STRATEGIES FOR REDUCING BRIDGE COSTS THROUGH THE USE OF HIGH-STRENGTH CONCRETE

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ABSTRACT

The use of high-strength concrete (HSC) for bridges has not gained the wide acceptance it has received in the building industry primarily due to the misconception that the benefits of the material do not justify its higher cost and the increased quality control requirements associated with its production. This paper is intended to help provide a clear economic incentive for precast concrete designers and producers to utilize HSC widely. Presented are some of the results of a study that has been recently concluded at the University of Calgary to assess the potential economic benefits from the utilization of concretes with compressive strengths of up to about 100 MPa for the design and construction of continuous, precast, prestressed I-girder bridges. It is shown that, as a result of the increase in girder capacity, the increase in the concrete cost associated with the use of HSC can be fully offset by the reduced number of girders required.

INTRODUCTION

For several years, the most common application of high-strength concrete (HSC) has been in the building industry where concrete strengths of 100 MPa and more have been frequently used in the lower columns of high-rise buildings. In comparison, strengths of about 60 MPa have been considered the maximum achievable in the precast, prestressed concrete industry [1]; this appears to be very timid especially considering that several research studies have indicated that the potential advantages from the utilization of HSC with its increased strength and improved durability for precast, prestressed highway bridges are quite promising [2,3]. In spite of this, there is still some disagreement within the precast industry that HSC is beneficial. The misconception that the benefits of the material do not justify its higher cost and the increased quality control requirements associated with its production seems to deter most designers and precast concrete producers from exploiting it. The use of HSC in bridges is unlikely to advance quickly without a clear economic incentive for precasters to utilize the material widely. This paper, which is based on the Ph.D. work of the first author [4], is intended to help provide such an incentive by developing some strategies for reducing the cost of precast, pretensioned I-girder bridges through the use of concretes with compressive strengths of up to about 100 MPa. These were chosen because they represent the most common type of prestressed concrete bridges constructed in North America [5]. The problem was formulated as an optimal design problem. Refined computer-oriented structural analysis methods combined with modern computational design optimization techniques were used to develop an optimization system that was utilized to perform economic studies on the type of bridges investigated.

RESEARCH APPROACH

Assumptions

The following assumptions were made in developing the optimization system:

1. Design conforms to the Ontario Highway Bridge Design Code (OHBDC) provisions [6].
2. The new Canadian Highway Bridge Design Code live load specifications [7].
3. Tendons are draped at the third-points of the girder span.
4. Unshored, composite construction.
5. 75 mm-thick future wearing surface will be placed on top of the concrete deck slab.
6. Each traffic barrier has a cross-sectional area of 0.3 m^2. This load is distributed equally among the girders.
7. Final effective prestressing force after losses is 0.80 of the prestressing force at transfer.
8. Concrete compressive strength at transfer is 0.70 of the 28-day strength.
9. 7-wire, low-relaxation, 15.2-mm strands having a tensile strength of 1860 MPa are used.
10. Unit cost estimates for materials, labour, etc., and for girder transportation and erection charges are as described by Hassanain [4].

Structural System and Method of Analysis

Bridges built of cast-in-place, reinforced concrete deck slabs on precast, pretensioned concrete I-girders were used for this investigation. These are commonly known as slab-on-girder bridges (see Fig. 1). The structural system selected has a configuration typical or representative of most existing bridges of this type. It consists of two continuous spans of equal lengths, and has three traffic lanes with an overall width of 12 m. Continuity is achieved by adding longitudinal reinforcing steel bars in the deck slab over the bridge piers. Compressive strength of the slab concrete was fixed at 35 MPa.

The OHBDC permits a simplified method of analysis for determining the live load longitudinal moments in continuous-span girder bridges. Recently, Jaeger *et al.*[8] have reported that this method is valid for the positive moment regions, but is inaccurate for negative moments. Because of the uncertainties that could be associated with this method, the semicontinuum method [9], which is approved by the OHBDC as a refined method of analysis, was used for live load analysis. In this method, a slab-on-girder bridge is represented by discrete longitudinal members and a continuous transverse medium. This idealization is a closer representation of this type of bridge.

Formulation of the Optimal Design Problem

General C Design of prestressed concrete girder bridges on a trial-and-error basis is commonly used. However, it is a tedious and time consuming process. Mathematical optimization techniques provide systematic procedures by which optimal designs can be obtained with substantially less time and effort. The formulation of an optimal design problem requires identification of design variables for the structural system, an objective function that needs to be minimized, and design constraints that must be imposed on the system. In prestressed concrete design optimization, both the objective function as well as most of the constraints are nonlinear functions of the design variables; thus, requiring nonlinear programming procedures to be used. Many numerical methods have been developed to solve nonlinear programming problems. The methods start from an initial design provided by the user which is iteratively improved until the optimum is reached. Several of these methods have been incorporated into general-purpose design optimization software packages. One such package is IDESIGN (*I*nteractive *DESIGN* Optimization of Engineering Systems) [10]. IDESIGN was used in this study to solve the nonlinear programming problem. The optimization process consists of cycling between two distinct phases defined as analysis and optimal design in an iterative fashion until the optimum is reached.

