A DESIGN STUDY OF SPLICED PRESTRESSED CONCRETE BRIDGES WITH I- AND BOX-GIRDERS

Pimpida Surakomol, P.E., Michigan State University
Rigoberto Burgueño, Ph.D., Michigan State University

ABSTRACT

Design of spliced precast/prestressed concrete girder bridges through continuous post-tensioning has gained renewed interest, however, preference has been given to the use of I-girders and exploration of the advantages of using box beam sections has been limited. Through the use of structural optimization analyses, a comparative design study was performed on single spans featuring longitudinally spliced precast/prestressed standard I-girders and spread box-girders. A comparison of the result outcomes confirms that box-beams allow for shallower depths and longer spans than I-girders with modest cost increase. These results indicate that more attention should be paid to exploring the use box-beams in spliced girder bridge projects.

Keywords: Spliced girders, Optimization, Analysis, Design, Precast, Prestressed, Post-tensioned, Box girders.

INTRODUCTION

Precast/Prestressed concrete bridges have become a very popular type of bridge construction, primarily for reasons of economy, savings in life-cycle costs and their fast construction. However, the use of precast/prestressed concrete bridges for longer spans (for instance, greater than 150 feet) has been limited primarily due to transportation restraints. In order to overcome these drawbacks and to have an alternative to compete with steel superstructures, methods to achieve continuity with precast/prestressed girders have become of great interest to increase the spanning capabilities of this bridge system.

Several methods have been proposed to achieve the continuity of precast/prestressed girders[1,2,3]. The most common are the use of mild steel reinforcement in the deck, the splicing of prestressed strands, and the use of longitudinal post-tensioning. Of these methods, splicing of girder segments through post-tensioning, commonly referred to as longitudinal girder splicing, appears to have the greatest potential for extending span ranges for precast prestressed concrete girder bridges[4]. The technique involves the fabrication of girders in segments that are then assembled into the final structure. While not new, interest in the development of standard design procedures and guidelines for these systems has recently gained wide attention in bridge engineering practice.

A spliced precast girder bridge is defined as a type of superstructure in which precast concrete beam-type elements are joined longitudinally, typically using post-tensioning, to form the complete girder. The resulting superstructure cross-section is a conventional beam-and-slab system with a composite cast-in-place deck or precast deck. Among the reasons to use spliced girders are the reduction of substructure units due to increases span lengths, reduction of girder units due to increased girder spacing, and functionality and aesthetic improvements by reducing superstructure depth. Clearly, similar benefits can also be gained by using high-strength concrete and lightweight concrete.

Spliced girder bridges have a proven track record, with more than 250 spliced girder bridges having been constructed in the US, some of them dating back as early as 1952[4]. In spite of their past and continued use, the application of this technique is not widespread. A significant reason for limited utilization of spliced girders is the ambiguity in their design and analysis, rooted in the consideration of various issues with which the designer of conventional precast prestressed concrete girders is typically not familiar. In addition, the information available in the literature regarding the design, analysis and construction of spliced girder bridges is limited, as the experience, information, and methods used on these projects have tended to be job-specific, and the knowledge gained has not been made widely available for use on similar projects[4]. A recently completed NCHRP research program on extending the span of bridges using precast prestressed concrete girders[4] will certainly improve the state of knowledge on the design of spliced girder bridges, as the effort has successfully lead to recommendations on LRFD design procedures, standard details, and design examples.

While the design requirements for spliced girder bridges are not significantly different from conventional prestressed concrete design, the analysis procedure must take additional considerations. Among the most relevant are staged construction, multiple stressing stages, and combined pre-tensioning and post-tensioning. Thus, the design of spliced girder bridges involves greater complexity than is required for conventional precast/prestressed concrete girder designs. The design is generally executed using a computer program or a series of spreadsheets. Unfortunately, design guides, aides, and examples of spliced girder bridges to help designers are not readily available or address only limited portions of the design[4].

The design of spliced girder bridges depends on several parameters that significantly influence performance and cost. The most relevant are time dependent effects, splicing locations, construction sequences, girder segment geometries, number of beams, and number or profiles of pre-tensioned and post-tensioned reinforcement. Normally most design variables are determined based on the designer's judgment and on a trial and error processes. Consequently there is no guarantee of obtaining the most economical design, which requires more time and effort to explore, and which typical projects usually cannot afford.

Since design of spliced girder bridges involves a number of design considerations, use of mathematical optimization methods can provide a systematic approach to arrive at appropriate design solutions. Computational optimization techniques[5] can thus be used as a tool to develop the configuration and sectional optimization of spliced girder bridges and serve as a guide to produce design aids[6]. Design aids can depict the relations between girder shape, girder spacing, pre-tensioning and post-tensioning requirements, splicing locations, pier segment geometries, initial and final concrete compressive strengths, and tendon profiles. Bridge engineers would find benefits in design charts and tables, which are based on optimal solutions that will help expedite the design process. As a result the system can be more widely used by state highway agencies and bridge consulting firms.

Current designs for longitudinally spliced girder bridges through continuous post-tensioning have shown preference to the use of I-girders and bulb-tee girders. Preference for these girder types seems justified due to their efficiency. However, exploration on the use of box-beam sections, which could lead to shallower depths and longer spans, has been limited, most likely due to concerns regarding cost efficiency. The use of I-girders or box-girders in spliced construction would clearly require consideration of some trade-offs. Splicing of box sections requires providing balanced post-tensioning to each of the webs and the provision of additional anchorage details, which can increase construction costs. However, this drawback can be conceptually overcome by incorporating the anchorage details into the end diaphragms. Conversely, the required thickening of the section webs, for both I- and box-beams, might be easier to achieve for box sections by simply modifying the Styrofoam blockouts typically used during casting. Solution of these limitations and the advantages of stiffer sections can make the use of box-beams feasible and attractive, both aesthetically and economically, for spliced construction.

In this paper, a comparative design study of a single-span spliced girder bridge with standard precast I- and box-girders built with a single-stage construction sequence is presented. The study is based on the results from structural optimization analyses with a common design objective of minimum cost. The designs follow the AASHTO LRFD Specifications[7] and the latest NCHRP recommendations[4]. A comparison the achievable span lengths, optimal splicing location and pre-tensioning and post-tensioning requirements are provided.

DESIGN STUDY

This section presents the prototype structure, design considerations, assumptions, and analysis procedures used to perform a design study aimed at evaluating and comparing the performance of standard precast I- and box-beams in spliced girder bridge construction. The system is assumed to be spliced in a single post-tensioning stage after casting of the concrete deck. Elastic analyses are used with considerations for construction staging and time dependent effects.

PROTOTYPE STRUCTURE

The prototype structure for the design study is a single-span spliced-girder bridge composed of three conventional girder segments as shown in Fig. 1. While the end girder segments may have different lengths, depending on the splicing location, the design study considers them to be equal. The bridge width is taken as 61 ft, enough to accommodate four traffic lanes, two 5 ft shoulders, and standard traffic barriers. The superstructure is taken to consist of standard precast/prestressed girders with a 9 in. composite cast-in-place concrete deck.

The girders used for the comparative study are AASHTO Type IV I-beams and 54”x48” box-beams. Although the girder sections most commonly used for spliced girder bridges of this type have typical depths of 72 in., the former-mentioned sections were chosen to allow an equivalent comparison of the two girder systems with the same overall height. Cross-sections of the prototype superstructures with I- and box-girders, including their reinforcement details are shown in Fig. 2 and Fig. 3. The remaining details defining the prototype structure, such as beam spacing, span length, and numbers of pre-tensioning and post-tensioning strands are design variables and parameters obtained through a design optimization process and are discussed later.

