

LIFE-CYCLE COST OPTIMIZATION OF PRESTRESSED SIMPLE-SPAN CONCRETE BRIDGES WITH SIMPLE AND SPLICED GIRDERS*

M. MADHKHAN^{1**}, A. KIANPOUR² AND M. E. TORKI HARCHEGANI³

^{1,2}Dept. of Civil Engineering, Isfahan University of Technology (I.U.T.), Postal code 8415683111, I. R. of Iran
Email: madhkhan@cc.iut.ac.ir

³ Dept. of Civil Engineering, Sharif University of Technology, Postal code 113658639, I. R. of Iran

Abstract– In this paper, determination of the design variable optimum values of simple and spliced pre-tensioned girders in bridges is addressed by considering the life cycle cost of the whole structure. To this end, a program called OBPG is written with FORTRAN, consisting of the analysis, design and optimization subroutines based on the feasible direction method. AASHTO principles are used, assuming initial values for the design variables such as the dimensions of the girders and the deck, and the strands number. Then, using the analysis outcomes and considering the design criteria, the total cost of the structure, including that of the girders and slabs concreting, pre-tensioning cables performance, reinforcement and frameworks, are minimized as functions of the design variables. Although the initial optimization cost using life cycle cost is slightly more than that using the initial cost, optimization of the life cycle cost proves to be more economical. Ultimately, a program capable of analyzing simple and spliced pre-tensioned bridge girders is developed which calculates the life cycle and gives an optimum design.

Keywords– Optimization, feasible-directions method, pre-tensioned girders, post-tensioned girders, life-cycle cost

1. INTRODUCTION

Among various criteria influencing designs, the overall cost is usually the most consequential because of the growing consideration of crude materials as well as the shortage of energy-resources. Numerical methods have recently been put to much work for optimizing and designing structures automatically. Among the pioneering researchers in this field are Frangopol, Haftka, Gurdal, and Vanderplaats [1, 2]. Gobel and Lapay [3] put forward the use of non-linear optimization methods in reinforced concrete structures and prestressed concrete bridge decks for the first time.

Many researchers have worked to develop optimization algorithms for the optimal design of the bridge initial costs, including Farkas [4], who offered a theoretical model for the lifetime cost of steel bridges considering long-term effects such as strength reduction with time. Lee et al. [5, 6] suggested a practical realistic method for designing steel bridges optimally taking into account the life-cycle cost by considering environmental stimulants such as corrosion and hefty traffic. They used the initial cost, the life-time maintenance cost, rehabilitation costs, the costs imposed by consumers, and the indirect socio-economical detriments as the life-time cost functions. An optimized structural design was presented for prestressed concrete railway sleepers by Sadeghi and Babaei [7] based on 5 design comparators. They evaluated the number and place of prestressing rebars and reached an optimum model minimizing the weight of concrete and tendons, and thus the whole construction cost. The simulation-based life-cycle cost

*Received by the editors December 29, 2010; Accepted December 1, 2012.

**Corresponding author

optimization of FRP bridge-deck panels under different discount factors (interest rates) was studied by Hong et al. [8]. They introduced an analytical model exerting the Monte Carlo simulation to estimate the service life and optimal life-cycle cost of deck panels. It was deduced that preventive maintenance is a very productive way to maintain not only conventional, but also FRP bridge deck panels. The application of a modified genetic algorithm as an optimization method for minimizing the production costs of reinforced concrete slabs with precast prestressed concrete joists was proposed by De Castilho et al. [9]. They identified, described, and then combined the multiple costs involved in the production of these slabs and introduced an objective function under 28 equality and inequality constraints. Their method was strongly dependent upon the initial value offered by the user. Thus, when there is little information about the range of solutions, this optimization method will be ineffective. The economic optimization of prestressed beam sections in pedestrian-traffic bridges was investigated with a heuristic method by Marti and G-Vidosa [10]. They used the simulated annealing and the threshold accepting algorithms and solved the problem for bridge beams with span lengths between 20 and 40 *m* and widths between 3 and 6 *m*. The constraints included flexure, shear, and deformations. The optimum economic design of multi-span earthquake-resistant reinforced concrete bridges was studied by Mitoulis et al. [11] with the scope of evaluating the cost range of different bridge structural systems designed in terms of low and high seismic actions. They considered four structural systems: an isolated bridge whose deck is supported on all piers and the abutments are supported on rubber bearings, a semi-integral, a quasi-integral, and a fully integral bridge with tall abutments rigidly connected to the deck. With respect to structural and serviceability costs, the semi-integral system was the most cost-effective. An optimum design for slab formworks was proposed by Kaveh and Shakouri [12] by using a numerical harmony search algorithm simulating a musical harmony searching procedure. They provided optimum designs for the cross section and spacing of joists and stringers while minimizing the total cost. The analysis and design of reinforced concrete tall bridge piers with hollow rectangular cross sections was proposed using the ant colony optimization by Martinez et al. [13]. Three different hollow rectangular cross sections were considered for tall road piers: a section with two side walls inclined, one with two frontal walls inclined, and one with all four walls inclined. Results demonstrated that the first specimen was the most economically efficient, both in column and foundation costs. Kaveh and Massoudi [14] presented an ant colony system model for cost optimization of composite floor systems based on the load and resistance factor (LRFD) design. They minimized the whole cost of the structure, subjected to serviceability and strength requirements, and investigated the effects of beam spans and loadings on the cost optimization of composite beams. Even though abundant research concerning the optimization of reinforced concrete bridge structures is reported in the literature, to the best of the authors' knowledge, work on the corresponding problem for the life-cycle cost of prestressed concrete bridge decks is meager. In this paper, the FORTRAN programming language is used to analyze the pre-tensioned bridge girders under dead and live loads. In order to obtain a proper algorithm for the optimization stages, the "Feasible Directions" method is used to optimize the pre-tensioned girders. The annual life-cycle cost is used as the objective function to compare the design alternatives with various geometric variables.

