Proposed Replacement of AASHTO Girders with New Optimized Sections

BASILE G. RABBAT and HENRY G. RUSSELL

ABSTRACT

Structural efficiency and cost-effectiveness of bridges built with pretensioned I-sections and T-sections and a cast-in-place deck were evaluated. Selected precast, prestressed sections produced in the United States were compared with AASHTO and Prestressed Concrete Institute (PCI) girders. Spans in excess of 80-ft (24.4 m) were considered. Bulb-T, Colorado, and Washington girders were more structurally efficient than AASHTO-PCI girders. Cost analyses were performed on existing Bulb-T, Colorado, Washington, and AASHTO girders, and on modified counterparts with 6-in.-thick (152-mm) webs. Bulb-T's were found to be the most cost-effective girder, with estimated cost savings of 17 percent on the in-place cost of girders and deck compared with the AASHTO girders. For equal span length, girder spacing, and truck loading, modified Bulb-T's required up to 25 percent less prestressing force than the AASHTO girders. Modified Bulb-T's are recommended for use as national standards. Ways of implementing the proposed new optimized sections are suggested.

The standard AASHTO and Prestressed Concrete Institute (PCI) girders, types I through VI, were developed in the late 1950s and early 1960s. Standardization has led to simplified design and economical bridge construction (1). In the past 25 years there have been significant advancements in the technology of prestressed-concrete design and construction. Individual state highway departments developed their own standards for improved efficiency and economy. With new designs entering the market, the question became: How efficient are the standard AASHTO girders?

OBJECTIVES

This investigation was undertaken to evaluate the latest prestressed-concrete bridge girder designs being used in the United States and to determine which designs represent optimum designs that could be promoted as national or regional standards. The investigation was limited to bridges built with pretensioned I-sections and T-sections, for spans in excess of 80 ft (24.4 m), and with concrete compressive strengths up to 7,000 psi (48.3 MPa).

SCOPE

The objectives were accomplished within the following scope:

1. Current precast, prestressed-concrete girders with composite cast-in-place deck designs being used in the United States were summarized;
2. Creative new concepts becoming available through research were reviewed;
3. Girders representing optimum designs and exhibiting strong potential for standardization were determined; and
4. Recommendations for standardization of the most practical and cost-effective designs were made.

RESEARCH APPROACH

The project was divided into two phases. In phase 1 information was collected throughout the United States on a regional basis from selected highway agencies and producers. Advantages and disadvantages of the concepts inventoried were assessed.

In phase 2 structural efficiency and cost-effectiveness of the best existing designs, as well as some modified ones, were evaluated relative to the efficiency of AASHTO sections. This included evaluation of structural parameters such as girder spacing, span length, concrete strength, and deck thickness.

A computer program was developed for use in the parametric studies. A relative unit cost index was assigned to girder and deck-slab concretes, prestressing strands, and reinforcing steel. The cost index reflected in-place relative costs for the finished girder and deck. Costs of materials and labor were included. Data generated by the computer program were used to determine the most cost-effective girders and to develop design charts.

Survey results of phase 1, computer program documentation, and results of phase 2 analyses are available in a detailed report (2). A summary of the cost-effectiveness analyses and a sample design chart are presented in this paper.

COST-EFFECTIVENESS ANALYSIS

Cross Sections Analyzed

Comparisons of the structural efficiency of existing girders indicated that the most efficient sections were Bulb-T, Washington series, and Colorado G54 and G68 sections (2). Bulb-T's have been used successfully in the Pacific Northwest. A set of Bulb-T sections was developed in 1959 by Anderson (3). These sections, as well as the Washington series and Colorado G68, have 5-in.-thick (127-mm) webs. Strands deflected within the webs of these sections are bundled. End blocks are also used in these girders.

Several survey participants expressed concern about possible difficulties in manufacturing and transporting girders with 5-in.-thick webs. The main concerns were consolidation of the concrete in thin and deep members and stability of such slender members during transport. On the other hand, some survey participants believed that current AASHTO girders can be improved by reducing their web thickness.

At a meeting held in April 1980, members of the PCI Committee on Bridges were asked about the minimum practical web width to place and consolidate the concrete in precast, prestressed I-sections. All committee members were in favor of a minimum web thickness of 6 in. (152 mm). (Note that these data are from the minutes of the PCI Committee on Bridges meeting held April 15, 1980, at the Ramada O'Hare Inn, Des Plaines, Illinois, J. Barker, chairman.)

Standard AASHTO bridge girders types I and II have 6-in.-thick webs. In all regions of the United States, concrete has been placed and consolidated in these sections without difficulty. Therefore, in phase 2 sections with 5-in.-thick webs were evaluated and compared with similar sections with 6-in.-thick webs. Sections with 6-in.-thick webs should be easier to manufacture and transport than sections with 5-in.-thick webs.

Existing and modified sections analyzed for cost-effectiveness are shown in Figures 1 and 2.