DESIGN ASSUMPTIONS

The most relevant design assumptions for the prototype bridges in the design study are summarized herein. The independent girder segments are spliced by cast-in-place concrete joints between segments and continuous post-tensioning along the entire bridge length. Temporary supports are provided underneath the splice locations before the girders are connected. The composite deck, splice joints, and end diaphragms have the same concrete compressive strength. No permanent intermediate diaphragms are used. Thus, only temporary bracing for girder stability during construction is required.

All girder segments are pre-tensioned to resist stresses resulting from handling, erection, and deck weight. The girder segments are connected through a single-stage post-tensioning construction procedure. The post-tensioned strands are stressed after the deck and splice concrete are placed and reach their specified design strength, typically 3,500-5,000 psi. Finally, the girders are taken as a single-span system after post-tensioning with the composite section resisting all loads after this stage. If post-tensioning is not enough to lift the girders from the intermediate temporary supports the system is considered continuous until these restraints are removed.

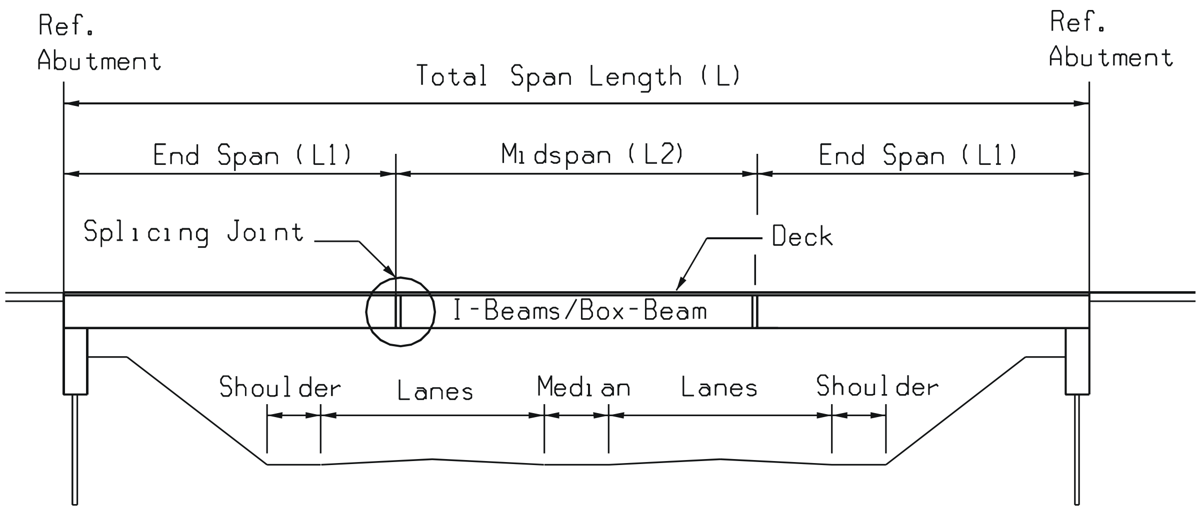
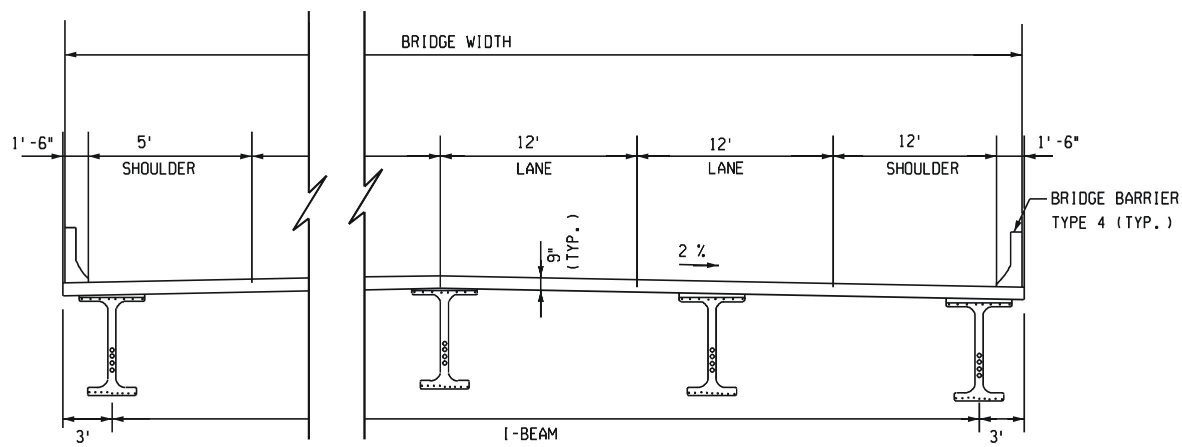
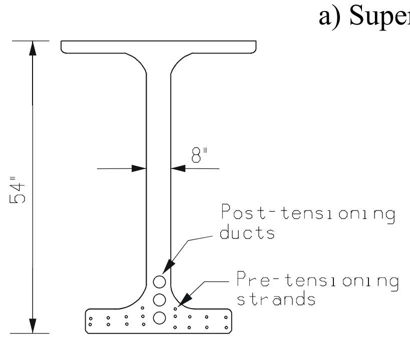


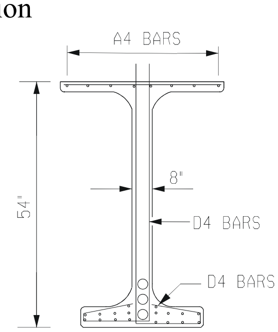
Fig. 1 Three-Segment Single-Span Spliced Precast/Prestressed Girder Bridge



