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Simplified computational methodology for analysis and studies on behaviour of incrementally launched continuous bridges

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Abstract. Incremental launching method is one of the highly competitive techniques for construction of concrete bridges. It avoids costly and time consuming form work and centralizes all construction activities in a small casting yard, thus saving in cost and time against conventional bridge construction. From the quality point of view, it eliminates the uncertainty of monolithic behaviour by allowing high repetitiveness and industrial environment. But, from analysis and design point of view, the most characteristic aspect of incrementally launched bridges is that, it has to absorb the stresses associated with the temporary supports that are gradually taken on by the deck during its launch. So, it is necessary to analyse the structure for each step of launching which is a tedious and time consuming process. Effect of support settlements or temperature variation makes the problem more complex. By using transfer matrix method, this problem can be handled efficiently with minimal computational effort. This paper gives insight into method of analysis, formulation for optimization of the structural system, effect of support settlement and temperature gradient, during construction, on the stress state of incrementally launched bridges.

Key words: incrementally launched continuous bridge; construction phase analysis; transfer matrix; optimisation; thermal analysis; settlement analysis; launching nose; nose-deck interaction; length ratio; load ratio; stiffness ratio.

1. Introduction

Incremental launching method is one of the highly competitive techniques for construction of concrete bridges. In segmental construction, which may be treated as the only alternative industrialized technique, there are several deficiencies like segment length which is the constraint by transportation requirement, number of joints in the structure which affect the structural continuity and additional requirement of prestressing to resist the joint from opening. These weak points of the segmental precasting technique have been efficiently eliminated in incremental launching technique. Leonhardt *et al.* (1966) developed incremental launching method and this technique was first used for the construction of bridge over Rio Caroni, in Venezuela. By avoiding costly and time consuming false work and by concentrating all construction activities in a small fabrication area, considerable amount of saving in cost and time against conventional bridge construction can be

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achieved. Incremental launching method combines the advantages of precast segmental construction with segmental cast-in-situ methods. Grant (1975) pointed out the specific advantages of incrementally launched concrete structures. From the quality point of view, it eliminates the uncertainty of monolithic behaviour by allowing high repetitiveness and industrial environment that makes this technique a unique one. Baur (1977) described incremental launching method for bridge erection as a fast, safe and efficient technique. Zellner and Svensson (1983) brought out the suitability and applicability of incremental launching technique for different structures, specifically bridges. Marchetti (1984) discussed specific design problems associated with the construction of incrementally launched bridges.

Nevertheless, from analysis and design point of view, the most characteristic aspect of incrementally launched bridges is that they have to absorb the stresses associated with the temporary supports that are gradually taken on by the deck during its movement. So, it is necessary to analyse the structure for each step of launching which is a tedious and time consuming process. Rajasekaran and Sankarasubramanian (2001) presented the transfer matrix method of solution for beams and suspension cables and brought out the simplicity of the method. During launching, each cross section of the deck passes mid span and support cyclically and therefore each section is subjected to maximum positive and negative moments and maximum shear. Rosignoli (1997) used ‘transfer-matrix’ method for solution of the launching stresses of the bridge during construction. Granath (1998) reported results from laboratory experiments, finite element analyses and analytical calculations, concerning the distribution of the reaction force against an I-shaped steel girder launched incrementally. Rosignoli (1999) presented the methodology for optimisation of launching nose. Rosignoli (2000) pointed out the importance of considering time dependent deformations, such as creep during incremental launching, particularly in the cases where there is intermediate stoppage of launching work, as had happened in the case of Serio river bridge. Sasmal *et al.* (2002) brought out the importance of construction phase analysis of incrementally launched continuous span bridges. The effect of support settlement or temperature variation makes the problem more complex and it is really difficult to carry out the parametric or optimisation study when these factors are also imposed. The studies dealing with analysis of bridges built by incremental launching method are literally very few. Further, optimisation of structural parameters and study of important parameters such as support settlement and temperature gradient on the stress state of incrementally launched bridges has not received much attention. This paper presents a simple and efficient algorithm to overcome repetitive and time consuming methods to carry out a detailed construction phase analysis. A computer program has been developed based on the formulations presented in this paper. Using this program, comprehensive parametric studies have been carried out to obtain nose-deck interaction with different influencing parameters, effect of support settlements and temperature gradient during launching. The methodology and computer program developed in this study provide designers an efficient and reliable tool to get the optimum parameters for designing incrementally launched continuous bridges.