Structural Parameters

The sections were evaluated through a detailed

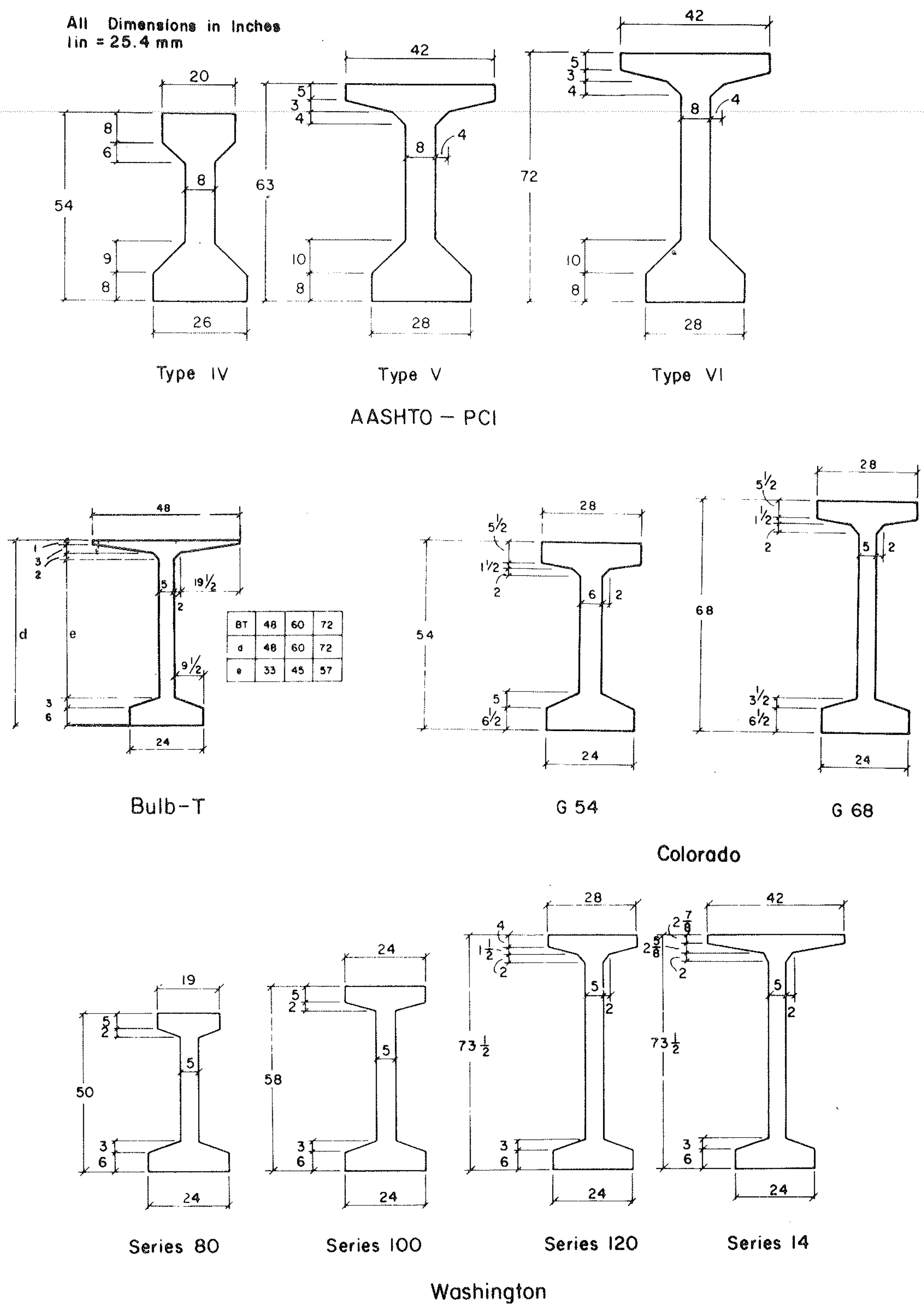

FIGURE 1 Existing girders analyzed.

structural analysis. Parameters considered in the analysis included girder spacing, span length, deck thickness, and concrete strength. Girder spacing was varied between 4.5 and 10 ft (1.37 and 3.05 m). Spans in excess of 80 ft (24.4 m) were considered. Deck thickness varied with girder spacing. Concrete strength for girders was varied between 5,000 and 7,000 psi (34.5 and 48.3 MPa).

Development of Computer Program

To evaluate the effect of each variable, a parametric study was carried out. The number of variables necessitated preparing a computer program to analyze each case and to generate cost data. This program, called BRIDGE, required input of girder span, spacing, and cross section; concrete and strand characteristics; and relative costs of materials. The program determined deck thickness and reinforcement, required number of strands, and cost index per unit surface area of bridge deck.