a) Superstructure Cross-Section

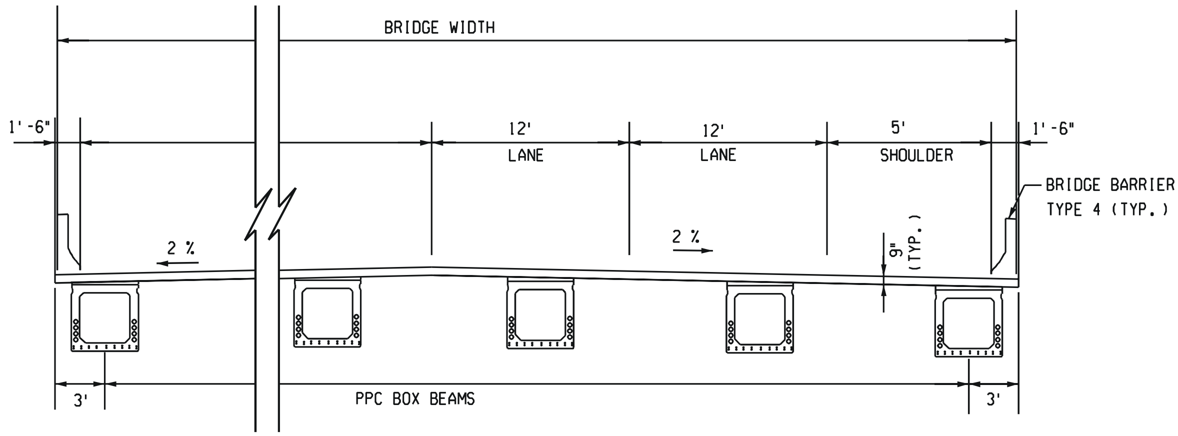


b) Girder Cross-Section

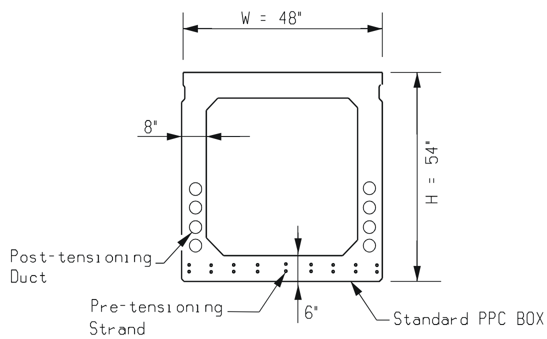


c) Girder Reinforcement Details

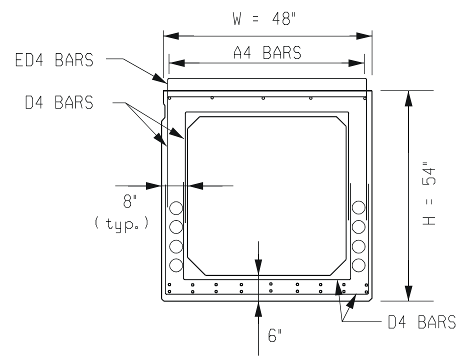
Fig. 2 I-Beam Bridge Cross-Sections and Reinforcement Details



a) Superstructure Cross-Section



b) Girder Cross-Section



c) Girder Reinforcement Details

Fig. 3 Box-Beam Bridge Cross-Sections and Reinforcement Details

CONSTRUCTION SEQUENCE

There are many aspects that can be altered in the design of spliced girder bridges. Of particular importance for this type of construction are the post-tensioning sequence and the sequence of construction stages. The sequence of construction stages for a single-staged post-tensioned spliced girder bridge is listed Table 1. Four events are considered critical for the pre-tensioning strands for a single staged post-tensioning splicing procedure applied after the deck is cast[4]. These critical construction stages are listed in Table 2. For the post-tensioned strands, there are three critical construction events for single stage post-tensioning[4] as listed in Table 3.

DESIGN CRITERIA

The design study was conducted using the simplified design method of the AASHTO LRFD Bridge Design Specifications with Interims through 2003[7]. Only flexural demands are evaluated. Thus, effective stresses were calculated only at the center of each girder segment and at the splice location at the top and bottom of the girder and at the top of the deck for each critical stage of the pre-tensioning strands (Table 2). For flexure, service limit state requirements govern the required prestressed force. The allowable concrete stress limits used are those prescribed by AASHTO LRFD, which are summarized in Table 4.

Table 1 Construction Sequence for Single-Staged Post-Tensioning in Spliced Girder Bridges

Construction Stages	Time (days)	Action Description
1	-	Pre-tensioning strands are stressed
2	0	Girder segments are cast
3	1	Pre-tensioning strands are released
4	50	Girder segments are erected
5	60	Deck and Splice concrete are placed
6	75	Post-tensioning strands are stressed
7	100	Barriers are added
8	140	Live load is applied to the bridge
9	15000	Future wearing surface is added
10	27500	At final condition after all prestress losses

Table 2 Critical Construction Stages due to Pre-Tensioning

Critical Construction Stage	Time (days)	Action Description
3	1	Pre-tensioning strands are released
5	60	Deck and Splice concrete are placed
6	75	Post-tensioning strands are stressed
10	27500	At final condition after all prestress losses

Table 3 Critical Construction Stages due to Post-Tensioning

Critical Construction Stage	Time (days)	Action Description
6	75	Post-tensioning strands are stressed
7	100	Barriers are placed
10	27500	Final condition after all prestress losses

Stresses at the girder top and bottom and for the top of deck are computed along the girder segments for each of the applied loads and stressing actions. Stresses from the applied (external) loads are computed from the bending moment demands and the composite and non-composite section properties. Stresses from the stressing actions are evaluated separately for each critical construction stage. The effects of prestress losses are included in the stress evaluation by using the effective prestress values for each of the four critical construction stages. The total service limit state stresses are determined by adding the stresses caused by the externally applied loads and those caused by the stressing operations. The stress evaluation just described is performed for all load combinations. Four limit states are considered, two follow the service limit states defined in the LRFD specifications[7] and the other two are based on recommendations from the NCHRP 12-517 report[4]. A description of the stress checks and the associated load combinations for these limit states follows.

Table 4 Allowable Concrete Stress Limits for Critical Construction Stages

Critical Construction Stages	Stress Type	Girder concrete	Deck concrete
Pre-Tensioning Strand Release (3)	Compression	$0.6f'_{ci}$ = 3000 psi	
	Tension	-200 psi	
Deck/Splice Concrete (5)	Compression	$0.6f'_{c}$ = 3900 psi	
	Tension	-200 psi	
Post-Tensioning (6)/Barriers (7)	Compression	$0.6f'_{c}$ = 3900 psi	$0.6f'_{cd}$ = 2700 psi
	Tension	$-0.24\sqrt{f'_{c}}$ =-612 psi	$0.24\sqrt{f'_{cd}}$ =-90 psi
Final Condition – after losses (10)	Compression	$0.6\phi_w f'_{c}$ = 3900 psi	$0.6\phi_w f'_{cd}$ = 2700 psi
	Compression 2	$0.45f'_{c}$ = 2925 psi	$0.45f'_{cd}$ = 2025 psi
	Compression 3	$0.4f'_{c}$ = 2600 psi	$0.4f'_{cd}$ = 1800 psi
	Tension	$-0.19\sqrt{f'_{c}}$ =-484 psi	

There are two load combinations considered for full service loads for the final construction stage as described in LRFD specifications[7]. These are the Service Limit State I (SLS-I) and Service Limit State III (SLS-III) conditions. The difference between these limit states is the live load factor, which is equal to one (1.0) for SLS-I and 0.8 for SLS-III. Service limit state I is used to check the concrete compression stress limit while service limit state III is used to check the concrete tension stress limit for full service loads. Full service loads includes the girder self-weight, deck load, non-composite dead load, temporary pier removal, construction dead load, and live load. The top deck stress, and top and bottom girder stresses on these limit states are checked against the allowable concrete stress[7] (Table 4).

The two additional limit states recommended by the NCHRP 12-517 report[4] deal with compression stress limits for partial service load at the final construction stage. The two limit states differ in the load combination to be considered. In the first one, referred to as Compression 2 by the NCHRP Report[4], considers all the loads from the SLS-I except the live load with all other loads using a unit load factor. The second additional limit state, named Compression 3[4], considers all the SSL-I loads but with a unit load factor for the live load and a load factor of 0.5 for all other loads. The limit stresses for the Compression 1 and Compression 2 checks (see Table 4) are those recommended by the NCHRP 12-517 report[4].

LOADS

Non-composite dead loads include the weights of girders, deck, optional stay-in-place deck forms, and construction loads. Beam spacing has a direct influence on the deck, built-up, and construction loads. Girder and haunch loads are taken as beam line loads. The unit weight of the girder and deck concrete are assumed to be 150 lb/ft^3. Uniform 2-in. haunches are assumed over the top flange width for construction tolerance and variation in camber. The corrugated metal stay-in-place deck forming is assumed to be 16 lb/ft^2. The construction design load is taken as 20 lb/ft^2 during placement of the deck concrete. Composite dead loads include splice and spliced diaphragms, removal of temporary support, barrier weight, and future wearing surface. Composite loads are distributed equally to all girders in the superstructure cross-section.

