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Cost optimization of segmental precast concrete bridges superstructure using genetic algorithm

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Abstract. The construction of segmental precast concrete bridge is an increase due to its superior performance and economic advantages. This type of bridge is appropriate for spans within 30 to 150 m (100 to 500 ft), known as mega-projects and the design optimization would lead to considerable economic benefits. A box-girder cross section superstructure of balanced cantilever construction method is assessed here. The depth of cross section, (variable along the span linearly), bottom flange thickness, and the count of strands are considered as design variables. The optimum design is characterized by geometry, serviceability, ductility, and ultimate limit states specified by AASHTO. Genetic algorithm (GA) is applied in two fronts: as to the saving in construction cost 8% and as to concrete volume 6%. The sensitivity analysis is run by considering different parameters like span/depth ratio, relation between superstructure cost, span length and concrete compressive strength.

Keywords: segmental precast posttensioned concrete bridge; box girder; optimization; genetic algorithm

1. Introduction

There exist a great number of numerical optimization algorithms regarding the basis of the simulation of a natural phenomenon, illustrating the significant increase in the development of non-deterministic search techniques (Hasançebi *et al*. 2010). Such algorithms do not need gradient calculating and initial point, while are capable of presenting reliable response required for desirable periods (Kaveh *et al*. 2016). According to the available literature, many studies regarding optimization problems are focused on solving small scale problems. During past decades, optimization methods are applied to solving large scale and complex structural problems (Kaveh *et al*. 2016, Hasançebi *et al*. 2011). Due to the numerous design variables and constraints in bridge design, the formulation of such optimization problems is relatively complex. Therefore, the optimization of bridges, especially the prestressed concrete, has been of less concern compared to other structures.

Due to their high durability and economically advantageous features, prestressed concrete bridges, especially those with post-tensioning box girder, are of shown promising practical applications (Podolny 1979). Because of the different variables affecting the design of such bridges, various design results for a definite span length and deck width are determined in a sense that selecting the most beneficial design with the assistant of conventional methods is not a simple task. Hence, a proper optimization technique is required in order to achieve the desired design results.

Optimization of prestressed bridges is initiated by Torres *et al*. (1966) where the construction costs applying linear planning method is minimized. But in recent decades, the meta-heuristic optimization methods like genetic algorithm (GA), neural networks, particle swarm and ant colony optimizations have been and are of major concern. The optimization of overall costs of prestressed concrete bridges of I-beam is assessed by Sirca and Adeli (2005), who applied a neural dynamics model in order to solve the optimization problem. A study on the cost optimization of bridges with I-section precast pretensioned concrete girder by the means of topology optimization where genetic algorithm is applied is run by Aydin and Ayvaz (2010). In this study, the construction cost of girders including both reinforced and prestressed concrete is considered as the optimization criteria. In a similar study, the optimization of the overall construction costs of the prestressed concrete bridges is of concern by Aydin and Ayvaz (2013), where the results indicate 12.6% reduction in construction costs. The optimum cost for the construction of both reinforced and prestressed concrete beams through GA is proposed by Alqedra *et al*.(2011), where the discrete variables are applied in this model and the results indicate that with regards to span length, 16.7% to 27% reduction in the reinforced concrete beams costs and 17.8% to 29.8% of the save in prestressed concrete costs is evident. An

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optimization of continuous post-tensioned concrete box girders with variable depth by applying gradient and confined linear programming methods is introduced by Al-Osta *et al*. (2012), where an arrangement of both short and long tendons leads to the most beneficial design. The effect of the high-strength concrete in the optimization of bridges with post-tensioned box girder according to AASHTO LRFD (2007) specifications is assessed by Chang *et al*. (2012), where it is concluded that by applying high-strength concrete, it is possible to design bridges with longer span length and less prestressing strands count. The application of evolutionary operation for the optimum cost of two-span continuous prestressed concrete bridge with I-girder, based on AASHTO (2002) standard specifications is assessed by Ahsan *et al*. (2011), where a 36% reduction in the cost is evident. The optimization of the simple and spliced prestressed concrete girders of bridges where its life cycle cost is of concern is assessed by Madhkhan *et al*. (2013), who the design constraints are considered, therefore, a significant reduction in the overall cost is achieved. Size optimization of continuous multi-span composite steel girders with box-section where the cuckoo search algorithm is introduced by Kaveh *et al*. (2014), where as a consequence of this optimization, which is based on AASHTO (2002) standard specifications, the weight of deck is reduced by 15%. The prestressed concrete beams by the virtue of improved constrained differential evolution (ICDE) algorithm is optimized by Quaranta *et al*. (2014), where the preferable performance of this method in comparison to GA and particle swarm optimization methods is evident. The optimization of bridges with post-tensioned concrete girders by applying modified colliding bodies optimization (MCBO) algorithm of considering 17 variables and 101 implicit constraints according to AASHTO (2002) standard specifications is proposed by Kaveh *et al*. (2016).

An optimized prestressing and local reinforcement design for a mixed externally and internally precast prestressed segmental bridge is proposed by Xu *et al*. (2016), where the safety, economic efficiency and constructability for two different tendon layouts of these bridges erected by the span by span method are assessed which led to the optimal tendon layouts; it should be noted that cost optimization is not addressed in this study. A generalized formulation for optimization of concrete beam reinforced with glass fiber reinforced polymer bars is proposed by Rahman *et al*. (2017) who applied a simple GA with constraints based on ACI code specifications. Their results indicate a significant reduction in the volume of the component materials in comparison with the classical design solutions. A multi-parameter optimization technique is proposed by Gao *et al*. (2017) to achieve an optimum PC cable-stayed bridge design. In this technique, the number of prestressing tendons in girder, cable forces, cable areas and cross-section sizes of the girders and the towers are design variables. The optimum design could achieve a 24% cost saving, compared with the traditional design.

