Effectiveness of I-Girder Splicing Alternatives

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ABSTRACT

The trend toward increased span capacity of girder bridges has continued in recent years due to the need for improved safety and rapid bridge replacement. Precast concrete members must now span further while minimizing the superstructure depth in order to compete favorably with a new breed of high performance structural steel I-beams. This paper presents four different systems for creating continuous spliced concrete I-girders. The first system is limited to spans where full-span segment lengths are spliced over the piers. Within the first system, three different methods of continuity are studied: (a) continuous for live load through mild deck bars, (b) continuous for deck weight using a new girder threaded rod detail, and (c) continuous for deck weight plus superimposed loads using post-tensioning. The remaining three systems utilize post-tensioning of partial span segments, with prismatic or haunched pier segments. The advantages and disadvantages of each system are examined. The capacity of each system is determined on the basis of four criteria: ultimate negative moment capacity, ultimate positive moment capacity, Service III positive moment capacity, and shear capacity. Sample charts using the Nebraska NU2000 (78.7 in (2000 mm)) deep beams are presented for illustration.

Keywords: Bridge , Concrete, Precast, I-Girder, Post-tensioning, Continuity, Splicing

INTRODUCTION

As the trend moves toward extending the span capacities of precast concrete bridges, the need for an optimum system increases. This paper presents four different systems for building concrete NU I-girder bridges (see Figure 1). Within the first system three different methods are studied. The actual bridge capacity of each system is the least of four different capacities: the ultimate negative moment capacity, the ultimate positive moment capacity, the service III positive moment capacity, and the shear capacity. The capacity of each system is carefully calculated and sample NU2000 span charts are presented. System capacities are compared, and recommendations for improving the capacity of each system are presented.

Experience has shown that the simplest and most economical system is when full span-length pieces are installed directly onto their permanent supports as in system I. When span lengths exceed the maximum shippable length or weight, however, girder segments must be spliced at intermediate locations in the girder away from the piers as in systems II through System IV, as shown in Figure 1[1].

