DESIGN OF ADJACENT PRECAST BOX GIRDER BRIDGES ACCORDING TO AASHTO LRFD SPECIFICATIONS

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ABSTRACT

Precast, prestressed concrete box girders are widely used in short and medium span bridges in North America. Adjacent box girder bridges are particularly popular due to their speed and ease of construction, favorable span-to-depth ratio, versatility, and aesthetic appeal. The majority of these bridges are still constructed of concretes with strengths of not more than 50 MPa. There is a lack of cost effectiveness studies, and preliminary design guidelines that would provide designers and precasters with the incentive to use high-performance concrete (HPC). To make matters even more complicated, designers will soon have to convert to AASHTO LRFD Specifications, which substantially change the way future bridges will be designed. Any economic studies and design guidelines will have to reflect the forthcoming changes in bridge design philosophy. This paper presents a rigorous and systematic procedure using mathematical optimization techniques for the design of adjacent box girder bridges. The procedure is used to develop a design optimization system that can be utilized to carry out cost effectiveness studies of the use of HPC in this type of bridges, and to develop preliminary design charts and guidelines according to AASHTO LRFD Specifications. A numerical design example is presented to demonstrate the application and capabilities of the developed optimization system.

INTRODUCTION

Precast, prestressed concrete box girders are widely used in short and medium span bridges in North America. Surveys indicate that close to 50% of bridges built in the U.S. are prestressed concrete (Dunker and Rabbat 1992). One third of these bridges are precast box girder type structures. In most of them, the box girders are placed adjacent to each other as shown in Figure 1. They are generally connected at their interfaces by grouted shear keys and, in some states, are provided with a nominal amount of full-width, transverse post-tensioning. Since their introduction in the 1950s, adjacent precast box girder bridges have generally performed very well. Tests done on girders that were more than 25 years old have shown that the girders have maintained their structural strength and ductility, and that their behavior at service loads and at factored ultimate loads was excellent (Shenoy and Frantz 1991). Bridges of this type have become popular and economical solution in more than 30 states throughout the U.S. (Miller *et al.* 1999) for a variety of reasons. When built as a non-composite structure, there is no need to cast and cure a deck slab, making erection fast, easy, and economical even in remote locations. The adjacent box girder also has a favorable span-to-depth ratio,

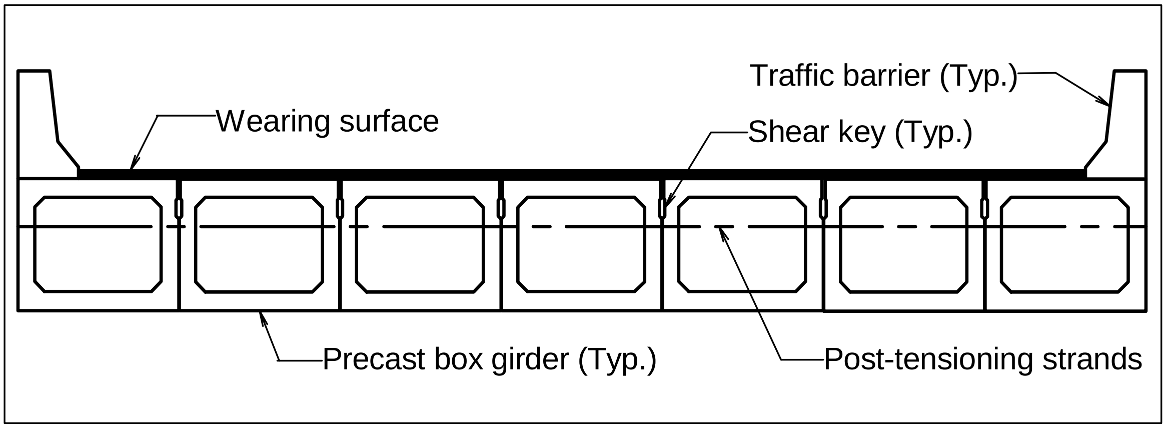

Figure 1. Cross section of a typical adjacent box girder bridge

which is important where vertical clearance is a design consideration. In addition, recent projects have demonstrated the versatility and aesthetic appeal of this type of bridges (Shutt 2001); the precast box girders can be produced in various sizes with or without haunches. Moreover, they eliminate the many visual break lines along the side face of the bridge that are associated with I-shaped girder bridges and are unsightly to some observers, especially in urban areas.