The optimum cost of partially prestressed concrete I crosssectioned beams design through GA is proposed by Turkeli *et al*. (2018), where 35~50 % cost reduction is evident in comparison with its own traditional design. An optimization and sensitivity analysis is run by Shariat *et al*. (2018), where the Lagrangian Multiplier Method (LMM) is applied on a rectangular reinforced concrete beams. Their objective is to achieve a minimum design cost as to the single and double RC beams where the specifications of three regulations of American Concrete Institute (ACI 318-14), British Standard (BS 8110), and Iranian concrete regulation are observed. The results indicate that the LMM could be applied in minimizing the single and double reinforced beams manufacturing cost with the different boundary conditions instead of applying complex optimization formulizations.

An optimization of Box Girder Bridge through GA Method is proposed by Morab and Fernandes (2018), where the PSC box girder of span 40m is applied. Here the loadings are subject to Indian Road Congress loadings (IRC: 6-2014) prestressed concrete code (IS: 1343-2012) and IRC: 18-2000 specifications. The limit state method is applied here and MATLAB software is used for optimization process through GA.

GA is applied by Baradaran and Madhkhan (2019) to optimize the mega bracing system configuration in steel frames, who obtained the optimum angle of mega bracing in steel frames. The results indicate that simultaneous utilization of various genetic operators leads to an increase in convergence rate of optimum frame weight as well as reduction of computations.

The economic optimization of high-performance post-tensioned concrete box girder pedestrian bridges is assessed by Yepes *et al.* (2019), where 33 discrete design variables that define the geometry, the concrete, the reinforcing steel bars and the post-tensioned steel are concern. Different acceptance criteria are proposed in modifying a variant of the simulated annealing algorithm with a neighborhood move based on the mutation operator of GA. The obtained results indicate that the cost of high-strength concrete decreases by 4.5% and the concrete volume by 26%. A geometrical structural optimization study for deck concrete arch bridges through GA is presented by AbdElrehim *et al.* (2019), where 30~35 % cost reduction is evident in comparison with traditional designs.

As observed above, the optimization of segmental precast concrete bridges is not comprehensively studied yet. Attempt is made in this study to introduce an optimization approach in the context. The considered bridge is a continuous three-span bridge with post-tensioned concrete box girder. The cross section of the girder varies linearly along the bridge spans. The segments of the bridge are assumed to be constructed and installed according to the balanced cantilever method. The metaheuristic algorithm of choice is the Genetic Algorithm (GA) (Holland 1975). Various span lengths and compressive strength of concrete are of concern for this optimization. A sensitivity analysis is run on the span/depth ratio, the relation between superstructure cost, span length, concrete compressive strength and superstructure weight.

2. Formulation

2.1 Objective function

The objective of this study is to optimize the cost of bridge superstructure construction. Precast concrete

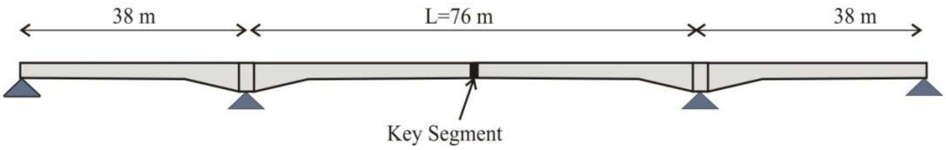

Fig. 1 Longitudinal section of the studied bridge

segments and prestressing steel cost are constitute two major parts which should be of concern, while can be mathematically expressed as follows

$$C_T = C_{PC} + C_{PS} \quad (1)$$

where, CPC and CPS are the costs of precast concrete segments and prestressing steel, respectively, and are calculated through

$$C_{PC} = UP_{PC} \times V_{PC} \quad (2)$$

$$C_{PS} = UP_{PS} \times W_{PS} \quad (3)$$

where, UP_{PC} is the material, construction and installation cost of precast concrete segments per volume, UP_{PS} is the V_{PC} material and construction cost of prestressing steel per weight, is the total volume of concrete, and W_{PS} is the total weight of prestressing steel. UP_{PC} and UP_{PS} are estimated at 950 (USD/m^3) and 9500 (USD/ton). These values are based on local assessments with regards to the concrete of 40 MPa (5.8 ksi) compressive strength. However, the unit prices for concrete and steel are in accordance with those in the literature (Aydın and Ayvaz 2010, Aydın and Ayvaz 2013).

The optimal design should satisfy the geometry, serviceability, ductility, and ultimate limit states requirements. In order to observe the available constraints, the following external penalty approach is applied

$$C_T = C_{PC} + C_{PS} + Penalty \quad (4)$$

where, Penalty is the constraint violation function expressed as

$$Penalty = \alpha_p \sum_{i=1}^{n_g} [g_j]^q \quad (5)$$

where, α_p is a constant penalty coefficient, q is a non-negative coefficient, n_g is the count of problem constraints, and g_j is calculated through Eq. 6, where if g_j violates the constraint it should be considered the same as the constraint, otherwise it should be assumed zero

$$if\ g_j \leq 0 \Rightarrow [g_j] = max\{0, g_j\} \quad (6)$$

2.2 Design variables

The cross-sectional dimensions of box girder and the prestressing strands count are considered as the design variables in this study and tabulated in Table 1. A longitudinal section of this studied bridge is illustrated in Fig.1. The bridge is a symmetric three-span bridge and the depth of the cross section varies linearly from the two interior supports to the quarter-span. A typical cross section of the deck bridge and the design variables are shown in Fig. 2. It should be mentioned that except for the design variables including D, D_0, t, and t_0, other parameters of the bridge cross section like the slope of the web are assumed constant.

