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Design of prestressed concrete I-girder bridge superstructure using optimization algorithm

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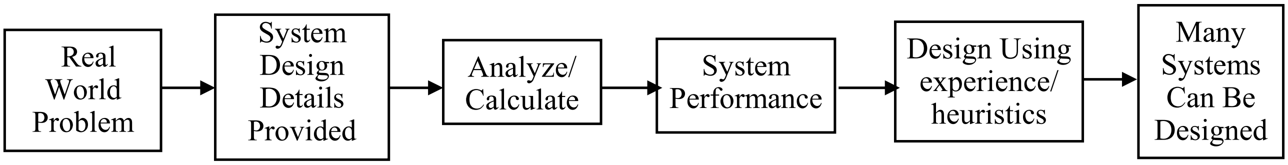
ABSTRACT: In Bangladesh, post tensioned simply supported prestressed concrete (PC) I-girder bridges are widely used bridge system for short to medium span (20m to 50m) highway bridges due to its moderate self weight, structural efficiency, ease of fabrication, low maintenance etc. In order to compete with steel bridge systems, the design of PC I-girder Bridge system must lead to the most economical use of materials. In this paper, cost optimization approach of a post-tensioned PC I-girder bridge system is presented. The objective is to minimize the total cost in the design process of the bridge system considering the cost of materials, fabrication and installation. For a particular girder span and bridge width, the design variables considered for the cost minimization of the bridge system, are girder spacing, various cross sectional dimensions of the girder, number of strands per tendon, number of tendons, tendons configuration, slab thickness and ordinary reinforcement for deck slab and girder. Design constraints for the optimization are considered according to AASHTO Standard Specifications. The optimization problem is characterized by having a combination of continuous, discrete and integer sets of design variables and multiple local minima. An optimization algorithm called Evolutionary Operation (EVOP) is used, that is capable of locating directly with high probability the global minimum. The proposed cost optimization approach is compared with an existing project which leads to a considerable cost saving while resulting in feasible design.

1 INTRODUCTION

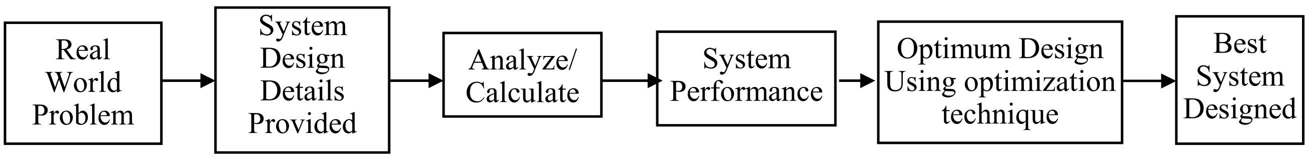
In conventional structure design process, the design method proposes a certain solution that is corroborated by mathematical analysis in order to verify that the problem requirements or specifications are satisfied. If such requirements are not satisfied, then a new solution is proposed by the designer based on his intuition or some heuristics derived from his experience (Fig. 1(a)). The process undergoes many manual iterations before the design can be finalized making it a slow and very costly process. There is no formal attempt to reach the best design in the strict mathematical sense of minimizing cost, weight or volume. The process of design is relied solely on the designer's experience, intuition and ingenuity resulting in high cost in terms of times and human efforts.

An alternative to the conventional design method is optimum design (Fig. 1(b)). An optimum design normally implies the most economic structure without impairing the functional purposes of the structure. An optimization technique transform the conventional design process of trial and error into a formal and systematic procedure that yields a design that is best of in terms of designer specified figure of merit – the goodness factor of design. It is a completely automated process that allows lesser skilled and experienced engineers to create optimum design.

Advances in numerical optimization methods, numerical tools for analysis and design of structures and availability of powerful computing hardware have all significantly aided the design process. So there is a need to perform research on optimization of realistic three-dimensional structures, especially large structures with hundreds of members where optimization can result in substantial savings (Adeli and Sarma 2006). Large and important projects containing PC I-girder bridge structures (Fig.2) which are widely used for short to medium span (20m to 50m) highway bridges have potential for substantial cost savings through the application of optimum design methodology and will be of great value to practicing engineers.



(a) Conventional design process



(b) Optimum design process

Figure 1. Comparison between (a) conventional design process and (b) optimum design process.

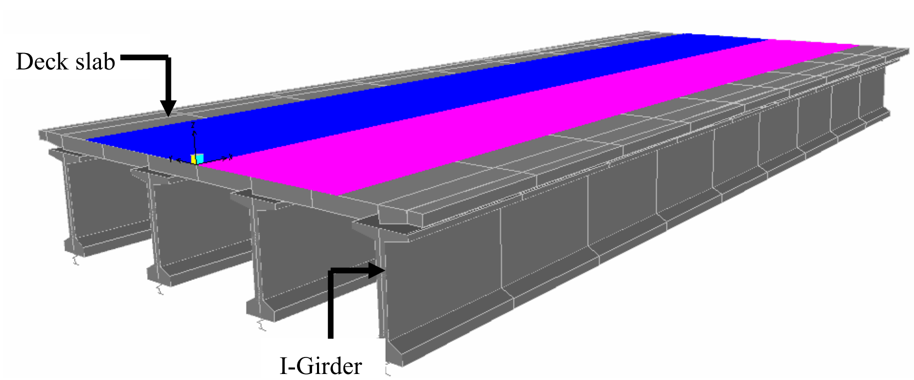


Figure 2. Prestressed concrete I-Girder bridge system.