The majority of precast box girder bridges are still made of concretes with strengths of not more than 50 MPa. Strengths of about 60 MPa are normally considered the maximum achievable in the precast, prestressed concrete industry. This appears to be very timid in comparison with the concrete strengths of 100 MPa and more, which have been used in the building industry for years. Despite the widely known potential advantages from the utilization of high-performance concrete (HPC) with its increased strength and improved durability, the higher initial cost and the increased quality control requirements associated with its production seem to deter many designers and precasters from implementing the material. Another reason may be that the current edition of the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (AASHTO 1998) do not allow the use of concrete strengths above 70 MPa without conducting tests “to establish the relationships between the concrete strength and other properties.”

A major benefit from the use of HPC for precast box girder bridges is that it allows significant increases in span lengths. When span lengths increase, the number of piers and foundations can be reduced in multi-span bridges, or they can even be entirely eliminated. This, in turn, reduces the cost of the substructure. Another benefit from the use of HPC is that it allows the use of a shallower girder cross-section for a given span length, which increases the vertical clearance when replacing an old bridge. A limited number of experimental HPC adjacent box girder bridges have been built in the U.S. to showcase the advantages of the material (Greuel *et al.* 2000, and Shing *et al.* 1997). However, there is still a lack of cost effectiveness studies and preliminary design guidelines that would provide a comfort level to designers and precasters that the material can be economically and confidently produced.

To complicate matters even more, various states are moving to convert from AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) to AASHTO LRFD Specifications (AASHTO 1998), which substantially change the way future bridges will be designed. Nearly half of the states have reviewed or implemented the LRFD methodology for bridge design. According to the Federal Highway Administration (FHWA), the target date for all states to have fully implemented LRFD is October 1, 2007 (Lwin *et al.* 2001). The FHWA will eventually mandate that states use the LRFD Specifications to qualify for significant federal matching of funds for new bridge construction projects. Any economic studies and design guidelines for adjacent box girder bridges will have to reflect the forthcoming changes in bridge design philosophy.

This paper summarizes the vehicular live load provisions in AASHTO LRFD Specifications, and reviews some of the problematic issues that might hinder the application of HPC in the design and construction of adjacent box girder bridges according to these Specifications. The paper then introduces a rigorous and systematic procedure that utilizes mathematical optimization techniques for the design of these bridges. The procedure is used to develop an optimization system that can be utilized to carry out cost effectiveness studies of the use of HPC for adjacent box girder bridges, and to develop preliminary design charts according to LRFD Specifications as a tool to obtain optimum bridge superstructure designs. The intent of the study presented herein is to look beyond current precast concrete production capabilities, and bridge design and construction methods. Although current practice is considered as the basis for the assumptions underlying the study, it is not used as a means to restrict potential applications of HPC in precast, prestressed concrete adjacent box girder bridges.

AASHTO LRFD BRIDGE DESIGN PROVISIONS

General

With the 2000 interim revisions, AASHTO archived the Standard Specifications (AASHTO 1996), which has been in use for highway bridge design in the U.S. for more than 70 years. These Specifications will no longer be maintained or enhanced by AASHTO. Future efforts will be concentrated on the LRFD Specifications (AASHTO 1998). The latter was created to respond to some assessments that the former had inconsistencies, discernible gaps, and even some conflicts (Lwin *et al.* 2001).

The LRFD Specifications use factored (increased) loads along with reduced resistance (capacity) factors. The major difference between the AASHTO Standard and the AASHTO LRFD design philosophies is that the latter has more uniform levels of safety along with more accurate bridge analysis methods. The primary objective of LRFD is to design and construct bridges with better serviceability, longer life spans, and improved maintenance characteristics. This is accomplished by using factors that were calibrated based on statistics and structural reliability theory to describe the uncertainties of load and resistance (Lwin *et al.* 2001).

LRFD uses limit states instead of load combinations like its AASHTO Standard predecessor. The limit states consist of various service, strength, fatigue, and extreme event states that must be satisfied. A limit state is defined in the Specifications as "a condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed." The design of pretensioned girders using the LRFD Specifications usually consists of satisfying the requirements of Service I, Service III, and Strength I limit states. Service I is used when checking compressive stresses, while Service III is applicable when checking allowable tensile stresses. Strength I is used for design at the Strength Limit State.