2. FORMULATION OF THE PROBLEM

The most important hint the designer ought to consider using optimization methods is to create a mutual relation between the analysis and the optimization program.

Formulation of the problem plays a great role in the design success. The design problem consists of an objective function and (several) inequality or equality constraint(s) influencing the objective function,

denoted with $g_j(\underline{X})$ and $h_k(\underline{X})$, respectively. The objective function and the constraints are all functions of the design variables X_i . Thus, the problem is to find the set of variables X_i such that the objective function becomes minimal under the inequality constraints. Mathematically, it can be written as:

$$\text{Minimize } F(\underline{X}) \quad (1)$$

such that:

$$g_j(\underline{X}) \leq 0 \quad j = 1, 2, \dots, M \quad (2)$$

$$h_k(\underline{X}) = 0 \quad k = 1, 2, \dots, L \quad (3)$$

$$x_i^l \leq x_i \leq x_i^u \quad i = 1, 2, \dots, N \quad (4)$$

in which X is the design variables vector containing the elements dimensions, the structure geometric properties, the material(s) used for elements, etc.

The objective function is usually considered to be the total weight of the structure since it can be easily defined based on the design variables, and it signifies the cost properly. Equality constraints are rarely used in design of structures. As an example for the inequality constraint, the stress in each element must not exceed the allowable stress. Therefore:

$$g = 1 - \frac{\sigma}{\sigma_a} \geq 0 \quad (5)$$

in which σ_a is the allowable stress. To begin with, a recommended set of design variables X^0 is given to the algorithm as an entry, either violating a number of constraints or satisfying them all. The minimization method adopted in this paper is the *feasible directions* method, primarily introduced by Zoutendijk [15] as a direct method. In this method, the aim is to find the S direction in which the objective function becomes minimum without violating any constraint:

$$\underline{X}^q = \underline{X}^{q-1} + \alpha \underline{s}^q \quad (6)$$

in which q is the iteration step number, \underline{s} is the search direction vector, and α is the numerical parameter. Thus, $\alpha \underline{s}$ will be the difference in \underline{X} in this iteration step. Therefore, the optimization problem comprises two parts: firstly, to find a search direction able to decrease the objective functions without violating any constraint; secondly, to determine the singular parameter α to enhance the previous design. The modified minimization problem using the feasible directions method can be written as follows:

$$\text{Maximize } \beta \quad (7)$$

$$\nabla f(\underline{X}) \cdot \underline{s} + \beta \leq 0 \quad (8)$$

$$\nabla g_j(\underline{X}) \cdot \underline{s} + \theta_j \beta \leq 0 \quad (9)$$

in which ∇ signifies the gradient of the functions, $g_j(X)$ is the set of active constraints, and β is an intermediate objective function. An active constraint takes place when it approaches the constraint boundary. This method is proposed by Vanderplaats [16, 17] in a computer program called CONMIN, used in this paper as the optimizer. Memari and Madhkhani [18] have also used this program successfully optimizing 2D steel frames with different types of braces, viz. cross and chevron, concentric and eccentric braces.

3. DESIGN AND OPTIMIZATION OF BRIDGES WITH SPLICED GIRDERS

In this research, a single-bay bridge with spliced girders is considered with the post-tensioning applied after concreting the deck. This construction method gives the maximum obtainable bay length in relation to that given by other methods. In this method, the constructed pre-tensioned girders are first set in place temporarily, and then the contact gaps, diaphragms, and the deck are concreted wholly. After the concrete reaches its serviceability strength, longitudinal post-tensioning is used throughout the bridge to complete the connection between elements. Finally, the temporary supports are removed abruptly, so that the deck and the girders act in composition. The simplified shape of this bridge is demonstrated in Fig. 1.

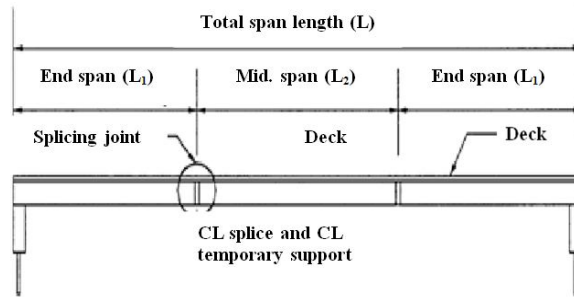


Fig. 1. Schematic shape of the bridge with single-bay spliced girder

a) Design variables

The design variables are: npr_{1i} , npr_{2i} , npr_{i1} , npr_{i2} , n_{po} , y_a , and y_{bb} , which are shown in Fig. 2. n stands for number, pr for pretensioned, and po for post-tensioned. The distance between the pre-tensioned cables at the top and bottom of the girder is taken to be 50 mm (1.968 in), and post-tensioning strands can be defined with a quadratic profile. Naturally, post-tensioning strands need several ducts to be passed through; however, to avoid using several duct profiles as design variables, an effective post-tensioning profile is used to signify the number of ducts. The distance between the single ducts is taken to be 152 mm (6 in). The depth of the effective post-tensioning profile is obtained by dividing the sum of vertical distances from the outer bottom cord to the center of each duct, by the number of ducts in that region.