The following assumptions were made in program BRIDGE:

1. Design conforms to AASHTO specifications (4).
2. Live load consists of HS20-44 loading.
3. Girders are simply supported.
4. A typical interior girder is considered.
5. Concrete deck is cast-in-place and acts compositely with the girder. Deck formwork is sup-

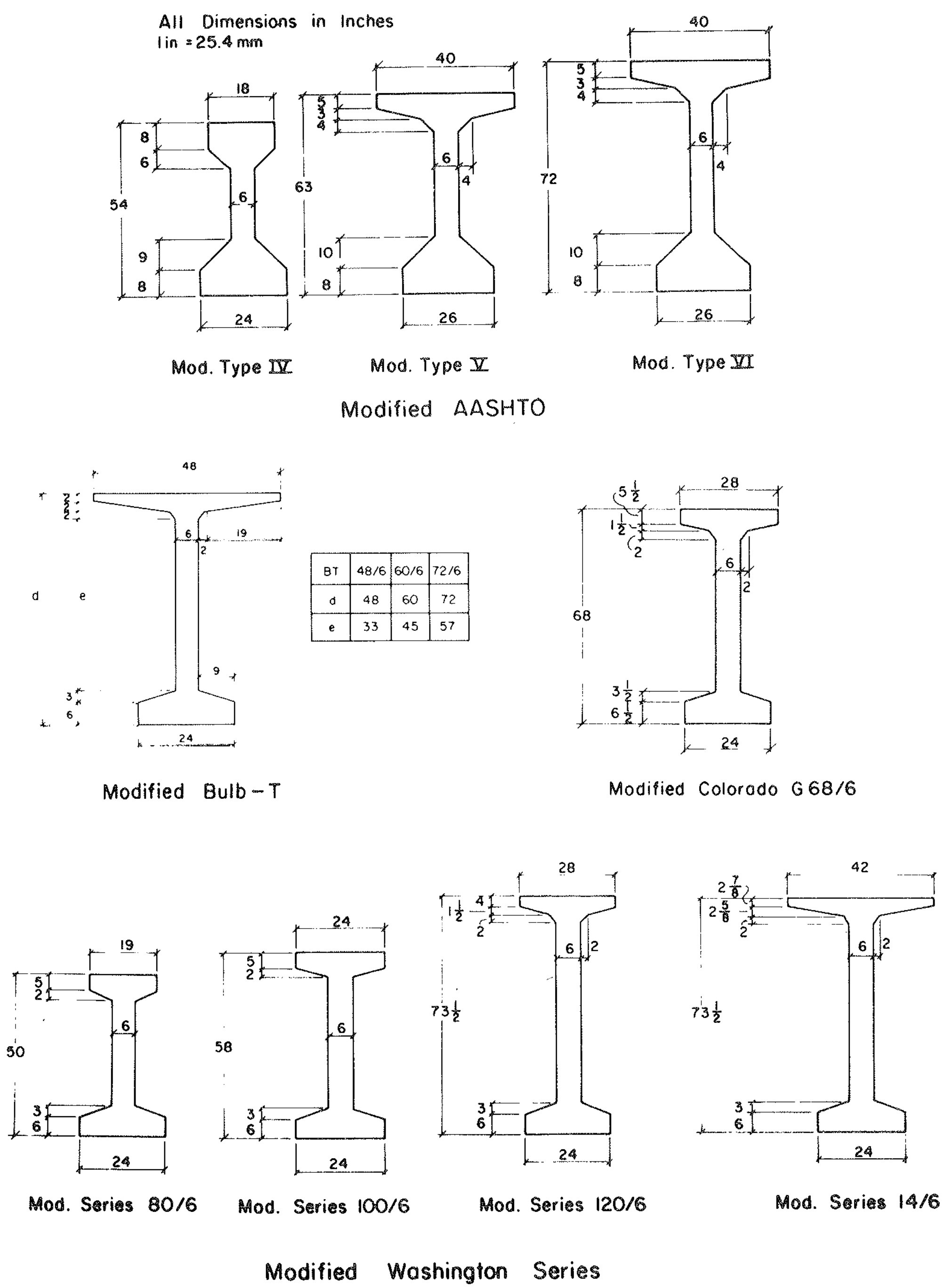

FIGURE 2 Modified girders analyzed.

ported on the girder. In calculations of the composite section properties, the transformed area of strands is neglected.

6. Concrete compressive strength of the deck is constant and equal to 4,000 psi (27.6 MPa) at 28 days.

7. Strands are grade 270 (1862 MPa) stress relieved, with 0.5-in. (12.7-mm) diameter and have an idealized trilinear stress-strain curve.

8. Total prestress losses are constant and equal 45,000 psi (310 MPa).

9. Initial or long-term camber or sag does not govern design, as the AASHTO specifications (4) do not specify deflection limits for concrete bridges.

10. Cost of materials, labor, transportation, and erection of girders with concrete compressive strengths between 5,000 and 7,000 psi (34.5 and 48.3 MPa) is assumed constant. The effect of increasing the girder concrete strength from 5,000 to 7,000 psi on the in-place cost of the girder is negligible.

11. Relative unit costs of materials and labor are constant for the cost analysis. All girders are compared on a common basis.