Table 1 Design variables

No.	Variable	Symbol	Type	Constraints
1	Girder depth in pier (m)	D_0	Continuous	$2.17 \leq D_0 \leq 5.0$
2	Girder depth in mid-span (m)	D	Continuous	$2.17 \leq D \leq 5.0$
3	Bottom slab thickness in pier (mm)	t_0	Continuous	$180 \leq t_0 \leq 500$
4	Bottom slab thickness in mid-span (mm)	t	Continuous	$180 \leq t \leq 500$
5	Number of strands per tendon	n	Discrete	$6 \leq n \leq 20$

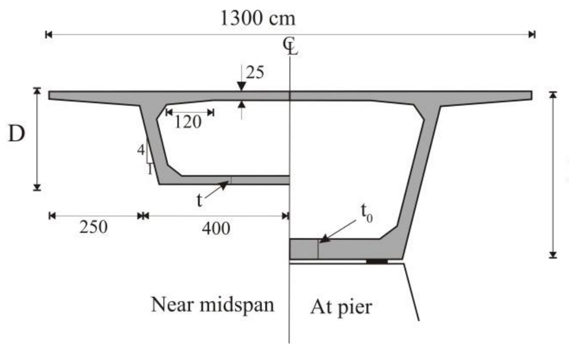

Fig. 2 Typical cross-section of deck bridge

2.3 Constant parameters

These parameters consist of: span length, deck width, post-tensioned anchorage system, live loads according to AASHTO (2002), superimposed dead loads, and material properties, Table 2. Class-C anchorage system and 7-wire strands with low relaxation having 12.7 mm (0.5 in) diameter are the post-tensioning tendons.

In this study, the concrete compressive strength is constant, equal to 40 MPa (5.8 ksi). AASHTO HS20-44 (2002) live load including truck and lane load is applied on three lanes. The impact factor applied to the live load is calculated as

$$Impact\ Factor = 1.3 - 0.005\ L \quad (7)$$

where, L is the span length (m).

Although the prestressing strands count varies, their

Table 2 Constant design parameters

Constant parameter	Values
Span length (L)	50, 60 ,66, 76, 83, 93, 100, 110, and 120 (m)
Deck width (W)	13 (m)
The tensile strength of prestressing steel (f_{pu})	1860 (MPa)
The yield strength of prestressing steel (f_y)	0.9 f_{pu}
The yield strength of reinforcement steel (f_y)	400 (MPa)
Unit weight of concrete	2.4 (kN/m^3)
Unit weight of Steel	78.5 (kN/m^3)
Modulus of elasticity of concrete	$4700\sqrt{f_c'}$
Modulus of elasticity of prestressing steel	1.93×10^5 (MPa)
Modulus of elasticity of reinforcement steel	2×10^5 (MPa)
Live loads	HS20-44 (Truck and lane load)
Design traffic lane width	3 (m)
Barrier load	5 (kN/m)
The thickness of asphalt wearing surface	70 (mm)

grouping and layouts are considered conventionally similar to that of (PCI 1984, Lacey and Breen 1975, Heins and Lawrie 1984, Duan and Chen 2014):

Group 1: cantilever tendons including 38 tendons (19 tendons in each web for the cantilever construction).

Group 2: continuity tendons in tail spans including two tendons, one tendon in each web

Group 3: continuity tendons in center span including two types of tendons; 3a and 3b. Group 3a consists of 8 tendons (4 in each web) and Group 3b consists of 4 tendons, located in the top slab. The strand count in each 3b tendon is half of other tendons count. The longitudinal layout of tendons is shown in Figs. 3-5. With respect to the constant cross-sectional area of strands, the prestressing force of each tendon (F_{pi}) only depends on the strand count in each tendon. Prestressing bending moment of each tendon groups (M_{pi}) at any section of superstructure is calculated by multiplying prestressing force into its eccentricity.

2.4 Design constraints

The design constraints are based on AASHTO (2002) standard specifications, like allowable compressive and tensile stress in all segment installation, completion, operational phases, deflection and geometrical constraints.

2.4.1 Allowable stress constraints

In accordance with AASHTO (2002) standard specifications (Article 9.15.2.2), the allowable stress constraint at the top and bottom fibers of the sections in different construction phases is applied as

$$-0.4 f_c' \le f \le 0.5\sqrt{f_c'} \tag{8}$$

where, f is the stress at any point of the section and f_c' is concrete compressive strength (MPa).

Stresses are calculated and controlled in each of the following Phases:

Phase 1. Cantilever installation of the segments and applying the group1 posttensioning.

Phase 2. Completion of tail spans by posttensioning of tendons group 2.

Phase 3. Completion of center span: in this phase, the key segment is constructed by applying cast-in-place concrete thus, the left and right-sides of cantilever segments of the center span are connected to each other and, afterwards, 3a and 3b post-tensioning tendons are installed, by which the span construction is completed.

Phase 4. Superimposed dead loads: after post-tensioning of all prestressing tendons, the superimposed dead loads like asphalt and railing loads could be applied.

Phase 5. Application of live load

The eight critical sections of the mid-bridge and numbering of the sections with respect to the order of the balanced cantilever installation of segments are shown in Fig. 6. The ID numbers of the segments are attributed according to the number of joints between the segments and orientation of each segment with respect to section zero. As to the symmetric configuration of bridge and applied loadings, stresses defined in previous phases are calculated in different sections. In each phase, 32 allowable tensile and compressive strength constraints, that is, 160 constraints constitute the five phases.