Optimization of bridge structures is not attempted extensively because of complexities such as a large number of variables, discrete values of variables, and difficulties in formulation. For prestressed concrete structures the approach to design takes the form of cost optimization problem because different materials are involved. A review of articles pertaining to cost optimization of prestressed concrete bridge structures is presented by Hassanain and Loov (2003). From this paper it can be observed that a few studies have been performed regarding the optimum design of the I-girder bridge structure considering the total cost of materials, fabrication and installation. Sirca and Adeli (2005) presented an optimization method for minimizing the total cost of the pretensioned PC I-beam bridge system by considering concrete area, deck slab thickness, reinforcement, surface area of formwork, number of beam as design variables. The problem is formulated as a mixed integer-discrete nonlinear programming problem and solved using the patented robust neural dynamics model. They did not consider cross-sectional dimensions as design variables; instead they used standard AASHTO sections. Ayvaz and Aydin (2009) presented a study to minimize the cost of pre-tensioned PC I-girder bridge through topological and shape optimization. The topological and shape optimization of the bridge system were performed together using Genetic Algorithm.

In the present study a cost optimum design approach of a simply supported post-tensioned PC I-girder bridge system is presented considering the cost of materials, labor, fabrication and installation. The bridge system consists of precast girders with cast-in situ reinforced concrete deck. Large number of design variables and constraints are considered for cost optimization of the bridge system. A global optimization algorithm named EVOP (Evolutionary Operation) (Ghani 1989) is used which is capable of locating directly with high probability the global minimum. The optimization algorithm solves the optimization problem and gives the optimum solutions or design. The formulation of optimization problem of the bridge system and linking of optimization problem to the optimization algorithm or method to obtain the optimum solution is shown in Fig.3. To formulate the optimization problem a computer program is developed in C++.

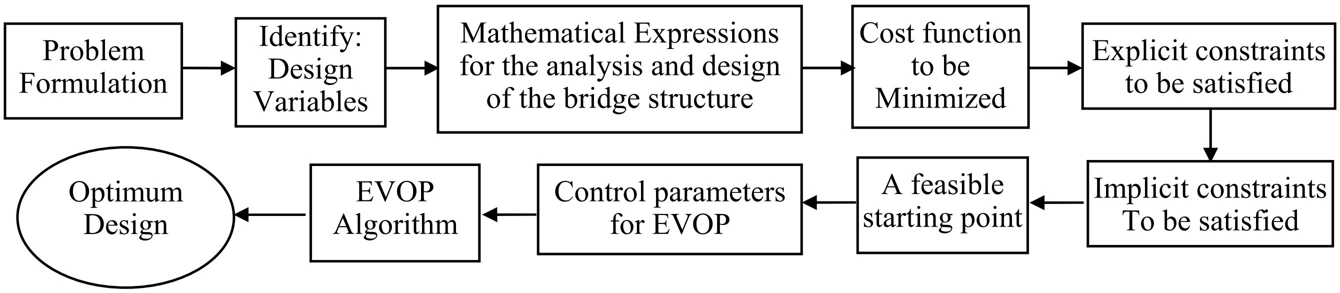


Figure 3. Optimization Problem Formulation and Linking with EVOP

2 PROBLEM FORMULATION

2.1 *Design variables and constant design parameters*

For a particular girder span and bridge width, a large number of parameters control the design of the bridge such as girder spacing, cross sectional dimensions of girder, deck slab thickness, number of strands per tendon, number of tendons, deck slab reinforcement, configuration of tendons, anchorage system, pre-stress losses, concrete strength etc. The design variables and variable type considered in the study are tabulated in Table 1. A typical cross-section of the PC I-girder is illustrated in Fig. 4 to highlight several of the design variables.

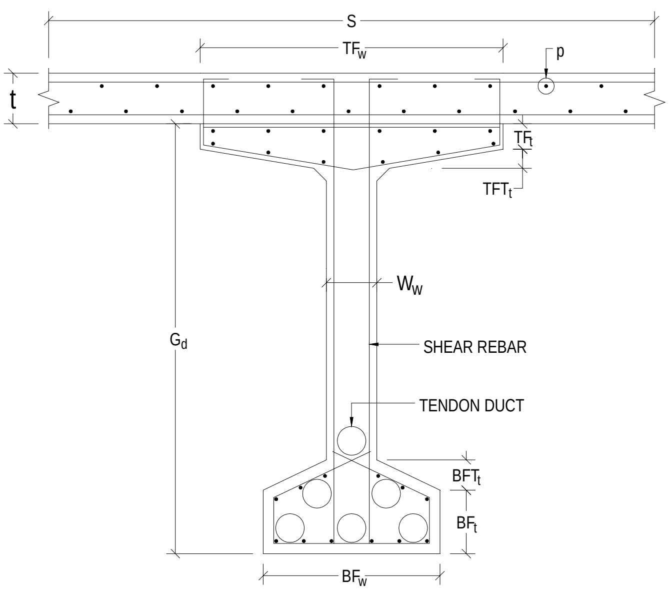


Figure 4. Girder composite section with design variables.