12. Cost analysis comparisons are for precast girders and a cast-in-place deck. Cost of substructure and approach fills are not considered.

13. Additional costs from the use of end blocks in all girders that have 5-in.-thick (127-mm) webs are ignored.

Relative Unit Cost Indexes

Several factors affect the cost of the superstructure. Costs of material and labor vary from region to region, between states of a region, between districts of a state, and within a district according to bridge location. An assessment of local and regional factors was not possible within the scope of this investigation. However, a cost analysis was possible by comparing the cost of the recommended sections on a common basis.

From survey data an average cost was determined for girder concrete, deck concrete, reinforcing steel, and prestressing strands. These average costs included materials and labor. For girder concrete, the cost also included transportation and erection. Average costs were then reduced to relative costs per pound of in-place material. The following relative unit costs for in-place materials (including labor) were used for the cost analyses: concrete (girders and deck), 1 unit per pound; strands, 8 units per pound; reinforcing steel, 9 units per pound; and epoxy-coated reinforcing steel, 12 units per pound.

Girders were compared based on the same unit costs. The relative costs of materials were taken as the product of material weight and relative unit costs. The summation of relative costs of materials was then divided by deck area to give a cost index per square foot. Additional weight and therefore cost of concrete required for the end blocks of girders that have 5-in.-thick webs were ignored.

Optimum Cost Index Charts

By using the BRIDGE program, a cost chart (2) was prepared for each of the sections shown in Figures 1 and 2. The same relative unit costs for in-place materials (material and labor), as well as material properties, were assumed for all girders and decks. A representative chart is shown in Figure 3. This figure shows the cost index per square foot of deck versus span length for an AASHTO type VI girder. The solid lines are for selected girder spacings. Maximum girder spacing was set at 10 ft (3.05 m). The dashed line is an optimum cost curve.

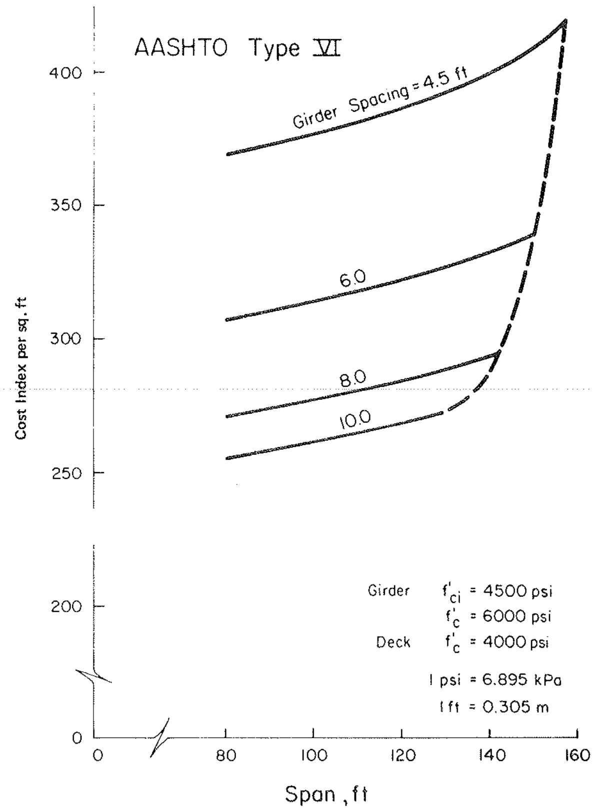

FIGURE 3 Cost chart for AASHTO type VI girder.

Figure 3 illustrates the effect of girder spacing on cost. For a given span, as girder spacing increases, unit cost per square foot of bridge deck decreases. For an AASHTO type VI section, if girders are spaced 10 ft apart, the cost per unit area of bridge deck is 30 percent less than if girders are spaced 4.5 ft (1.37 m) apart. Therefore, it is most economical to place girders at the largest practical girder spacing. This fact has already been suggested by Scott (5) and Jacques (6).

COST-EFFECTIVENESS COMPARISONS

Optimum cost curves were used to compare the cost-effectiveness of selected girders. Girder spacing ranged between 4.5 and 10 ft (1.37 and 3.05 m). A few selected cases are compared here.

Overall Comparisons

Optimum cost curves for AASHTO type VI, Colorado G68, Washington series 14, and Bulb-T BT72 girders are compared in Figure 4. These girders are intended to be used for spans in excess of 100 ft (30.5 m). The data in Figure 4 indicate that the Bulb-T BT72 is the most economical girder for spans up to 135 ft (41.2 m), and that the AASHTO type VI girder is the most expensive.

Modified girders G68/6, series 14/6, and BT72/6 are compared with an AASHTO type VI girder in Figure 5. For spans up to 140 ft (42.7 m), the modified Bulb-T BT72/6 is the most economical and is, on average, about 3 percent cheaper than a modified Washington series 14/6 girder.