LRFD Live Load Model

Developed in 1993, the LRFD live load model (HL-93) consists of a combination of the design truck or design tandem applied together with the design lane load. The LRFD design truck is identical to the Standard HS20 truck (AASHTO 1996). The LRFD design tandem consists of two axels (110 kN each) spaced 1.2 m apart. In either case, the transverse spacing of wheels is taken as 1.8 m. The LRFD design lane consists of a uniformly distributed load of 9.3 kN/m in the longitudinal direction; it is distributed transversely over a 3.0-m width. The static effects of the design truck or design tandem are multiplied by (1+IM/100), where IM is the dynamic load allowance. For all limit states, except Fatigue, IM is taken as 33%. For Fatigue Limit State, it is taken as 15%. The dynamic load allowance is not applied to the design lane load.

LRFD Live Load Distribution

For many years, AASHTO has kept the lateral distribution of wheel loads to a simple factor, S/D, where S is the spacing of the girders, and D is a constant based on the bridge type. This formula has been found to generate valid results for bridges of typical geometry (i.e. non-skewed bridges with girder spacing near 1.8 m and span length about 18 m), but would lose accuracy very quickly when the bridge parameters were varied (e.g. when relatively short or long span bridges were considered) (Zokaie 2000). AASHTO LRFD Specifications present more accurate, but more complex formulas for live load distribution that account for parameters such as span length, girder spacing, and cross-sectional properties of the bridge deck. LRFD computes the live load distribution factor per lane rather than per wheel as in the Standard Specifications.

For example, for one design lane loaded, the live load moment distribution factor, g, per lane in an interior adjacent box girder is expressed as

$$g = k\left(\frac{b}{2.8\,L_g}\right)^{0.5}\left(\frac{I}{J}\right)^{0.25}$$

where

$$k = 2.5\left(N_g\right)^{-0.2} \geq 1.5,$$

N_g = number of girders, b = width of girder (mm), L_g = girder span (mm), I = girder moment of inertia (mm^4), and J = girder St. Venant torsional constant (mm^4).

For two or more design lanes loaded, g in an interior adjacent box girder is defined as

$$g = k\left(\frac{b}{7600}\right)^{0.6}\left(\frac{b}{L_g}\right)^{0.2}\left(\frac{I}{J}\right)^{0.06}$$

It should be noted that the live load distribution factors given in the LRFD Specifications take into consideration the multiple presence factor.

Flexural (Longitudinal) Design

AASHTO LRFD Specifications limit the allowable concrete compressive stress to 60% of the concrete's breaking strength at release. This limitation has not often been a factor in the past because the allowable tensile stress at service conditions usually controlled cross-section size and prestressing force requirements. However, the use of HPC permits longer span bridges with shallower girder depths. Frequently, these design cases are limited by the allowable compressive stress at release. Some tests have indicated that this limit could be increased to 70% of the concrete's breaking strength at release (Russell and Pang 1997). Of less significance is the allowable tensile stress limit. Some studies have suggested an increase of 33% in this limit (Bridge 1997). More comprehensive studies are still needed, however, to develop the knowledge of allowable stress design and the impact of increasing the limits on allowable stresses.

For box sections, it is common that the flexural strength requirement controls the design. Approximate formulas for pretensioning steel stress at ultimate flexure are given in the LRFD Specifications. Use of these formulas simplifies the process of calculating the flexural strength by eliminating consideration of nonlinear material properties of both concrete and prestressing steel at ultimate conditions. However, due to their simplified nature, these formulas should be used with caution, especially beyond the limits for which they were developed. Another area of concern is that the LRFD Specifications may considerably overestimate the neutral axis depth, especially for a cross-section with non-rectangular geometry (Badie and Tadros 1999). This results in underestimating the flexural capacity of the section. The general strain compatibility approach can be used to avoid the difficulties in applying the approximate formulas or the inaccuracies associated with their use. LRFD Specifications recognize the strain compatibility approach but do not give any guidance relative to its application in practice.

Transverse Design

The major problem associated with adjacent precast, prestressed box girder bridges has been cracking of the grouted shear keys, which connect the girders at their interfaces (Huckelbridge *et al.* 1995). The grout used to make the keys has been observed to crack and allow water to leak between the girders. This water, in many cases, contains deicing salts and, over time, it penetrates the girders and causes corrosion of the prestressing strands (Miller *et al.* 1999).