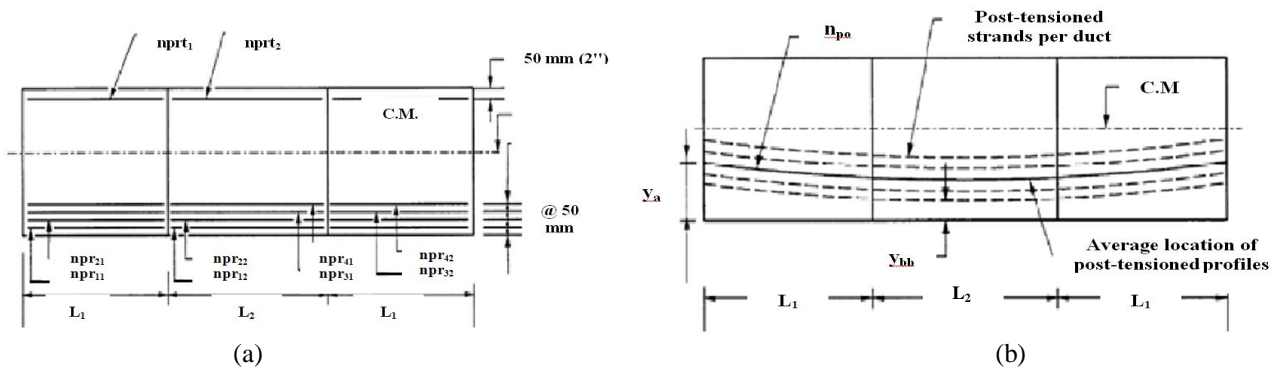


Fig. 2. (a) pre-tensioning cables, (b) post-tensioning cables

b) The objective function

The objective function in this study is considered to be the superstructure cost, defined as stated in Eq. (13). The components of Eq. (10) are defined in Table 1.

$$F(x_i) = C_{PS} + C_{PT} + C_{CD} + C_{TS} + C_B + C_R \quad (10)$$

Table 1. Components of Eq. (10)

Material	Component cost
Pre-tensioning, C_{PS}	$C_p \cdot N_g \cdot [2L_1 (npr_{i1} + nprt_1) + (npr_{i2} + nprt_2)]$
Post-tensioning, C_{PT}	$C_{PO} \cdot N_g \cdot n_{po} (2L_1 + L_2)$
Beam, C_B	$C_b \cdot N_g (2L_1 + L_2)$
Concrete, C_{CD}	$C_c \cdot L \cdot W \cdot t_s$
Reinforcement, C_R	$C_r \cdot W (2L_1 + L_2) W_{st}$
Temporary support, C_{TS}	$2C_{ts} \cdot N_g$

Since it is difficult to distinguish the actual costs, approximate values have been adopted considering typically-constructed decks and pre-tensioned girders with additional costs for temporary supports and the pre-tensioning itself. The typical costs are collected in Table 2 [19].

Table 2. Assumed unit costs of materials

Component item	Unit cost	
	SI Units	US Units
Cost of concreting (C_c)	176 \$/m ³	230 \$/yd ³
Cost of pre-tensioning (C_p)	50 \$/strand-m	15 \$/strand-ft
Cost of post-tensioning (C_{po})	100 \$/strand-m	30 \$/strand-ft
Cost of temporary support (C_{ts})	1000 \$/beam	1000 \$/beam
Cost of PCI-BT 96 (C_b)	900 \$/m	300 \$/ft
Cost of I-shaped beam: Type IV	420 \$/m	140 \$/ft
Cost of epoxy reinforcement (C_r)	2 \$/kg	0.908 \$/lbm

c) Design constraint functions

The design constraints for the optimization problem are the ultimate-state strength and serviceability-state criteria, expressed in the AASHTO-LRFD design criteria and NEHRP 517. The ultimate-state constraints are the control of flexural and shear capacities, and the serviceability constraints are the limits of the decks' and girders' bending stresses. In the latter case, the serviceability-state criterion controls the pre-tensioning force demand. It is worth being stating that the neutral axis relative depth c/d_e was less than 0.42 in all cases. The concrete elements stresses are calculated diversely in each construction stage. Namely, when using pre-tensioned cables in girders, the girders do not become sufficiently strong; thus, the initial compressive strength is used for concrete. All the same, when the deck has been placed on the girders, the girders have gained enough strength; thus, the ultimate compressive strength is used for the ultimate-state limit. In each critical step, the stresses are calculated for the top and bottom cords of the girder as well as for the top cord of the deck.

4. LIFE-CYCLE COSTS IN SPLICED PRESTRESSED CONCRETE BRIDGES

Precast pre-tensioned bridges have proved to be preferential in comparison to other concrete bridges for economical issues including reduction of life-cycle costs, and for their construction convenience. The life-cycle cost comprises the initial cost and the maintenance cost. The maintenance cost of a newly-constructed bridge ought to be much less than its initial cost; thus, the designer had better consider the durability as part of the design problem. In this research, only parametric studies have been performed concerning increase in the slabs and concrete cover to consider the durability in advance. The annual life-cycle cost can be obtained by dividing the total life-cycle cost by the life-cycle period by year. Then, the obtained cost can be used as an objective function to compare the design alternatives with various concrete covers.

5. PARAMETRIC STUDIES

In order to calculate the internal forces of girders, including the maximum shear and moment throughout the girders induced by trucks' and tanks' live loads in conformity with Iranian loading standards, a program known as *OBPG (Optimization of Bridge Pre-tensioned Girders)* has been developed by the authors which is capable of calculating the internal forces based on the bay-lengths of the different parts of the bridge. There are several factors inducing damage in concrete bridges. One of the prevailing causes is chlorides, which trigger the commencement of reinforcement corrosion. Of the events increasing the so-called corrosion is the fading of concrete reinforcement cover, since in this case chlorides can better reach the reinforcing steel rebars. Another factor increasing the penetration of chlorides is the concrete infinitesimal holes, acting as penetration paths for the chloride ions. Other factors influencing the chlorides penetration include the concrete type and quality, the *W/C* ratio, construction circumstances, etc. The commencement time of corrosion is dependent upon the concrete covering value and quality, the concrete setting, chloride effective propagation properties, the chlorides concentration on the concrete surface, and the chloride threshold for reinforcement corrosion.

