			Material Cost			Total Cost
	Hourly Rate	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Drawing strands	$40	3	$120	$1500 per ton	3.3 ton	$4,950	$5,070
Harping strands	$40	3	$120				$120
Stressing strands	$40	6	$240				$240
Placing steel formwork	$40	3	$120				$120
Placing flange formwork	$40	6	$240				$240
Placing flange steel	$40	3	$120	$1200 per ton	0.28	$336	$456
Placing U-hoops	$40	2	$80	$650 per ton	0.1	$65	$145
Casting concrete	$40	12	$480	$650 per cu.yd.	40	$26,000	$26,480
Finishing concrete	$40	9	$360				$360
Removing formwork	$40	6	$240				$240
Cutting tendons	$40	3	$120				$120
Lifting the girder out and tidying up	$40	6	$240				$240
Site charges	$2,000	Total Labor Cost	$2,480	Total Material Cost		$31,351	$35,831
Contingency cost at 20%		$7,166	**Grand Total Rounded off to the Nearest Hundred**			**$43,000**	

The different steps involved in the girder manufacturing process involves drawing, harping and stressing the strands, assembling the girder and flange formwork, placing transverse and flange steel for girders that demand them based on design considerations, casting concrete and curing. After a period of curing which is usually around 16 hours, the formwork is removed and the strands are cut to release the prestress, after which the girder is removed from the casting bed.

In the precast plant, the casting of a girder takes place within 1 workday cycle so that the bed is free the next day to cast the next girder. The Girder is typically removed within 24 hours of starting the manufacturing process and is stored for curing and strengthening within the precast plant. The period of storage depends on the timeline of the construction schedule and is assumed to be 40-60 days.

Similar to Table 4-1, cost analysis was done for the other design options and the results obtained are summarized in Table 4-2. The labor and material cost involved in the construction of one girder is multiplied by the number of girders in each design (6 for Design 1 and 5 for the remaining five designs) in Table 4-2.

Design 1 which uses normal concrete requires additional transverse reinforcement for shear and bursting forces in the end region in the girder due to which there is an increase in the labor cost. Another aspect of importance is that Design 1 uses six girders instead of five.

Table 4-2: Cost analysis summary – Total Girder manufacturing

Design Option	Cost		
	Labor Cost	Material Cost	Total Cost with Contingencies
Design 1	$40,800	$81,200	$146,000
Design 2	$20,000	$107,000	$152,000
Design 3	$21,800	$156,800	$214,000
Design 4	$22,400	$156,800	$215,000
Design 5	$22,000	$168,500	$229,000
Design 6	$22,600	$182,200	$246,000

4.4 Prestress Plant Schedule

As mentioned in the previous section, for the purpose of optimum utilization of the girder casting beds in any precast plant, the entire casting process may follow a sequential pattern with minimum overlap of activity, finishing off within 24 hours (with about 16 hours given for curing the girder before lifting it off the bed). This is necessary to prevent the site charges from doubling for individual girders.

Table 4-3 demonstrates the hourly breakdown of the girder manufacturing schedule for Design 1 which uses normal concrete for the casting of Tx54 girders. This time schedule accounts for the placement of shear reinforcement at regular intervals in the girder as the material used for casting is 4 ksi normal concrete.

Table 4-4 shows the breakdown of schedule for Design 2 which uses UHPC for the casting of Tx54 girders. Table 4-5 shows the time schedule applicable for Designs 3 and 4 which has a wide flange attachment and U-hoops for composite action with the deck. And Table 4-6 presents the time schedule for Designs 5 and 6 with wide flanges but no U-hoops.

Activities that overlap over one another may be done simultaneously or be completed within the same hour. As the number of days saved in the precast plant only influences the site charges, the determining factor when analyzing the girder manufacturing process is the cost of the raw materials used as shown in the cost analysis in the previous section.

Table 4-3: Design 1 - Girder manufacturing schedule in precast plant

Activities	Hours																							
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Drawing strands	■																							
Harping strands	■																							
Stressing strands	■	■																						
Placing steel stirrups			■	■	■																			
Placing steel formwork					■																			
Casting concrete						■	■																	
Curing (16 Hrs.)								■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	
Finishing Concrete									■															
Removing formwork																							■	
Releasing Prestress																								■
Removing girder and cleaning																								■

Table 4-4: Design 2 - Girder manufacturing schedule in precast plant

Activities	Hours																							
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Drawing strands	■																							
Harping strands	■																							
Stressing strands	■	■																						
Placing U-hoops			■																					
Placing steel formwork			■																					
Casting concrete				■	■																			
Curing (18 Hrs.)						■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	■	
Finishing Concrete							■																	
Removing formwork																							■	
Releasing Prestress																								■
Removing girder and cleaning																								■

Table 4-5: Design 3 and 4 - Girder manufacturing schedule in precast plant

Activities	Hours																							
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Drawing strands																								
Harping strands																								
Stressing strands																								
Placing U-hoops																								
Placing flange steel																								
Placing steel formwork																								
Placing flange formwork																								
Casting concrete																								
Curing (18 Hrs.)																								
Finishing Concrete																								
Removing formwork																								
Releasing Prestress																								
Removing girder and cleaning																								

Table 4-6: Design 5 and 6 - Girder manufacturing schedule in precast plant

Activities	Hours																							
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
Drawing strands																								
Harping strands																								
Stressing strands																								
Placing flange steel																								
Placing steel formwork																								
Placing flange formwork																								
Casting concrete																								
Curing (18 Hrs.)																								
Finishing Concrete																								
Removing formwork																								
Releasing Prestress																								
Removing girder and cleaning																								

4.5 On-site Cost Analysis

Table 4-7 presents the cost analysis for Design 4 – 5-Tx54-UHPC-Girders with 4-inch wide flange, 2.5-inch site-cast UHPC deck slab reinforcement. As mentioned in the assumptions before, the cost of UHPC is considered to be higher on-site because of the limited quantity in which it is procured and the difficulty in mixing the concrete and the steel fibers in the right proportion which could be much simpler inside the precast plant when done by experienced laborers in a controlled environment.

The activities involved in the on-site construction process involves site-mobilization, transportation of girders from the precast plant to the construction site, lifting and placing the girders and precast panels (if required), placing deck steel, erecting formwork and casting the concrete. After these steps, the deck is tidied up, barriers are installed the road is striped. Erecting formwork for the overhanging ends of the deck is only required for Designs 1 and 2, taking an entire day's work. For the other designs, the casting only requires side forms which could be installed faster.

The cost associated with the casting of the longitudinal seams is broken down into the cost of placing the formwork and the cost for casting the UHPC. It should be noted that longitudinal seams are connected using UHPC irrespective of the type of concrete used for deck overlay. No additional longitudinal seam formwork cost is considered for Designs 3 and 6 as the girders touch each other. These designs only require end-forms placed at the ends of the span.