Several recent studies (El-Remaily *et al.* 1996, and Lall *et al.* 1998) have indicated that the shear key connection can be significantly strengthened, and cracking can be reduced or eliminated by providing adequate transverse post-tensioning. To make the box girders act together, AASHTO LRFD recommends a minimum average transverse prestressing of 1.7 MPa be used. However, it does not specify the contact area over which this post-tensioning force should be introduced. It is unclear whether it should be the top shear key area, the diaphragm-to-diaphragm contact area, or the full girder side face. Also, it does not specify the spacing between diaphragms nor the diaphragm size. El-Remaily *et al*. (1996) indicated that the recommendation given by AASHTO LRFD is not precise. They suggested that the LRFD Specification indicates that the bridge deck be designed as a rigid assembly of gridwork, and that post-tensioning acting on the transverse members of that grid, i.e. the diaphragm lines, be designed for not less than 1.7 MPa. A chart was developed for the determination of the required amount of transverse post-tensioning for the standard AASHTO box girder depths of 685, 840, 990, and 1065 mm. It is noted that the 1.7 MPa prestress specified in the LRFD Specification appears to be somewhat arbitrary. Testing is needed to determine a more refined value for this force.

FORMULATION OF THE DESIGN OPTIMIZATION PROBLEM

General

Most structures are designed on a trial-and-error basis. A preliminary design is estimated and analyzed. If it is satisfactory, it is considered a feasible design. If the trial design is not satisfactory, the designer has to change it and repeat the analysis until a feasible one is obtained. Usually, there are an infinite number of feasible designs, and designers strive to find the best (optimum) within the time and resources they have available.

Mathematical optimization techniques provide systematic procedures by which optimum designs can be obtained with substantially less time and effort. The formulation of an optimum design problem requires identification of a set of design variables that describe the structure, an objective function that measures the merits of alternate designs, and design constraints that must be satisfied (Arora 1989). The objective function and constraints must be functions of the design variables. Prestressed concrete design optimization problems, in general, are nonlinear because the objective function and/or most of the constraints are nonlinear functions of the design variables; thus, requiring nonlinear programming procedures to be used. Some previous studies have adopted linear programming methods (Fereig 1994, and 1999), or have used a simple numerical iterative method (Schemmel and Zia 1990).

Design Variables

The design of a structure can be completely described by a set of pre-assigned parameters, and a set of design variables. Only the design variables are modified during the optimization process. For standard AASHTO precast box girders, cross-sectional dimensions are known and become pre-assigned parameters. Design variables include the required prestressing force (or alternatively, the required amount of prestressing steel), the tendon profile defined by its eccentricity, and the girder concrete compressive strength at 28 days.

Objective Function
The objective function to be minimized is taken as the superstructure cost / deck area, i.e.

$$\text{Cost} = N_g \, C_g \, / \, WL$$

where C_g = cost of each girder (including materials, production, transportation, and erection costs), W = width of the bridge, and L = total length of the bridge. Items with fixed costs do not need to be included in the objective (cost) function because they are included in all alternative feasible designs and, therefore, have no effect on the optimum design. Examples of such items are the wearing surface, barriers and guardrails, drains, lighting, and signs.

Design Constraints
The cost function is minimized under all relevant constraints according to AASHTO LRFD Specifications. These include the flexural constraints at service limit states, and at strength limit states. In addition, practical constraints, which constitute limits for the design variables, are included. Other constraints (e.g. shear and deflection) could be added. However, in general, they have marginal effect on the design and were left to be checked at the final design stage. For brevity, only the active constraints (governing design criteria) are presented herein (Compressive stresses are positive).

1. Concrete allowable compressive stress at prestress transfer at the transfer length section (Service I), i.e.

   $$\frac{P_i}{A} + \frac{P_i \, e}{S_b} - \frac{M_g + M_d}{S_b} \leq 0.6 \, f'_{ci}$$

   where P_i = initial prestressing force, A = girder cross-sectional area, e = tendon eccentricity, S_b = girder cross-section modulus with respect to its bottom surface, M_g = moment due to self-weight of the girder, M_d = moment due to weight of diaphragms, and f'_{ci} = girder concrete strength at transfer.

2. Maximum eccentricity, i.e.

   $$e \leq e_{max}$$

   where e_{max} = maximum practical eccentricity that can be accommodated within the girder.

3. Flexural strength of girders at Strength Limit State (Strength I), i.e.

   $$M_r \geq M_u$$

where M_r = factored flexural resistance of the section, and M_u = factored moment.