2. Formulation of transfer matrix for incrementally launched bridges

The transfer-matrix method is an approach to matrix structural analysis that uses a mixed form of the element force-displacement relationship and transfers the structural behaviour parameters (state array). The advantage of the transfer matrix method is that it produces a system of equations to be

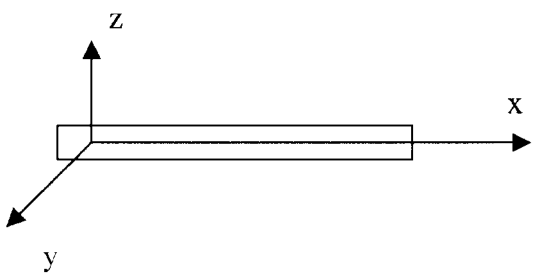

Fig. 1 Axis system of the continuous girder

solved that are quite small in comparison with those produced by stiffness method. Considering the case of a continuous beam in the x-y plane (as shown in Fig. 1), the state of stress at each cross section, can be defined by three forces [axial force (P), shear along y (V_y) and shear along z (V_z)] and three moments [M_x, M_y and M_z]. By considering only the presence of external forces orthogonal to the x-y plane and of external couples about in-plane axes, at each cross section six components of the load effect will get reduced to three, as orthogonal shear force V_z, bending moment M_y and twisting moment M_x whose internal work develops through the orthogonal displacement η_z, the flexural rotation ϕ_y and torsional rotation ϕ_x respectively. If axial force and the torsional deformations are not considered, the elastic system of the beam at each cross section can be described by a state array

$$\{S\}^T = \{\eta_z \;\; \phi_y \;\; M_y \;\; V_z\} \tag{1}$$

and the relation between the state arrays of two subsequent sections can be linked by a matrix $[T]$ which transfers the definition of the state array, hence called as transfer matrix. Therefore, the relation between state arrays of i) an origin section 0 and successive section 1 and ii) section 1 and section 2 of the beam, can be expressed as

$$\{S\}_1 = [T]_0^1 \times \{S\}_0 \tag{2}$$

$$\{S\}_2 = [T]_1^2 \times \{S\}_1 = [T]_1^2 \times [T]_0^1 \times \{S\}_0 \tag{3}$$

Thus, in the beam composed of 'n' subsequent segments, the relation between the state arrays of its two end sections can be obtained as

$$\{S\}_n = [T]_{n-1}^n \times [T]_{n-2}^{n-1} \times \ldots \times [T]_1^2 \times [T]_0^1 \times \{S\}_0 \tag{4}$$

$$= [T]_0^n \times \{S\}_0. \tag{5}$$

The governing equilibrium equation of the beam subjected to uniformly distributed load (p), can be written as

$$\frac{d^4\eta}{dx^4} = -\frac{p}{EI} \tag{6}$$

For obtaining the homogeneous solution of Eq. (6), it is assumed that

$$\eta = \alpha_1 + \alpha_2 x + \alpha_3 x^2 + \alpha_4 x^3 \tag{7}$$

where α_1 ... α_4 are the constants which can be solved by applying the boundary conditions.

Particular integral has been considered to incorporate the influence of loads. So, the final solution in matrix form, linking the state variables between two sections $i + 1$ and i, can be written as:

$$\{S\}_{i+1} = \begin{Bmatrix} \eta_z \\ \phi_y \\ M_y \\ V_z \\ 1 \end{Bmatrix}_{i+1} = \begin{bmatrix} 1 & l & -\dfrac{l^2}{2EI} & -\dfrac{l^3}{6EI} & k_1(p, l, E, I) \\ 0 & 1 & -\dfrac{l}{EI} & -\dfrac{l^2}{2EI} & k_2(p, l, E, I) \\ 0 & 0 & 1 & l & k_3(p, l) \\ 0 & 0 & 0 & 1 & k_4(p, l) \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \times \begin{Bmatrix} \eta_z \\ \phi_y \\ M_y \\ V_z \\ 1 \end{Bmatrix}_i \tag{8}$$

where the fifth column is obtained from particular integral and the fifth row has been introduced to keep the transfer matrix as a square matrix. All the terms of first four columns depend only on the mechanical and geometrical properties of the girder segment and the terms of the fifth column depend on external loads. For distributed load (p) applied uniformly on one full segment i.e. i^{th} to $i + 1^{\text{th}}$ station, the integral constants (k_1, k_2, k_3 and k_4) can be obtained as

$$k_1 = \frac{pl^4}{24EI}; \quad k_2 = \frac{pl^3}{6EI}; \quad k_3 = -\frac{pl^2}{2}; \quad k_4 = -pl \tag{9}$$

The cases of distributed load applied only to some portion of the segment or non-uniform EI can be handled by dividing the segment into sub-segments which are fully subjected to distributed load or having approximately uniform EI. If the load intensity or flexural rigidity value is uniform between 'x' and 'i', and has different magnitudes between 'x' and '$i + 1$', then the segment can be divided into two sub-segments, i.e. one from i to x and the other from x to $i + 1$, the composite transfer matrix $[T_c]_i^{i+1}$ can be obtained as,

$$\{S\}_{i+1} = [T]_x^{i+1} \times [T]_i^x \times \{S\}_i \tag{10}$$

2.1 Modification of transfer matrix to account for thermal gradient

The integration constants have been modified to incorporate the effect of temperature gradient. Thus the state variables can be related as