2.4.2 Ultimate flexural strength constraints

These constraints based on ultimate strength design method (Article 9.17.2) are determined in different sections

$$M_u \le \phi M_n \tag{9}$$

where, M_u is the available bending moment, M_n is the nominal flexural strength of the section and ϕ is the strength reduction factor, equal to 0.9 according to AASHTO (2002) standard specifications.

2.4.3 Ductility constraints

The minimum flexural prestressing steel in the critical cross sections is determined through the following

$$1.2M_{cr} \le \phi M_n \tag{10}$$

where, M_{cr} and ϕM_n are the cracking and ultimate bending moments, respectively.

This design is carried subject to the under-reinforced conditions to provide ductile failure. For this purpose, according to AASHTO (2002) standard specifications, reinforcement index should not exceed 0.36β. The maximum prestressing steel in different sections is defined as

$$\omega \le 0.36\beta_1 \tag{11}$$

where, ω is the reinforcing index and β_1 is the concrete compressive strength factor in case of compressive strength at 40 MPa (5.8 ksi) is 0.75 according to AASHTO (2002) standard specifications.

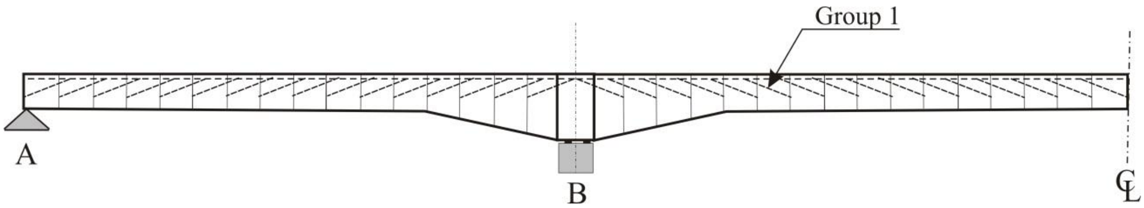

Fig. 3 Group1: Cantilever tendon layout

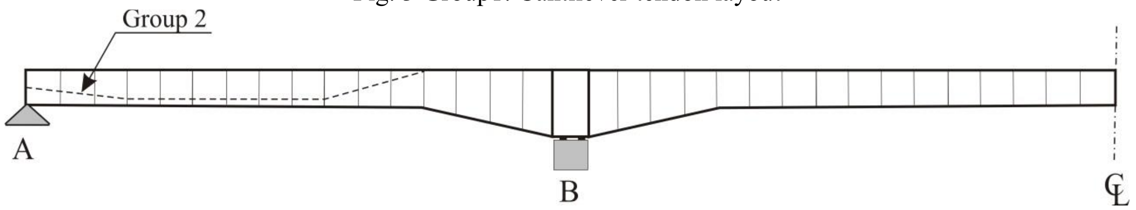

Fig. 4 Group2: Tail span continuity tendons

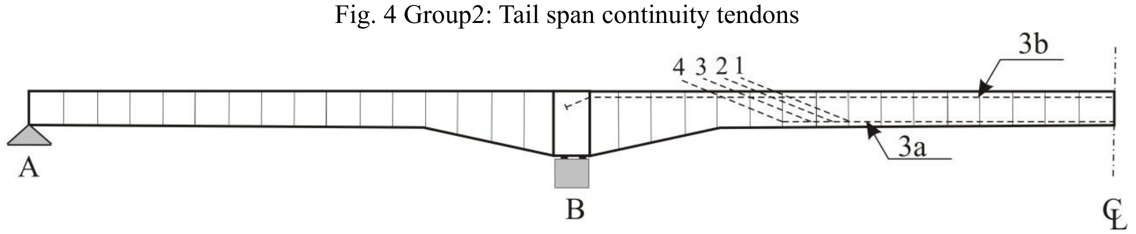

Fig. 5 Group3: Center span continuity tendons

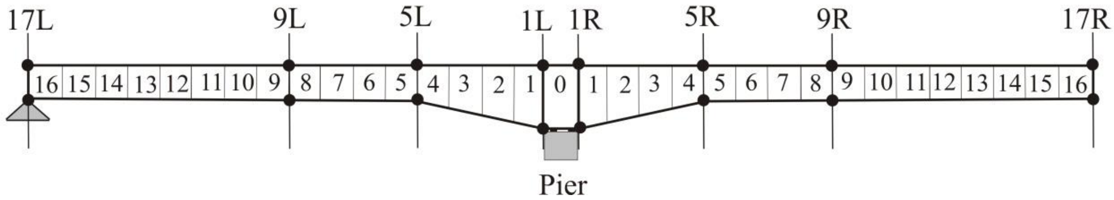

Fig. 6 Installation sequence of segments and critical sections

2.4.4 Serviceability constraints

According to Article 9.11.3.1, the deflection due to live load shall not exceed 1/800 span length (*L*). Deflection is calculated by considering the maximum bending moment at the midpoint of the center span subject to live load, expressed as

$$\Delta \leq \frac{L}{800} \tag{12}$$

where, Δ is the deflection at mid-span and L is the center span length**.**

2.4.5 Shear constraints

Shear forces should be limited by the allowable shear as follows

$$V_u \leq \phi V_n \tag{13}$$

where, V_u is the ultimate shear force, φ is the shear strength reduction factor equal to 0.85, according to AASHTO (2002) standard specifications, and V_n is the nominal shear strength of the cross section. It is notable that the nominal shear strength is the sum of the shear strength of concrete (V_c) and the transversal steels (V_s). If the shear strength lack is observed in a section, transversal reinforcement is applied, where the shear constraints based on maximum allowable transverse reinforcement would be expressed as

$$V_s = \frac{V_u}{\varphi} - V_c \leq 0.67\sqrt{f_c'}bd \tag{14}$$

$$V_c = min\{V_{ci}, V_{cw}\} \tag{15}$$

where, *b* is the total width of the webs, *d* is distance from extreme compressive fiber to centroid of the prestressing force, V_{ci} and V_{cw} are the nominal shear strength of the concrete based on the bending-shear cracking and shear cracking of the prestressed beam section, respectively.