Modified Bulb-T's are compared with AASHTO sections in Figure 6. For spans from 80 to 120 ft (24.4 to 36.6 m), modified Bulb-T's yield savings of about 17 percent when compared with the AASHTO

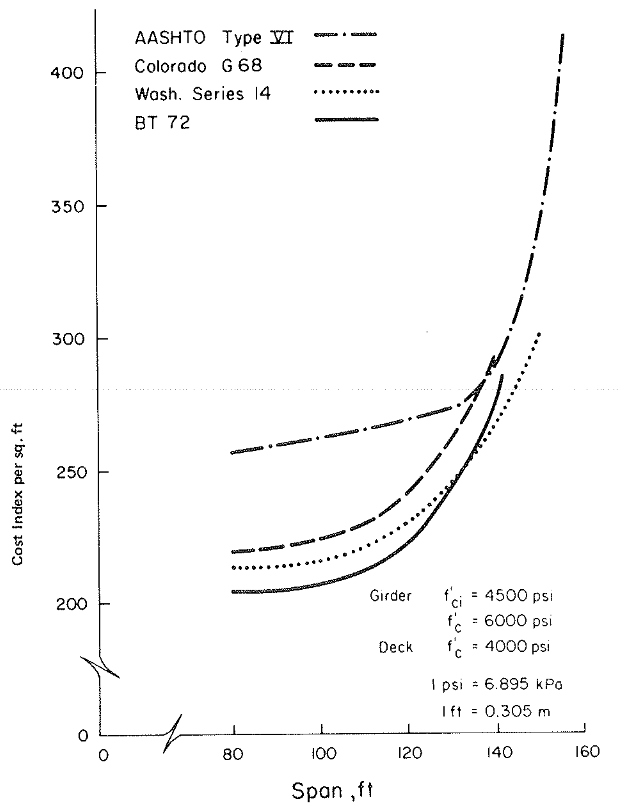

FIGURE 4 Comparison of optimum cost curves for AASHTO type VI, Colorado G68, Washington series 14, and Bulb-T BT72.

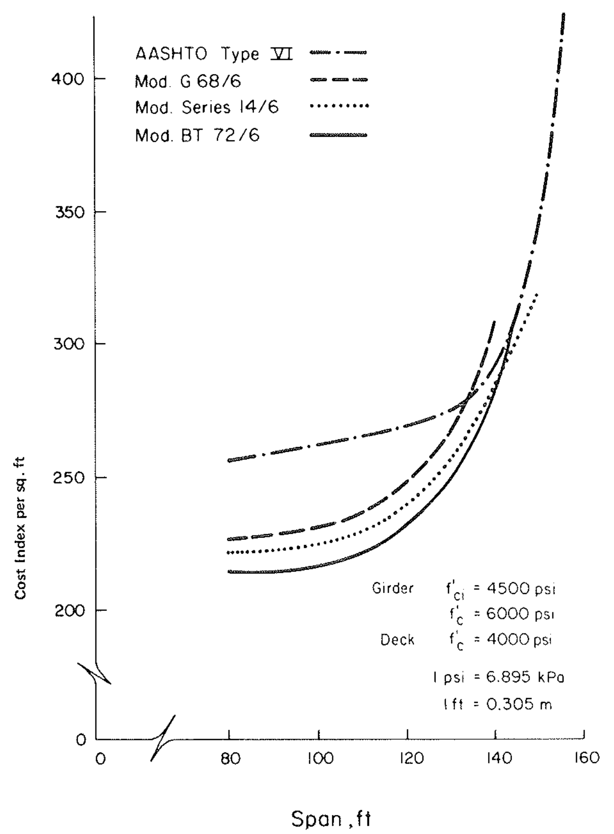

FIGURE 5 Comparison of optimum cost curves for AASHTO type VI and modified G68/6, series 14/6, and BT72/6 girders.

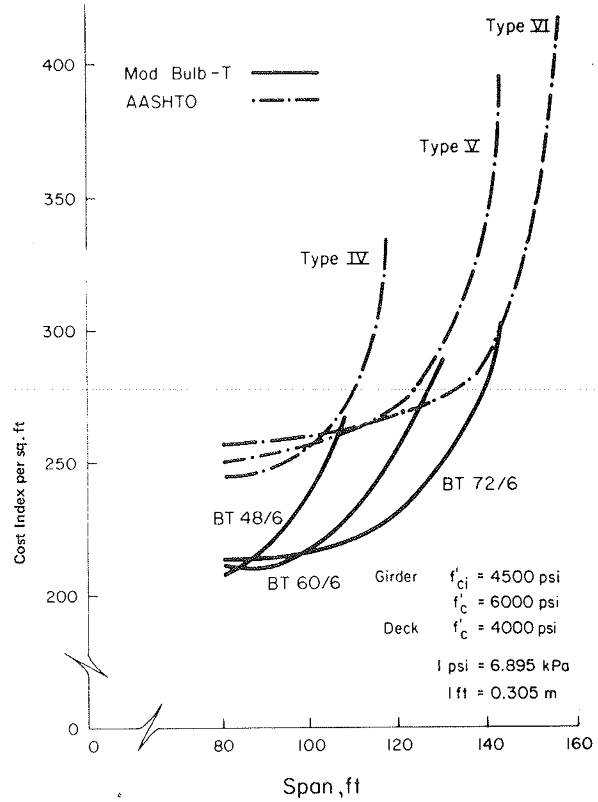

FIGURE 6 Comparison of optimum cost curves for modified Bulb-T's and AASHTO girders.

girders. For spans of 120 to 140 ft, cost savings vary from 17 down to 2 percent.