Design Variables C For standard CPCI precast I-girders, cross-sectional dimensions were known and became preassigned parameters instead of design variables. Composite action of the precast girder and cast-in-place deck slab was assumed, and thus the slab thickness was taken as a design variable. Other design variables included the required amounts of prestressing and non-prestressing flexural reinforcements in the girders and slab, and the tendon profile defined by the eccentricities at girder mid-span and over piers. Moreover, the girder concrete compressive strength at 28 days was also taken as a design variable.

Objective Function C The relevant objective function in the design of bridges with a fixed number of spans is the minimum superstructure cost / deck area. This assumes that the cost of piers and abutments is relatively unaffected by changes in the number of girders. The objective function was taken as the material (concrete and steel) costs plus overhead and waste, in addition to the labour, transportation and erection costs. Slab formwork cost may vary slightly with the change in the number of girders. For example, as the number of girders decreases, the actual slab formwork area increases; however, the labour cost of placing this formwork decreases in a way that may partially or fully compensate for the increase in material cost. Therefore, it was decided to exclude the formwork cost from the objective function. Furthermore, costs of some other items that are relevant to the superstructure such as wearing surfaces, barriers, drains and diaphragms were not included in the objective function because their costs are not affected by the use of HSC. The effect of girder section depth was not directly included in this study although it can be very important. Increased depth will often increase costs because additional earthwork may be needed to increase the embankment height or excavation depth. When clearance under a bridge is not a factor, increased girder depth may actually reduce costs marginally because of the resulting lower pier heights.

Design Constraints C The objective function was minimized under all relevant constraints according to the OHBDC. These included the flexural constraints at transfer, during construction, at serviceability limit states and at ultimate limit states, in addition to the practical constraints. Other constraints (e.g. shear and deflections) could be added. However, in general, they have marginal effect on the design and were left to be checked at the final design stage. It should be mentioned here that the OHBDC does not have explicit criteria for limiting deflections in prestressed concrete bridges. More details can be found elsewhere [4].

ECONOMIC STUDIES

Investigation Layout

To assess the potential economic advantages from the use of HSC for precast, prestressed concrete slab-on-girder bridges, the developed optimization system was used to generate 115 optimal bridge superstructure designs. There were three main parameters that were varied throughout the generated designs due to their potentially significant impact on any economic analysis. These parameters were girder cross section, number of girder lines (referred to here, for simplicity, as just number of girders) or, alternatively, transverse girder spacing, and girder span length.

Five standard CPCI girder cross sections were used for this study. These were CPCI Type 1200, 1400, 1600, 1900 and 2300 [11]. They are commonly used in Canada for highway bridges with span lengths of up to about 50 m. It is believed that the girder types utilized for this investigation constitute an adequate range over which it would be possible to generalize the conclusions obtained. Transverse spacings of the girders were varied as 3.0, 4.0 and 6.0 m. These spacings were chosen because they optimized the design of the exterior and interior girders when 4, 3 and 2 girders were used, respectively. Design was optimized by increasing the girder spacing until the service load stresses in the exterior girders are nearly equal to those in the interior ones. The minimum number of girders that may be used for a bridge depends to a large extent on design philosophy. In some jurisdictions, such as Nebraska, at least 4 girders are required in order to allow traffic to be diverted on to half of the bridge in case of future deck replacement on the other half [12]. In other situations, a minimum of 3 girders is considered necessary for highway overpasses. This limit is chosen to prevent catastrophic collapse in the event that one girder is severely damaged by impact from a high vehicle. In order to use 3 girders without increasing the deck slab thickness, the girder spacing to slab thickness ratio of 15 stipulated by the OHBDC was exceeded in this study; however, this ratio was less than the limit of 18 specified in the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications [13]. In this case, the slab overhang length of 2.0 m was marginally greater than the limit of 1.8 m in the OHBDC. It should be noted that in this study, the optimal deck thickness was found to always correspond to the 225-mm minimum thickness specified in the OHBDC. For stream crossings, where impact from vehicles is not a consideration, 2 girders may be feasible. This is generally the optimum when construction costs represent a very high percentage of the total bridge superstructure cost. However, it should be recognized that traffic will have to be diverted during future bridge deck replacement. The present limits on deck span and slab overhang length in both the OHBDC and the AASHTO LRFD specifications are exceeded if only 2 girders are used. However, box-girder bridges have routinely been built with such proportions without any problems. The last major parameter varied throughout the generated optimal designs was girder span length. Spans were varied over a range from 20 to 52 m. The incremental increase for the span length was taken as 2 m for all cases. This small increment allowed accurate trends to be obtained and made it easier to identify maximum span lengths within an acceptable range.