Table 4-7: Design 4 – On-site construction cost analysis

Activity	Labor Cost					Material Cost			Total Cost
	Number of Workers	Hourly Rate	Hours	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Site mobilization	4	$50	4	16	$800				$800
Transportation	4	$60	8	32	$1,920	$7 per mile	1000	$8,400	$10,320
Cranes lifting and placing the girders	6	$50	10	60	$3,000	$1100 per crane day	2	$2,200	$5,200
Placing seam formwork	8	$50	4	32	$1,600				$1,600
Casting longitudinal seam	4	$50	4	16	$800	$2000 per cu.yd.	3.3	$6,600	$7,400
Placing deck steel	24	$50	24	576	$28,800	$850 per ton	3.5	$2,975	$31,775
Erect deck formwork and screed	8	$50	8	64	$3,200				$3,200
Casting concrete	4	$50	8	32	$1,600	$750 per cu.yd.	42.6	$31,950	$33,550
Screed and finish concrete	16	$50	8	128	$6,400				$6,400
Tidy up	8	$50	8	64	$3,200				$3,200
Curing time	2	$50	15	30	$1,500				$1,500
Removing formwork	4	$50	4	16	$800				$800
Install barrier	12	$50	8	96	$4,800	$270 per 4-ft	60	$16,200	$21,000
Stripe road	4	$50	4	16	$800				$800
Site clean-up	10	$50	8	80	$4,000				$4,000
			Total Labor Cost		$63,200	Total Material Cost		$68,300	$131,500
Contingency cost at 20%		$26,300			**Grand Total Rounded off to the Nearest Thousand**			**$158,000**	

Most of the activities involved in the on-site construction process are similar for the designs except when it comes to particular items such as placing deck steel for Designs 1, 2 and 4, and placing post-tensioned mono-strands for Design 3. Also Design 6 does not having additional side or overhang formwork. These peculiarities for each of the designs are captured and accounted for in the cost analysis performed.

Table 4-8 summarizes the cost associated with the different design options considered. The costs are broken down into labor and material costs and a contingency of 20% is considered for the analysis. The assumptions based on which Table 4-7 was prepared are discussed in detail in section 4.2. Similar estimates are prepared for the other five designs considered and are summarized below.

It can be observed that Designs 1 and 2 which uses 4-inch precast panels and 4.5-inch conventional concrete deck, has the largest labor force among all the options. This is due to the additional expense accounted for the installation of longitudinal and transverse reinforcement. For conventional concrete deck with no post-tensioned strands, #4 rebars placed at 8-inch center-to-center spacing is considered as reinforcement in either direction. This increases the labor force required for the tying of the reinforcement and increases the amount of mild steel to be procured.

The difference of $600 observed between the labor costs for Design 1 and Design 2 is caused due to the extra amount required for transporting and erecting an additional girder for the first design option (Design 1 considers six conventional concrete girders while Design 2 considers five UHPC girders).

Table 4-8: Cost analysis summary – On-site Construction

Design Option	Construction cost		
	Labor Cost	**Material Cost**	**Total Cost with Contingencies**
Design 1	\$75,800	\$84,400	\$192,000
Design 2	\$75,200	\$82,800	\$190,000
Design 3	\$47,600	\$51,000	\$118,000
Design 4	\$63,200	\$68,300	\$158,000
Design 5	\$34,400	\$54,400	\$107,000
Design 6	\$19,200	\$37,400	\$68,000

The remaining 4 designs have varying thicknesses for the top flanges, thus it may be observed that for Designs 5 and 6, the labor charges and the material cost decrease proportionally with an increase in the flange thickness. As the top flange thickness increases, the amount of UHPC required for deck casting correspondingly reduces and hence the site-work material cost required also reduces.

Table 4-9 shows the cost comparison for the six different design options considered. The total girder procurement cost is obtained by multiplying the cost of manufacturing one girder by the total number of girders. The on-site construction charges are added with it and shown in the table. Graphical representation of Table 4-9 is shown in Figure 4-1.

4.6 On-site Construction Schedule

The number of days spent for construction on-site may have a significant impact on the overall project delivery for highway bridges (Khaleghi et.al. 2012). The different steps involved in the construction process are analyzed and arranged in sequential order following principles of accelerated bridge construction.

Tables 4-10 through 4-14 depict the on-site schedule of construction for Designs 1 to 6. Activities taking place at some point of time during a day is marked for the whole day. The events are assumed to take place with an aggressive schedule in alignment with the ABC. Among the tables, Designs 1 and 2 have a similar schedule. Also, Designs 4 and 5 have similar schedules due to the similarity in the execution of on-site construction steps.

Table 4-9: Total cost analysis summary

Design Option	Cost		
	Girder Cost	**On-site Cost**	**Total Cost**
Design 1	$146,000	$192,000	$338,000
Design 2	$152,000	$190,000	$342,000
Design 3	$214,000	$118,000	$332,000
Design 4	$215,000	$158,000	$373,000
Design 5	$229,000	$107,000	$336,000
Design 6	$246,000	$68,000	$314,000

Table 4-10: Design 1 and 2 – On-site construction schedule

Activities	Days											
	1	2	3	4	5	6	7	8	9	10	11	Days 12-35
Lifting and placing the girders												
Placing precast panels												
Erect overhang formwork												
Placing deck steel												
Cast concrete												
Screed and finish concrete												
Tidy up												
Curing time (28 days)												
Removing formwork												
Install barrier												
Stripe road												
Site clean-up												

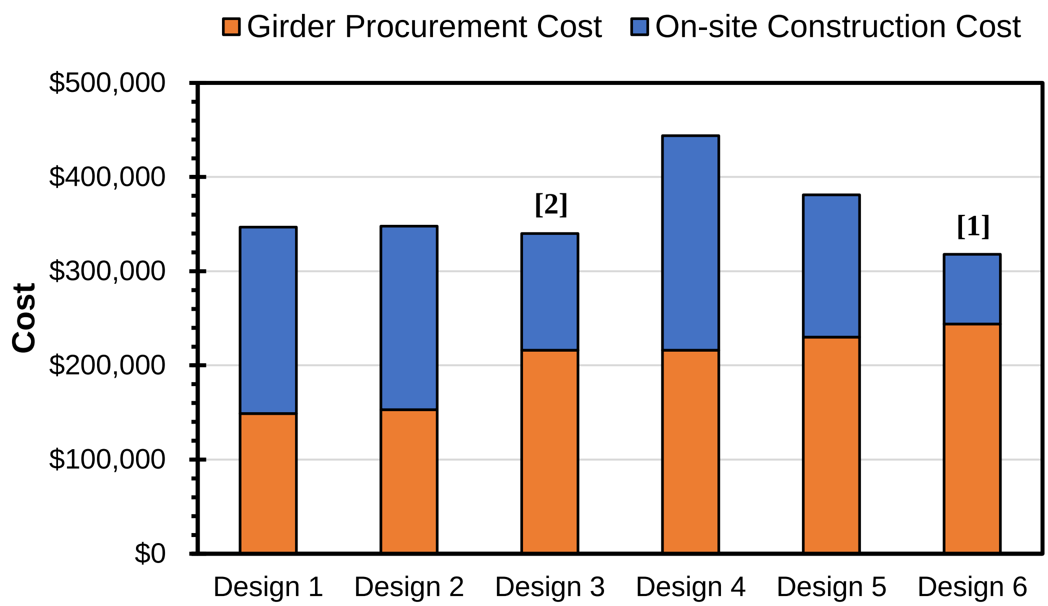


Figure 4-1: Comparative cost analysis summary for the six different design options considered. Most promising alternatives:[1]. Fastest construction time schedule;[2]. Most likely the most enduring