Web Thickness

Comparisons of Bulb-T, Washington series, and Colorado G68 sections with 5-in.-thick (127-mm) webs and similar sections with 6-in.-thick (152-mm) webs indicate that girders with 6-in.-thick webs cost 3 to 5 percent more than similar girders with 5-in.-thick webs. However, survey results from phase 1 indicate that sections with 6-in.-thick webs are easier to manufacture throughout the United States. All sections with 5-in.-thick webs have end blocks. Additional labor and material costs and girder weight resulting from the use of end blocks were ignored. However, if costs of end blocks are considered, differences in costs of sections with 5-in.-thick web versus sections with 6-in.-thick web would be somewhat less than indicated.

Comparisons between AASHTO sections and modified AASHTO sections with 6-in.-thick webs indicated that modified AASHTO sections yield cost savings of 6 percent when compared with AASHTO sections.

Effect of Concrete Strength

In all comparisons the concrete compressive strength of the girder was assumed to be 6,000 psi (41.4 MPa). Some girders were analyzed assuming 5,000 and 7,000 psi (34.5 and 48.3 MPa) concrete. The effect of concrete compressive strength on optimum cost curves is shown in Figure 7 for an AASHTO type VI girder. Comparisons indicate that by increasing the concrete compressive strength of the girder from 5,000 to 7,000 psi, maximum span capability of a section was increased by about 15 percent.

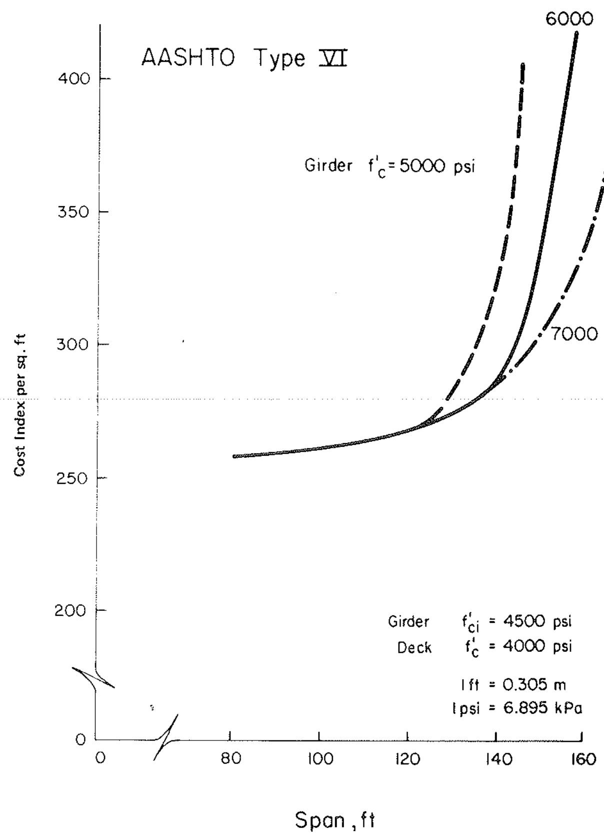

FIGURE 7 Variation of optimum cost curves with concrete compressive strength for AASHTO type VI girder.

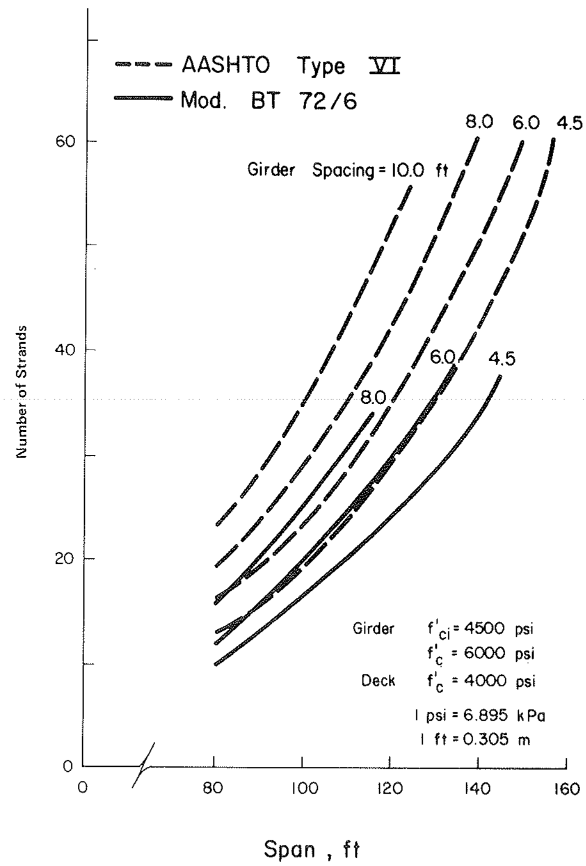

FIGURE 8 Required number of strands in AASHTO type VI girder and modified Bulb-T BT72/6.