Interpretation and Discussion of Results

Due to space limitations, only the results of the 3-girder bridge are reported in this paper. Further results are presented in Ref. [4]. Table 1 contains the optimal girder concrete strength, f_c', along with the value of the objective (cost) function for the different combinations of span length and girder cross section. It can be noted that the optimal values of f_c' are distributed in unequal intervals for the equal increments of span length used. It is desirable to have equal increments of f_c' because it would make it possible to interpret the results more readily. An increment of 10 MPa is reasonable within the range of strengths considered. The approach used to equalize the increments of f_c' was to plot the data contained in the table as shown in Figs. 2 and 3, and then to interpolate on the graphs to find the corresponding values of span length and cost for every 10-MPa increment of concrete strength. The resulting data is plotted in Fig. 4. This figure illustrates the variation in the minimum superstructure cost with span length for the various girder sections in the 3-girder bridge. Girder concrete strength associated with the first point on each curve is shown in the figure. Strengths corresponding to subsequent points on each curve are increased in 10-MPa increments. Based on these curves, it is possible to identify the most cost effective designs for various ranges of girder span length, assuming there was no practical limit on the level of concrete strength that can be achieved.

Current production capabilities in some precasting plants might preclude producing concretes of very high strength. In this case, Fig. 4 along with similar figures plotted for the 2-girder bridge and the 4-girder bridge can be used to establish minimum cost curves for certain maximum concrete strengths. These curves can be used for design purposes when f_c' is specified. Two such curves are plotted in Fig. 5 for maximum available f_c' of 60 MPa and 80 MPa. The explanation of how these curves were plotted can be found in Ref. [4]. Fig. 5 reveals that, in general, as a result of the increase in girder capacity, the increased concrete mix constituent and quality control costs associated with the use of concretes of higher strengths can be fully offset by the reduced number of girders required for a given span length. The economic advantage of using HSC is quite apparent for the longer span lengths. For example, it can be seen that a designer would have to utilize 4 CPCI Type 2300 girders made with 60-MPa concrete to span 44 m at a total superstructure cost of $209/m^2. If 80-MPa concrete was used to manufacture the girders, then only 3 of them would be needed resulting in a total superstructure cost of $176/m^2. This would translate into a saving of about $33/m^2. For this two-span bridge, the cost savings associated with the use of the higher-strength concrete would have been close to $35,000 or about 16% of the total superstructure cost. Fig. 5 reveals also that the use of a shallower girder cross section manufactured with a higher-strength concrete can be more economical than the use of a deeper section made with a lower-strength concrete. For example, for the range of spans from 27.8 m up to about 32 m, it is more economical to use 2 Type 1900 girders made with 80-MPa concrete than it is to use 2 Type 2300 girders made with 60-MPa concrete. While the saving in the superstructure cost associated with the use of the shallower section, in this case, did not exceed 3% within the range of spans considered, it is important to keep in mind that there may be other savings from the reduced substructure height.

Fig. 6 shows the minimum cost curves for maximum available girder concrete strengths of 40 MPa, 60 MPa, 80 MPa and 100 MPa. It should be noted that these are Aidealized@ curves in the sense that they do not show the girder type as was the case in Fig. 5. From Fig. 6, it can be appreciated that both the optimal number of girders and the girder concrete strength vary markedly with girder span length. In general, the optimum is reached with 2 girders if a sufficiently high concrete strength can be achieved. This requires a girder spacing of 6.0 m and a slab overhang length of 3.0 m. As mentioned earlier, this spacing and overhang length exceed the limits in current bridge design codes [6,13]. Thus, the use of 2 girders is contingent on further analytical and experimental studies which would verify satisfactory behaviour of such a configuration. The optimal concrete strengths for different girder span length limits are summarized in Tables 2, 3 and 4 for the 2-girder bridge, 3-girder bridge and 4-girder bridge, respectively. If the spacing and slab overhang length limits can be extended to allow the use of 2 girders, it seems clear that strengths greater than 100 MPa are likely to be worth exploring for spans in excess of 42 m, as suggested by Table 2. Higher-strength concretes, however, are unlikely to be advantageous for shorter girders unless shallower depths result in significant savings in earthwork costs. From Fig. 6 and Table 4, it can be noted that for the bridge with 4 girders, there seems to be little advantage to the use of 100-MPa concrete for spans less than 52 m which was the limit on span lengths used for this study.