2.4.6 Geometry constraints

According to AASHTO (2002) standard specifications (Articles 9.9.1 and 9.9.2) the minimum top flange thickness should be 1/30 of the clear distance between fillets or webs but not less than 150 mm (6 in). This constraint is applicable for the bottom flange, given the difference where the minimum allowable thickness is 140 mm (5.5 in). By considering these specifications into account, the top flange thickness is taken constant, equal to 250 mm (9.8 in), while the bottom flange thickness varies along the span with a minimum value of 180 mm (7.1 in).

To the best knowledge of the authors, there exists no limitation on the depth of the box girder specified in AASHTO (2002) standard specifications, therefore, a single cell box would be preferably practical when depth to width ratio is equal to or greater than 1/6 according to AASHTO LRFD (2017) specifications (Article 5.12.5.3.11d); consequently, the minimum depth of the girder section is 2170 mm (85.4 in), which is equal to 1/6 width of the top flange is applied here.

3. Optimization algorithm

In addition to the direct mathematical methods, there exist a great number of other methods for optimization problems categorized as approximation, probabilistic, and metaheuristic algorithms. The focus of metaheuristics is on the combination of a heuristic concept with a mathematical planning method. In this study, the GA is applied for optimization purposes. GA is an evolutionary optimization method where the principle of Darwin's natural selection theory is applied (Holland, 1975). GA begins with the introduction of a count of initial solutions, which are first randomly selected within defined scopes of design variables and the next, sorted according to their fitness. The fitness of each solution is determined by the proximity to the optimal solution. In GA, fittest solutions have more chance to become combined and reproduced.

3.1 Genetic operators

In GAs, during the reproductive stage, the following genetic operators are applied with the impact of which on a population, the next generation of that population is produced:

- Reproduction operator, the selection
- Mating operator, the crossover
- Mutation operator

3.1.1 Selection

Based on the theory of the survival of the best, the best should be selected to generate a better next generation. For this reason, this operator is named the selection. This operator selects a number of chromosomes from a population for reproduction.

3.1.2 Crossover

The most important operator in the GA is the crossover operator consists of a process where the old generations of the chromosomes are combined to generate a new generation. The couples considered at the selection stage as the parent, exchange their genes and generate new members. The crossover in the GA leads to the loss of genetic diversity or dispersion, by allowing each parent other to find good genes. This operator consists of three steps:

- Selecting two strings randomly
- Selecting the location for random action
- Replacing, the volume of the two strings

Among the many types of crossover, here the single point crossover is applied.

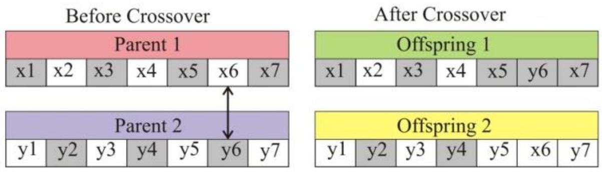

Fig. 7 Single point crossover

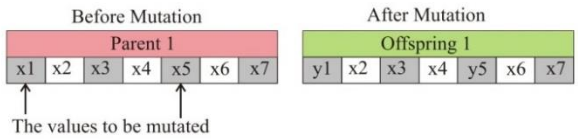

Fig. 8 Mutation

For this purpose, one point is selected randomly and the values of which are displaced, Fig. 7.

3.1.3 Mutation

Mutation is a phenomenon in genetic science that rarely occurs in some chromosomes during which the children are endowed with characteristics that are not owned by any of their parents. The contribution of the mutation in the GA is to restore the genetic material lost or not found in the given population, to avoid early convergence of the algorithm to local optimal solutions. In the binary mutations, some genes are randomly selected and converted into zero and one, vice versa. One of the mutation methods is as follows: with a number lower than one, named the probability of mutation, a random number is recalled for each gene in a population; if this random number is less than the probability of mutation, gene mutation occurs, a rare phenomenon in nature. If the characters are continuous numbers, the mutation can occur in the form of positive or negative random variations around the preceding character, Fig. 8.

By applying the crossover and mutation operators, the initial solutions are improved in order to produce new solutions with greater fitness (in this case the minimum cost (or weight) of the superstructure). These new solutions replace the older improper solutions. The above process is repeated until the stop criterion (the convergence or the count of iterations) is met. The overall flowchart of the GA based optimization procedure is shown in Fig. 9.