Loads from splice joints and temporary diaphragms are not transferred to the superstructure during construction since they are supported by temporary supports. After post-tensioning is applied and the temporary supports are removed, these loads are applied to the composite beam. Reactions at the temporary supports include all non-composite dead loads and the splice joint weight. They are estimated as half of the load from both the end and middle segments. These support loads are based on the assumption that the bridge is lifted off the temporary supports after post-tensioning. The three continuous segments are then analyzed as a simply supported system with two point loads applied at the temporary support locations.

Design live loads are selected either from the design truck loads in combination with design lane loads or design tandem loads in combination with design lane loads[7]. The live load used in the design is based on the maximum load combination effect. The dynamic allowance (IM) is only applied to the design truck as specified in AASHTO LRFD[7]. Live load demands are based on the load distribution factors for interior beams as given in the AASHTO LRFD specifications. Barriers are assumed to weigh 371 lb/ft each. A load of 25 lb/ft^2 is considered for future wearing surfacing.

MATERIAL AND SECTION PROPERTIES

Material properties are typically project-specific, thus generalization is difficult. For the reported study, the assumed concrete material properties for the girders, deck and splice joints are summarized in Table 5. The pre-tensioning and post-tensioning strands are assumed to be 0.6-in. diameter and made from low-relaxation steel, with properties as given in the AASHTO LRFD specifications[7] (Table 5). The sectional properties of the non-composite I- and box-beam used in the prototype bridges are summarized in Table 6.

Table 5 Assumed Material Properties

Girder Concrete	Deck & Splice Concrete	Pre-tensioning Strands	Post-Tensioning Strands
f'_c = 6500 psi	f'_{cd} = 4500 psi	A_{ps} = 0.217 in.^{2}	A_{ps} = 0.217 in.^{2}
f'_{ci} = 5000 psi	f'_{cdi} = 3500 psi	f_{pu} = 270 ksi	f_{pu} = 270 ksi
E_c = 4888 ksi	E_{cd} = 4067 ksi	f_{py} = 243 ksi	f_{py} = 243 ksi
E_{ci} = 4287 ksi	E_{cdi} = 3587 ksi	f_{po} = 202.5 ksi	f_{pj} = 218.7 ksi
		E_p = 28500 ksi	E_p = 28500 ksi

Table 6 Sectional Properties of I- and Box-Beams

	AASHTO Type IV I-Beam	54" x 48" Box Beam
Section Height, H (in.)	54	54
Cross-Sectional Area, A (in.^{2})	789	930
Moment of Inertia, I (in.^{4})	260,730	390,000
Bottom Section Modulus, S_b (in.^{3})	10,543	14,607
Top Section Modulus, S_t (in.^{3})	8,908	14,286

PRE-TENSIONING AND POST-TENSIONING DETAILS

All pre-tensioned strands are straight without any draping since they will interfere with the post-tensioning ducts. No debonding of pre-tensioned strands is used in the study. Strands are placed in the top and bottom flanges of girder section and they are fully stressed to $0.75f_{pu}$. The number of required pre-tensioned strands is selected to provide enough prestress in the precast beam not to have concrete stresses exceed allowable limits at release and upon placing the concrete deck. Both end segments are provided with the same number of pre-tensioned strands.

A parabolic post-tensioning strand layout is assumed to counteract the bending moment demand for the simply supported beam. The number of post-tensioned tendons is selected to provide enough pre-compression to resist live and dead loads when they are transferred from the individual girder segments to the full simple span. Multiple strand tendons are assumed and the distance between each duct varies along the girder length. Post-tensioning design requirements are evaluated at four critical locations: at the middle of both end segments, at the center of the middle segment, and at the center of the splice joint.

PRESTRESS LOSSES

The method to calculate prestress losses for spliced girder bridges is slightly different than that followed in conventional prestressed concrete designs. In spliced girder bridges, concrete compressive forces are applied at two different times and in different forms, i.e., pre-tensioning followed by post-tensioning. These two stages clearly affect each other. Additional elastic shortening losses take place in the pre-tensioning strands as a consequence of post-tensioning. Furthermore, the effective compressive force in the section is no longer concentrated within a small region, but it is distributed depending on the relative location of the pre-tensioning strands and the post-tensioning tendons.

Prestress losses for the pre-tensioned strands are calculated at the mid-span of each girder segment, while prestress losses for the post-tensioning tendons are calculated at both ends and at mid-span. The prestress losses are then used along the entire length of the girder segment. The resulting stress due to prestress is computed at the centroid of the pre-tensioned strands and at the centroid of the post-tensioned strands.

Prestress losses in the spliced girder can be classified as instantaneous or time-dependent. Instantaneous prestress loss from pre-tensioning includes those from concrete elastic shortening and those from relaxation of the pre-tensioning steel at release. Instantaneous post-tensioning losses include those from friction and anchorage seating. Time dependent losses for the pre-tensioning and post-tensioning operations include losses due to relaxation of the steel, due to concrete shrinkage, and due to concrete creep.

Time dependent losses can be determined either by using the time-step method, the simplified method or the lump sum method. The simplified approach[4], based on the AASHTO-LRFD[7] provisions, was used in this study. The simplified approach is adequate [4,7] since the total span length is expected to be less than 250 ft, normal weight concrete was assumed to be used, and the minimum concrete strength at prestressing is assumed to be 5,000 psi. This method permits determination of effective prestress using the time interval of each construction stage. There are four critical construction stages for evaluation of prestress losses for the pre-tensioned strands as given in Table 7. Similarly, there are three critical construction stages of prestress losses for the post-tensioned strands as listed in Table 8.

Table 7 Prestress Losses of Pre-Tensioned Strands at Critical Construction Stages

Critical Construction Stages for Pre-Tensioned Strands	Prestress Losses
At release of pre-tensioned strands	1) Elastic shortening at release 2) Relaxation at release
At placement of deck and splice concrete	1) Elastic shortening at release 2) Final relaxation 3) Intermediate creep and shrinkage at 60 days
At stressing of post-tensioning tendons	1) Elastic shortening at release 2) Final relaxation 3) Intermediate creep and shrinkage at 75 days 4) Elastic shortening loss due to post-tensioning
At final conditions after losses	1) Elastic shortening at release 2) Final relaxation 3) Final creep and shrinkage 4) Elastic shortening loss due to post-tensioning

Table 8 Prestress Losses of Post-tensioned Strands at Critical Construction Stages

Critical Construction Stages for Post-Tensioned Strands	Prestress Losses
At post-tensioning	1) Friction 2) Anchorage 3) Elastic shortening loss due to post-tensioning
At placement of barrier and sidewalks	1) Friction 2) Anchorage 3) Elastic shortening loss due to post-tensioning 4) Intermediate creep and shrinkage at 100 days 5) Final relaxation
At final conditions after losses	1) Friction 2) Anchorage 3) Elastic shortening loss due to post-tensioning 4) Final creep and shrinkage 5) Final relaxation

ANALYSIS

Analysis of the spliced girder bridge structure is based on 2-D elastic frame analyses including the effects of time-dependent material behavior and staged construction. A custom frame analysis program using the stiffness method is used. Each span segment is divided into multiple elements to obtain stresses, internal forces and deflection results at the member ends. The beam-line model is subjected to the demands of an interior girder according with the load distribution factors specified in the AASHTO-LRFD Specifications[7]. Interior girders are used in the analysis since their demands usually control the superstructure design. In addition, usually all beams are designed as interior beams to allow future bridge widening.