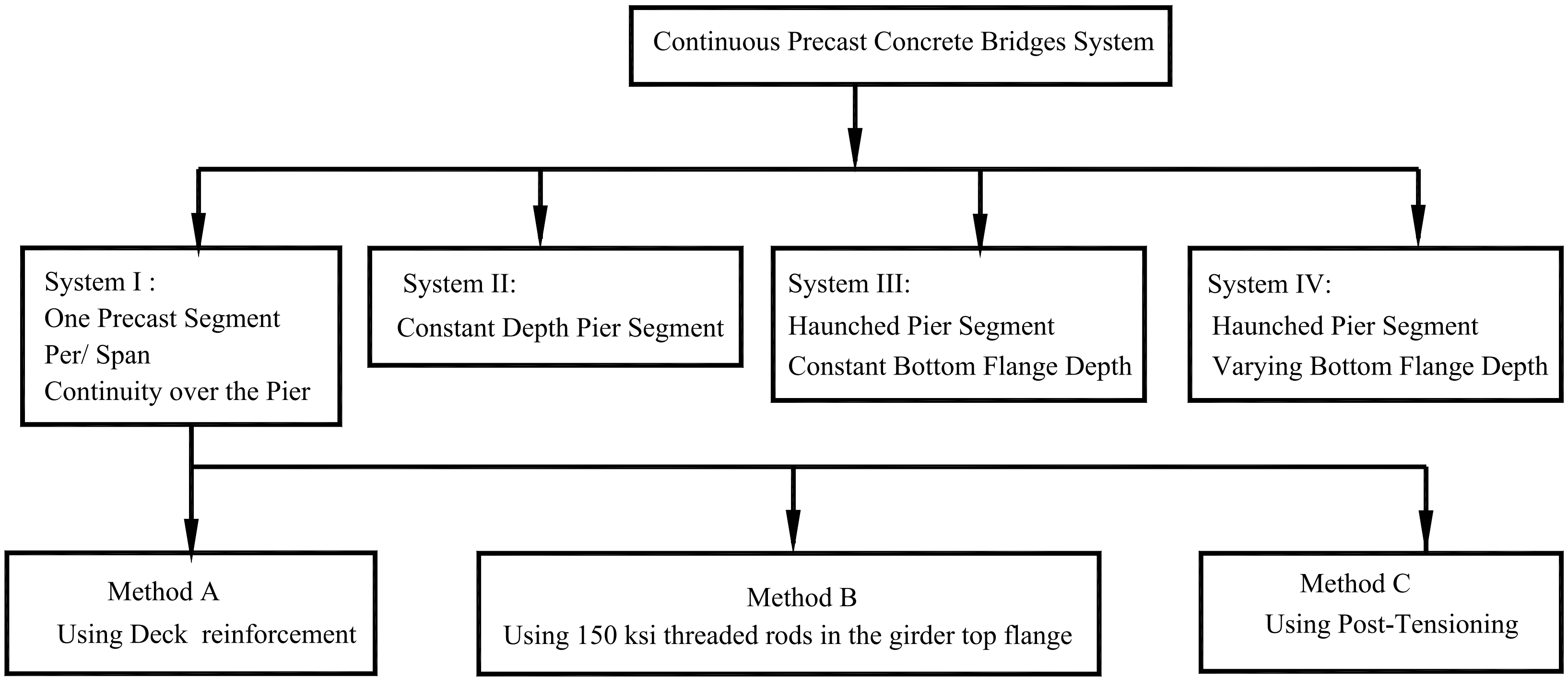

Figure 1 Splicing Precast Concrete Bridges Flowchart

DESIGN SYSTEMS

Bridge assumptions are summarized in Appendix A. A sample span chart is also presented.

System I: Full Span Segment

System I is the easiest and most economical system. However, bridges in this system are limited by shipping and handling capacities. The precast pieces in this system span between the permanent supports (pier, abutment). Three methods of creating continuity are studied to optimize this system.

Method A: Conventional Deck Reinforcement[2,3]

This method is the simplest and perhaps the least costly of existing methods. Continuity is created by placing mild reinforcement in the deck over the piers. Girder self-weight and deck slab weight are carried by the simple span precast segments. However, super imposed dead load and live load are carried by the continuous composite girder/ slab system. This method does not require extra equipment or a specialized contractor. But the superstructure is continuous only for superimposed dead loads and live loads, which is approximately only one third of the total loads[4]. Consequently, method A has small negative moments and relatively high positive moments, leading to relatively high pretension force which causes high prestress losses and bottom cracking at the piers.

Method B: Threaded Rod Splicing

In this method, I-girders are fabricated with 150 ksi high strength threaded rods embedded in the top flange. The threaded rods are mechanically spliced in the field at the diaphragms over the piers. The diaphragm concrete is then placed, and the deck slab is cast after the diaphragm gains the required strength. For more details, see Ma et al (1998)[5]. This is a relatively new system. The first bridge using this system has been designed and was scheduled for construction near Clarks, Nebraska in the fall of 2002. As opposed to Method A, this method allows for the superstructure to be continuous for deck slab in addition to super imposed dead load and live load, which is almost 70% of the total load. Accordingly, Method B can improve the span capacity of a given girder size by 10 to 15% over Method A. The negative moment created by deck slab weight, super imposed dead load and live load reduces the need for crack control bottom reinforcement over the piers.

Method C: Full Length Post-tensioning[2]

This method is more expensive than the previous methods. It requires full-length ducts and usually necessitates widening the girder webs. It also requires end blocks to resist bursting stresses at the anchorage zones.

Continuity in this method is created through post-tensioning the full length of the bridge. This method, like Method B, allows for the superstructure to be continuous for deck slab in addition to superimposed dead load and live load which is almost 70% of the total load. This is an effective method, especially if spliced segmental I-beams are needed for spans longer than the shipping capabilities of single-piece spans. A sample span chart for this method is presented in Appendix A.

Concrete NU I-Beam Capacities

Within the first system, the threaded rod continuity method gives the largest span capacity without changing the web width. The reinforcement steel in the deck slab method, method A, gives higher capacity than post-tensioning in beams beyond 10.25 ft girder spacing using NU2000 girders. The reinforcement steel in the deck slab method is mostly controlled by the positive moment, concrete tension at service. The threaded rod continuity method is controlled by the ultimate negative moment. Adding a steel plate at the bottom flange of the NU I-girder at the negative moment section can improve the capacity of this system.

Post-tensioning method results in a high gap between the capacities of the positive moment section (service and ultimate) and those of the ultimate negative moment and shear. The negative moment capacity controls the design of the NU2000 girder.

System II: Segmental Construction with Constant Cross Section[6]

The precast pieces used in this system are spliced with post-tensioning tendons away from the pier, as shown in Figure 2. The system allows for larger bridge spans than the maximum transportable concrete precast beams. The field segments are pretensioned to carry the beam self-weight during shipping and construction and to contribute to the flexure capacity of the positive moment section. The pier segments can have some pretensioning to carry the beam self-weight during shipping and construction, and top convention reinforcement to contribute to the flexure capacity of the negative moment section. All the precast pieces have the standard NU cross-section.