CONCLUSIONS

Based on the results of the study reported in this paper, it was possible to develop the following strategies for reducing slab-on-girder bridge costs through the use of HSC:

1. For a given span length, it is more economical to use fewer girders made with HSC than it is to use a larger number of them manufactured with normal-strength concrete. This implies that it is most economical to place girders at the largest practical transverse spacing. This economic advantage of using HSC becomes more apparent for longer span lengths.
2. The use of a shallower girder cross section manufactured with a higher-strength concrete to span a given length can be more economical than the use of a deeper section made with a lower-strength concrete.
3. If 4 girders are required for a bridge, concrete strengths ranging from 40 to 80 MPa will be the optimum depending on the girder span length. A girder strength greater than 100 MPa is unlikely to be advantageous unless the span is longer than 52 m or shallower cross sections are desired.
4. If 3 girders are allowed for a bridge, concrete strengths up to 100 MPa will be the optimum, and a lower superstructure cost will be achieved as compared to the same span length with 4 girders. To enable the use of 3 girders, a girder spacing to slab thickness ratio of about 18, as allowed in AASHTO LRFD specifications, is necessary along with a permissible slab overhang length of 2.0 m.

5. For span lengths up to 42 m, 2 girders will result in the lowest superstructure construction cost. The optimal concrete strength varies from 40 to 100 MPa. For situations where the 2-girder design option is considered appropriate, it would be worthwhile to determine whether the deck will function satisfactorily with a transverse girder spacing of 6.0 m. For bridges of the width used for this study, and a slab thickness of 225 mm, this requires verification that a girder spacing to slab thickness ratio of about 27 will be satisfactory. A slab overhang length of 3.0 m is also needed for this configuration.

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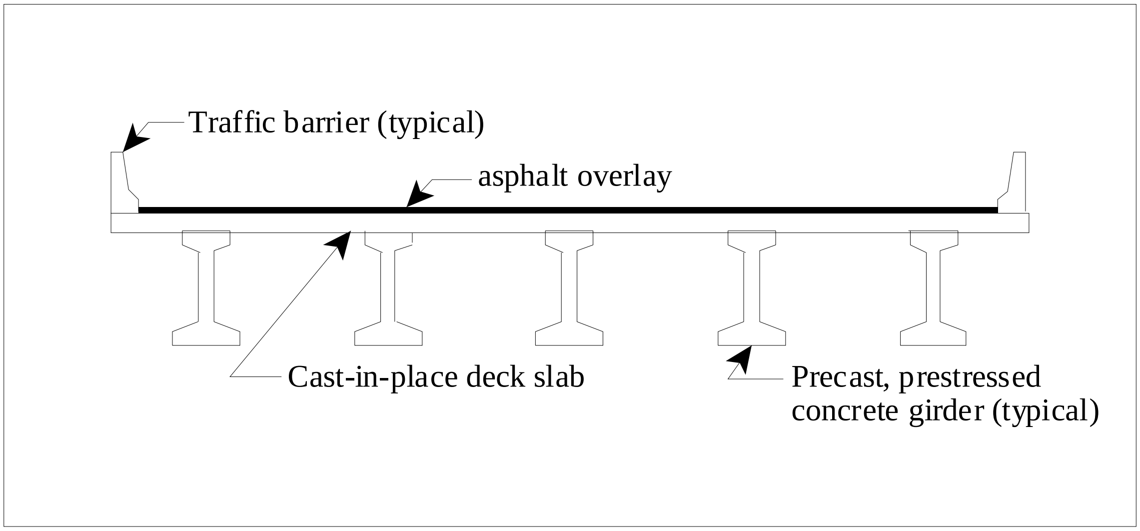

Fig. 1 Cross section of a typical slab-on-girder bridge

Table 1 Results of the optimal design solutions for the 3-girder bridge

Span, m	CPCI 1200		CPCI 1400		CPCI 1600		CPCI 1900		CPCI 2300	
	f_c', MPa	Cost, $/m^2	f_c', MPa	Cost, $/m^2	f_c', MPa	Cost, $/m^2	f_c', MPa	Cost, $/m^2	f_c', MPa	Cost, $/m^2
20	54.2	97								
22	66.7	100	44.3	105						
24	79.8	103	53.3	108						
26	93.4	108	62.0	111	46.5	118				
28	108.1	113	71.7	115	53.7	121				
30	123.5	121	81.9	119	61.3	125	48.0	127		
32			92.6	124	69.3	129	54.2	130	41.3	136
34			103.9	131	77.8	133	60.8	134	46.3	140
36			115.8	140	86.6	140	67.7	140	51.5	145
38			128.2	152	95.9	149	74.9	148	57.0	152
40					105.7	160	82.5	156	62.7	159
42					115.8	171	90.4	165	68.7	167
44							98.6	175	75.0	176
46							107.2	186	81.5	184
48									88.2	194
50									95.3	204
52									102.5	214