4. Results and discussion

4.1 Optimization results

After the formulation of bridge analysis based on design variables, it is verified through manual calculations for a practical example designed by conventional design procedure. GA is coded in MATLAB and is added to the analysis and design codes. The optimization algorithm is run independently for 3 times. The convergence history of the penalized objective function for the best run is shown in Fig. 10, where, the consecutive step-like movements demonstrate the efficiency of GA in optimization. GA needs

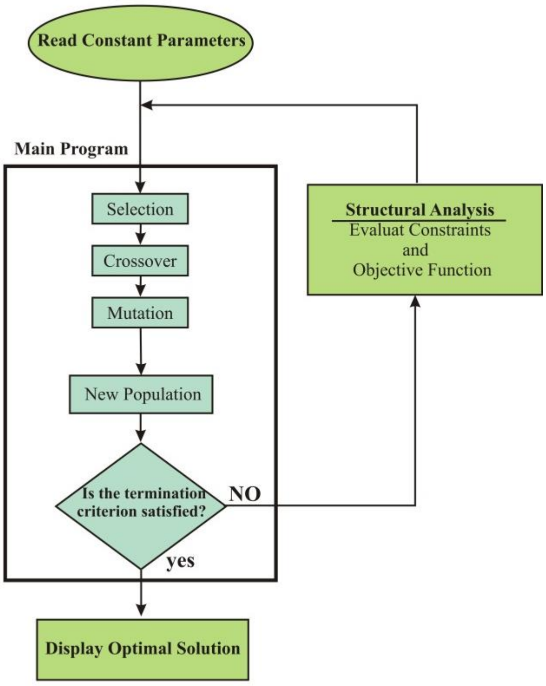

Fig. 9 General flowchart of optimization algorithm

Table 3 Optimum values of design variables and cost

Design	*D (m)	D_0 (m)	t (m)	t_0 (m)	n	*Weight* (kN)	*Cost* (USD)
Practical example	2.45	4.57	0.22	0.457	14	27330	1,417,400
GA based	3.13	4.56	0.18	0.18	10	27600	1,306,732
						Total cost saving	110,668

*D, D_0, t, t_0 and n are the design variables shown in Table 1.

90 iterations or 900 structural analyses to converge to the optimum design.

The optimum results of the best run are shown in Table 3. It should be noted that GA is fixed at the bottom flange thickness (t and t_0) at its lower bound, 180 mm (7.1 in), which is its lowest allowable value. This finding is in agreement with that of (Ahsan *et al*. 2011).

This proposed optimization procedure yields a total cost saving of \$110,668.00. The obtained results indicate an 8% reduction in the superstructure construction cost. Due to the high ratio of the unit price of prestressing steel in relation to concrete, this optimization reduces the required prestressing steel volume by 30%. In this context, the weight of the required concrete increases less than 1%, if more portions of the cost are assigned to concrete, then the results would have revealed a reduction in the required concrete volume.

The results of superstructure weight optimization are tabulated in Table 4, from the content of which, the optimization is obtained at 5.5% reduction in superstructure weight, while the prestressing steel volume is increased by 14 %.

A review of available studies in the context reveals a cost saving within 5-30% range subject to the girder shape (I-shape or box), the construction method (cast in place, precast, segmental) and the number of design variables in each prestressed bridge optimization.

There exists no record of segmental bridge optimization with balanced cantilever erection method. The low cost saving on segmental bridge optimization in this study is due to the fact that in this type of bridges, it is not possible to make a change in the longitudinal layouts of prestressing tendons, and this fact limits cost savings.

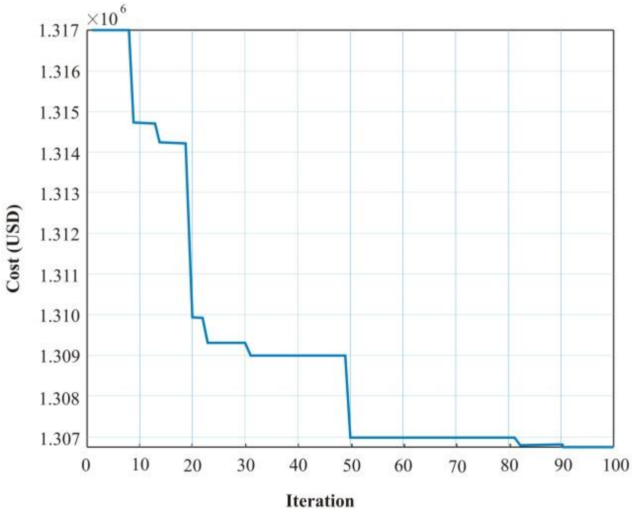

Fig. 10 Convergence curve recorded for the best run

4.2 Sensitivity analysis

In addition to the optimization, the sensitivity analysis, considering different parameters, is run and the results are presented as follows:

4.2.1 Span/depth ratio

The results of the optimization of superstructure costs for different span length varying from 50 to 120 m (164 to 394 ft) are tabulated in Table 5, where L/D_0 and L/D are the span/depth ratios at piers and mid-span, and *O.C* is the optimum cost of deck construction per m^2.The average of L/D_0 and L/D ratios are obtained as 17.7 and 24.7, respectively. The values for span/depth ratios are pointed out in Fig. 11.

The obtained results state that the values for span/depth ratio after optimization are within the range proposed by AASHTO LRFD (2017) specifications that is D_0/L is within 1/20 to 1/16 in all cases and D/L is within 1/28 to 1/24 in all cases. Moreover, the average values for D_0/L and D/L for different span length are approximately equal to 1/18 and 1/24, respectively, which are in accordance with AASHTO LRFD (2017) recommendations. In span length of 50 m (164 ft.), the variable value of D is equal to its lower bound. It is notable that the lower bound of the section depth is equal to the 1/6 of section width according to AASHTO LRFD (2017) specifications.