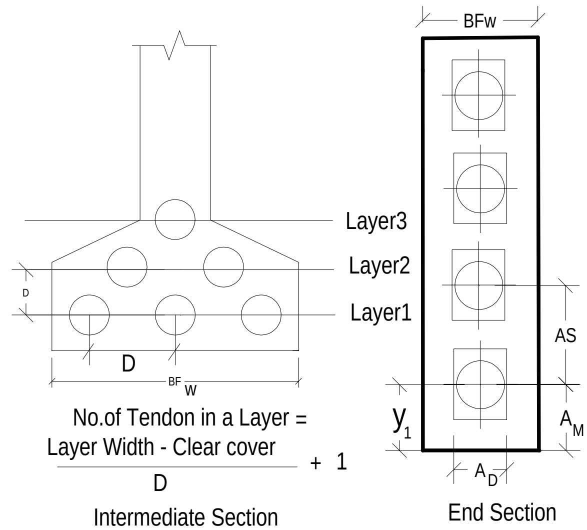


Figure 5. Tendons arrangement in the girder

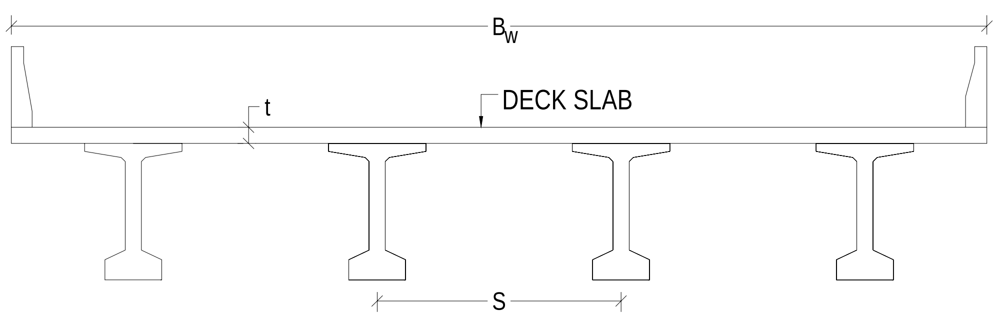


Figure 6. Girders arrangement in the bridge

The constant design parameters under consideration are various material properties, superimposed dead loads, AASHTO live load, strand size, post-tensioning anchorage system and unit costs of materials including fabrication and installation etc. Optimization is based on the analysis of an interior girder arranged as shown in Fig. 6. The girder and the deck are assumed to act as a composite section during service condition.

Prestress is considered to be applied in two stages, a percentage of total prestress at initial stage to carry only the girder self weight and stress produced during lifting and transportation and full prestress during casting of deck slab. In the present study the tendons arrangement is not assumed as fixed rather it is considered as design variable as it has significant effects on prestress losses and flexural stress at various sections along the girder. Tendons layout along the span is assumed as parabolic. The vertical and horizontal arrangement of tendons depends on various cross sectional dimensions of girder such as depth, bottom flange and web. Typical arrangements of tendons at various sections are shown in Fig. 5. The arrangement of tendons also depends on duct size and spacing, anchorage spacing and anchorage edge distance. These parameters depend on a design variable, namely, number of strand per tendon and on a constant parameter, namely, concrete strength etc.

Table1. Design variables with Explicit Constraints

Design variables	Variable type	Explicit Constraint
Girder spacing (S) (m)	Discrete	$B_W/10 \leq S \leq B_W$
Girder depth (G_d) (mm)	Discrete	$1000 \leq G_d \leq 3500$
Top flange width (TF_w) (mm)	Discrete	$300 \leq TF_w \leq S$
Top flange thickness (TF_t) (mm)	Discrete	$75 \leq TF_t \leq 300$
Top flange transition thickness (TFT_t) (mm)	Discrete	$50 \leq TFT_t \leq 300$
Bottom flange width (BF_w) (mm)	Discrete	$300 \leq BF_w \leq S$
Bottom flange thickness (BF_t) (mm)	Discrete	$a \leq BF_t \leq 600$
Web width (W_w) (mm)	Discrete	$b \leq W_w \leq 300$
Number of strands per tendon (N_s)	Integer	$1 \leq N_s \leq 27$
Number of tendons per girder (N_T)	Integer	$1 \leq N_T \leq 20$
Lowermost tendon position at the end from bottom (y_1) (mm)	Continuous	$A_M \leq y_1 \leq 1000$
Initial stage prestress (% of full prestress) (η)	Continuous	$1\% \leq \eta \leq 100\%$
Slab thickness (t) (mm)	Discrete	$175 \leq t \leq 300$
Slab main reinforcement ratio (ρ)	Continuous	$\rho_{min} \leq \rho \leq \rho_{max}$

a = clear cover + duct diameter; b = clear cover + web rebars diameter + duct diameter; A_M = Anchorage minimum vertical edge distance

2.2 *Objective function*

In this study, the objective is the cost minimization of the present bridge systems by taking into account the cost of all materials, fabrication and installation. The total cost of a bridge system is formulated as:

$$C_T = C_{GC} + C_{DC} + C_{PS} + C_{OS} \quad (1)$$

where, C_{GC}, C_{DC}, C_{PS} and C_{OS} are the cost of materials, fabrication and installation of Girder Concrete, Deck slab Concrete, Prestressing Steel and Ordinary Steel for deck reinforcement and girder's shear reinforcement respectively. Costs of individual components are calculated as Eq. (2) to Eq. (5):

$$C_{GC} = (UP_{GC} V_{GC} + UP_{GF} SA_G) N_G \quad (2)$$
$$C_{DC} = (UP_{DC} V_{DC} + UP_{DF} (S - TF_w)) N_G \quad (3)$$
$$C_{PS} = (UP_{PS} W_{PS} + 2\, UP_{ANC} N_T + UP_{SH} N_T L) N_G \quad (4)$$
$$C_{NS} = UP_{OS} (W_{OSD} + W_{OSG}) N_G \quad (5)$$

where, UP_{GC}, UP_{DC}, UP_{PS} and UP_{OS} are the unit prices including materials, labor, fabrication and installation of the precast girder concrete, deck concrete, prestressing steel and ordinary steel respectively. UP_{GF}, UP_{DF}, UP_{ANC}, UP_{SH} are the unit prices of girder formwork, deck formwork, anchorage set and metal sheath for duct respectively; V_{GC}, V_{DC}, W_{PS}, W_{OSD} and W_{OSG} are the volume of the precast girder concrete and deck slab concrete, weight of prestressing steel and ordinary steel in deck and in girder respectively; L is the girder span; N_G is number of girders; S is girder spacing.

2.3 *Explicit Constraints*

These are specified limitation (upper or lower limit) on design variables which are derived from geometric requirements (superstructure depth, clearances etc.), minimum practical dimension for construction, code restriction etc. The constraint is defined as

$$X_L \le X \le X_U \tag{6}$$

where X = Design variable, X_L = Lower limit of the design variable, X_U = Upper limit of the design variable.

Explicit constraints for girder spacing: Lower and upper limit of girder spacing is considered such that number of girder in the bridge can vary from 1 to 10.