Prestress losses are computed using the simplified AASHTO-LRFD method[7] with consideration of the effects of combined pre-tensioning and post-tensioning on the girder segments, as recommended by the recent NCHRP study on spliced girder bridge design[4]. Design lifetime of the spliced girder bridge system is taken to be 75 years after all prestress losses have occurred. The section design is performed using a custom program coded in MATLAB[8], which enforces compliance with service limit states as previously discussed.

The analysis program was verified against Design Example 1 from the NCHRP 517 report[4]. The design example uses the modified 96-in. PCI bulb-tee with a beam spacing of 9 ft and a span length of 196 ft. All moments at each stage and stresses at the top and bottom of the beam and at the top of the deck for all critical construction stages and load combinations were checked along the length of the span. The only difference between the analysis used in this study and that used in the NCHRP 517 example is the post-tensioning profile. The NCHRP example assumes as straight lines connected by segments to develop a parabolic curve, whereas the developed analysis tool for this study uses a continuous parabolic curve. The results from the custom program were found to be within 5% of the NCHRP 517 results.

OPTIMIZATION OF SPLICED GIRDER BRIDGES

Much progress has been made over the past two decades to facilitate the analysis of spliced girder bridges by the development of research and commercial programs[9,10]. Unfortunately, in spite of these powerful tools, design guides, aides, and examples of spliced girder bridges to help designers are not readily available or address only limited portions of the design[4].

As previously mentioned, the design of spliced girder bridges depends on several parameters that significantly influence both performance and cost. On the spliced girder projects executed to date, these design variables are determined based on the designer's judgment and on a trial-and-error basis. Consequently there is no guarantee of obtaining the most economical design, which requires more time and effort to explore and which typical projects usually cannot afford.

Implementation of mathematical optimization algorithms[5] can eliminate trial-and-error design procedures and thus lead to the design of more efficient systems. Design optimization tools are particularly suitable for systems that depend on several controlling variables in unknown or non-intuitive forms. These methods can result in significant cost savings, performance improvements and ease of design by providing guidance and insight into optimal solutions.

Computational optimization techniques have been effectively used for the configuration and sectional optimization of bridges and serve as a guide to produce design aids[6]. Optimization for conventional prestressed concrete girders and bridges has been broadly used in research efforts and has been successfully used for development of standard precast girder sections. However, only limited design optimization work has been done to improve the design of spliced girder bridges. The most significant work known to the authors is that of Lounis et al.[6], who investigated the feasibility of using existing I-beam sections for both pre-tensioned and post-tensioned construction. Their results showed that no standard section can be considered optimal for all structural systems and a new I-beam section type was developed.

OBJECTIVE FUNCTION DEFINITION

Typically there can be several design solutions that satisfy all requirements of safety and serviceability imposed by a design code and a number of specified merit criteria. Thus, the designer makes a difficult decision in selecting the best design among the possible alternative solutions that adequately satisfy all the governing design criteria. A common design criterion in bridge design is the minimization of total structural cost.

In this study, the objective function is the minimization of structural cost, which is defined as:

Cost = Concrete Cost + Pre-tensioning Strand Cost + Post-Tensioning Strand Cost + Temporary Supports Cost + Beam Cost + Reinforcement Cost

Concrete Cost = (Concrete Cost per cubic yard) x (Slab Volume)

$$= Cc \cdot L \cdot W \cdot ts$$

Pre-Tensioning Cost = (Pre-Tensioned Strand Cost) x (No. Beams) x [(No. Pre-Tensioned Strands in End Segment) x (End Segment Length) x (2) + (No. Pre-Tensioned Strands in Middle Segment) x (Middle Segment Length)]

$$= Cp \cdot Ng \cdot \left[(npr1 + nprt) \cdot L1 \cdot 2 + \cdot(npr2 + nprt) \cdot L2\right]$$

Post-Tensioning Cost = (PT Strand Cost) x (No. Beams) x (No. PT Strands) x [(End Segment Length) x (2) + (Middle Segment Length)

$$= Cpo \cdot Ng \cdot npo \cdot (L1 \cdot 2 + L2)$$

Beam Cost = (Beam cost) x (No. Beams) x (Total Span Length)

$$= Cb \cdot Ng \cdot (L1 \cdot 2 + L2)$$

Reinforcement Cost = (Reinforcement cost) x (Bridge Width) x (Total Span Length) x (Reinforcement weight per unit length)

$$= Cr \cdot W \cdot (L1 \cdot 2 + L2) \cdot Wst$$

Temporary Supports Cost = (Temporary Support Cost) x (No. Beams) x (2)

$$= Cts \cdot Ng \cdot 2$$

DESIGN VARIABLES

The design variables for the optimization procedure are a mixed set of integer and continuous geometric parameters that define the dimension and shape of the precast/prestressed girder segments, the material layout, and the span arrangement of the spliced girder bridge system. The design variables include: *npr1* = number of bottom pre-tensioning strands on end segments, *npr2* = number of bottom pre-tensioning strands on mid-span segments, *nprt* = number of top pre-tensioning strands, *npo1*, *npo2* = number of post-tensioning strands on 1st and 2nd duct, respectively, *ep01* = start drape of post-tensioning profile of 1st duct, and *ep02* = middle drape of post-tensioning profile of 2nd duct. These and the remaining design variables are noted in Fig. 4.

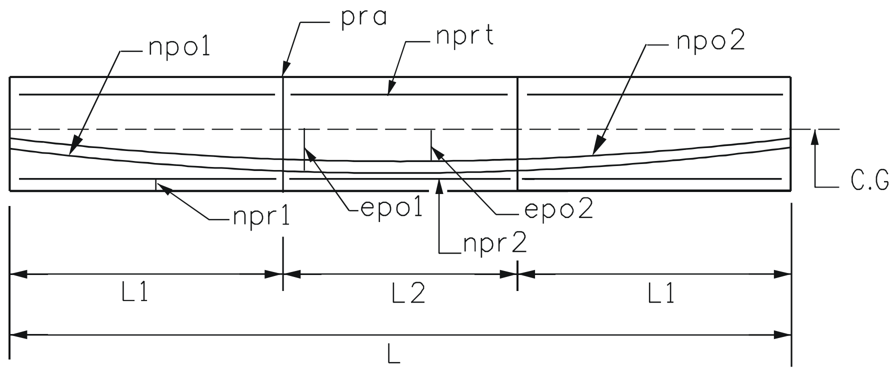


Fig. 4 Variables for the Design Optimization of Spliced Girder Bridges

DESIGN PARAMETERS

Design of spliced girder bridges requires the definition of many design parameters, i.e., values that remain fixed in the optimization process. These parameters have a significant role in defining the problem and they consequently significantly influence the resulting value for the design variables. The most important parameters in this study are: *L1* = end span segment length (= *pra·L*), *L2* = mid-span segment length (=*L* – 2·*L1*), *L* = total span length (starting at 110 ft), *pra* = ratio of end span segment length over total span length (0.2, 0.25, and 0.3), *W* = bridge width (60 ft), *Wo* = overhang width (3 ft), *S* = beam spacing (6ft., 8ft., and 10ft.), *Ag* = beam area, *Sb* = bottom sectional modulus, *St* = top sectional modulus.