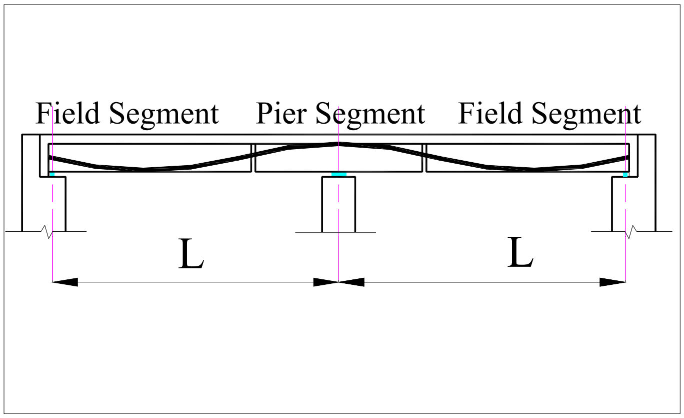

Figure 2 System II layout

Concrete NU I-Beam Capacities

The ultimate negative moment capacity controls the design. The maximum segmental span length for this system is higher than the span capacities. Consequently, the system is not optimized for this reason.

System II Discussion and Recommendations

The positive service tension capacity is close to the positive ultimate moment capacity, indicating that this system is efficient for these design criteria. However, the large difference between the negative and the positive capacities makes this system an inefficient overall.

Generally for this system the ultimate shear and the ultimate negative moment capacity are lower then the positive ultimate moment capacity and the tension service capacity, respectively. Significant capacity in the positive region remains unused. For example, for NU2000 girder spacing of 10 feet the ultimate positive span capacity permits a span of 270 feet while the ultimate negative capacity limits the span to 165 ft. That is why the pier segment needs to be deepened to optimize the structure as in system III or IV.

Improving the Efficiency of Systems I and II

In long-span spliced bridges, the sections over the pier are often subject to high shear and bending moment. In most cases, these sections may limit the span capacities of the system as we see in system II and system I methods B and C. In such cases, designers often use deeper sections at the pier in order to satisfy shear and flexure design requirements. Usually this is done by varying the web depth as in system III or by increasing the thickness of the girder's bottom flange at the pier section.

System III: Segmental Construction with Haunched Pier Segment

The third system is the first alternative of deepening the pier segment by using a curved haunched girder. For a two-span bridge, three precast pieces are used in this system: two field segments and a one-piece haunched pier segment. This system is the same as system II, with the exception of the pier segment's variable depth. See Kamel (1996) [7].

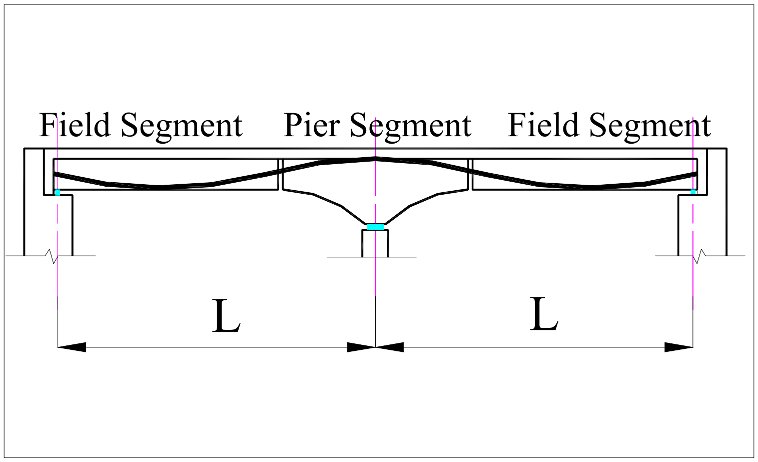

Figure 3 System III Layout

Concrete NU I-Beam Capacities

Here we see little improvement from System II. The ultimate negative moment capacity still controls the design. The ultimate shear capacity is improved from system II. The maximum segmental span length for this system is much higher than the span capacities.

System III Discussion and Recommendations

For system III, the ultimate negative moment capacity controls the design. The critical section for the ultimate negative moment is found to be three quarters of the distance from the pier center line to the end of the curved portion of the pier segment.

The tension positive service capacity is close to the ultimate positive moment capacity, indicating that this system is efficient for these design criteria. However, the large difference between the negative and the positive capacities makes this an inefficient system overall.