As observed in Table 5, for the spans equal to or less than 76 m (249 ft.), the thickness of bottom flange is obtained as the defined lower bound 180 mm (7.1 in). This result is due to the constraint type which directs the

Table 4 Optimum values of design variables and cost considering the weight as objective function

Design	*D (m)	D_0 (m)	t (m)	t_0 (m)	n	Weight (kN)	Cost (USD)
Practical example	2.45	4.57	0.22	0.457	14	27330	1,417,400
GA based	3.13	4.56	0.18	0.18	10	27600	1,306,732
				Reduction		1480	12,300

*D, D_0, t, t_0 and n are the design variables shown in Table 1

Table 5 The optimum designs for different span length

L (m)	D (m)	D_0 (m)	t (m)	t_0 (m)	n	L/D	L/D_0	$O.C$(USD/m^2)
50	2.17	2.67	0.18	0.18	6	22.81	18.54	559
60	2.31	3.16	0.18	0.18	8	25.71	18.80	593
66	2.54	3.78	0.18	0.18	9	25.98	17.46	635
76	3.13	4.56	0.18	0.18	10	24.25	16.64	662
83	3.18	4.3	0.20	0.32	12	25.94	19.19	703
93	3.8	5.62	0.20	0.38	13	24.32	16.44	739
100	4	5.24	0.22	0.46	15	24.75	18.89	794
110	4.43	6.26	0.27	0.48	17	24.58	17.40	860
120	4.99	7.25	0.30	0.68	20	23.81	16.39	933

optimization process. For the spans equal to or less than 76 m (249 ft.), the tensile strength at the top of the section at location 5*L* in the Fig. 6 is the active constraint. On the contrary, in the spans with length higher than 76 m (249 ft.), the compressive strength at the bottom of the section is the active constraint.

4.2.2 Span length effect on superstructure cost

The optimum cost of the superstructure with respect to the span length is shown in Fig. 12, where the optimum cost of the superstructure per unit area increases as the span length increase. The superstructure cost can be linearly estimated with respect to the span length with a correlation factor of 0.97963; for example, a 40 and 55 % increase in superstructure cost occurs when the span length changes from 50 to 100 m (164 to 328 ft.) and 60 to 120 m (197 to 394 ft.), respectively.

4.2.3 The Relation between superstructure cost and concrete compressive strength

Applying high strength concrete in segmental precast concrete bridge could minimize the superstructure geometry and girder weight, which could increase the construction speed (Jiang *et al.* 2016). In order to assess this relation, first, an estimation of the concrete price per unit volume with respect to its compressive strength is run through the following equation

$$U_{pc} = 830 + 3f'_c \tag{16}$$

where, U_{pc} is the reinforced concrete price per unit volume including material, construction, transportation, and installation costs, and f'_c is the compressive strength of concrete (MPa). According to AASHTO (2002) provisions,

Table 6 The optimum designs for different concrete strength

f'_c (MPa)	L (m)	D (m)	D_0 (m)	t (mm)	t_0 (mm)	A_{ps}	L/D	L/D_0	$O.C$ (USD/m^2)
25	76	3.56	4.89	23	47	9.87	21.3	15.5	690
30	76	3.23	4.57	20	34	10.86	23.5	16.6	678
35	76	2.9	4.54	20	24	10.86	26.2	16.7	668
40	76	3.13	4.56	18	18	9.87	24.2	16.6	662
45	76	2.61	4.04	18	18	11.85	29.1	18.8	669
50	76	2.4	3.92	18	18	12.83	31.6	19.4	679

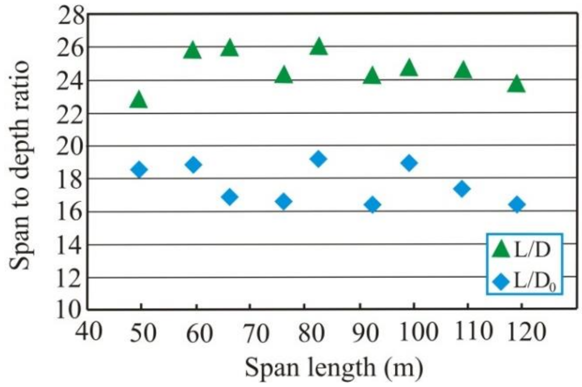

Fig. 11 Span/depth ratio at piers and mid-span for different span length

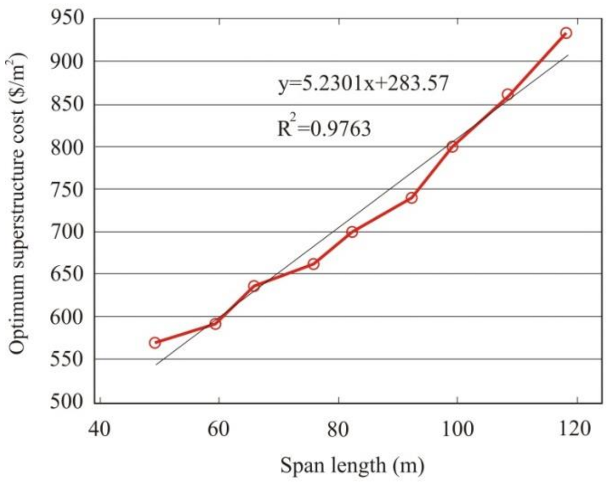

Fig. 12 Relation between optimum cost and span length

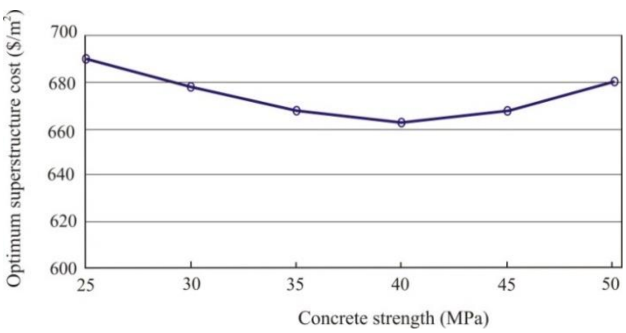

Fig. 13 Effect of concrete strength on optimum superstructure cost

the allowable stress specifications are applicable for precast prestressed concrete segments with compressive strength up to 42 MPa (6.1 ksi).