Effect of Bundling Strands

In the comparisons given in the previous subsection strands were assumed to be spaced 2 in. (50.8 mm) on center at midspan. Strands were positioned as low as practical in the section to obtain maximum eccentricity of the prestressing force. Some girders were analyzed by assuming that strands were bundled at midspan. Comparisons indicated that overall cost savings and an increase in span capability from bundling of strands were negligible.

REQUIRED PRESTRESSING FORCE

Both modified Bulb-T BT72/6 and AASHTO type VI girders are 72 in. (1.83 m) deep. The number of seven-wire 0.5-in.-diameter (12.7-mm) stress-relieved strands needed in these girders is shown in Figure 8. Plotted are the number of required strands versus girder span for different girder spacings.

Because the modified Bulb-T BT72/6 is 35 percent lighter than the AASHTO type VI girder, it requires fewer strands. Therefore, the magnitude of the initial prestressing force is smaller. Consequently, existing prestressing abutments for AASHTO type VI girders would be adequate for prestressing the modified Bulb-T BT72/6.

The data in Figure 8 indicate that AASHTO type VI girders can be used at a girder spacing of 10 ft (3.05 m). Maximum girder spacing for the modified Bulb-T BT72/6 is approximately 8.5 ft (2.6 m) (2). For equal span, girder spacing, and truck loading, the modified Bulb-T BT72/6 requires 15 to 25 percent fewer strands than the AASHTO type VI girder.

SI CONVERSION

Recently, new International System of Units (SI), metric, sections were adopted in Canada under an arrangement agreed to by the prestressed-concrete producers. For an unspecified period of time, bridges in Canada will be designed by using the new metric sections, but alternate designs will be produced based on existing nonmetric sections. Because the new sections are more efficient than the existing ones, it was believed that the changeover would be accelerated by the competitive need to use the new sections. Some of the new metric sections are currently being produced in Ontario.

CONCLUSIONS

Based on the cost-analysis results discussed previously, the following conclusions have been drawn.

1. For girders with 5-in.-thick (127-mm) webs, the most cost-effective sections are Bulb-T's. For spans from 80 to 120 ft (24.4 to 36.6 m), Bulb-T's have 20 percent less in-place cost for girder and deck compared with AASHTO girders. For spans of 120 to 135 ft (36.6 to 41.2 m), the cost reduction for Bulb-T's varies from 20 to 5 percent. The next most cost-effective sections with 5-in.-thick webs are the Washington series.
2. In most regions of the United States it may not be easy to consolidate the concrete in girders with 5-in.-thick webs. Moreover, in these girders the strands must be bundled at midspan, and end blocks are needed to conform with minimum concrete cover requirements.

3. By using girders with 6-in.-thick (152-mm) webs, it will be possible to economically consolidate girder concrete in all regions of the United States. However, the use of girders with a 5-in.-thick web will be beneficial where experience has demonstrated the thickness to be satisfactory.

4. For girders with 6-in.-thick webs, the most cost-effective sections are modified Bulb-T's. For spans of 80 to 120 ft, modified Bulb-T's have 17 percent less in-place cost for girder and deck compared with AASHTO girders. For spans of 120 to 140 ft (36.6 to 42.7 m), the cost reduction varies from 17 to 2 percent.

5. Reduction of top and bottom flange widths and web thicknesses of AASHTO types IV, V, and VI girders by 2 in. (50.8 mm) reduces the overall inplace cost of girders and deck by about 6 percent. Span capability of the modified sections is not affected by these changes in width.

6. The overall in-place cost of girders and deck is decreased substantially by placing girders at the largest practical girder spacing.

7. An increase in the concrete compressive strength of the girder from 5,000 to 7,000 psi (34.5 to 48.3 MPa) increases the span capability of AASHTO girders by about 15 percent.

8. Bundling of strands at midspan to increase eccentricity of prestress does not lead to any significant overall cost reduction for the girders considered.

9. For equal span and girder spacing, modified Bulb-T's require up to 25 percent less prestressing force than the AASHTO girders.

RECOMMENDATIONS

Based on the conclusions in the previous section, the following actions are recommended:

1. Modified Bulb-T girders with 6-in.-thick (152-mm) webs are recommended for use as national standard precast, prestressed-concrete bridge girders in the United States for spans from 80 to 140 ft (24.4 to 42.7 m);
2. Girder spacing should be as large as possible; and
3. If metrication is adopted in the United States, modification of the previous sections to SI units should be considered as part of any standardization.