Table 4-11: Design 3 – On-site construction schedule

Activities	Days										
	1	2	3	4	5	6	7	8	9	10	Days 11 to 31
Lifting and placing the girders	■										
Casting longitudinal seams		■									
Placing side-form and screed		■									
Laying PT strands		■									
Casting concrete			■								
Screed and finish concrete			■								
Tidy up				■							
Curing time (7 days)				■	■	■	■	■	■	■	Optional curing
Removing formwork					■						
Stressing PT strands						■					
Install barrier						■					
Stripe road						■					
Site clean-up							■				

Table 4-12: Design 4 – On-site construction schedule

Activities	1	2	3	4	5	6	7	8	9	10	11
Lifting and placing the girders	■										
Placing seam formwork		■									
Casting longitudinal seams		■									
Placing side-form and screed		■									
Placing deck steel			■	■	■						
Casting concrete						■					
Screed and finish concrete						■					
Tidy up							■				
Curing time (5 days)							■	■	■	■	■
Removing formwork								■			
Install barrier									■		
Stripe road									■		
Site clean-up										■	

Table 4-13: Design 5 – On-site construction schedule

Activities	Days							
	1	2	3	4	5	6	7	8
Lifting and placing the girders	■							
Placing seam formwork		■						
Casting longitudinal seams		■						
Placing side-form and screed		■						
Casting concrete			■					
Screed and finish concrete			■					
Tidy up				■				
Curing time (5 days)				■	■	■	■	■
Removing formwork					■			
Install barrier						■		
Stripe road						■		
Site clean-up							■	

Table 4-14: Design 6 – On-site construction schedule

Activities	Days						
	1	2	3	4	5	6	7
Lifting and placing the girders	■						
Casting longitudinal seams		■					
Finish concrete		■					
Tidy up		■					
Curing time (5 days)			■	■	■	■	■
Install barrier				■			
Stripe road					■		
Site clean-up						■	

Table 4-15 presents the summary of the on-site construction time schedule for the six designs considered. For the purpose of representation, weekends are avoided and only workdays are considered for the Tables.

Designs 1 and 2 takes a longer time to complete because of the additional overhang formwork placement, longitudinal and transverse deck reinforcement and the extra normal concrete that needs to be cast and which then takes longer to cure. As the decks are made of conventional concrete for both these options, laying longitudinal and transverse reinforcement is necessary. This slow bridge construction approach considers 28 days for the curing of normal concrete. The assembly of an extra girder also slows down the construction of Design 1 which uses six conventional concrete girders. Design 3 uses post-tensioned strands which eliminates the requirement for slab reinforcement. Instead, the deck is prestressed, the number of days required for curing could be cut short to 7 days unlike the 28 days required for a normal reinforced deck. However, if the bridge construction takes place at locations where accelerated bridge construction is not essential such as an off-line renewal, providing 28 days for curing is the suggested procedure.

The most rapid construction schedules are for Designs 4 and 5 which use site-cast UHPC. Among them, Design 6 only needs the connection between the girders cast on site taking away the need to have any side-forms or screeding.

4.7 Summary and Findings

This chapter has explored cost estimates and construction scheduling for each of the six different bridge deck designs. Based on the assumptions adopted, cost estimates and construction schedule results, the following observations are made:

Table 4-15: Schedule Summary – On-site construction

Design Option	Number of Days	Comment
Design 1	35	NC Deck
Design 2	35	NC Deck
Design 3	10	PT Deck
Design 4	11	Reinforced UHPC
Design 5	8	Site-cast UHPC
Design 6	7	Site-cast UHPC

1. The girder manufacturing cost was found to be minimum for the conventional concrete Tx54 girder (Design 1 at $146,000). Most of the cost associated with girder manufacturing is from the cost of procuring the concrete used. The cost of 8.5 ksi conventional concrete is only 40% of the cost of UHPC and this difference drives the total girder manufacturing cost up for the other design options considered even though they use one girder less. However, the cost of Design 2 – UHPC Tx54 girders came close at $152,000. Compared to design option 1, this is only a 4.1% increase in the cost. Even though UHPC is much costlier than conventional concrete, the transverse reinforcement required for conventional concrete and the extra girder required for Design 1 results in this small difference.
2. The on-site construction cost was found to be minimum for Design 6 at $68,000 which only involves the placing of girder and the casting of four girder-to-girder seam connections with UHPC; this eliminates multiple steps required for the other designs. The on-site construction cost for Design 6 is only 36% of cost for the TxDOT standard - Design 1. This is due to the additional formwork erection, precast panel assembly and deck steel placement required for the 4-inch precast panel + 4.5-inch NC deck.
3. The total cost of construction including the girder procurement was found to be least for Design 6 at $314,000. Compared to Design 1 at $338,000, there is an 8.3% reduction in the total cost of construction. It could be argued that this slight difference is within the bounds of uncertainty of estimating which makes a difficult

case to be convincing to switch to UHPC. What can be more compelling is the time schedule.

4. The schedule summary for on-site construction shown in Table 4-13 shows the effectiveness of using a full-thickness precast top flange. While the conventional concrete option takes 35 days to complete construction assuming no delays, Design 6 only needs some 7 days to complete the construction of the span. This is a significant advantage when considering the lane-closure costs, traffic diversions and fuel wastage associated with the additional 28 days for construction and curing.
5. Comparing the cost and schedule for the different design options considered, it is evident that Design-6: 5-Tx54-UHPC-Girders with 6.5-inch wide flange, site-cast UHPC connection; has a considerable advantage over the other designs. Design 6 was found to be the most economical in terms of both cost and time. Another important consideration is the lifetime cost of the UHPC bridge. While normal concrete bridges are expected to have a nominal lifespan of 75 years, UHPC bridges are claimed to be adequate for 150 years due to the low moisture permeability of the UHPC. Factoring this in makes Design-6 the most attractive option.

CHAPTER 5

SUMMARY, CONCLUSION AND RECOMMENDATIONS

5.1 Summary

This research investigated the design and constructability of slab-on-girder Ultra High-Performance Concrete (UHPC) bridge spans using different configurations. In order to reduce the number of on-site construction days, the use of wide flanges of varying thicknesses were investigated. The thesis evaluates five UHPC design scenarios, which are compared to the standard TxDOT way of constructing highways with Tx54 girders using normal concrete. The different longitudinal girder-to-girder connection options were also investigated. For each design, detailed estimates were made together with construction engineering schedules. Among the five alternatives compared to the normal concrete with the precast panelized deck benchmark, it is evident that a full thickness UHPC precast deck slab top flange, identified in the thesis as Design 6 is the cheapest alternative and competitive with the conventional construction alternative. This design was also identified as the most rapid to construct requiring only one week of on-site construction time. By contrast, the most enduring solution had a half-depth field cast topping which was transversely post-tensioned to actively tie all units together to mitigate the possibility of longitudinal cracking at the girder-to-girder connections.