a) Optimization problem for bridges with simple pre-tensioned girders

The design variables for the optimization problem of simple pre-tensioned girders are the girder cross sectional dimensions and the slab thickness, as shown below:

- $x_1 = h$; Section height of the pre-tensioned girder
- $x_2 = b_1$; Top flange width of the pre-tensioned girder
- $x_3 = b_2$; Bottom flange width of the pre-tensioned girder
- $x_4 = t_1$; Top flange thickness of the pre-tensioned girder
- $x_5 = t_2$; Bottom flange thickness of the pre-tensioned girder
- $x_6 = t_w$; Web thickness of the pre-tensioned girder
- $x_7 = t_c$; Thickness of the deck slab
- N ; Number of pre-tensioned cables

The design variables are demonstrated in Fig. 3. The assumed parameters used for the bridge are shown in Table 3. The optimization results using a number of different design variables are included in Table 4 (The objective function is the useful life cost of the bridge). Correspondingly, the objective function vs. iteration-number diagram is shown in Fig. 4. In each example, a different initial value has been considered for each variable. These initial values do not affect the convergence iteration number significantly. As seen in Fig. 4, the fourth trial has more quickly reached the minimum-cost design.

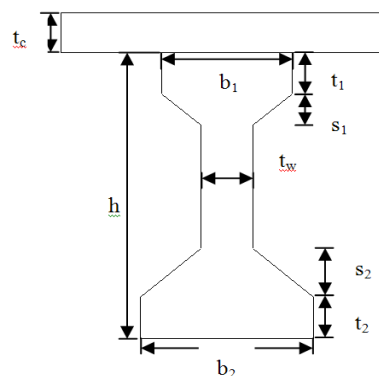


Fig. 3. Design variables for the optimization problem of bridges with pre-tensioned girders

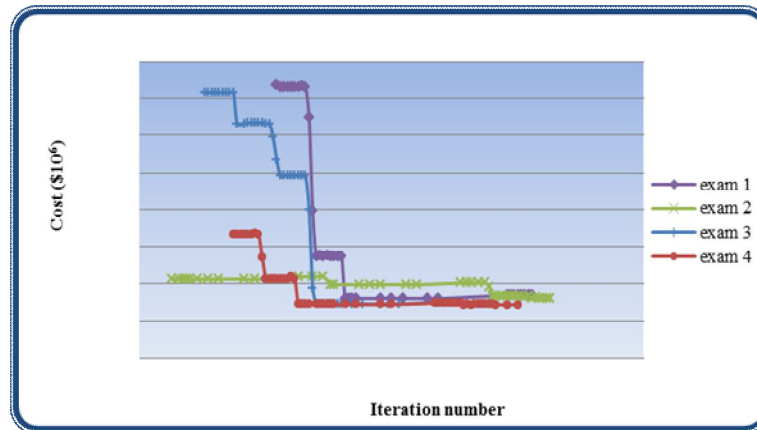


Fig. 4. The cost vs. iteration number diagram

Table 3. Properties of the design bridge components

Simple-span bridge properties		Girder material properties	Deck slab material properties	Pre-tensioned cables properties
Bay length	$L = 26.5m$ (85.48ft)	$f'_c = 35MPa$ (5.07ksi)	$f'_{cd} = 24MPa$ (3.48ksi)	$a_{pa} = 140mm^2$ (0.22in ²)
Deck width	$W_D = 9m$ (29.03ft)	$f'_{ci} = 31.5MPa$ (4.57ksi)	$f'_{cdi} = 18.5MPa$ (2.68ksi)	$E_p = 206215.9MPa$ (29901.3ksi)
Distance between girders	$S_D = 2.4m$ (7.74ft)	$E_D = 28.635MPa$ (4.6921ksi)	$E_{cd} = 23237.9MPa$ (3369.5ksi)	$f_{pu} = 1890MPa$ (274.05ksi)
		$E_{pi} = 26622.4MPa$ (3860.25ksi)	$E_{dci} = 20391.2MPa$ (2956.72ksi)	$f_{py} = 0.9 f_{pu}$
		$W_C = 2400kg/m^3$ (157.44lbm/ft ³)	$W_{cd} = 2400kg/m^3$ (157.44lbm/ft ³)	$f_{p0} = 0.75 f_{pu}$

Note: *i* stands for *initial*

Table 4. Optimization outcomes based on various initial design variable values

Variable	Example 1		Example 2		Example 3		Example 4	
	First	Final	First	Final	First	Final	First	Final
X(1)	80	144.7	180	152.6	120	143	137	142.3
X(2)	20	44.5	60	56.2	40	47.3	50	50.6
X(3)	20	77.7	80	62.3	60	70.8	66	68.8
X(4)	10	21.5	25	27.4	15	23	20	25.2
X(5)	10	34.1	25	27.9	15	24.3	20	23.1
X(6)	10	30	30	30	20	30	25	30
X(7)	5	59	100	56	10	52	30	49
Objective function	1161205.217		1150303.858		1146740.187		1144772.850	
Error percentage	1.44%		0.48%		0.17%		0.00%	

b) Optimization problem for bridges with spliced pre-tensioned girders

The properties of the simple-span bridge used in this study are collected in Table 5. The objective function in this problem is again the useful life cost, expressed based on 6 simple design variables including the pre-tensioned and post-tensioned strands. There are 21 design constraints including constraints due to stress, ultimate strength, shear, and deflection. The results demonstrate that the minimum cost pertaining to Examples 5 and 6 is $\$2.983 \times 10^6$ and $\$2.84 \times 10^6$, respectively.