OPTIMIZATION CONSTRAINTS

There are several constraints to consider depending on the state of the design being optimized. Only the service limit states were checked in this study since most of the time this limit state governs over the strength limit criteria. Constraints for the service limit state include: concrete girder stress limits, prestressing stress limits, deflection limits, and concrete slab stress limits. Stresses at each critical construction stage are checked with the allowable compression and tension stresses listed in Table 4.

COST DATA

The recent NCHRP project on spliced girder bridges[4] was able to gather cost data for a sampling of projects around the US. However, detailed construction information for post-tensioned spliced girder bridges was difficult to obtain since most states only track cost data on a project basis. In addition, the research team found costs to vary significantly between regions, methods of project delivery, and local consulting practices. Thus, it was found that it was difficult to assign meaningful cost parameters that are generally relevant.

It follows that it was also difficult to determine realistic cost values to use in the proposed cost minimization objective function. Cost estimates were thus based on values used for conventional prestressed girders with supplemental costs for temporary supports and post-tensioning. While this type of incremental approach is rarely valid[4], it is considered adequate for the comparative study being reported here. The assumed unit cost materials for the optimization analysis are summarized in Table 9.

Table 9 Assumed Unit Cost of Materials

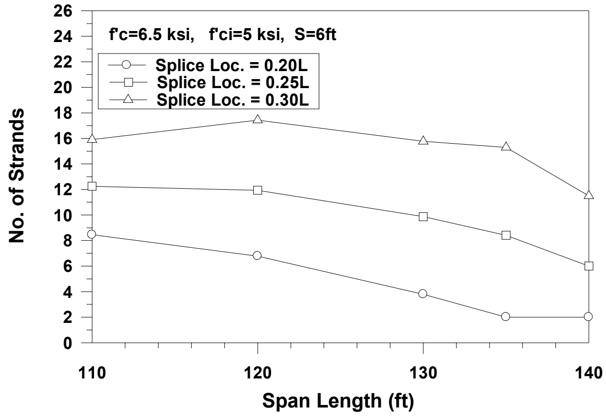
Component Item	Unit Cost
Cost of Concrete (Cc)	300 \$/yd^3
Cost of Pre-Tensioning (Cp)	50 \$/strand-ft
Cost of Post-Tensioning (Cpo)	100 \$/strand-ft
Cost of Temporary Support (Ctemp)	1000 \$/beam
Cost of Box-Beam: 54"x48" (Cb)	240 \$/ft
Cost of I-Beam: Type IV (Cb)	140 \$/ft
Cost of Epoxy Reinforcement (Cr)	0.90 \$/lb

RESULTS

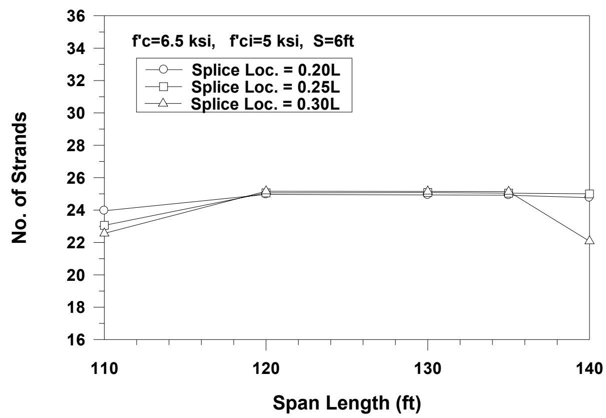
Analysis results of single span spliced girder bridges featuring AASHTO Type-IV I-beams and 48”x54” box-beams using design optimization algorithms are shown in Fig. 5 and Fig. 6. The results are grouped in tabular form with the left column corresponding to the I-beams and the right column to the box-beams. The figures provide traces that relate the optimal requirements of pre-tensioning strands in the end and middle girder segments and the amount of continuous post-tensioning to the achievable span length. These results were direct output from the optimization procedure, where the span length was kept as a design parameter. Optimum designs were developed by incrementing the span length until no feasible solution was obtained. Thus, the results shown in Fig. 5 and Fig. 6 also indicate the maximum achievable span length for the current girder types.

Fig. 5 compares the optimal pre-tensioning and post-tensioning requirements for I-beams and box-beams as a function of span length. Girder spacing and splice location were kept as design parameters (fixed values) in the optimization process. The girders were spaced at 6 ft on center and the three traces in each plot within the figure correspond to different splice locations. Designs with three splice locations were evaluated: 0.20L, 0.25L, and 0.30L, where L is the total system span length. For I-beams, the splice location only significantly

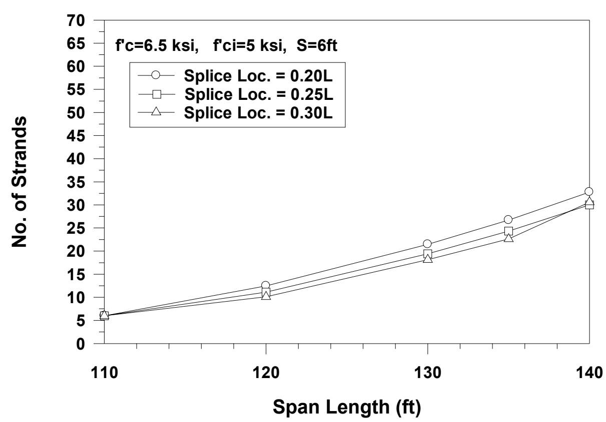
I-Beam: AASHTO Type IV



a) Pre-tensioning Strands in end-segment

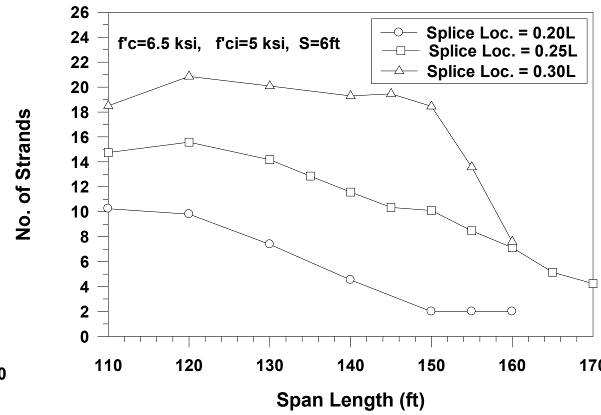


b) Pre-tensioning Strands in mid-segment

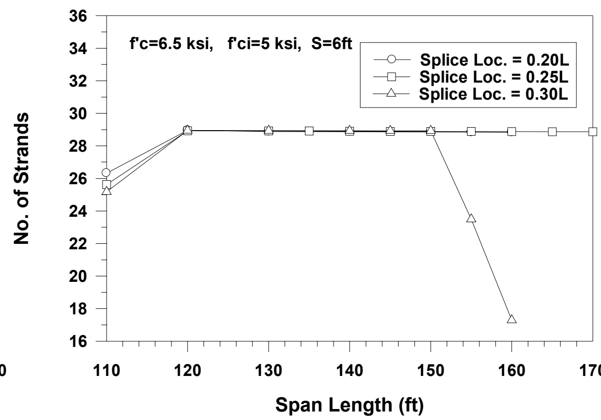


c) Post-tensioning Strands per web

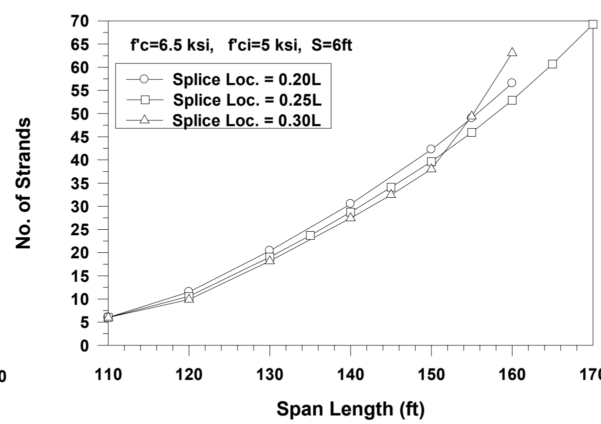
Box-Beam: 48”x 54”