IMPLEMENTATION

Construction of the Interstate highway system has been completed in some states. In most states it is close to completion. Therefore, the rate of bridge construction on the Interstate system is much slower than it was from the late 1950s to the early 1970s. Nevertheless, according to statistics prepared by the Bridge Division of the FHWA, considerable new bridge construction and major reconstruction is ongoing.

The cost of new prestressed-concrete bridge construction and bridge rehabilitation with participation of federal funds authorized during calendar year 1982 totaled $767 million. Based on bridge inventory and inspection records, it is anticipated that "in the next 20 to 30 years, we will have over $30 billion worth of bridge construction based on the value of the dollar today" (7). Revenue from the recently legislated $0.05 tax on gasoline has increased the funds allocated for bridge construction by about 25 percent.

As previously mentioned, selection of bridge type is based on economy. Safety standards for Interstate and other high-speed highways require greater clearances. Therefore, there is need for construction of bridges with spans of 110 to 130 ft (33.5 to 39.6 m). In all states surveyed, except California, the most economical bridges for spans of approximately 70 to 130 ft (21.3 to 39.6 m) are constructed with pretensioned bridge girders.

Cost analyses indicate that modified Bulb-T's can yield savings of 17 percent on the overall cost of girder and deck compared with AASHTO girders. Also, the modified Bulb-T's are about 35 percent lighter than AASHTO girders for comparable spans. A 140-ft (42.7-m) AASHTO type VI girder is extremely heavy and therefore difficult to transport on highways. Lighter sections with 140-ft spans have been transported on highways with no difficulty.

Although steel forms constitute a capital investment, their life span is limited to about 10 years. Where new forms are needed, new plants built, or improved sections sought, optimized sections should be considered. Capacity of existing stressing beds or abutments will be adequate for the modified Bulb-T's, as these sections require up to 25 percent less prestressing force than the AASHTO girders.

The implementation of new sections should be gradual over a period of time. It will require effort on the part of both departments of transportation and producers. Preparation of design aids for the new sections will encourage and facilitate implementation of the new sections. A sample preliminary design chart is shown in Figure 8, where design curves have been superimposed for the modified Bulb-T BT72/6 and the AASHTO type VI girder.

Highway agencies should be informed of the economic benefits that can be achieved with optimized sections. Departments of transportation will have to design with old and new sections over a transition period. The Canadian experience in switching to new metric sections sets an example of implementation of new sections under an arrangement agreeable to producers and highway agencies.

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Proposed Limit State Strength Evaluation of Existing Reinforced-Concrete Bridges

ROY A. IMBSEN and ROBERT A. SCHAMBER

ABSTRACT

Because of several catastrophic bridge failures, bridge safety has been emphasized during the past decade. As a result there has been a concerted effort to develop and disseminate procedures for systematic bridge inspection and rating. Although bridges with concrete superstructures rarely fail catastrophically, gradual deterioration and increased loads can affect their structural capacity. Existing procedures for inspecting and rating bridges with concrete superstructures are limited. A summary of a methodology proposed for rating reinforced-concrete bridges is presented. The methodology was developed in the first phase of an NCHRP project to improve strength evaluations of existing reinforced-concrete bridges. The methodology is presented in a limit-states format by using approximate load and resistance factors. By using this format a basis is provided on which probability theory and engineering judgment can be rationally combined to allow for independent consideration of each of the major variables that can affect the determination of the load capacity of a bridge. This methodology includes consideration of the level of effort in maintenance and inspection, the degree of load-limit enforcement, the quality of construction, the refinement used in simulating the bridge, the effects of deterioration on the load-carrying capacity, and the degree of refinement in determining the load-distribution factors.

Currently, the procedure for evaluating reinforced-concrete bridges in the United States is based on AASHTO guidelines published in the Manual for Maintenance Inspection of Bridges (1). Experience has demonstrated that the structural capacity of reinforced-concrete bridges usually exceeds the capacity calculated by the conventional techniques presented in this manual.

Many engineers recognize the built-in conservatism in the current approach to the evaluation of bridge strength. Factors that tend to cause the capacity of reinforced-concrete bridges to be underestimated include

1. Material strengths that exceed nominal values used for evaluation,
2. Conservative assumptions used in calculating structural resistance (i.e., zero tension in concrete),
3. Interaction of structural components in resisting and distributing the loads,
4. Structural redundancies, and
5. Overestimation of the loads.

INTRODUCTION

To make improvements in the bridge evaluation process that will lead to more realistic evaluations while still preserving public safety requires a rational consideration of each of the five factors previously mentioned. One method for making such improvements is through a limit-states approach based on probabilistic concepts. This approach was used in the recently developed Clause 12 of the Canadian Standards Association Bridge Code (2-4).

The proposed methodology for evaluating existing bridges incorporates such a limit-states approach. Although the methodology represents a significant change in the current philosophy, from the user's