Explicit constraints for top flange: The lower limit of top flange width is assumed as 300 mm from lateral stability and bearing considerations and upper limit equal to girder spacing. The lower limit of top flange thickness is considered as 75 mm to resist damage during handling and proper placement of transverse reinforcement and upper limit is assumed as 300 mm. The lower limit of top flange transition thickness is considered as 50 mm to facilitate placement and consolidation of concrete and upper limit is assumed as 300 mm. The haunch thickness and width is assumed as 50 mm.

Explicit constraints for web: The lower limit of web width is equal to diameter of duct plus web rebars and clear cover and upper limit is assumed as 300 mm.

Explicit constraints for bottom flange: The lower limit of bottom flange width is assumed as 300 mm to accommodate anchorage setup and upper limit equal to girder spacing. The lower limit of thickness is equal to clear cover and duct diameter to fit at least one row of tendons. The upper limit is assumed as 600 mm. The width to thickness ratio of bottom flange transition area is assumed as 2 to 1 from practical construction point of view.

Explicit constraints for girder depth: The lower limit of girder depth is considered as 1000 mm and upper limit 3500 mm which is common range of girder depth to minimize the cost of substructure and approach roads and from aesthetics and limited clear space criterion.

Explicit constraints for number of strand per tendon: Within the available anchorage system one tendon may consist of several seven-wire strands like 1 to 55. Here the effect of number of strands in a tendon is studied. For this study it is considered that each tendon may consist of 1 to 27 strands.

Explicit constraints for number of tendon: The amount of pre-stressing force required for cost optimum design are directly associated with the number of tendons required in the girder. For this study it is considered that the number of tendon may vary from 1 to 20.

Explicit constraints for lowermost tendon position: To vary the profile of tendon along the girder span the lower most tendon position from bottom at the end section is considered as a design variable and the other tendon positions are determined from anchorage spacing. The lower limit of vertical position of the tendon is considered equal to anchorage minimum vertical edge distance and upper limit is assumed as 1000 mm.

Explicit constraints for deck slab: The lower limit of deck slab thickness is considered as 175 mm to control deflection and excessive crack and upper limit as 300 mm. The lower and upper limits of deck slab reinforcement are considered according to AASHTO standard specification. The explicit constraints for all the above design variables are shown in Table 1.

2.4 *Implicit constraints*

These constraints represent the performance requirements or response of the bridge system. A total 46 implicit constraints are considered according to the AASHTO Standard Specifications (AASHTO 2002). These constraints are categorized into eight groups:

1. Flexural working stress constraints
2. Flexural ultimate strength constraints
3. Shear constraints (ultimate strength)
4. Ductility constraints
5. Deflection constraint
6. Lateral stability constraint
7. Tendons eccentricity constraint and
8. Deck slab design constraint

These constraints are formulated as below:

2.4.1 *Flexural working stress constraints:*

These are the allowable stresses in concrete and are given by:

$$f^L \le f_j \le f^U \tag{7}$$

$$f_j = -\frac{F_j}{A} \pm \frac{F_j e_j}{S_j} \pm \frac{M_j}{S_j} \tag{8}$$

where, f^L = allowable compressive stress (lower limit), f^U = allowable tensile stress (upper limit) and f_j = the actual working stress in concrete; F_j, e_j, S_j, M_j = prestressing force, tendons eccentricity section modulus and moment at j^{th} section respectively. These constraints are considered at three critical sections along the span of the girder as shown in Fig. 7 and for various loading stages (initial stage and service conditions). The three critical sections are mid section, section immediately after anchor set where the prestress is its maximum value and section at the end of anchorage and transition zone. The end of anchorage and transition zone is assumed as 1.5 times the girder depth.

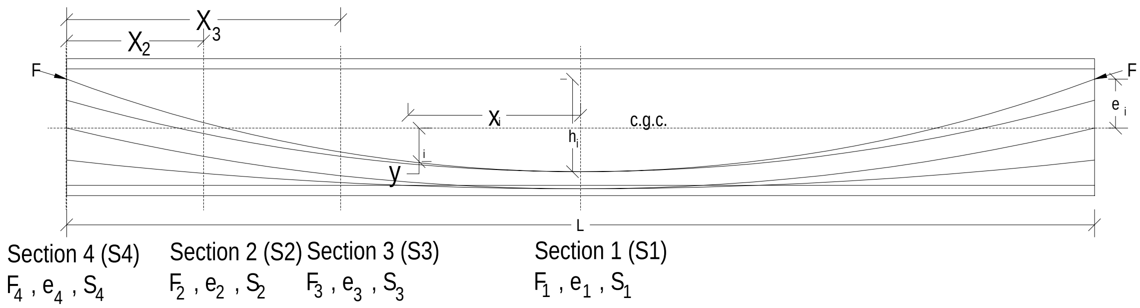


Figure 7. Tendons profile along the girder

The initial loading stage includes the girder dead loads in addition to the prestress after instantaneous losses (friction loss, anchorage loss and elastic shortening loss). In this stage net cross sectional properties of precast girder are used excluding duct. At initial stage a portion of total prestress is applied only to carry girder self weight. At service the first loading stage includes initial loading stage in addition slab and diaphragm weight. In this stage transformed cross sectional properties of precast girder are used and full prestress is applied. The second loading stage includes first loading stage in addition loads due to wearing course and median strip superimposed on composite section and prestress force after total losses is considered. The third loading stage includes live load and impact load superimposed on composite section in addition to second loading stage. The fourth loading stage consists of half of dead load and prestress force plus full live load. Loading stages are summarized in Table 3.

Prestress losses are estimated according to AASHTO Standard Specification instead of using lump sum value for greater accuracy because prestress losses are also implicit functions of some of design variables. The instantaneous losses depend on jacking equipment and anchorage hardware used and the design variables (number of tendons, number of strands per tendon, layout of tendon in the girder, prestressing of tendon and girder cross sectional properties). The long term losses are loss due to creep of concrete, loss due to shrinkage of concrete and loss due to steel relaxation and are also implicit functions of some of design variables. In post-tensioned girder, variation of prestressing forces are considered along the length of the girder due to friction losses and anchorage losses and are shown in Fig.8.