d) Pre-tensioning Strands in end-segment



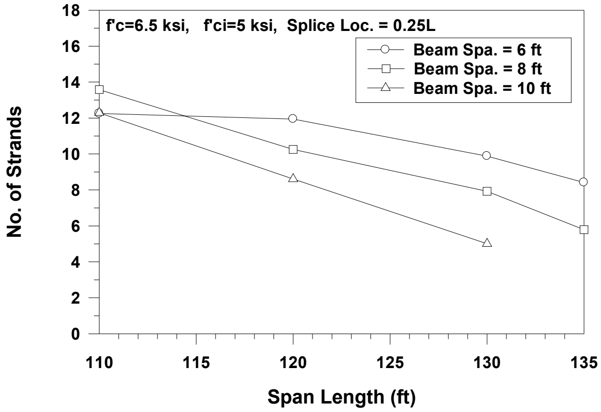
e) Pre-tensioning Strands in mid-segment



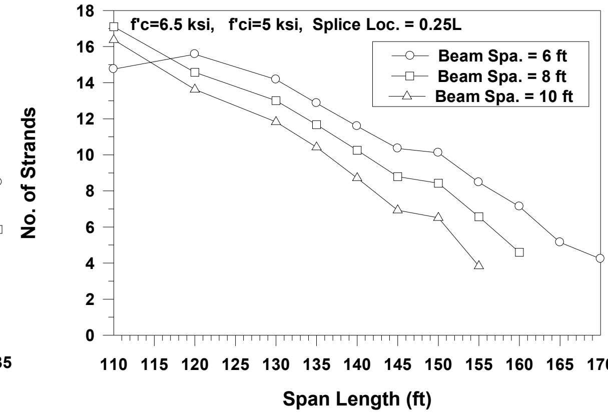
f) Post-tensioning Strands per two web

Fig. 5 Comparison of achievable span length and pre-tensioning and post-tensioning strands requirements of Box-Beam and I-Beam for different splicing locations

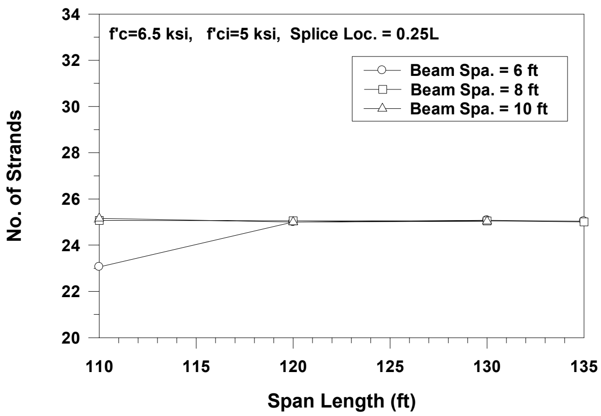
I-Beam: AASHTO Type IV *Box-Beam: 48”x54”*



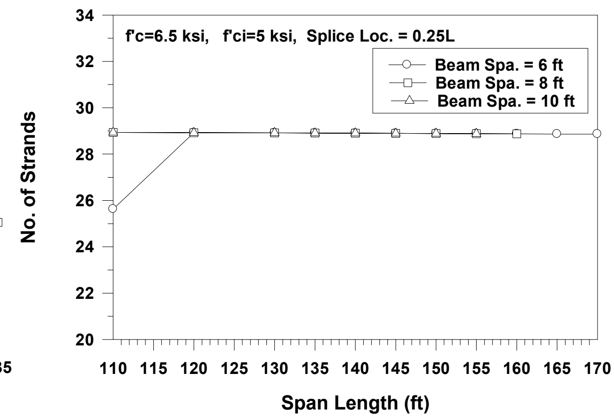
a) Pre-tensioning Strands in end-segment



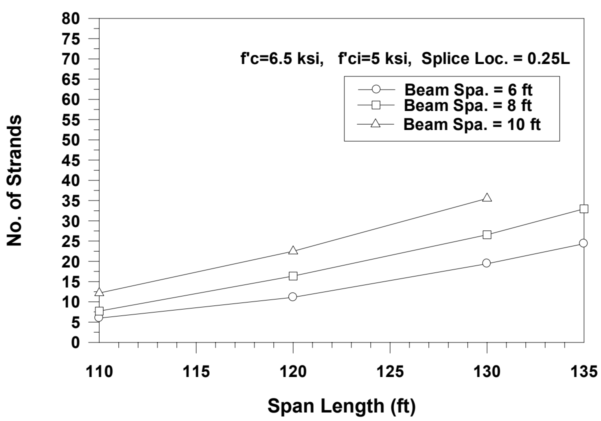
d) Pre-tensioning Strands in end-segment



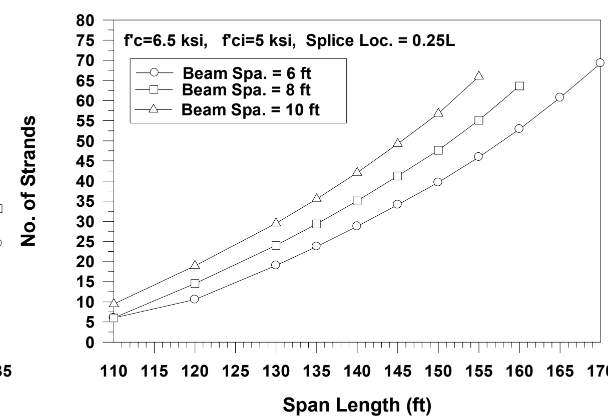
b) Pre-tensioning Strands in mid-segment



e) Pre-tensioning Strands in mid-segment



c) Post-tensioning Strands per web



f) Post-tensioning Strands for two webs

Fig. 6 Comparison of achievable span length and pre-tensioning and post-tensioning strands requirements of Box-Beam and I-Beam for different beam spacing influences the amount of pre-tensioning strands in the end girder segments. However, for the box beams, splice location also affects the middle girder segment pre-tensioning. In addition, for box-beams, the splice location seems to limit the achievable span length, indicating that a splice located at 0.25L permits a longer span than the other two splicing locations.

Fig. 6 correlates the optimal pre-tensioning and post-tensioning requirements for the I- and box-beams for variations in beam spacing. Each of the three traces in the plots of Fig. 6 corresponds to a beam spacing of 6, 8, and 10 ft. It can be seen that beam spacing noticeably affects the amount of pre-tensioning on the end girder segments but not in the middle segment. Post-tensioning requirements are affected by girder spacing, consistently increasing for greater girder separation. As expected, Fig. 6 shows that girder spacing has a strong effect in achievable span length, which influences the response of both I- and box-beam systems.