Table 5. Properties of the design simple-span bridge

Properties of the simple-span bridge		Properties of the girder material	Properties of the deck slab material	Properties of the pre-tensioned strands
Span length	$L = 60.35m$ (198 ft)	$f'_c = 45.7MPa$ (6.63ksi)	$f'_{cd} = 31.64MPa$ (4.59ksi)	$a_{pa} = 140mm^2$ (0.22in ²)
Deck width	$W_D = 18.59m$ (61ft)	$f'_{ci} = 35.15MPa$ (5.1ksi)	$f'_{cdi} = 24.61MPa$ (3.57ksi)	$E_p = 200375MPa$ (29054.37ksi)
Distance between girders	$S_D = 2.74m$ (9 ft)	$E_D = 34366.1MPa$ (4983.1ksi)	$E_{cd} = 26885.4MPa$ (3898.38ksi)	$f_{pu} = 1898.3MPa$ (275.25ksi)
Deck slab thickness	$t_C = 216mm$ (8.5in)	$E_{pi} = 30141MPa$ (437.44ksi)	$E_{cdi} = 23707.5MPa$ (3437.59ksi)	$f_{py} = 0.9f_{pu}$
		$W_C = 2400kg/m^3$ (157.44lbm/ft ³)	$W_{cd} = 2400kg/m^3$ (157.44lbm/ft ³)	$f_{p0} = 0.75f_{pu}$

Note: *i* stands for initial

c) Effect of specific weights of materials

In this study, the effect of concrete volumetric weight has been evaluated in three stages: the effect of the girder concrete specific weight, the deck slab specific weight, and both the girder concrete and deck slab specific weights. To assess the effect of the girder concrete specific weight, the deck specific weight is kept constant, and that of the girder is varied from 1600 to 2400 kg/m³ (104.96 to 157.44 lbm/ft³). All the same, for the second part, the girder specific weight is kept constant, and that of the deck slab ranges between 1600 and 2400 kg/m³ (104.96 to 157.44 lbm/ft³). Thirdly, both the specific weights of the girder and the deck slab have been equally changed in the 1600 to 2400 kg/m³ (104.96 to 157.44 lbm/ft³) range. The corresponding figure is Fig. 5. The utmost difference between the results of OBPG and that of Ref. [19] is 13.38%, which demonstrates the high accuracy of the results. Moreover, this figure proves that the allowable span length of the girder can be increased by 9 m, i.e. 14 percent by reducing the specific weight by 33 percent.

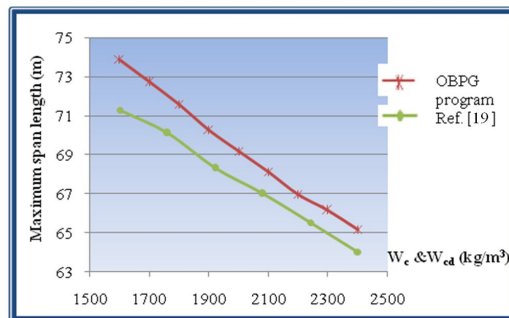


Fig. 5. Comparison between Ref.[19] and the OBPG program results in the span length based on both the deck slab's and the girder's concrete specific weight

d) Effect of the girder concrete compressive strength

In this study, the compressive strength of girders ranges between 46 and 70 MPa (6.67 to 10.15 ksi), and as before, the allowable span-lengths and the corresponding increase percentages are obtained. The pertaining figure is Fig. 6. The outcomes demonstrate the negligibility of the differences between the results obtained from OBPG and those of Ref. [19]. The utmost difference between the results is 2.52%. Also, it can be observed that increasing the concrete compressive strength by 52 percent contributes to only 2.52 percent increase in the allowable girder span length. Namely, the concrete compressive strength does not have a major effect on the span length.

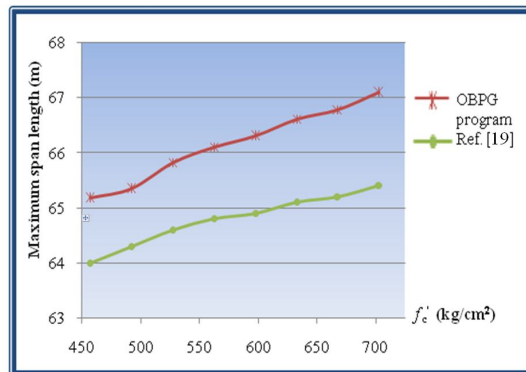


Fig. 6. Comparison between Ref.[19] and the OBPG program results in the span length based on the girder concrete compressive strength

e) Effect of lateral distances between the girders

Obviously, reduction of the lateral distance between each two girders leads to having longer permissible spans. For evaluation, the lateral distance is changed from 1.83 to 2.74 m (5.90 to 8.84 ft). The corresponding figure is Fig. 7. The cost vs. girders lateral spacing is shown in Fig. 8, demonstrating that the total cost will increase in case of reducing the lateral spacing. The abrupt changes in the cost pertain to the addition of one girder to the system, imposing a large cost, mainly due to pre-tensioning and post-tensioning. Thus, equilibrium should be reached between the increasing cost and the decreasing lateral spacing.

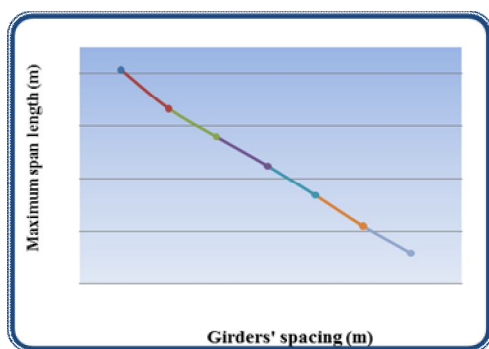


Fig. 7. Maximum span length based on the girders spacing

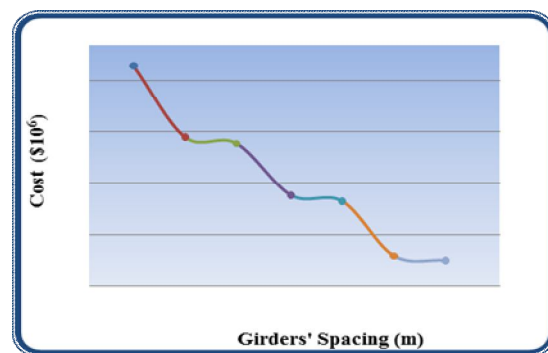


Fig. 8. Total cost based on the girders spacing

f) Comparison between the optimization of simple and spliced pre-tensioned girders