Solution of the Optimization Problem

Once the design optimization problem has been formulated, it is transcribed into the following standard nonlinear constrained optimization model: Find the set of n design variables contained in the vector {b} that will minimize the objective function

$$f(\{b\}) = f(b_1, b_2, ..., b_n)$$

subject to the constraints

$$h_i(\{b\}) = 0, \quad i = 1, ..., k$$

$$g_i(\{b\}) \le 0, \quad i = 1, ..., m$$

$$b_i^l \le b_i \le b_i^u, \quad i = 1, ..., n$$

where k is the number of equality constraints, m is the number of inequality constraints, b_i^l and b_i^u are the lower and upper bounds on the ith design variable, respectively.

Many numerical methods have been developed to solve nonlinear constrained optimization problems. The methods start from an initial design provided by the user, which is iteratively improved until the optimum is reached. In this study, the sequential quadratic programming (SQP) method (Arora 1989) (also known in the literature as the recursive quadratic programming method) is used to solve the nonlinear constrained optimization problem. The use of this method is justified because an extensive comparative study of nonlinear programming methods presented by Schittkowski (1980) ranked the performance of classes of algorithms more or less in the following descending order: SQP method, generalized reduced gradient method, method of multipliers, and other penalty methods. The SQP method generates a sequence of quadratic programming sub-problems that are to be solved sequentially (Arora 1989). The method requires the first-order partial derivatives of the objective function and all constraints with respect to the design variables. Analytical expressions for such partial derivatives are generally difficult to obtain for practical engineering problems, and are, thus, usually calculated numerically using the finite difference method. The SQP method is incorporated into several general-purpose design optimization software packages that are available commercially (e.g. GENESIS and IDESIGN), or available in the public domain.

NUMERICAL DESIGN EXAMPLE

Assumptions

The following structural design assumptions were made:

1. Analysis and design conform to the AASHTO LRFD Bridge Design Specifications (AASHTO 1998) except where otherwise noted.
2. Single span bridge is investigated. A typical interior simply supported girder is considered; the reader is reminded that both the LRFD Specifications and current practice require that exterior girder capacity be at least equal to interior girder capacity. It may be desirable to provide the same amount of prestressing steel for all girders in order to equalize cambers and haunches (Chen and Aswad 1996).
3. Flexure governs the design of girders. In general, constraints other than those pertaining to flexure (e.g. shear and deflection) have marginal effect on the design. These are left to be checked at the final design stage.
4. A 75-mm thick bituminous non-composite wearing surface is placed on top of the girders.
5. Two traffic barriers each with 0.3 m^2 cross-section distributed equally to all girders.
6. Girders are transversely post-tensioned through 200-mm thick full-depth diaphragms located at quarter points (El-Remaily *et al*. 1996).
7. Concrete compressive strength at transfer is on average 70 percent of 28-day strength (Bridge 1997).
8. Effective prestressing force after losses is 80 percent of prestressing forces at transfer.
9. Seven-wire, low-relaxation, 15.2-mm diameter strands with an 1860-MPa tensile strength are used.
10. Only straight strands are used; neither debonding (shielding) nor draping (harping) of strands is considered in the present study.

Case Study

To illustrate the application and capabilities of the developed optimization system, a 38-m, single span, AASHTO Type BIII-1220 box girder bridge with no skew is considered. The girder cross-section is shown in Figure 2 (Bridge 1997). The properties of this section are as follows: A=0.522 m^2, y_b=0.490 m, I=0.069974 m^4, S_t=0.139865 m^3, S_b=0.142892 m^3, and J=0.118497 m^4. Note that y_b is the distance from centroid of the