As previously stated, the optimization designs are the result of minimizing the system overall cost. Thus, the total cost of the bridge system is defined by the optimal variable solution, i.e., amount of pre-tensioning and post-tensioning, design parameters such as span length and girder spacing, and the assumed unit costs (Table 9). It was found that the costs of for the resulting systems increased almost linearly with span length. This implies that a constant cost per square foot can be assumed for each girder type. For bridge systems with a girder spacing of 6 ft, the average cost of the spliced I-beam solution was found to be $40/ft^2, while the average spliced box-girder bridge cost was $56/ft^2. This implies a cost difference of only 40% between the two girder types. The implications of this cost difference are discussed in the next section.

DISCUSSION

The data in Fig. 5 and Fig. 6 can be used to compare the maximum achievable spans between the spliced I- and box-girders and the gains that are made with respect to conventional designs for each of these girder types. Clearly, the estimated achievable span length of a bridge system depends on the analysis method and assumptions. For comparison purposes, the maximum span length of conventional pre-tensioned girder designs are taken from current design aids from state transportation agencies. A summary of the maximum span length for conventional and spliced AASHTO Type-IV I-girders for different beam spacing is given in Table 10. The analysis results show that, for a beam spacing of 6 ft, simply supported span lengths of 140 ft can be achieved with by splicing Type-IV AASHTO I-beam. This spliced span is 25% longer than the maximum achievable span of a single segment. The results also indicate that the gains in span length increase for larger beam spacing.

Results summarizing the achievable span lengths for single-span spliced girders using either I- or box-beams found through the design optimization study are given in Table 11. It can be observed that the spliced 48"x54" box-beams allow for 16% to 21% longer spans that the spliced AASHTO Type-IV I-beams. Thus, spliced box-beams can further stretch the spanning capabilities of precast/prestressed beams. Additionally, the results indicate that for the same span length, a spliced girder bridge with box-beams would result in a shallower section than an I-beam solution.

Table 10 Maximum Span Lengths for Conventional and Spliced I-Girder Bridges

Beam Spacing (ft)	Pre-tensioned Bridge Max. Span Length (ft)	Spliced Girder Bridge Max. Span Length (ft)	Increase in Max. Span Length
6	112	140	25%
8	101	135	34%
10	92	130	41%

Table 11 Maximum Span Lengths for Spliced Girder Bridges with Box- and I-Girders

Beam Spacing (ft)	Spliced Box-Beam Bridge Max. Span Length (ft)	Spliced I-Beam Bridge Max. Span Length (ft)	Increase in Max. Span Length
6	170	140	21%
8	160	135	16%
10	155	130	16%

The comparison given in Table 10 and Table 11 is based on equal height and equal beam spacing. Therefore, it should be noted that this result does not mean that I-girders are limited to those span lengths. It is clear that reduced girder spacing can yield longer span.

It is recognized that the evaluated cost values may not be realistic. Errors in the cost estimate can be diverse, but particular uncertainty exists in the assumptions made for the cost of intermediate piers and the post-tensioning operation. Also, the additional cost for the box-beams cannot be severely criticized since the girder spacing was fixed for the comparison. Nonetheless, what is probably of higher importance is that the cost difference between the spliced I-beams and spliced box-beam solutions can be modest. While the used unit cost values need to be refined, a slight cost difference between these two systems could further highlight the efficiency of box beams for spliced girder bridge construction.

Splicing of box beams seems to provide greater spanning capabilities and/or allow for shallower superstructure depth for a modest cost increase. However, further considerations need to be studied to reach general conclusions, such as the feasibility and practicality of modifying box-beam geometries and special detailing requirements for the box-beam ends.

It can be noted from Fig. 5 and Fig. 6 that the number of pre-tensioned strands decrease as the overall system span length increases. This goes against the expected trend. However, rational judgment of this behavior is difficult due to limited experience with these systems. A preliminary explanation can be that the post-tensioning strands are cost-wise more efficient than pre-tensioning strands and are thus given preference by the structural optimization algorithm. In addition, while the authors have made a determined effort to verify the analysis program against documented examples, further verification will be conducted to ensure that these results are not the consequence of errors in the developed program.

CONCLUSIONS

The presented work has utilized structural optimization analyses to perform a comparative study on single span bridges featuring longitudinally spliced precast/prestressed I-girders and box-girders. Results based on a single cost minimization objective function and a single post-tensioning construction stage confirm that box-girders can allow for shallower depths and longer spans (16% to 21%) than achieved with equal depth I-girders. The box-beam solution was estimated to cost 40% more than the I-girders when both systems have equal beam spacing. The real cost difference can be smaller if refined unit cost estimates are used and box-beam spacing is increased due to their higher spanning capability. Thus, the design study showed that box-girders are a viable and efficient option for use in spliced girder construction, thus increasing the spanning capability of this bridge concept even further.

Recognition of the above results and their consequences was possible due to the integration of mathematical optimization algorithms and sequential time-dependent analysis tools. The optimization procedure allowed the determination of design solutions for both girder systems in a way that satisfies a common objective. An even comparison of the two girder types was then possible. Furthermore, the results and plots provided in this study are a preview of how design optimization tools can be useful to develop design aids for the preliminary design of spliced girder bridges. Given the added complexity in the design and analysis of spliced girder bridges, the availability of design aids can be a great asset to bridge engineers to expedite the design process of this bridge type, which can then result in its wider use by state highway agencies.

REFERENCES

1. Anderson, A.R, " Stretched-out AASHTO-PCI Beams Types III and IV for Longer Span Highway Bridges," *PCI Journal*, Vol.18, No. 5, 1973, pp 32-49.
2. Tadros, M.K., Ficenec, J.A., Elinea, A., and Holdsworth, S., "A new Technique to Create Continuity in Prestressed Concrete Members," *PCI Journal*, Vol.38, No.5, 1993, pp30-37.
3. Saleh, M.A., Elnea, A., and Tadros, M.K, "Creating Continuity in Precast Girder Bridges," *Concrete International*, American Concrete Institute, Detroit, Michigan, Vol.17, No.8, August 1995, pp 27-32.
4. Castrodale, R.W. and White, C.D., "Extending Span Ranges of Precast Prestressed Concrete Girders," NCHRP Report 517, National Cooperative Highway Research Program, Transportation Research Board, Washington, D.C., 2004.
5. Arora, J. S., Introduction to Optimal Design, Second Edition, Elsevier Academic Press, San Diego, CA, 2004.
6. Lounis, Z., Mirza, M.S., and Cohn, M.Z. "Segmental and Conventional Precast Prestressed Concrete," *Journal of Bridge Engineering*, Vol.2, No.3, 1997, pp 73-81.
7. AASHTO, LRFD Bridge Design Specifications, 2nd Edition, American Association of State Highway and Transportation Officials, Washington, D.C., 2003.
8. Mathworks Inc., *MATLAB 6*, Release 13, http://www.mathworks.com. (July 2004).
9. Karim, A. M., and Tadros, M.K., "Computer Analysis of Spliced Girder Bridges," *ACI Structural Journal*, Vol.90, No.1, 1993, pp 21-31.
10. Leap Software Inc., *CONSPLICE PT®*, http://www.leapsoft.com (July 2004).