5.2 Answering the Research Questions

This section restates the research questions posed at the end of the literature review and seeks to answer them in light of the research performed.

Question 1 *- What impact does a UHPC wide flange cast in the prestress plant have on the overall design strength and long-term deflection associated with the construction of the single span bridge?*

Having a UHPC wide flange cast in the prestress plant increases the design strength and the flexural capacity of the girder as it increases the moment of inertia of the section and effectively the flexural rigidity of the girder. This in turn reduces the initial camber built into the girder. As the design was performed to minimize the deflections, good evidence to represent this argument is the fact that the long-term deflection reduced from 0.58-inch to 0.11-inch with the addition of the 6.5-inch-wide flange for the same number of strands.

Question2 *- What thickness of the top wide flange is the most economical solution in comparison with the conventional 4-inch precast panels and 4.5-inch cast-in-place topping alternative?*

Among the alternatives with wide flange considered, the thickness of the top wide flange considered were 4-inch, 5-inch and 6.5-inch. The cost of construction of the girder in the precast plant steadily increases with the increase in the thickness of the wide top flange. However, this additional cost is largely offset by minimizing tasks to be conducted on-site. Moreover, the time savings are substantial.

The most rapid construction schedule was for the full-thickness top flange where ABC principles could be applied. As the entire deck casting is completed inside the precast plant and only the longitudinal seam connections had to be cast-on-site, the construction time for the full-thickness flange solution was found to be only 7 days. The considerable time saving on-site also leads to reduction of indirect costs from lane-closure delays, additional traffic rerouting expenses, fuel charges etc.

Question 3 *– Is it possible to eliminate or at least control cracks along longitudinal seams for the girder-to-girder connections?*

Multiple options are discussed to control or mitigate potential longitudinal cracks along the girder-to-girder seams. Using UHPC instead of conventional concrete by itself will help prevent (but not necessarily eliminate) the evolution of cracks due to its high strength and superior bond strength. Transverse post-tensioning used in Design-3 is identified as the best alternative to inhibit the evolution of longitudinal cracks. The prestress serves as a surrogate for the absence of bond strength at the normal concrete to UHPC interface. A deck prestressed in the transverse direction should have high resistance to the formation of cracks throughout the lifespan of the structure. The connection used for Design 4 using a reinforced topping slab is the next best alternative to prestressing as the UHPC reinforced diaphragm is intended to provide integrity throughout and specifically along the girder-to-girder connection. The full-thickness flange with strong UHPC-reinforced connection used in Design 6 should also limit cracking but not necessarily eliminate it due to the absence of transverse prestress.

5.3 Conclusion

Based on the research conducted, the following conclusions may be drawn:

1. UHPC is a viable alternative to conventional concrete in the field of highway bridge construction. The enhanced structural properties of UHPC includes very

high compressive strength, ductility, toughness, as well as exceptional durability and flowability. While the tensile capacity of UHPC is only marginally higher compared to normal concrete, it is the additional toughness that makes a principal difference. However, as quality control is a very important factor for materializing these properties, the best construction practice is to bring more of the UHPC casting inside the precast plant where strict quality control can be enforced.

2. A major insight from the thesis was the comparison of the cost and time involved with the construction of the different design options. Breaking down the step involved in the manufacturing of the girder and the assembly on site showed that the overall cost for Design 6 was 8.7% less than the benchmark design. Considering the multitude of additional benefits from using UHPC including additional resistance to damages, almost twice-long life span and a 80% decrease in the time required to assemble the bridge on-site, Design 6 is a better alternative than the TxDOT standard - Design 1.
3. A possible concern that might evolve during the life span of constructing wide flange girders with a thin cast-on-site deck is the potential evolution of longitudinal cracks along the seams of the girder-to-girder connections. An approach to mitigate this possibility is to install post-tensioned strands every 18-inches throughout the length of the bridge.
4. It is beneficial to have a wider flange attached to the top of the girder cast in the precast plant as the wide flange considerably pulls up the center of gravity

of the concrete section and provides a larger eccentricity for the strands at the bottom. The effectively reduces the initial camber required to counter the long-term deflections of the girder as was observed in Design 6. It was seen that using a 6.5-inch-wide flange on a traditional Tx54 resulted in the initial camber reducing 77% and long-term deflection reducing 81% in comparison with the benchmark design.

5.4 Recommendations for Future Research

Based on the research conducted, the following research recommendations are made:

1. This work considered only simply supported spans. The next step is to investigate the viability of full-thickness top flange girders for continuous spans. The effectiveness of using UHPC for continuous spans shall also be compared with normal concrete. Negative moments are expected over the interior supports which can lead to cracking in the deck, because the full-thickness UHPC flanges are pretensioned, cracking of the deck slab can be avoided, by design.
2. Life-cycle costs of UHPC v/s normal concrete is another area that needs to be explored. UHPC has only been in the market for a few decades but is considered to have a lifespan of over 150 years according to many researchers. Corroboration of this thinking is needed through analysis and accelerated testing. Looking into the cost associated with the maintenance UHPC girders and deck will help push it further into the market as a viable option in comparison with normal concrete.
3. The long-term multipliers considered for the deflection analysis are universal averages and there might be localized variations. Also, the equations and constants

were developed for conventional concrete and the design assumptions considers the same constant multipliers to be valid irrespective of the strength of the concrete considered. For future research, these constants could be revisited, and changes could be made for analyzing the deflection of Ultra High-Performance Concrete structures.

4. For the cost analysis and schedule calculations, several assumptions are considered to be valid which includes highly variable parameters like site conditions and employee wages which vary from site-to site. The completion times for each of the activities included in the construction process is also based on multiple assumptions and an aggressive work schedule. These assumptions might be altered in course of time to have a more accurate prediction of the cost and time required to execute each of the construction activities. Therefore, a comprehensive sensitivity study should be performed accordingly.
5. Analysis and design, testing and evaluation of different connection options using UHPC to prevent the propagation of cracks along longitudinal girder-to-girder interfaces for single span bridges and transverse interfaces where the cross-section of one girder meets another girder for multi-span bridges is also worth exploring. Exploiting the properties of UHPC for these connections would save bridge maintenance charges by preventing potential crack propagation.

REFERENCES

Agarwal, H. B. (2020). Deflection Minimization of Prestressed Concrete Girders. MS Thesis, Zachry Department of Civil Engineering, Texas A&M University.