In this study, in order to assess the sensitivity of the cost to the compressive strength of concrete, optimization is conducted for concrete with a compressive strength between 25 to 50 MPa (3.6 to 7.2 ksi) similar to studies in the literature (Kaveh *et al*. 2016, Al-Osta *et al*. 2012, Ahsan *et al*. 2011). The results are tabulated in Table 6 and shown in Fig. 13.

According to Fig. 13, 40 MPa (5.8 ksi) compressive strength leads to a minimum superstructure cost. This finding is in agreement with that of (Morab and Fernandes, 2018). The results indicate that application of such compressive strength leads to 2.5 % reduction in superstructure cost in comparison to that of 30 or 50 MPa (4.4 or 7.2 ksi). Although the results show a low sensitivity of cost to concrete compressive strength, at 40 MPa (5.8 ksi) or less, the active constraint for optimization is the compressive strength at the bottom of the section on joint 5L, Fig. 6. At in higher compressive strength, due to a reduction in cross-sectional area, the tensile stress at the top of the section of the same joint is of dominant effect.

A reduction in the required concrete volume at 40 MPa (5.8 ksi) compressive strength or less, compensates its price increase per unit volume. In higher compressive strength, the tensile stress constraint limits the reduction in the required concrete volume; therefore, in of 40 to 50 MPa (5.8 to 7.2 ksi) compressive strength range, the superstructure cost increases.

As shown in Table 6, the optimum thickness of the bottom flange at 40 MPa (5.8 ksi) compressive strength or more is equal to its lower bound at both supports and mid-span locations, while in compressive strength less than 40 MPa (5.8 ksi), the thicknesses are different in the mentioned locations. This occurrence is due to the fact that the bottom flange thickness in positive bending moment zone is controlled with respect to the dead load effect, while in negative bending moment zone, the allowable compressive stress determines the required thickness (Heins and Lawrie 1984).

4.2.4 Relation between span length and prestressing tendons cross-sectional area

The normalized values of different parameters with respect to span length are tabulated in Table 7. The prestressing tendons cross-sectional areas (Aps) in terms of span length are shown in Fig. 14. According to the results, there exists a linear relation between these two parameters which can be determined with an increase in span length from 50 to 100 m (164 to 328 ft). The required prestressing force is increased by 50 %.

5. Conclusions

The optimization of the construction costs of the superstructure of a three-span segmental precast concrete bridge, with variable cross sections, is conducted here. The balanced cantilever method is considered as the construction method and GA is applied for these purposes.

Table 7 The normalized value of different parameters with respect to span length

L (m)	W/W_0	$O.C/O.C_0$	n/n_0	L/L_0	Δ/Δ_0
50	1.00	1.00	1.00	1.00	1.00
60	1.21	1.04	1.33	1.20	1.56
66	1.41	1.12	1.50	1.33	1.69
76	1.71	1.16	1.67	1.53	1.81
83	1.87	1.24	2.00	1.67	2.31
93	2.25	1.30	2.17	1.87	2.38
100	2.46	1.40	2.50	2.00	2.63
110	2.89	1.51	2.83	2.20	2.75
120	3.36	1.64	3.33	2.40	2.81

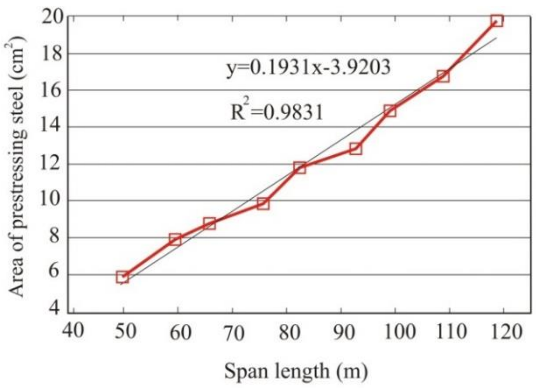

Fig. 14 Relation between prestressing tendons cross-sectional area and span length

Design of such bridges is relatively complicated due having many prestressing phases and variations in deck cross section. The optimization procedure, considering five design variables and approximately 200 design constraints according to AASHTO (2002) standard specifications, is provided by applying GA in MATLAB software.

The results are briefed as follows:

- Cost optimization is leads to an 8% reduction in superstructure construction costs, mostly due to the reduction in requiring prestressing tendons.
- Regarding the high unit price of prestressing steel in comparison with concrete unit price, the cost optimization obtained a 30% reduction in the required prestressing steel volume.
- By replacing the weight instead of the cost in the objective function, the superstructure weight optimization leads to a 5.5% reduction in superstructure weight, consequently, a 14% increase in prestressing steel.
- A sensitivity analysis of different parameters is run and the following results are obtained:
- The span/depth ratio in the optimization of different span lengths remains within the appropriate range proposed by AASHTO LRFD (2017) specifications, that is D_0/L and D/L ratios are obtained in the ranges of 1/20 to 1/16 and 1/28 to 1/22, respectively.
- With an increase in span length, optimum superstructure cost per unit area, prestressing tendons

cross-sectional area and the maximum deflection at mid-span increases.
- The variation trend of relative deflection is similar to that of span/depth ratio.

According to the superstructure cost optimization in terms of concrete compressive strength, its optimum value is 40 MPa (5.8 ksi), while sensitivity of the superstructure cost to concrete compressive strength is less than 3%.

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