The maximum segmental span length for this system is higher than the span capacities. Generally for this system the ultimate shear and the ultimate negative moment capacity are lower then the ultimate positive moment capacity and the tension service capacity. Significant capacity remains unused. That is why the pier segment needs to be deepened more so that all the capacities are equal. However if the pier segment was deepened to optimize the system capacities, the height and the weight of the pier segment would be greater than the shipping and handling capacity. It is therefore recommended that a two-piece pier segment be used. This pier segment consists of a straight haunch block and I-girder to optimize the structure, shown as System IV.

System IV: Segmental Construction with Two Pier Segment Pieces: A Straight Haunch Block and an NU I-Girder[1].

The fourth system is the second alternative of deepening the pier segment. The system utilizes a two-piece pier segment: a straight haunch block and an NU-I girder. Refer to figure 4. Dividing the pier segment into two pieces allows constructing a deeper pier segment within the allowable shipping and handling capacities.

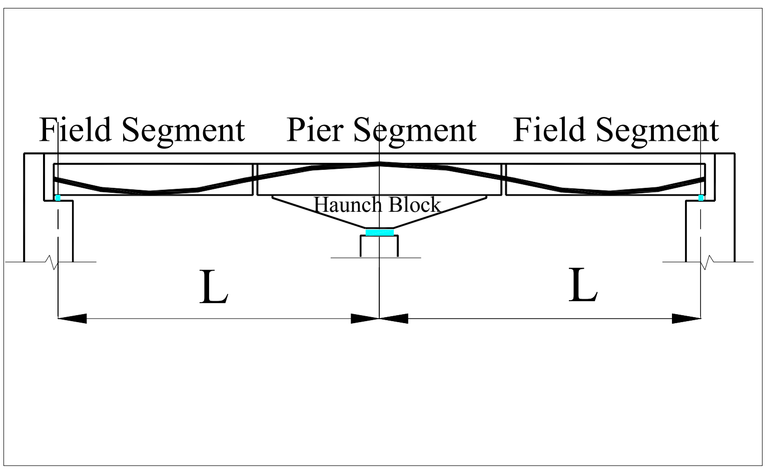

Figure 4 System IV Layout

Concrete NU I-Beam Capacities

The ultimate negative moment capacities, the ultimate positive moment capacities, the service III positive moment capacities, and the ultimate shear capacities are almost the same for all girders (with some modification for the web).

Optimizing the haunch block

The system is most efficient because all the capacities are equal. In order to achieve this goal, a two-piece pier segment is used, with an NU I-girder and a straight haunch block underneath. Haunch block dimensions of 0.50(L) in length and 0.9(h) deep were found to be the most efficient haunch block size, equalizing the ultimate negative, the ultimate positive, the service III positive, and the shear capacities with some modifications as shown in Table 1 (Where h is the girder height and L is the span length)

Table 1 Two-Span Bridge NU I-Girder Web Width Modifications for System IV

	8 ft Girder Spacing	10 ft Girder Spacing	12 ft Girder Spacing
	Web width (in.)	Web width (in.)	Web width (in.)
NU2000	7.0	7.5	8.0

With minor adjustments for the web width, it is clear that system IV is superior in terms of span capacity. The maximum segmental span length for shipping is lower than the span capacities for system IV. For this system it is recommended to barge the girder or splice more than one piece together in the field.

CONCLUSIONS

When the negative moment, positive moment, and shear capacities are far apart, the lower capacity controls the design, leading to under-utilized capacities. The second system has a large gap of up to 130 ft between the span capacities of the positive moment section (service and ultimate) and those of the ultimate negative moment and shear, while the first system has only a 60 ft gap.

The third system has a smaller gap than the second system, which reaches 100 ft between the span capacities of the positive moment section (service and ultimate) and those of the ultimate negative moment and shear.

With the suggested haunch block dimensions (0.5 L and 0.9 h), and using the modifications in table 1, the fourth system was found to be the most efficient system. All capacities of the system are equal. The gaps between the capacities that existed in the previous systems were avoided. In conclusion, ranking the four systems according to span capacities, the fourth system received the highest rank, followed by the third, the first, and the second system, in that order.

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APPENDIX A

BRIDGE ASSUMPTIONS

In order to study the capacities of each system and perform the comparison, assumptions need to be made, as shown in Table A-1.