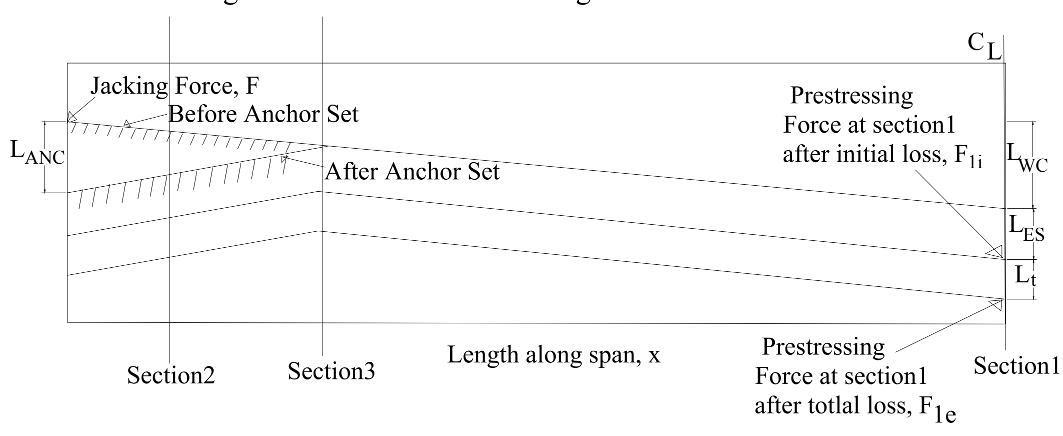


Figure 8. Variation of prestressing force along the length of girder

The prestress forces after all losses at three sections are F_{1e}, F_{2e} and F_{3e} respectively. For post-tensioned members according to AASHTO allowable prestress immediately after seating at anchorage $0.7 f_{su}$, at the end of the seating loss zone $0.83f_y^*$ and stress at service load after losses $0.80f_y^*$. In the present study tensioning to $0.9f_y^*$ (jacking stress) for short period of time prior to seating is considered to offset anchorage and friction losses and implicit constraints are applied such that the stresses in the tendon remain within the allowable limit. The implicit constraints are as follows:

$$0 \le F_{4i} \le 0.7 f_{su} A_s \quad (9) \qquad 0 \le F_{3i} \le 0.83 f_y^* A_s \quad (10) \qquad 0 \le F_{3e} \le 0.80 f_y^* A_s \quad (11)$$

where F_{4i}, F_{3i} = Force after initial loss at section 4 and section 3 respectively; A_s = Area of prestressing steel;

Table3. Loading stages and implicit constraints

Load stage	Resisting section	Section properties	Load Combination	Implicit constraint	Load stage	Resisting section	Section properties	Load Combination	Implicit constraint
Initial stage	Precast section	A_{net}, e_i, S_{net}	$\eta F+G$	Eq. (12)					
1	Precast section	A_{tf}, e, S	$F_i+G+SB+DP$	Eq. (13)	3	Precast section + Composite section	A_{tf}, e, S + S_C	$F_e+G+SB+DP$ + $SD+L+I$	Eq. (15)
2	Precast section + Composite section	A_{tf}, e, S + S_C	$F_e+G+SB+DP$ + SD	Eq. (14)	4	Precast section + Composite section	A_{tf}, e, S + S_C	$0.5(F_e + DL)$ + $L+I$	Eq. (16)

F = Jacking Force; G = Girder self weight; SB = slab weight; DP = diaphragm weight; SD = super-imposed dead load for wearing coarse and curb weight; DL = total dead load; L = live load; I = impact load; S = Section Modulus; A_{tf} = Transformed area;

$$-0.55 f'_{ci} \le f_i \le 0.25\sqrt{f'_{ci}} \quad (12) \qquad -0.60 f'_c \le f_i \le 0.5\sqrt{f'_c} \quad (13) \qquad -0.40 f'_c \le f_i \le 0.5\sqrt{f'_c} \quad (14)$$

$$-0.60 f'_c \le f_i \le 0.5\sqrt{f'_c} \quad (15) \qquad -0.40 f'_c \le f_i \le 0.5\sqrt{f'_c} \quad (16)$$

2.4.2 *Ultimate flexural strength constraints*

The ultimate flexural strength constraints for the precast section and composite section are considered as:

$$0 \le M_{pu} \le \varphi M_{pn} \quad (17) \qquad 0 \le M_{cu} \le \varphi M_{cn} \quad (18)$$

where, M_{pu} and M_{cu} are factored bending moments; φM_{pn} and φM_{cn} are flexural strength of the precast and composite section respectively.

2.4.3 *Ductility (maximum and minimum prestressing steel) constraints*

The maximum prestressing steel constraint for the composite section is given in Eq.(19) and the constraint which limit the minimum value of reinforcement is shown in Eq.(20).

$$0 \le w \le w_u \quad (19) \qquad 1.2 \le M_{cr}^* \le \varphi M_n \quad (20)$$

Where, w = Reinforcement index and w_u = Upper limit to reinforcement index = $0.36\beta_1$; M_{cr}^* = Cracking Moment.

2.4.4 *Ultimate and horizontal shear strength constraints*

The ultimate shear strength is considered at two sections, section at the end of transition zone and section where the prestress is maximum and the related implicit constraint on is defined as,

$$\varphi V_s = (V_u - \varphi V_c) \le 0.666\sqrt{f'_c} W_w d_s \quad (21)$$

where, V_u = ultimate shear strength, V_c = the concrete contribution taken as lesser of flexural shear, V_{ci} and web shear, V_{cw},, V_s = shear carried by the steel in kN. These two shear capacity are determined according to AASHTO specification.

The constraint for horizontal shear for composite section is considered as:

$$V_u \le \varphi V_{nh} \quad (22)$$

where V_{nh} = nominal horizontal shear strength.