Balakumaran, S.S.G., Weyers, R.E., Brown, M.C. (2018). Linear Cracking in Bridge Decks. Virginia Transportation Research Council. Report No. FHWA/VTRC 18-R13

Culmo, M. P. (2011). Accelerated Bridge Construction: Experience in Design, Fabrication and Erection of Prefabricated Bridge Elements and Systems. Federal Highway Administration Report No. FHWA-HIF-12-013.

Deng, S., Shao, X., Yan, B., Wang, Y., and Li, H. (2020). On Flexural Performance of Girder-To-Girder Wet Joint for Lightweight Steel-UHPC Composite Bridge. Applied Sciences. 10. 1335. 10.3390/app10041335.

El-Helou, R. G. and Graybeal, B. A., (2019) The Ultra Girder: A Design Concept for a 300-foot Single Span Prestressed Ultra-High Performance Concrete Bridge Girder, International Interactive Symposium on Ultra High-Performance Concrete 2(1).

Graybeal, B.A. (2006). Material Property Characterization of Ultra High-Performance Concrete. Report No. FHWA-HRT-06-103. 1-176.

Graybeal, B.A. (2014). Design and Construction of Field-Cast UHPC Connections. 10.13140/RG.2.2.25221.76001. Report No. FHWA-HRT-14-084.

Graybeal, B.A, and Zhang, G. (2014). Development of a Family of Ultra-High Performance Concrete Pi-Girders. FHWA Tech Brief: PB2014-1100626, 1-8.

Gunasekaran, D. (2020). Mechanics and Design of Prestressed Girder Bridges Using Ultra High-Performance Concrete. MS Thesis, Zachry Department of Civil Engineering, Texas A&M University.

Hueste, M. B. D., Mander, J. B. and Parkar, A. S. (2012). Continuous Prestressed Concrete Girder Bridges Volume 1: Literature Review and Preliminary Designs. Texas Department of Transportation. Report No. FHWA/TX-12/0-6651-1.

Jiang, D. (2015). Experiments, Analysis and Design Models for Slab on Prestressed Concrete Girder Bridge Structures. Doctoral dissertation, Texas A & M University.

Kakizaki, M. (1992). Effect of Mixing Method on Mechanical Properties and Pore Structure of Ultra High-Strength Concrete, Katri Report No. 90, Kajima Corporation, Tokyo.

Khaleghi, B., Schultz, E., Seguirant, S., Marsh, L., Eberhard, M., and Stanton, J. (2012). Accelerated Bridge Construction in Washington State-From Research to Practice. PCI J. 57(4), 34-49

Kim, J.Y. (2018). Development of Cost-Effective Ultra High-Performance Concrete (UHPC) for Colorado's Sustainable Infrastructure. Colorado Department of Transportation – Research Report No. CDOT-2018-15

Martin, L. D. (1977). A Rational Method for Estimating Camber and Deflection of Precast Prestressed Members. PCI Journal.

Naaman, A. E. (2004). Prestressed Concrete Analysis and Design: Fundamentals (2nd ed.) Ann Arbor, Michigan, USA: Techno Press 3000.

Park, J. S., Kim, Y. J., Cho, J. R., and, Jeon, S. J. (2015). Early-age strength of ultra-high-performance concrete in various curing conditions. Materials, 8 (8): 5537–5553.

Perry, V.H., and Seibert, P.J. (2011). Working with Ductal ultra high-performance concrete, Concrete Plant International, 1(11), 2-5.

Russell, H.G., and Graybeal, B.A. (2013). Ultra High-Performance Concrete: A State-of-the-Art Report for the Bridge Community. Report No. FHWA-HRT-13-060.

Wang, J., Liu, J., Wang, Z., Liu, T., and Zhang, J. (2021). Cost-Effective UHPC for Accelerated Bridge Construction: Material Properties, Structural Elements, and Structural Applications. ASCE Journal of Bridge Engineering, 26 (2): 04020117.

APPENDIX A

DESIGN CALCULATIONS

A.1 Overview

Adjoined in the coming section is the MATHCAD worksheet used as a template to perform the stress calculations for each of the designs considered. For representation purposes, the calculations shown corresponds to Design 4 used as an example in the thesis. The section properties of the girder cross section considered are shown in Table 3-1.

The force and eccentricity for the design is obtained from an Excel worksheet which calculates different strand layout and checks it against all the limiting condition equations used to prepare the stress blocks.

The various bridge design parameters used, maximum live load deflection limits, calculation of the section properties, and moment calculations are also detailed in the calculation worksheet.

A.2 MATHCAD Worksheet

Input Parameters:

Length, $L := 120\ ft$

$W := 46\ ft$

$Overhang := 3\ ft$

Number of girders, $N := 5$

Distributed Load, $w := 0.64\ \frac{kip}{ft}$

Point Load, $P_1 := 20\ kip$

Point load, $P_2 := 32\ kip$

Length between loads, $k := 14\ ft$

Distance between support and load, $a := \frac{L}{2} - k = 46\ ft$

$f'_c := 20\ ksi$

$E := 0.8 \cdot 57000 \cdot \sqrt{f'_c \cdot 1\ psi} = (6.449 \cdot 10^6)\ psi$

$\gamma_{eq} := 155.1\ \frac{lb}{ft^3}$

$Area_{TX54} := 817\ in^2$

Thickness of wide flange, $T_f := 4\ in$

Thickness of cast on site deck, $T_d := 2.5\ in$

Moment of Inertia of Girder, $I := 620337\ in^4$

Moment of Inertia of Composite Shape, $I_c := 757772\ in^4$

Centroid of Girder, $y_b := 35.51\ in$

Centroid of Composite Shape, $y_{bc} := 39.97\ in$

$F_i :=$ READEXCEL ("..\Excel FIles\Eccentricity-Force Calculator.xlsx", "UHPC-4" Flange!L41:L41")

$F_i := \det(F_i)$

Initial prestressing force, $F_i := F_i \cdot kip = (2.46 \cdot 10^3)\ kip$

$e_1 :=$ READEXCEL ("..\Excel FIles\Eccentricity-Force Calculator.xlsx", "UHPC-4" Flange!L38:L38")

$e_1 := \det(e_1)$

Eccentricity at endspan of girder, $e_1 := e_1 \cdot in = 12.977\ in$

$e_2 :=$ READEXCEL ("..\Excel FIles\Eccentricity-Force Calculator.xlsx", "UHPC-4" Flange!L34:L34")

$e_2 := \det(e_2)$

Eccentricity at midspan of girder, $e_2 := e_2 \cdot in = 18.843\ in$

Eccentricity at endspan of composite section, $e_{c1} := e_1 + y_{bc} - y_b = 17.437\ in$

Eccentricity at midspan of composite section, $e_{c2} := e_2 + y_{bc} - y_b = 23.303\ in$

$\gamma_{asphalt} := 145\ \frac{lb}{ft^3}$

Thickness of asphalt layer, $t_{asphalt} := 2.5\ in$

Bridge Design Parameters

$$S := \frac{(W - 2 \cdot Overhang)}{(N-1)} = 10\ ft$$

$$LLDF := \frac{S}{12\ ft} = 0.833$$

Concrete Section Properties

$Area_{girder} := Area_{TX54} + S \cdot T_f = \left(1.297 \cdot 10^3\right)\ in^2$