In case of using spliced girders, the allowable span-length increases. However, the longer the span becomes, the more difficult the construction of the pre-tensioned girders will be. Fortunately, using spliced girders resolves this problem due to the fact that only one half of the span-length need be constructed. This will allow the taskmaster to have the girders constructed in precast segments, which can

be made with very high accuracy. The comparison between the simple and spliced girders of the cross sections designed in Ref. [19] obtained from OBPB and Ref [19], are shown in Fig. 9 to Fig. 11. The horizontal axes are the proportion L_1/L , as shown in Fig. 1. The results reveal that spliced girders have an allowable span length 35 to 68 percent more than that of ordinary girders, depending on the L_1/L proportion and the type of cross section (either AASHTO III, AASHTO IV, or PCI-BT 96). This fact demonstrates the usefulness of spliced girders as compared to ordinary ones. Furthermore, increasing the L_1/L proportion (from 0.2 to 0.25) leads to increase in the allowable span length in all three cross section types. It can also be deduced that, as far as the allowable span length is concerned, PCI-BT 96 cross section proves more efficient than the other two, and AASHTO IV proves better than AASHTO III.



Fig. 9. Comparison of the optimization of simple and spliced girders for the AASHTO III section

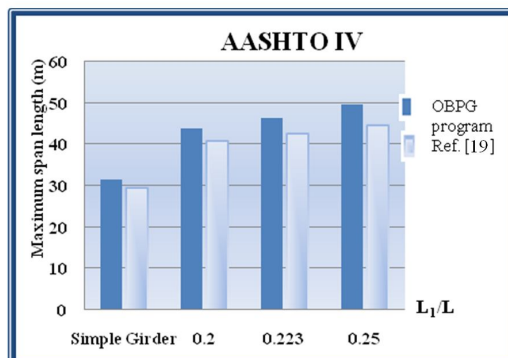


Fig. 10. Comparison of the optimization of simple and spliced girders for the AASHTO IV section

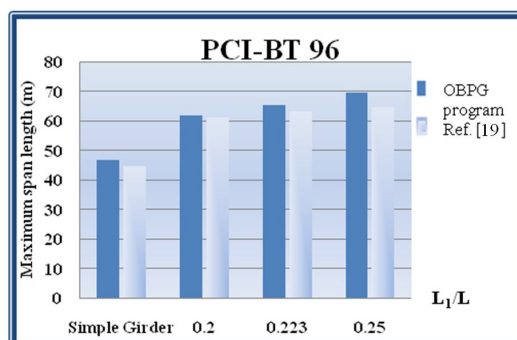


Fig. 11. Comparison of the optimization of simple and spliced girders for the PCI – BT 96 section

g) Effect of concrete cover on the spliced pre-tensioned girders throughout the useful life

It is recommended by ACI-318-05 that at least a 40 mm (1.57 in) concrete cover over the pre-tensioned cables be provided. However, to avoid being hazarded by chlorides, increasing the cover is

inevitable, especially when the useful life is rather long, e.g. at least 100 years. In this study, the cross-sections put forward in Ref. [19] are put to assessment. To do so, the chloride concentration on the concrete boundary is kept at 19 kg/m^3 (1.25 lbm/ft^3), then the concentration in the depth x after t seconds is calculated using Eq. (11) [19].

$$C(x, t) = C_0 \left(1 - \operatorname{erf} \frac{x}{2\sqrt{D_c t}} \right) \tag{11}$$

where C_0 is the initial concentration, and D_c is the propagation coefficient, which is 0.0728 and 0.0293 for cold and hot climates, respectively. The chloride concentration vs. service-life diagram is shown in Fig. 12, assuming the weather is cold. The allowable chloride concentration in Asian and American countries is 1.2 and 0.7 kg/m^3 (78.72×10^{-3} and $45.92 \times 10^{-3} \text{ lbm/ft}^3$), respectively. By drawing a horizontal line, the ordinate of which equals the allowable chloride concentration, the serviceable life of each concrete covering can be obtained. For instance, the serviceable life of the 50 mm (1.97 in) cover in Asia is 16 years. Consequently, it is possible to determine the influence of concrete covering in the total cost. The design assumptions for this problem are collected in Table 6. The influence of concrete covering is included in Table 7. Therefore, the total-cost vs. the covering-thickness diagram is shown in Fig. 13. The total cost includes the initial, maintenance, and life-cycle costs. Therefore, the initial and life-cycle costs of the bridge can be obtained depending on the concrete covering. By increasing the cover from 50 to 90 mm (1.97 to 3.54 in), the initial cost increases by 7.43% whereas the life-cycle cost decreases by 14.78% . Thus, the so-obtained design will be more cost-effective in terms of the life-cycle cost.