2.4.5 *Deflection constraint*

Deflection due to live load (AISC Mkt 1986) is calculated as Eq. (23) and The live load deflection constraint is as Eq. (24).

$$\Delta_{LL} = \frac{324}{E_c I_c} P_T (L^3 - 555L + 4780) \quad (23) \qquad \Delta_{LL} \le L/800 \quad (24)$$

2.4.6 *End section tendon eccentricity constraint:*

Eccentricity of tendons at the end section becomes a constraint because eccentricity has to remain within the kern distances of the section to avoid extreme fiber tension both at initial stage and at final stage. The following constraint limits the tendon eccentricity at end section so that the eccentricity remains within the kern distances,

$$\frac{G_d}{6} + 0.25\sqrt{f_{ci}^{'}}\frac{A_4 G_d}{6F_{4i}} \le e_4 \le \frac{G_d}{6} + 0.5\sqrt{f_c^{'}}\frac{A_4 G_d}{6F_{4e}} \quad (25)$$

2.4.7 *Lateral stability constraint:*

The following constraint according to PCI (PCI 2003) limits the safety and stability during lifting of long girder subject to roll about weak axis,

$$FS_C = \frac{1}{\frac{Z_o}{y_r} + \frac{\theta_i}{\theta_{\max}}} \ge 1.5 \quad (26)$$

where FS_c = factor of safety against cracking of top flange when the girder hangs from lifting loop.

2.4.8 *Deck slab constraints:*

The constraint considered for deck slab thickness according to design criteria of ODOT (ODOT 2000) is shown in Eq.(27) and the constraint which limit the required effective depth for deck slab is shown in Eq.(28).

$$t \ge \frac{S_d + 17}{3} \quad (27) \qquad d_{min} \le d_{req} \le d_{prov} \quad (28)$$

where, S_d = effective slab span in feet = $S\text{-}TF_w/2$; t = slab thickness in inch.

3 OPTIMIZATION METHOD

In the present optimization problem a large number of design variables and constraints are associated. The design variables are classified as combination of continuous, discrete and integer variables. Expressions for the objective function and the constraints are non linear functions of these design variables. So the optimal design problem becomes highly nonlinear and non-convex having multiple local minima which requires an optimization method to derive the global optimum. As a result the global optimization algorithm named EVOP (Ghani 1989) is used. It has the capability to locate directly with high probability the global minimum. It is also capable to deal with possible finite number of discontinuities in the nonlinear objective and constraining functions. It has the ability to minimize directly an objective function without requiring information on gradient or subgradient. It can also deal with objective functions having combination of integer, discrete and continuous variables as arguments. There is no requirement for scaling of objective and constraining functions. It has the capability for optimization even when there are more than one of the above difficulties simultaneously present. It has facility for automatic restarts to check whether the previously obtained minimum is the global minimum. The procedure EVOP has successfully minimized a large number of internationally recognized test problems (Ghani 1995). The problems were categorized as unconstrained, constrained, multiple minima and mixed variable problems.

The algorithm can minimize an objective function

$$F(X) = F(x_1, x_2 \ldots x_n) \tag{29}$$

where, $F(x)$ is a function of n independent variables $(x_1, x_2 \ldots x_n)$. The n independent variables x_i's $(i = 1, 2 \ldots n)$ are subject to explicit constraints

$$l_i \leq x_i \leq u_i \tag{30}$$

Where, l_i's and u_i's are lower and upper limits on the variables. They are either constants or functions of n independent variables (movable boundaries). These n independent variables x_i's are also subject to m numbers of implicit constraints

$$L_j \leq f_j(x_1, x_2 \ldots x_n) \leq U_j \tag{31}$$

Where, $j = 1, 2 \ldots m$. L_j's and U_j's are lower and upper limits on the m implicit constraints. They are either constants or functions of n independent variables.

The Procedure

The method is subdivided into six fundamental processes (Fig.10) which are fully described in the reference (Ghani 1989). They are,

1. Generation of a 'complex',
2. Selection of a 'complex' vertex for penalization,
3. Testing for collapse of a 'complex',
4. Dealing with a collapsed 'complex',
5. Movement of a 'complex' and
6. Convergence tests.

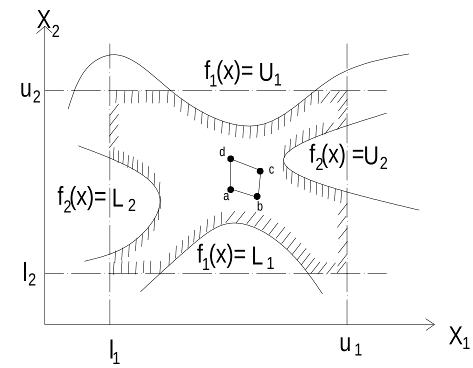


Figure 9. A "complex" with four vertices

A 'complex' is a 'living' object spanning an n-dimensional space defined by k ≥ (n+1) vertices inside the feasible region. It has the intelligence to move towards a minimum located on the boundary or inside the allowed space. It can rapidly change its shape and size for negotiating difficult terrain. Fig.9 shows a 'complex' with four vertices in a two dimensional parameter space. The 'complex' vertices are identified by lower case letters 'a', 'b', 'c' and 'd' in an ascending order of function values, i.e. f(a)<f(b)<f(c)<f(d). Straight line parallel to the co-ordinate axes are explicit constraints with fixed upper and lower limits. The curved lines represent implicit constraints set to either upper or lower limits. The hatched area is the two dimensional feasible search spaces.

The algorithm EVOP requires three user written functions the objective function, the explicit constraint function and implicit constraint function, some user input control parameters and a starting point inside the feasible space. Given the coordinates of a feasible point in an N-dimensional space the objective function calculates the functional value. Explicit constraint function evaluates the upper and the lower limits of the explicit constraints. Implicit constraint function evaluates the implicit constraints values and their upper and lower limits. The input control parameters of EVOP with their default values and ranges are, $\alpha = 1.2$ (1.0 to 2.0); $\beta = 0.5$ (0 to 1.0); $\Delta = 10^{-12}$; $\gamma = 2.0$ (greater than 1.0 to upwards), $\Phi = 10^{-14}$ (10^{-16} to 10^{-8}) ($\Phi = 10^{-12}$ will yield higher accuracy for convergence compared to $\Phi = 10^{-14}$) and $\Phi_{cpx} = 10^{-9}$ (10^{-16} to 10^{-8}).