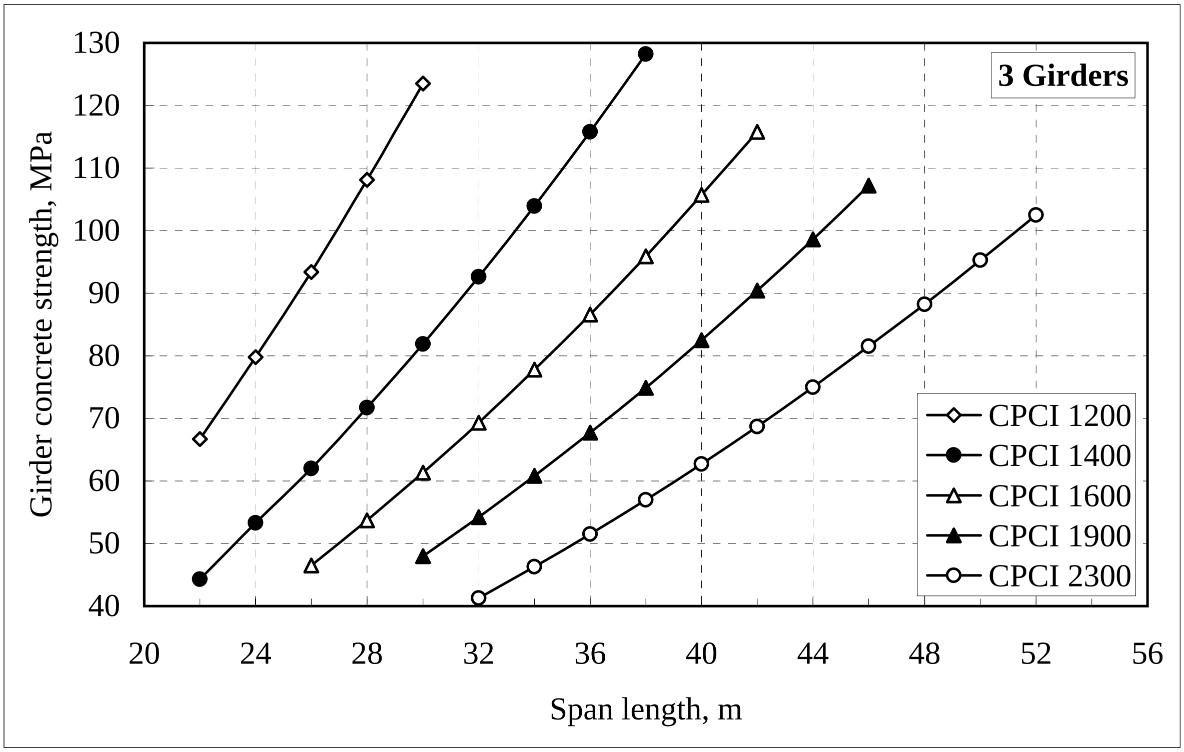

Fig. 2 Girder concrete strength vs. span length for the 3-girder bridge

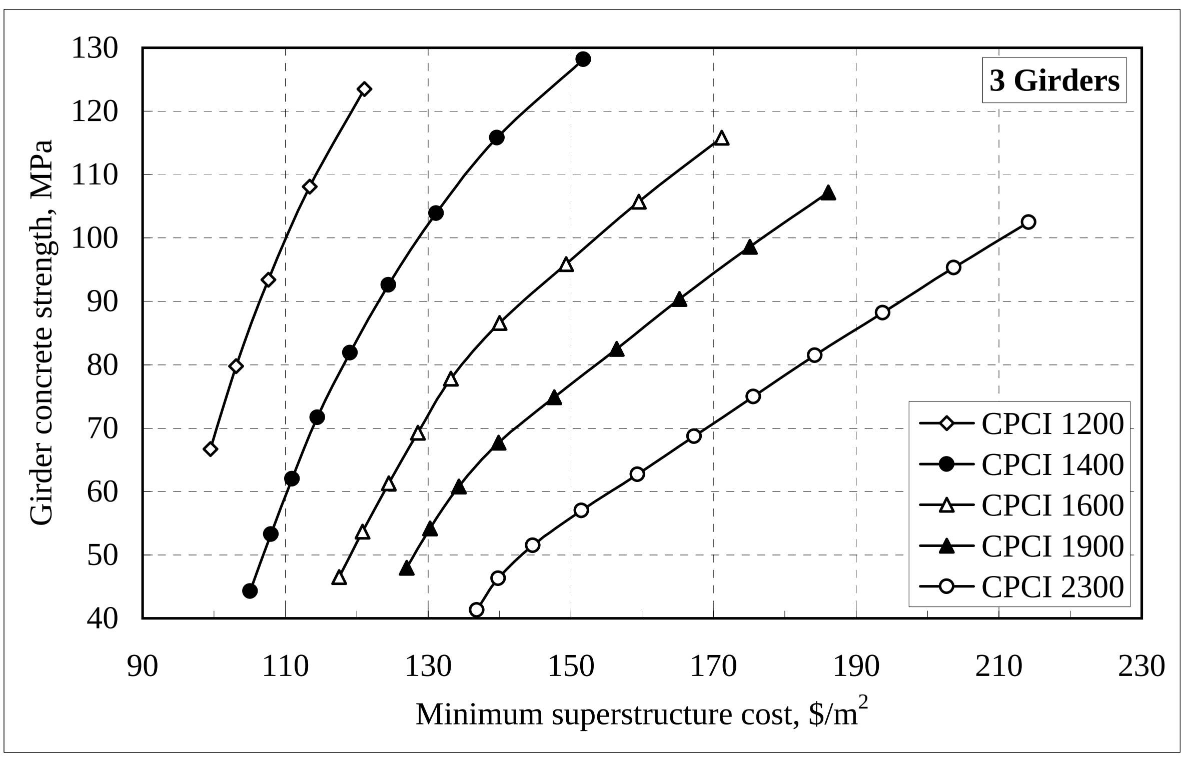

Fig. 3 Girder concrete strength vs. minimum superstructure cost for the 3-girder bridge