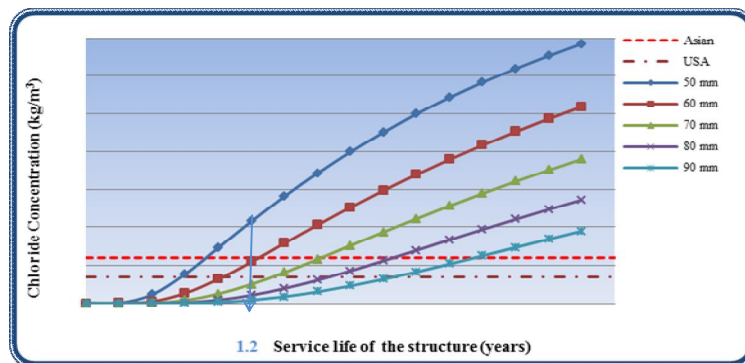


Fig. 12. Effect of concrete cover on the chloride concentration based on the service life of the bridge

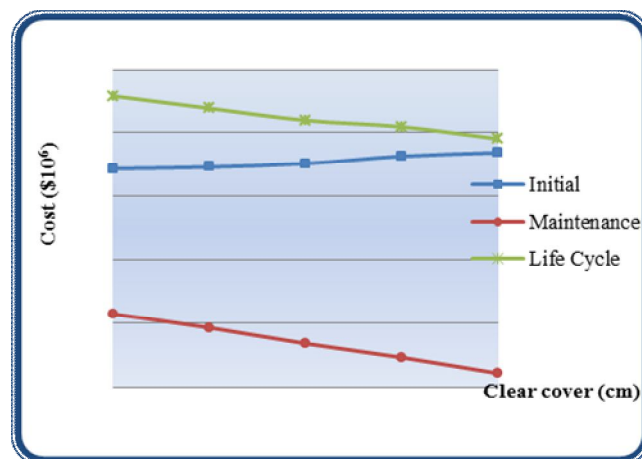


Fig. 13. Initial, maintenance, and life-cycle costs based on different concrete covers

Table 6. Design assumptions of the service-life problem

Bridge width	18.6 m	(61 ft)
Bridge length	36.3 m	(119 ft)
Beam spacing	2.1 m	(7 ft)
No. of beams	8	
Slab thickness cover	152.4 mm	(6 in)
Approximate reinforcement	96 km/m ³	(6 lb/ft ³)
Lifetime estimate	90 Years	
Superstructure concrete deck	1589 \$/m ³	(45 \$/ft ³)
Epoxy steel reinforcement	2.2 \$/kg	(1 \$/lb)
AASHTO Type III	400 \$/m	(122 \$/ft)
Deep concrete overlay	183 \$/m ²	(17 \$/ft ²)
Shallow concrete overlay	162 \$/m ²	(15 \$/ft ²)
Pre-tensioned strands cost	53 \$/m	(16 \$/ft)
Post-tensioned strands cost	108 \$/m	(33 \$/ft)

Table 7. Effect of concrete covering change in the life-cycle cost

Initial cost					
Clear cover (cm)	5	6	7	8	9
Clear cover difference (cm)	0	1	2	3	4
Deck area (m ²)	675.18	675.18	675.18	675.18	675.18
New slab thickness (cm)	20.32	21.32	22.32	23.32	24.32
Deck concrete volume (m ³)	137.2	143.9	150.7	157.5	164.2
Reinforcement steel	13171	13819	14467	15115	15764
No. of pre-tensioned strands	22	20	21	22	21
No. of post-tensioned strands	36	37	37	38	39
Initial deck cost (\$)	246981	259136	271290	283445	295600
Initial beam cost (\$)	1467682	1468262	1483654	1530408	1546380
Total initial cost (\$)	1714663	1727398	1754944	1813853	1841980
Maintenance and repair cost (of the cool area)					
Superstructure service life (years)	16	23	33	48	63
No. of deep overlay	2	2	1	1	0
No. of shallow overlay	3	2	2	1	1
Deep concrete cost overlay per time (\$)	123558	123558	123558	123558	123558
Shallow concrete cost overlay per time (\$)	109379	109379	109379	109379	109379
Deep concrete overlay cost (\$)	247116	247116	123558	123558	0
Shallow concrete overlay cost (\$)	328137	218758	218758	109379	109379
Total maintenance and repair cost (\$)	575253	465874	342316	232937	109379
Life-cycle cost					
Life cycle cost (\$)	2289916	2193272	2097260	2046790	1951359
Percent increased from the initial cost	34%	27%	20%	13%	6%

5. CONCLUSION

In the present paper, the life-cycle cost of simple-span spliced RC girders with prestressed bars is evaluated by using the feasible directions method. To do so, the well-known OBPG program is extended to consider all costs pertaining to life-cycle cost. At first, internal forces due to live loads, caused by hefty vehicles such as trucks and tanks, were calculated for girders using the OBPG program and Ref. [19].

When compared, the results obtained from OBPG had acceptably small differences with those of Ref. [19]. Then, optimization of prestressed girders is performed through the following steps:

- 1- Pre-tensioned girders are optimized based on the combined section dimensions and the number of pre-tensioned strands. It is concluded that the time consumed to reach the minimal objective function depends on the initial value taken for the design variables set. Nevertheless, all trials finally approach almost equal objective functions with infinitesimal differences. It is worth noting that the best initial values are the ones recommended in *AASHTO* for reaching the minimal function more quickly and producing the least objective function.
- 2- Connected pre-tensioned girders (the connection is longitudinal and made with post-tensioned strands) are optimized using the OBPG program based upon the number of pre-tensioned and post-tensioned strands; then the results are compared to those obtained from Ref.[19], and negligible differences are observed between the two results. In this part, a constant cross section is used for girders.
- 3- The effect of reducing the concrete specific weight was assessed in three states:
 - Reducing the girder concrete specific weight from 2400 to 1600 kg/m^3 (157.44 to 104.96 lbm/ft^3) leads to 7.9 % increase in the allowable bridge span-length.
 - Reducing the deck slab concrete specific weight from 2400 to 1600 kg/m^3 (157.44 to 104.96 lbm/ft^3) leads to 5.6 % increase in the allowable bridge span-length.
 - Reducing the girders and deck slabs specific weights simultaneously from 2400 to 1600 kg/m^3 (157.44 to 104.96 lbm/ft^3), using equal values for both, leads to 13.4 % increase in the allowable bridge span length.
- 4- By increasing the girder concrete compressive strength from 46 to 70 MPa (6.67 to 10.15 ksi), the allowable bridge span-length can be increased by 2.95%, which is quite negligible.
- 5- By decreasing the lateral spacing between the girders from 2.74 to 1.83 m (8.84 to 5.90 ft), the allowable bridge span-length increases by 10.68 % while the total cost is increased by 36.63 %.
- 6- Initial and life-cycle costs of the bridge are determined depending on the concrete covering. This study demonstrated that, by raising the covering from 50 to 90 mm (1.97 to 3.54 in), the initial cost increases by 7.43 % whereas the life-cycle cost gets decreased by 14.78 %. This contributes to having a more cost-effective design.