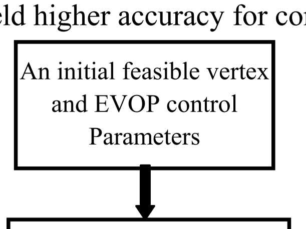


Figure 10. General outline of EVOP Algorithm

A computer program coded in C++ is used to input control parameters and to define three functions: an objective function, an explicit constraint function and an implicit constraint function. First the values of the control parameters are assigned with their default values and other input parameters are set to specific numerical values. These other input parameters for the present optimization problem are: number of complex vertices, K = 15; maximum number of times the three functions can be collectively called, limit = 100000; dimension of the design variable space, N = 14; number of implicit constraint, NIC = 46 and number of EVOP restart, NRSTRT = 10. NRSTRT is the number of automatic restart of EVOP to check that the previously obtained value is the global minimum. If NRSTRT = 5, the EVOP program will execute 5 times. For first time execution a starting point of the complex inside the feasible space has to be given. For further restart the complex is generated taking the coordinates of the previous minimum (values obtained from previous execution of EVOP) as the starting point of the complex.The EVOP Algorithm is called. Next suitable values of the control parameters are obtained by varying the parameters within the range sequentially and setting Φ to highest value that would still yield convergence and number of function evaluation becomes lowest with least function value. The program is rerun using optimum design variables obtained previously as starting point with same values of control parameters and checked whether a better minimum is obtained.

4 RESULT AND DISCUSSION

In this section, an example is presented to demonstrate the practical application of the approach presented in this thesis paper. The present example is a real life project named “Teesta Bridge” which is to be built in northern Bangladesh. It is a prestressed concrete I Girder Bridge of medium span (50 m) made composite with the cast in situ deck slab (BRTC 2007). The input constant design parameters used are summarized in Table 4. The cost data for materials, labor, fabrication and installation used for the optimum design are same as that for the existing design. The cost data are obtained from RHD cost schedule (RHD 2006). The comparative values of the design variables, cost of the existing design and the cost optimum design are presented in Table 5. The cross-sections showing the values of design variables of the bridge are given in Fig. 11. It can be seen from Table 5 that the cost optimum design produces the optimal I-girder bridge system configuration that yields the least overall cost and is 35% more economical than the existing design. There are significant differences in almost all of the design variables between the two designs. Girder spacing is greater in the optimum design so the number of girders in the bridge obtained in the optimal design is less than that of the existing design (Fig. 11). In the cost optimum design girder depth, top flange width, bottom flange thickness and slab thickness are comparatively greater and top flange thickness, bottom flange width, web width, prestressing steel and deck slab reinforcement are lesser than the existing design.

Table 4. Constant design parameters

Category	Parameter
Relative Cost Data:	UP_{GC} = 12,500 BDT per m^3 UP_{GF} = 415 BDT per m^2 UP_{DC} = 6,000 BDT per m^3 UP_{DF} = 400 BDT per m^2 UP_{PS} = 90,000 BDT per ton UP_{ANC} = 4,500 BDT per set UP_{SH} = 90 BDT per lin. meter UP_{OS} = 45,000 BDT per ton
Material properties:	Ultimate Strength of Prestressing steel, f_{pu} = 1861 MPa; Yield strength of ordinary steel, f_y = 410 MPa; Girder concrete strength, f'_c=40 MPa; Girder initial concrete strength f'_{ci} = 30 MPa Deck slab concrete strength, f'_{cdeck} = 25 MPa
Bridge design data:	Girder Length = 50 m (L = 48.8 m) Bridge Width, B_W = 12.0 m (3 Lane) Live Load= HS20-44 No of diaphragm = 4 Diaphragm width = 250 mm Wearing coarse = 50 mm Curb height = 600 mm Curb width = 450 mm
General Constant:	7 wire low-relaxation strand. Freyssinet anchorage system.

Table 5. Existing design and Cost optimum design

Design Variables	Existing Design	Optimum Design
Girder spacing (S)(m)	2.4	3.0
Girder depth (G_d)(mm)	2500	2700
Top flange width (TF_w) (mm)	1060	1250
Top flange thickness (TF_t) (mm)	130	75
Top Flange transition width (mm)	270	500
Top flange transition thickness(mm)	75	50
Top Flange Haunch width (mm)	150	50
Top Flange Haunch thickness (mm)	150	50
Web width (W_w) (mm)	220	150
Bottom flange width (BF_w) (mm)	710	360
Bottom flange thickness (BF_t) (mm)	200	250
Bottom Flange transition width (mm)	245	105
Bottom Flange transition thickness(mm)	250	52.5
Number of strands per tendon (N_s)	12 (0.5"dia)	8(0.6"dia)
Number of tendon per girder (N_T)	7	6
Lowermost tendon position (y_t)	400	930
initial stage prestress (η)	42.8%	53%
Slab thickness (t) (mm)	187.5	210
Slab main reinforcement ratio (ρ)	0.82%	0.63%
Total cost per square meter of deck slab (BDT)	12,250	7,925
% SAVING = $\frac{12,250-7,925}{12,250}\times 100$	35.0%	