$Area_{deck} := S \cdot T_d = 300\ in^2$

$Area_c := Area_{girder} + Area_{deck} = \left(1.597 \cdot 10^3\right)\ in^2$

$Area_{asphalt} := S \cdot t_{asphalt} = 300\ in^2$

$y_t := \left(54\ in + T_f\right) - y_b = 22.49\ in$

$y_{tc} := \left(54\ in + T_f + T_d\right) - y_{bc} = 20.53\ in$

Section Modulus of Girder from top, $S_{xt} := \dfrac{I}{y_t} = \left(2.758 \cdot 10^4\right)\ in^3$

Section Modulus of Girder from bottom, $S_{xb} := \dfrac{I}{y_b} = \left(1.747 \cdot 10^4\right)\ in^3$

Section Modulus of Composite Shape from top, $S_{xtc} := \dfrac{I_c}{y_{tc}} = \left(3.691 \cdot 10^4\right)\ in^3$

Section Modulus of Composite Shape from bottom, $S_{xbc} := \dfrac{I_c}{y_{bc}} = \left(1.896 \cdot 10^4\right)\ in^3$

Allowable Stress Limit:

$f'_{ci} := 0.9 \cdot f'_c = 18\ ksi$

Initial Stage at Transfer:

$f_{ci} := -0.6 \cdot f'_{ci} = -10.8\ ksi$

$f_{ti} := 0.19 \cdot \sqrt{f'_{ci} \cdot ksi} = 0.806\ ksi$

Final Stage at Service:

$f_c := -0.6 \cdot f'_c = -12\ ksi$

$f_t := 0.19 \cdot \sqrt{f'_c \cdot ksi} = 0.85\ ksi$

Moment Calculation

Girder Self Weight, $W_{DG} := \gamma_{eq} \cdot Area_{girder} = \left(1.397 \cdot 10^3\right)\ \dfrac{lb}{ft}$

Deck Self Weight, $W_{DD} := \gamma_{eq} \cdot Area_{deck} = 323.125\ \dfrac{lb}{ft}$

Superimposed Dead Load due to Asphalt,

$W_{sidl} := \gamma_{asphalt} \cdot Area_{asphalt} = 302.083\ \dfrac{lb}{ft}$

Moment due to girder, $M_{DG} := W_{DG} \cdot g \cdot \dfrac{L^2}{8} = \left(2.515 \cdot 10^3\right)\ kip \cdot ft$

Moment due to deck, $M_{DD} := W_{DD} \cdot g \cdot \frac{L^2}{8} = 581.625\ kip \cdot ft$

Moment due to Live Load, $M_{LO} := 3590\ kip \cdot ft$ (point loads amplified by a factor of 1.3)

Adjusted moment due to Live load,
$M_{LL} := M_{LO} \cdot LLDF = \left(2.992 \cdot 10^3\right)\ kip \cdot ft$

Moment due to Superimposed DL, $M_{sidl} := W_{sidl} \cdot g \cdot \frac{L^2}{8} = 543.75\ kip \cdot ft$

Live Load Deflection Calculation

(i) Deflection due to the distributed load:

$$\Delta_1 := \frac{5}{384} \cdot \frac{w \cdot L^4}{E \cdot I_c} = 0.611\ in$$

(ii) Deflection due to Concentrated Point load:

$$\Delta_2 := \frac{1.3 \cdot P_2 \cdot L^3}{48 \cdot E \cdot I_c} = 0.53\ in$$

(iii) Deflection due to 2 Concentrated Point loads:

$$\Delta_3 := \frac{1.3 \cdot P_1 \cdot a}{24 \cdot E \cdot I_c} \cdot \left(3 \cdot L^2 - 4 \cdot a^2\right) = 0.612\ in$$

Total Deflection, $\Delta_{max} := \left(\Delta_1 + \Delta_2 + \Delta_3\right) \cdot LLDF = 1.461\ in$ (1.3 to account for the dynamic amplification factor)

Deflection Limit according to AASHTO is $\frac{L}{800} = 1.8\ in$

$$Result := \mathbf{if}\left(\Delta_{max} < \frac{L}{800}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Stress Diagram Calculations

1. At Transfer:

End Span

$$\sigma_{1te} := \frac{-0.95 \cdot F_i}{Area_{girder}} + \frac{0.95 \cdot F_i \cdot e_1}{S_{xt}} = -0.702\ ksi$$

$$Result := \mathbf{if}\left(f_{ci} < \sigma_{1te} < f_{ti}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\sigma_{1be} := \frac{-0.95 \cdot F_i}{Area_{girder}} - \frac{0.95 \cdot F_i \cdot e_1}{S_{xb}} = -3.538 \ ksi$$

$$Result := \text{if}\left(f_{ci} < \sigma_{1be} < f_{ti}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Midspan

$$\sigma_{1tm} := \frac{-0.95 \cdot F_i}{Area_{girder}} + \frac{0.95 \cdot F_i \cdot e_2}{S_{xt}} - \frac{M_{DG}}{S_{xt}} = -1.299 \ ksi$$

$$Result := \text{if}\left(f_{ci} < \sigma_{1tm} < f_{ti}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\sigma_{1bm} := \frac{-0.95 \cdot F_i}{Area_{girder}} - \frac{0.95 \cdot F_i \cdot e_2}{S_{xb}} + \frac{M_{DG}}{S_{xb}} = -2.595 \ ksi$$

$$Result := \text{if}\left(f_{ci} < \sigma_{1bm} < f_{ti}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

2. At Construction:

Endspan

$$\sigma_{2te} := \sigma_{1te} + \frac{0.075 \cdot F_i}{Area_{girder}} - \frac{0.075 \cdot F_i \cdot e_1}{S_{xt}} = -0.647 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{2te} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\sigma_{2be} := \sigma_{1be} + \frac{0.075 \cdot F_i}{Area_{girder}} + \frac{0.075 \cdot F_i \cdot e_1}{S_{xb}} = -3.259 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{2be} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Midspan

$$\sigma_{2tm} := \sigma_{1tm} + \frac{0.075 \cdot F_i}{Area_{girder}} - \frac{0.075 \cdot F_i \cdot e_2}{S_{xt}} - \frac{M_{DD}}{S_{xt}} = -1.536 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{2tm} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\sigma_{2bm} := \sigma_{1bm} + \frac{0.075 \cdot F_i}{Area_{girder}} + \frac{0.075 \cdot F_i \cdot e_2}{S_{xb}} + \frac{M_{DD}}{S_{xb}} = -1.855 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{2bm} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

2. At Service State:

Endspan

$$\sigma_{3te} := \sigma_{2te} + \frac{0.075 \cdot F_i}{Area_c} - \frac{0.075 \cdot F_i \cdot e_{c1}}{S_{xtc}} = -0.619 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{3te} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\sigma_{3be} := \sigma_{2be} + \frac{0.075 \cdot F_i}{Area_c} + \frac{0.075 \cdot F_i \cdot e_{c1}}{S_{xbc}} = -2.973 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{3be} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