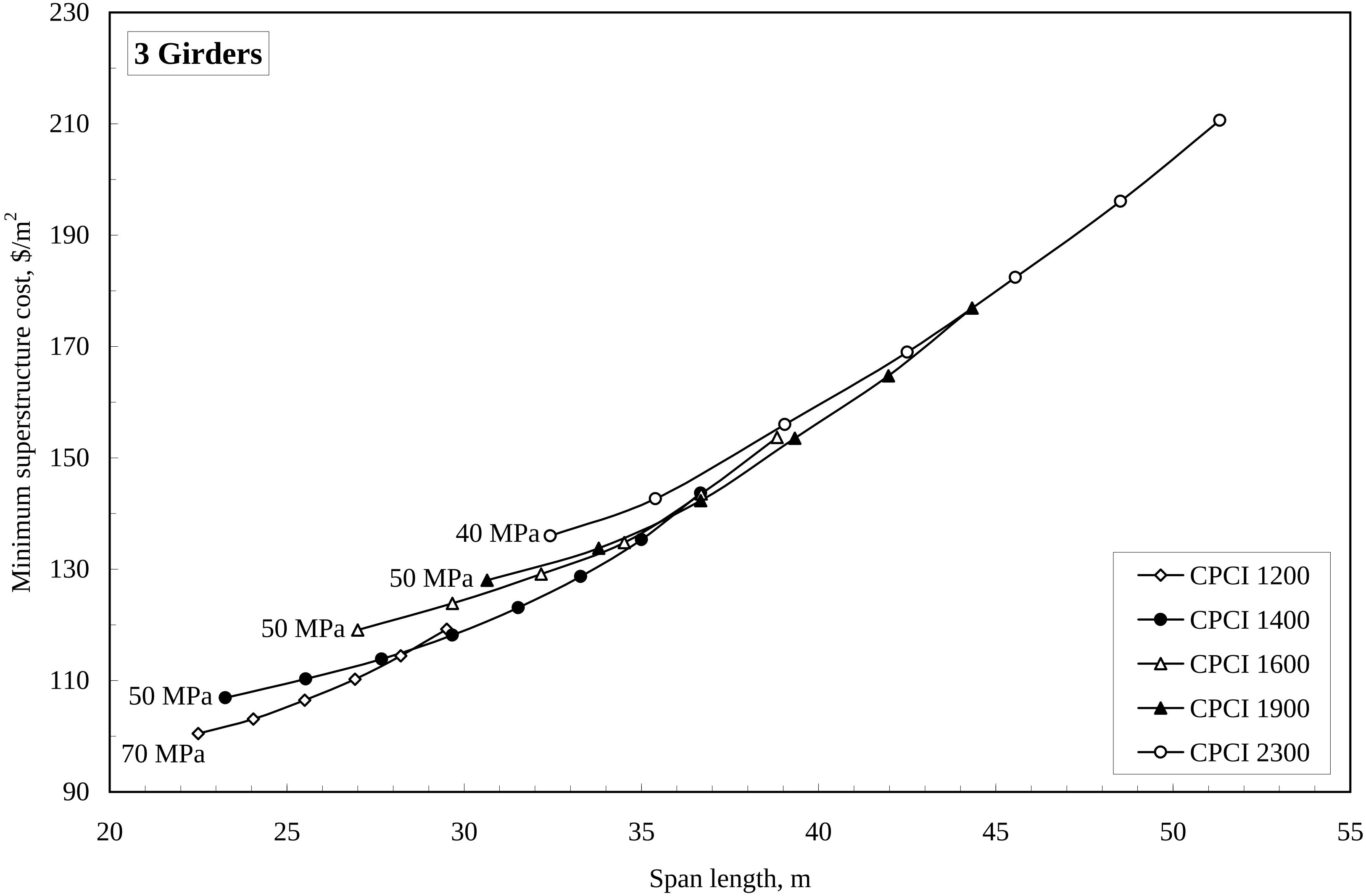

Fig. 4 Minimum superstructure cost vs. span length for the 3-girder bridge

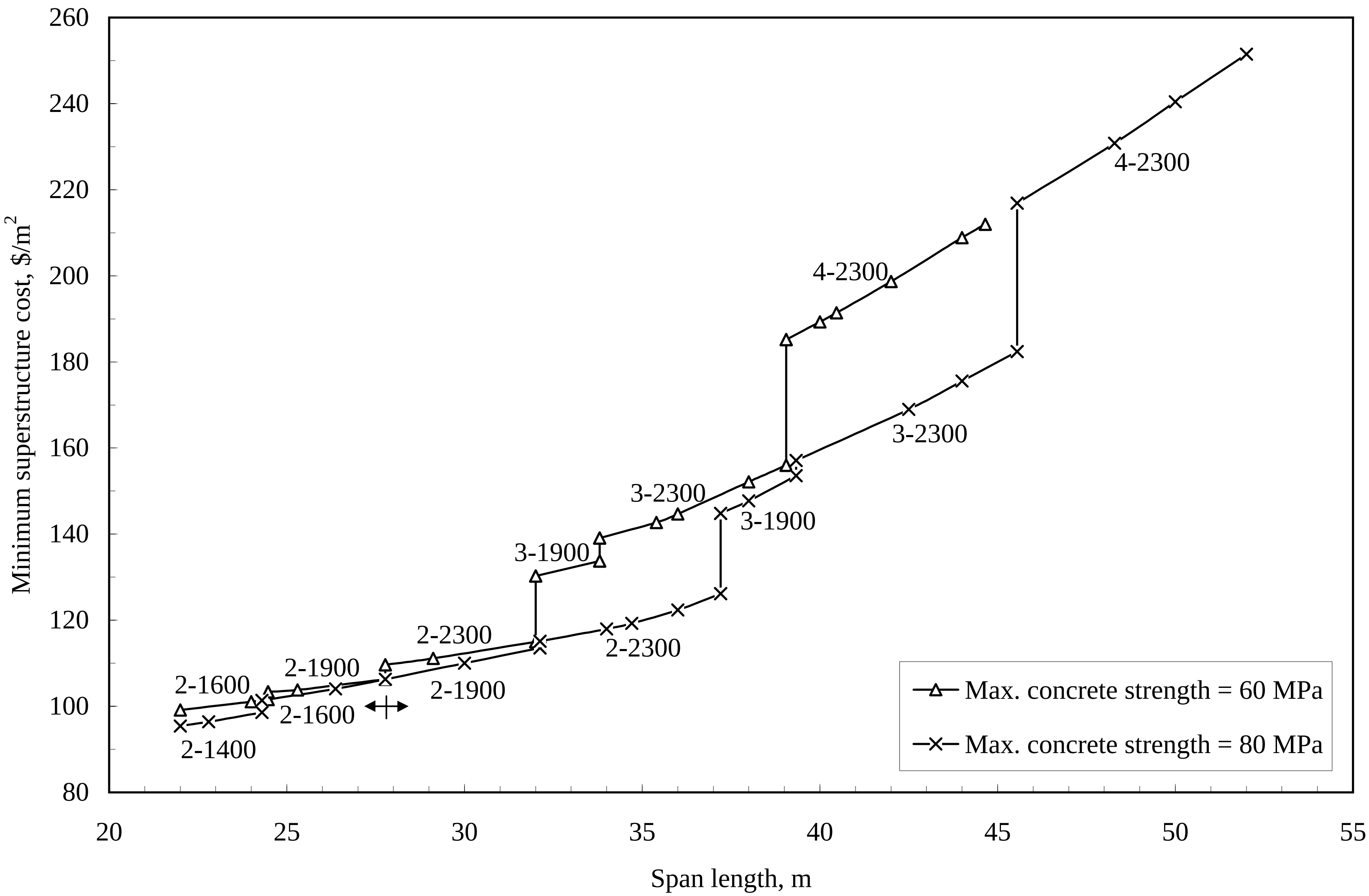

Fig. 5 Cost curves for maximum girder concrete strengths of 60 MPa and 80 MPa

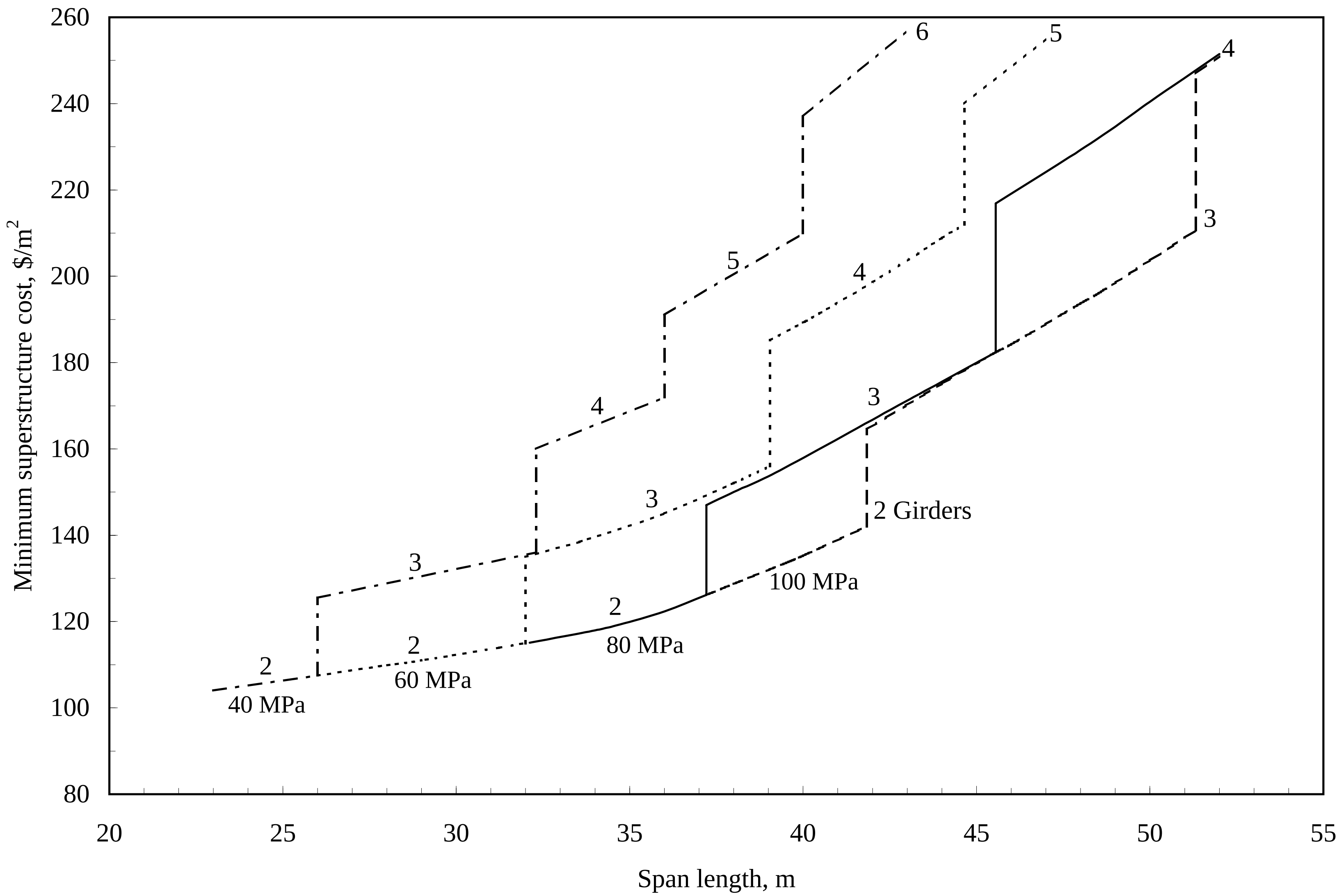

Fig. 6 Cost of optimal solutions with different span lengths and limiting concrete strengths

Table 2 Optimal concrete strengths for the 2-girder bridge

Span limit	Optimal f_c'
to 26 m	40 MPa
to 32 m	60 MPa
to 37 m	80 MPa
to 42 m	100 MPa

Table 3 Optimal concrete strengths for the 3-girder bridge

Span limit	Optimal f_c'
to 32 m	40 MPa
to 39 m	60 MPa
to 46 m	80 MPa
to 51 m	100 MPa

Table 4 Optimal concrete strengths for the 4-girder bridge

Span limit	Optimal f_c'
to 36 m	40 MPa
to 45 m	60 MPa
to 52 m	80 MPa