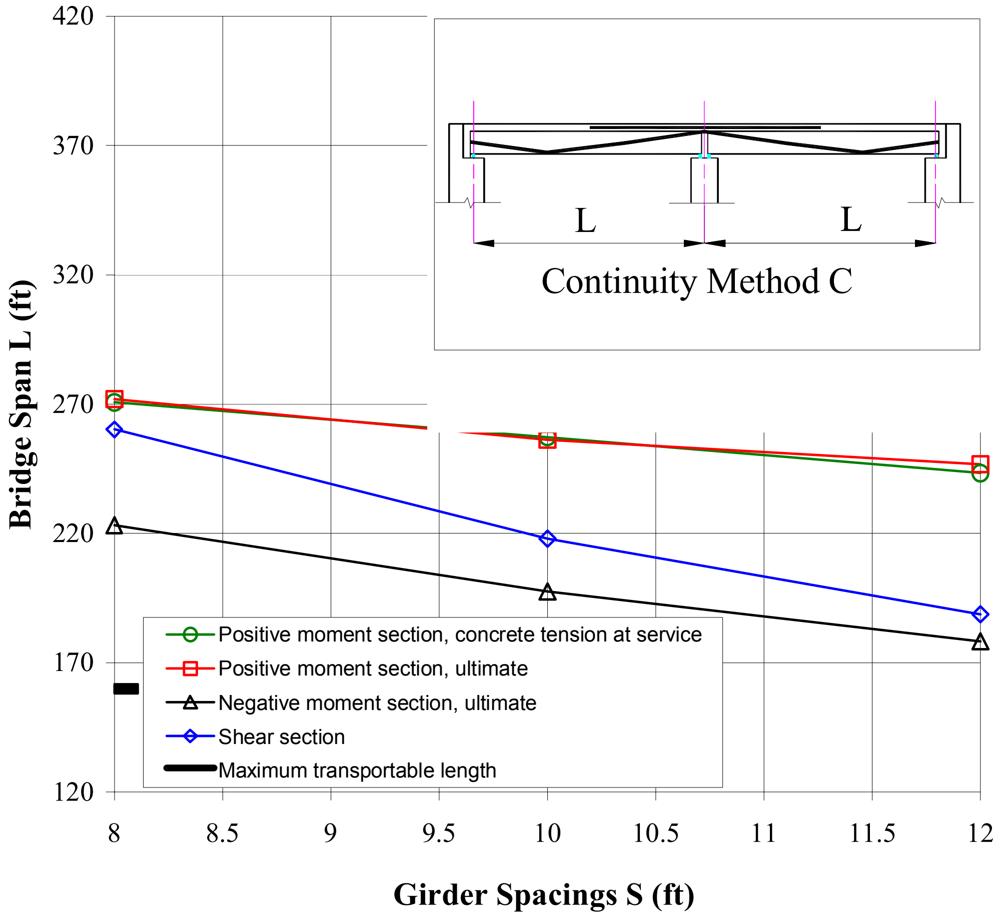
Table A-1 Bridge Assumptions

Bridge Data	Girder Spacing	8-10-12 ft
	Overall Width	46 ft-6 in.
	Number of Spans	2
	Deck Slab Thickness	8-10 ft girder spacing = 7.5 in.
		12 ft girder spacing = 8.0 in.
Span Data	Two Span Bridge	L-L
Concrete Data	Precast Concrete	28-days strength = 8,000 psi
		Release strength = 5,500 psi
		Unit weight =150 pcf*
	Cast in Place Concrete	28-days strength = 4,000 psi
		Unit weight =150 pcf*
Reinforcement Data	Steel bars	Yield strength = 60 ksi E_s =29,000 ksi
	Strands: See table 2 for prestress losses Low–Relaxation Strands 0.6 in.	Ultimate strength = 270 ksi E_s =28,500 ksi
	Threaded Rods	Min. yield stress** = 120 ksi Ultimate stress*** = 150 ksi
Load Data	S.I.D.L.	Future wearing surface= 25 pcf
		Barrier load = 0.3 kips/ft
	Live Loads	HL93[4]
Post-tensioning****	Stages: Applied at one stage	After casting the diaphragm
	Tendons: 3, 3.75 in. diameter, 15-0.6 in. strands each	Inside duct area > 2.5 strands area

* 148 pcf for young's modulus calculations and 150 pcf for weight calculations
** Elongation for 20 bar diameter 4% for yield stress
*** Reduction in area is 20% for ultimate stress
**** For initial and time dependent losses, please refer to Table A-2

Table A-2 Assumed Effective Prestressing at Each Construction Stage

Construction Stage	Stress in Pretensioning Strand	Stress in Post-Tensioning Strand
Pretensioning Strands	$0.92(0.75)f_{pu}$	---
Post-Tensioning Strands	$0.87(0.75)f_{pu}$	$0.92(0.78)f_{pu}$
Service Loads	$0.82(0.75)f_{pu}$	$0.82(0.78)f_{pu}$



Method C NU2000 Span Capacities- Sample Span Chart