Midspan

$$\sigma_{3tm} := \sigma_{2tm} + \frac{0.075 \cdot F_i}{Area_c} - \frac{0.075 \cdot F_i \cdot e_{c2}}{S_{xtc}} - \frac{M_{LL}}{S_{xtc}} - \frac{M_{sidl}}{S_{xtc}} = -2.686 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{3tm} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\sigma_{3bm} := \sigma_{2bm} + \frac{0.075 \cdot F_i}{Area_c} + \frac{0.075 \cdot F_i \cdot e_{c2}}{S_{xbc}} + \frac{M_{LL}}{S_{xbc}} + \frac{M_{sidl}}{S_{xbc}} = 0.726 \ ksi$$

$$Result := \text{if}\left(f_c < \sigma_{3bm} < f_t, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

APPENDIX B

COST ESTIMATE TABLES

Table B-1: Design 1 – Precast plant cost analysis per girder

Activity	Labor Cost			Material Cost			Total Cost
	Hourly Rate	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Drawing strands	$40	3	$120	$1500 per ton	3.3	$4,950	$5,070
Harping strands	$40	3	$120				$120
Stressing strands	$40	6	$240				$240
Placing Steel Stirrup	$40	72	$2,880	$1200 per ton	1.9	$2,280	$5,160
Placing steel formwork	$40	3	$120				$120
Casting concrete	$40	12	$480	$250 per cu.yd.	25.2	$6,300	$6,780
Finishing concrete	$40	9	$360				$360
Removing formwork	$40	3	$120				$120
Cutting tendons	$40	3	$120				$120
Lifting the girder out and tidying up	$40	6	$240				$240
Site charges	$2,000	Total Labor Cost	$4,800	Total Material Cost		$13,530	$20,330
Contingency cost at 20%		$4,066	**Grand Total Rounded off to the Nearest Hundred**			**$24,400**	

Table B-2: Design 2 – Precast plant cost analysis per girder

Activity	**Labor Cost**			**Material Cost**			**Total Cost**
	Hourly Rate	**Man Hours**	**Total Labor Cost**	**Unit Price**	**Number**	**Total Material Cost**	
Drawing strands	$40	3	$120	$1500 per ton	3.3	$4,950	$5,070
Harping strands	$40	3	$120				$120
Stressing strands	$40	6	$240				$240
Placing U-hoop	$40	2	$80	$650 per ton	0.1	$65	$145
Placing steel formwork	$40	3	$120				$120
Casting concrete	$40	12	$480	$650 per cu.yd.	25.2	$16,380	$16,860
Finishing concrete	$40	9	$360				$360
Cutting tendons	$40	3	$120				$120
Removing formwork	$40	3	$120				$120
Lifting the girder out and tidying up	$40	6	$240				$240
Site charges	$2,000	Total Labor Cost	$2,000	Total Material Cost		$21,395	$25,395
Contingency cost at 20%		$5,079	**Grand Total Rounded off to the Nearest Hundred**			**$30,500**	

Table B-3: Design 3 – Precast plant cost analysis per girder

Activity	Labor Cost			Material Cost			Total Cost
	Hourly Rate	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Drawing strands	$40	3	$120	$1500 per ton	3.3	$4,950	$5,070
Harping strands	$40	3	$120				$120
Stressing strands	$40	6	$240				$240
Placing steel formwork	$40	3	$120				$120
Placing flange formwork	$40	3	$120				$120
Placing U-hoops	$40	2	$80	$650 per ton	0.1	$65	$145
Placing flange steel	$40	3	$120	$1200 per ton	0.28	$336	$456
Casting concrete	$40	12	$480	$650 per cu.yd.	40	$26,000	$26,480
Finishing concrete	$40	9	$360				$360
Removing formwork	$40	6	$240				$240
Cutting tendons	$40	3	$120				$120
Lifting the girder out and tidying up	$40	6	$240				$240
Site charges	$2,000	Total Labor Cost	$2,360	Total Material Cost		$31,351	$35,711
Contingency cost at 20%		$7,142	**Grand Total Rounded off to the Nearest Hundred**			**$42,900**	

Table B-4: Design 5 – Precast plant cost analysis per girder

Activity	Labor Cost			Material Cost			Total Cost
	Hourly Rate	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Drawing strands	$40	3	$120	$1500 per ton	3.3	$4,950	$5,070
Harping strands	$40	3	$120				$120
Stressing strands	$40	6	$240				$240
Placing steel formwork	$40	3	$120				$120
Placing flange formwork	$40	6	$240				$240
Placing flange steel	$40	3	$120	$1200 per ton	0.28	$336	$456
Casting concrete	$40	12	$480	$650 per cu.yd.	43.7	$28,405	$28,885
Finishing concrete	$40	9	$360				$360
Removing formwork	$40	6	$240				$240
Cutting tendons	$40	3	$120				$120
Lifting the girder out and tidying up	$40	6	$240				$240
Site charges	$2,000	Total Labor Cost	$2,400	Total Material Cost		$33,691	$38,091
Contingency cost at 20%		$7,618	**Grand Total Rounded off to the Nearest Hundred**			**$45,700**	

Table B-5: Design 6 – Precast plant cost analysis per girder

Activity	Labor Cost			Material Cost			Total Cost
	Hourly Rate	**Man Hours**	**Total Labor Cost**	**Unit Price**	**Number**	**Total Material Cost**	
Drawing strands	$40	3	$120	$1500 per ton	3.3	$4,950	$5,070
Harping strands	$40	3	$120				$120
Stressing strands	$40	6	$240				$240
Placing steel formwork	$40	3	$120				$120
Placing flange formwork	$40	6	$240				$240
Placing flange steel	$40	6	$240	$1200 per ton	0.56	$672	$912
Casting concrete	$40	12	$480	$650 per cu.yd.	47.4	$30,810	$31,290
Finishing concrete	$40	9	$360				$360
Removing formwork	$40	6	$240				$240
Cutting tendons	$40	3	$120				$120
Lifting the girder out and tidying up	$40	6	$240				$240
Site charges	$2,000	Total Labor Cost	$2,520	Total Material Cost		$36,432	$40,952
Contingency cost at 20%		$8,190	**Grand Total Rounded off to the Nearest Hundred**			**$49,100**	