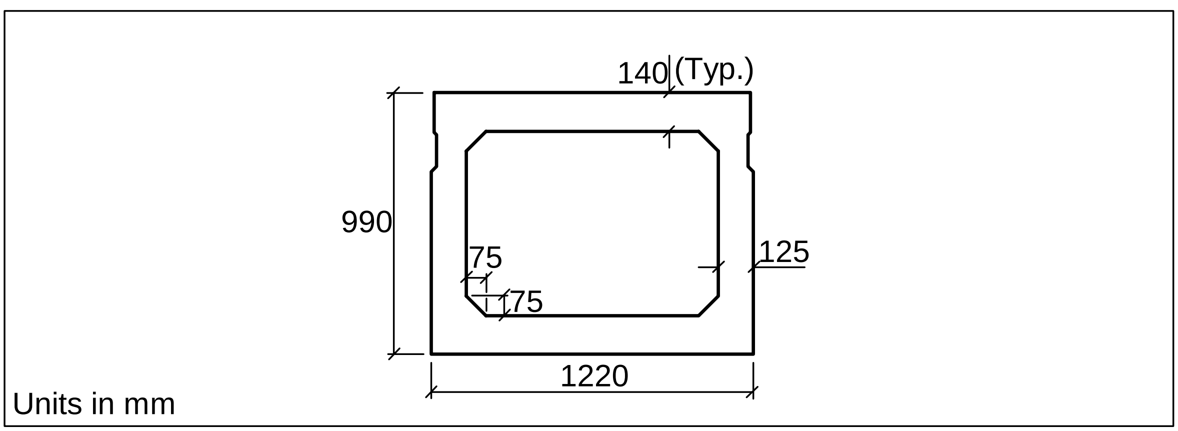

Figure 2. AASHTO box girder Type BIII-1220

girder cross-section to the extreme bottom fiber, and S_t is the girder cross-section modulus with respect to its top surface. The superstructure consists of seven adjacent girders (as shown in Figure 1) carrying two traffic lanes. In this study, design of this bridge implies finding values for all of the design variables.

The cost function requires unit costs for material, labor, and construction costs. Because girder concrete strength is a design variable that could change at each iteration of the design optimization process, it is necessary to have a continuous function for the concrete mix cost. Hassanain and Loov (1999) presented such a function. It relates the cost of any concrete mix to the cost of a 40-MPa mix by a ratio defined as

$$\mathrm{CMCR} = 0.936 + \left(\frac{f'_c}{100\ \mathrm{MPa}}\right)^3$$

where CMCR is the concrete mix cost ratio, and f'_c is the concrete compressive strength at 28 days. CMCR can be obtained from the above equation for any level of concrete strength, and then multiplied by the cost of a 40-MPa concrete. In the present study, the cost of a 40-MPa mix is assumed to be $78/m^3 including an overhead rate of 18%. In addition, labor and curing have been estimated to cost an additional $28/m^3. Seven-wire, 15.2-mm diameter prestressing strands cost $1.5/m for material and labor including wastage of 10% and an overhead rate of 18%.

Although a nominal overhead has been included, it should be recognized that the intent of this paper is to provide information, which can lead to a minimum cost to the construction company bidding a project. The company can apply overhead rates and establish profit margins as it sees fit, depending on expected competition.

A detailed approach for estimating transportation and erection costs for the precast girders is presented by Hassanain and Loov (1999), and is not included here because of space limitations.

At the optimum point, the following values for the design variables were obtained:

- The initial prestressing force in each girder is 6.34 MN, i.e. the effective prestressing force after losses is 5.07 MN. This translates into 32 strands per girder.
- Tendon eccentricity is 352 mm.
- Girder concrete compressive strength at 28 days is 64 MPa.

Final value of the cost function at the optimum point is $312.54/m^2.

It was found that if the girder concrete strength at 28 days were set to 40 MPa, which is a commonly used strength in precast, prestressed concrete plants, the span length of this AASHTO Type BIII-1220 box girder would be limited to 27 m compared to 38 m when using a 64-MPa concrete. This shows the effect of HPC on extending span capabilities.

The developed design optimization system is currently being used to carry out cost effectiveness studies of the use of HPC for adjacent precast box girder bridges. In addition, the system is being utilized to develop preliminary design charts and guidelines according to AASHTO LRFD Specifications as a tool to obtain optimum bridge superstructure designs.

CONCLUSION

Precast, prestressed concrete box girders are widely used in short and medium span bridges in North America. Adjacent box girder bridges are popular because of their economy, ease and speed of construction, favorable span-to-depth ratio, versatility, and aesthetic appeal. There are several benefits to the use of HPC in this type of bridges. To provide the economic incentive for designers and precasters to use the material widely, cost effectiveness studies, and design guidelines that reflect the forthcoming changes in bridge design philosophy are needed. This paper presented a design optimization system developed to provide these products. A numerical design example was presented to illustrate the application and capabilities of the system. Cost effectiveness studies are being carried out and preliminary design charts and guidelines are being developed to obtain optimum bridge superstructure designs.

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