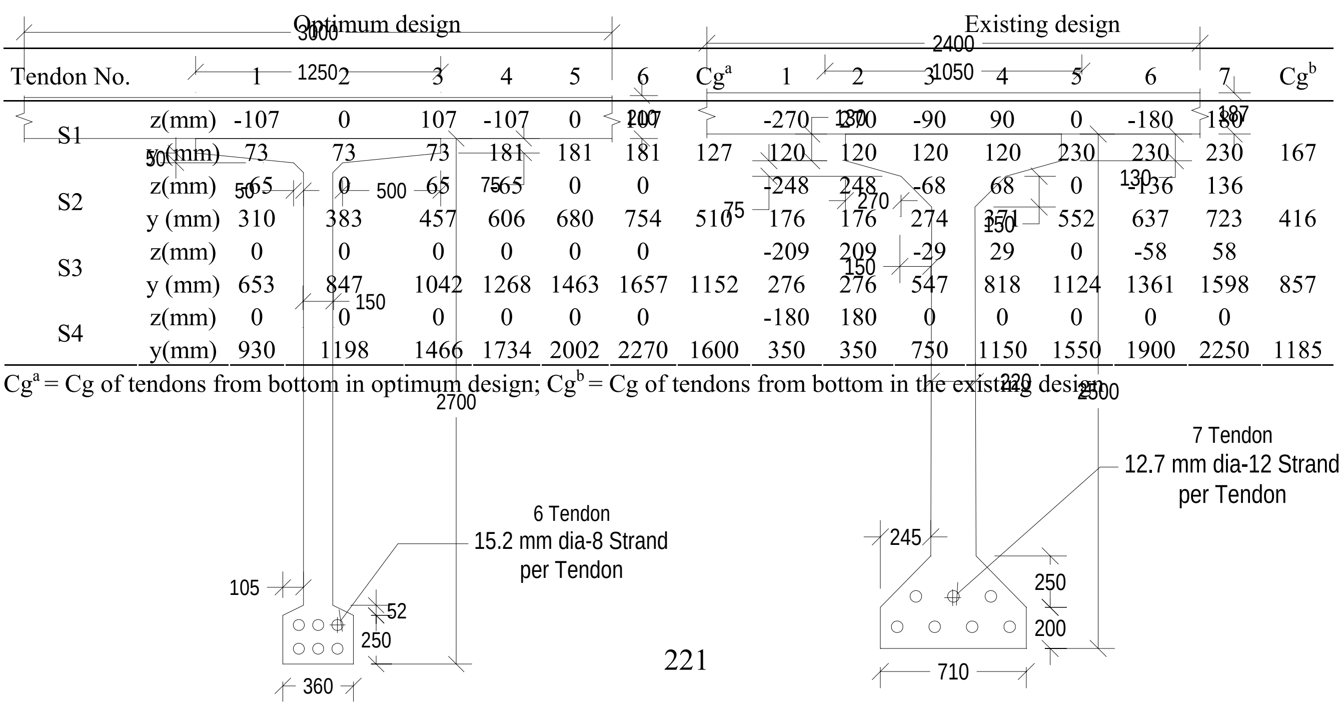
Tendons arrangements are also tabulated in Table 6. In the optimum design c.g. of tendons from bottom at various sections along the girder are significantly different and more than the existing design which indicates that consideration of tendons arrangement as a design variable is important because it affects prestress losses and flexural stress at various sections along the girder to a great extent.

The optimization problem with 14 mixed type design variables and 46 implicit constraints converges with just 459 number of function evaluations. Intel COREi3 processor has been used in this study and computational time required for optimization by EVOP is about only 2 seconds which indicates the design becomes fully automated.

Table 6. Tendon configuration in optimum and existing design

		Optimum design							Existing design							
Tendon No.		1	2	3	4	5	6	Cg^a	1	2	3	4	5	6	7	Cg^b
S1	z(mm)	-107	0	107	-107	0	107		-270	180	-90	90	0	-180	180	
	y (mm)	73	73	73	181	181	181	127	120	120	120	120	230	230	230	167
S2	z(mm)	-65	0	65	-65	0	0		-248	248	-68	68	0	-136	136	
	y (mm)	310	383	457	606	680	754	510	176	176	274	371	552	637	723	416
S3	z(mm)	0	0	0	0	0	0		-209	209	-29	29	0	-58	58	
	y (mm)	653	847	1042	1268	1463	1657	1152	276	276	547	818	1124	1361	1598	857
S4	z(mm)	0	0	0	0	0	0		-180	180	0	0	0	0	0	
	y(mm)	930	1198	1466	1734	2002	2270	1600	350	350	750	1150	1550	1900	2250	1185

Cg^a = Cg of tendons from bottom in optimum design; Cg^b = Cg of tendons from bottom in the existing design



(a) The optimum design (b) The existing design

Figure 11. Cross-section of bridge superstructure for the optimum design (a) obtained in this study and for the existing design (b). (All dimensions are in mm)

The present study commenced with an aim to achieve the cost minimization of the design of post-tensioned prestressed concrete I-girder bridge system by adopting an optimization approach to obtain the optimum design. To achieve the objectives a cost optimum design of a simply supported post-tensioned prestressed concrete I-girder bridge system is performed. A global optimization algorithm named EVOP (Evolutionary Operation) is used which is capable of locating directly with high probability the global minimum. A program is developed for the optimization which may be beneficial to designers and contractors interested in cost optimization to the design of I girder bridge system. The proposed cost optimum design approach is applied on a real life project (Teesta Bridge, Bangladesh) which shows that a considerable cost saving while resulting in feasible and acceptable optimum design. Following conclusions can be made from the study:

1. The obtained design obtained by the optimization approach is 35% more economical than the existing design of the real life project. Girder spacing is greater in the optimum design so the number of girders in the bridge obtained in the optimal design is less than that of the existing design. In the cost optimum design girder depth, top flange width, bottom flange thickness and slab thickness are comparatively greater and top flange thickness, bottom flange width, web width, prestressing steel and deck slab reinforcement are lesser than the existing design.
2. It is difficult to solve the present constrained global optimization problems of 14 numbers of mixed integers, discrete design variable and a large number of implicit constraints by using gradient based optimization methods, where as it can be easily solved with EVOP with a relatively small number of function evaluations by simply adjusting the EVOP control parameters.

It is recommended that the study be further extended to perform various parametric studies for the constant design parameters of the bridge system to observe the effects of such parameters on the optimum design.

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