Table B-6: Design 1 – On-site construction cost analysis

Activity	Labor Cost					Material Cost			Total Cost
	Number of Workers	**Hourly Rate**	**Hours**	**Man Hours**	**Total Labor Cost**	**Unit Price**	**Number**	**Total Material Cost**	
Site mobilization	4	$50	4	16	$800				$800
Transportation	4	$60	16	64	$3,840	$7 per mile	1200	$8,400	$12,240
Cranes lifting and placing the girders	6	$50	12	72	$3,600	$1100 per crane day	3	$3,300	$6,900
Lifting and placing precast panels	8	$50	8	64	$3,200	$170 per panel	150	$25,500	$28,700
Placing deck steel	24	$50	24	576	$28,800	$1200 per ton	6.5	$5,525	$34,325
Erect deck formwork and screed	12	$50	8	96	$4,800				$4,800
Casting concrete	4	$50	8	32	$1,600	$250 per cu.yd.	102	$25,500	$27,100
Screed and finish concrete	16	$50	8	128	$6,400				$6,400
Tidy up	8	$50	8	64	$3,200				$3,200
Curing time	2	$50	84	168	$8,400				$8,400
Removing formwork	4	$50	8	32	$1,600				$1,600
Install barrier	12	$50	8	96	$4,800	$270 per 4-ft	60	$16,200	$21,000
Stripe road	4	$50	4	16	$800				$800
Site clean-up	10	$50	8	80	$4,000				$4,000
			Total Labor Cost		$75,800	Total Material Cost		$84,400	$160,200
Contingency cost at 20%		$32,040			**Grand Total Rounded off to the Nearest Thousand**			**$192,000**	

Table B-7: Design 2 – On-site construction cost analysis

Activity	Labor Cost					Material Cost			Total Cost
	Number of Workers	Hourly Rate	Hours	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Site mobilization	4	$50	4	16	$800				$800
Transportation	4	$60	16	64	$3,840	$7 per mile	1000	$8,400	$12,240
Cranes lifting and placing the girders	6	$50	10	60	$3,000	$1100 per crane day	3	$3,300	$6,300
Lifting and placing precast panels	8	$50	8	64	$3,200	$230 per panel	120	$27,600	$30,800
Placing deck steel	24	$50	24	576	$28,800	$850 per ton	6.5	$5,525	$34,325
Erect deck formwork and screed	12	$50	8	96	$4,800				$4,800
Casting concrete	4	$50	8	32	$1,600	$250 per cu.yd.	87	$21,750	$23,350
Screed and finish concrete	16	$50	8	128	$6,400				$6,400
Tidy up	8	$50	8	64	$3,200				$3,200
Curing time	2	$50	84	168	$8,400				$8,400
Removing formwork	4	$50	8	32	$1,600				$1,600
Install barrier	12	$50	8	96	$4,800	$270 per 4-ft	60	$16,200	$21,000
Stripe road	4	$50	4	16	$800				$800
Site clean-up	10	$50	8	80	$4,000				$4,000
			Total Labor Cost		$75,200	Total Material Cost		$82,800	$158,000
Contingency cost at 20%		$31,600			**Grand Total Rounded off to the Nearest Thousand**			**$190,000**	

Table B-8: Design 3 – On-site construction cost analysis

Activity	Labor Cost					Material Cost			Total Cost
	Number of Workers	Hourly Rate	Hours	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Site mobilization	4	$50	4	16	$800				$800
Transportation	4	$60	8	32	$1,920	$7 per mile	1000	$8,400	$10,320
Cranes lifting and placing the girders	6	$50	10	60	$3,000	$1100 per crane day	2	$2,200	$5,200
Casting longitudinal seam	4	$50	4	16	$800	$2000 per cu.yd.	1.6	$3,200	$4,000
Placing PT strands	8	$50	4	32	$1,600	$1500 per ton	1.2	$1,800	$3,400
Erect deck formwork and screed	12	$50	8	96	$4,800				$4,800
Casting concrete	4	$50	8	32	$1,600	$250 per cu.yd.	76.6	$19,150	$20,750
Screed and finish concrete	16	$50	8	128	$6,400				$6,400
Tidy up	8	$50	8	64	$3,200				$3,200
Curing time	2	$50	84	168	$8,400				$8,400
Removing formwork	4	$50	8	32	$1,600				$1,600
Stressing strands	8	$60	8	64	$3,840				$3,840
Install barrier	12	$50	8	96	$4,800	$270 per 4-ft	60	$16,200	$21,000
Stripe road	4	$50	4	16	$800				$800
Site clean-up	10	$50	8	80	$4,000				$4,000
			Total Labor Cost		$47,600	Total Material Cost		$51,000	$98,600
Contingency cost at 20%		$19,720			**Grand Total Rounded off to the Nearest Thousand**			**$118,000**	

Table B-9: Design 5 – On-site construction cost analysis

Activity	Labor Cost					Material Cost			Total Cost
	Number of Workers	Hourly Rate	Hours	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Site mobilization	4	$50	4	16	$800				$800
Transportation	4	$60	8	32	$1,920	$7 per mile	1000	$8,400	$10,320
Cranes lifting and placing the girders	6	$50	10	60	$3,000	$1100 per crane day	2	$2,200	$5,200
Placing seam formwork	8	$50	4	32	$1,600				$1,600
Casting longitudinal seam	4	$50	4	16	$800	$2000 per cu.yd.	4.2	$8,400	$9,200
Place deck formwork and screed	8	$50	8	64	$3,200				$3,200
Casting concrete	4	$50	8	32	$1,600	$750 per cu.yd.	25.6	$19,200	$20,800
Screed and finish concrete	16	$50	8	128	$6,400				$6,400
Tidy up	8	$50	8	64	$3,200				$3,200
Curing time	2	$50	15	30	$1,500				$1,500
Removing formwork	4	$50	4	16	$800				$800
Install barrier	12	$50	8	96	$4,800	$270 per 4-ft	60	$16,200	$21,000
Stripe road	4	$50	4	16	$800				$800
Site clean-up	10	$50	8	80	$4,000				$4,000
			Total Labor Cost		$34,400	Total Material Cost		$54,400	$88,800
Contingency cost at 20%		$17,760			**Grand Total Rounded off to the Nearest Thousand**			**$107,000**	

Table B-10: Design 6 – On-site construction cost analysis

Activity	Labor Cost					Material Cost			Total Cost
	Number of Workers	Hourly Rate	Hours	Man Hours	Total Labor Cost	Unit Price	Number	Total Material Cost	
Site mobilization	4	$50	4	16	$800				$800
Transportation	4	$60	8	32	$1,920	$7 per mile	1000	$8,400	$10,320
Cranes lifting and placing the girders	6	$50	10	60	$3,000	$1100 per crane day	2	$2,200	$5,200
Casting longitudinal seam	4	$50	4	16	$800	$2000 per cu.yd.	5.3	$10,600	$11,400
Finishing concrete	4	$50	4	16	$800				$800
Tidy up	4	$50	4	16	$800				$800
Curing time	2	$50	15	30	$1,500				$1,500
Install barrier	12	$50	8	96	$4,800	$270 per 4-ft	60	$16,200	$21,000
Stripe road	4	$50	4	16	$800				$800
Site clean-up	10	$50	8	80	$4,000				$4,000
			Total Labor Cost		$19,200	Total Material Cost		$37,400	$56,600
Contingency cost at 20%		$11,320			**Grand Total Rounded off to the Nearest Thousand**			**$68,000**	