NOMENCLATURE

C_B	Girder construction cost
\tilde{C}_C	Cost of concreting
C_{CD}	Deck concreting cost
C_p	Unit cost of pre-tensioning
C_{po}	Unit cost of post-tensioning
C_R	Cost of epoxy reinforcement
\tilde{C}_{ps}	Total cost of pre-tensioning
C_{PT}	Total cost of post-tensioning
C_{TS}	Cost of temporary supports
\bar{E}	Modulus of elasticity
f_c	Final compressive strength of the prestressed concrete
f_{cd}	Final compressive strength of the deck concrete
f_u	Ultimate tensile stress
f_y	Yield tensile stress
L	Bridge length
N_g	Number of girders

n_{po}	Total number of pre-tensioned or post-tensioned cables on the girder
n_{pr_1}	Number of bottom pre-tensioned cables on the end parts of the i 'th row
n_{pr_2}	Number of bottom pre-tensioned cables on the middle parts of the i 'th row
n_{pr_i}	Number of top pre-tensioned cables on the end parts of the i 'th row
n_{pr_i}	Number of top pre-tensioned cables on the middle parts of the i 'th row
W	Bridge width
W_{st}	Total weight of reinforcing steels in unit area
ϕ_w	Strength reduction coefficient

REFERENCES

1. Frangopol, D. M. & Lin, K. Y. (1997). Life-cycle cost design of deteriorating structures. *Journal of Structural Engineering*, Vol. 123, No. 10, pp. 123–141, ASCE.
2. Haftka and Gurdal, (1993). *Elements of structural optimization*. Kluwer Academic publishers, p. 612.
3. Gobel, G. G. & Lapay, W. S. (1971). Optimum design of prestressed beams. *Journal of ACI*, Vol. 69, No. 9.
4. Farkas, J. (1996). *Optimum design of welding bridges. Welding in word Le Soudage dans le Monde*. Pergamen Press Ltd publishers, Oxford, U. K., p. 295.
5. Lee, K. M., Cho, H. N. & Choi, Y. M. (2004). Life–cycle cost effective optimum design of steel bridge. *Journal of Constructional Steel Research*, Vol. 60, No. 11, pp. 1585–1613.
6. Lee, K. M., Cho, H. N. & Choi, Y. M. (2006). Life–cycle cost effective optimum design of steel bridges considering environmental stressors. *Journal of Engineering Structures*, Vol. 28, No. 9, pp. 1252–1265, ASCE.
7. Sadeghi, J. M. & Babae, A. (2006). Structural optimization of B70 railway prestressed concrete sleepers. *Iranian Journal of Science & Technology, Transaction B: Engineering*, Vol. 30, No. B4, pp. 461-473.
8. Hong, T., Han, S. & Lee, S. (2007). Simulation-based determination of optimal life-cycle cost for FRP bridge deck panels. *Journal of Automation in Construction*, Vol. 16, pp. 140–152.
9. De Castilhoa, V. C., El Debsa, M. K. & Nicoletti, M. C. (2007). Using a modified genetic algorithm to minimize the production costs for slabs of precast prestressed concrete joists. *Journal of Engineering Applications of Artificial Intelligence*, Vol. 20, pp. 519–530.
10. Marti, J. V. & Vidos, F. G. (2010). Design of prestressed concrete precast pedestrian bridges by heuristic optimization. *Journal of Advances in Engineering Software*, Vol. 41, pp. 916-922.
11. Mitoulis, S. A., Tegos, I. A. & Stylianidis, K. C. (2010). Cost-effectiveness related to the earthquake resisting system of multi-span bridges. *Journal of Engineering Structures*, Vol. 32, pp. 2658-2671.
12. Kaveh, A. & Shakouri Mahmud Abadi, A. (2010). Harmony search algorithm for optimum design of slab formwork. *Iranian Journal of Science & Technology, Transaction B: Engineering*, Vol. 34, No. B4, pp. 335-351.
13. Martinez, F. J., Vidos, F. G., Hospitaler, A. & Alcalá, J. (2011). Design of tall bridge piers by ant colony optimization. *Journal of Engineering Structures*, Vol. 33, pp. 2320–2329.
14. Kaveh, A. & Massoudi, M. S. (2012). Cost optimization of a composite floor system using ant colony system. *Iranian Journal of Science & Technology, Transactions of Civil Engineering*, Vol. 36, No. C2, pp. 139-148.
15. Zoutendijk, G. (1960). *Methods of feasible directions*. Elsevier, Amsterdam.
16. Vanderplaats, G. N. (1973). CONMIN – A Fortran Program for Constrained Function Minimization. NASA TM X–62282.
17. Vanderplaats, G. N. (1984). Numerical optimization techniques for engineering design: with applications. Japan Soc. of Precision Engineers publication, p. 648.
18. Memari, A. M. & Madhkhan, M. (1999). Optimal design of steel frames to resist seismic loads. *Structural and Multidisciplinary Optimization Journal*, Vol. 18, No.1, pp. 56–66.
19. Surakomol, P. (2005). Optimal design and performance of longitudinally spliced prestressed concrete bridge. Dissertation Submitted to Michigan State Univ. for the Degree of Doctor of Philosophy, UMI Number: 3204803.