$$\{S\}_{i+1} = \begin{Bmatrix} \eta_z \\ \phi_y \\ M_y \\ V_z \\ 1 \end{Bmatrix}_{i+1} = \begin{bmatrix} 1 & l & -\dfrac{l^2}{2EI} & -\dfrac{l^3}{6EI} & k_1(p, l, E, I) - k_1'(l, \alpha_T, \Delta t, h) \\ 0 & 1 & -\dfrac{l}{EI} & -\dfrac{l^2}{2EI} & k_2(p, l, E, I) - k_2'(l, \alpha_T, \Delta t, h) \\ 0 & 0 & 1 & l & k_3(p, l) \\ 0 & 0 & 0 & 1 & k_4(p, l) \\ 0 & 0 & 0 & 0 & 1 \end{bmatrix} \times \begin{Bmatrix} \eta_z \\ \phi_y \\ M_y \\ V_z \\ 1 \end{Bmatrix}_i \tag{11}$$

where, $k_1' = l^2\alpha_T\Delta t/2h$; $k_2' = l\alpha_T\Delta t/h$ and l is span length; α_T is the coefficient of thermal expansion and h is the depth of concrete deck. Eq. (11) has four linear equations with eight unknowns. So, another four equations can be obtained by imposing the boundary conditions at two end sections. The number of redundants progressively increases with the progress of launching as new supports affected by the deck add more unknowns. Since the additional unknowns are concentrated in the support section state-arrays, where they constitute the only possible discontinuity, the transfer matrix method can be improved so as to operate only on the continuous terms of these arrays i.e. the rotation (ϕ) for compatibility and the bending moment (M) for equilibrium.

2.2 Formulation of reduced transfer-matrix

The state array matrix at $(i + 1)^{th}$ support section (Eq. 11) can be expanded as

$$\eta_{i+1} = \eta_i + T_{12}\phi_i + T_{13}M_i + T_{14}V_i + T_{15} \tag{12}$$

$$\phi_{i+1} = \phi_i + T_{23}M_i + T_{24}V_i + T_{25} \tag{13}$$

$$M_{i+1} = M_i + T_{34}V_i + T_{35} \tag{14}$$

$$V_{i+1} = V_i + T_{45} \tag{15}$$

where, the coefficients $T_{12} \ldots, T_{45}$ are the terms of transfer matrix $[T]_i^{i+1}$. T_{ij} is the element of i^{th} row and j^{th} column of the transfer matrix $[T]$. By extracting V_i from Eq. (12) and substituting the same in the other three Eqs. (13)-(15), one can obtain direct relation between the rotations and the bending moments at the two support sections (i and $i + 1$) of the span as:

$$V_{i+1} = -\frac{T_{12}}{T_{14}}\phi_i - \frac{T_{13}}{T_{14}}M_i + \frac{1}{T_{14}}(\eta_{i+1} - \eta_i - T_{15}) + T_{45} \tag{16}$$

$$\phi_{i+1} = \left(1 - T_{24}\frac{T_{12}}{T_{14}}\right)\phi_i + \left(T_{23} - T_{24}\frac{T_{13}}{T_{14}}\right)M_i + \frac{T_{24}}{T_{14}}(\eta_{i+1} - \eta_i - T_{15}) + T_{25} \tag{17}$$

$$M_{i+1} = -T_{34}\frac{T_{12}}{T_{14}}\phi_i + \left(1 - T_{34}\frac{T_{13}}{T_{14}}\right)M_i + \frac{T_{34}}{T_{14}}(\eta_{i+1} - \eta_i - T_{15}) + T_{35} \tag{18}$$

So, the reduced state-array matrix of the support section is

$$\{S'\}^T = \{\phi_y \;\; M_y \;\; 1\} \tag{19}$$

For a rigid support condition, the deflection terms may be either constant or null. Hence, the reduced transfer matrix for rigid support condition can be obtained as follows:

$$[T']_i^{i+1} = \begin{bmatrix} 1-\dfrac{T_{24}T_{12}}{T_{14}} & T_{23}-\dfrac{T_{24}T_{13}}{T_{14}} & -T_{15}\dfrac{T_{24}}{T_{14}}+T_{25} \\ -\dfrac{T_{34}T_{12}}{T_{14}} & 1-\dfrac{T_{34}T_{13}}{T_{14}} & -T_{15}\dfrac{T_{34}}{T_{14}}+T_{35} \\ 0 & 0 & 1 \end{bmatrix} \tag{20}$$

2.2.1 Reduced transfer matrix to account for settlements

While replacing the neo-flon plates or the launching bearing pads just after launching, it may induce change in support alignment. During launching of n^{th} span, if the settlements of two consecutive supports i and $i+1$ are denoted as η_i^{ns} and η_{i+1}^{ns} respectively, the reduced transfer matrix can be written as

$$[T']_i^{i+1} = \begin{bmatrix} 1-\dfrac{T_{24}T_{12}}{T_{14}} & T_{23}-\dfrac{T_{24}T_{13}}{T_{14}} & \dfrac{T_{24}}{T_{14}}(\eta_{i+1}^{ns}-\eta_i^{nx}-T_{15})+T_{25} \\ -\dfrac{T_{34}T_{12}}{T_{14}} & 1-\dfrac{T_{34}T_{13}}{T_{14}} & \dfrac{T_{34}}{T_{14}}(\eta_{i+1}^{ns}-\eta_i^{ns}-T_{15})+T_{35} \\ 0 & 0 & 1 \end{bmatrix} \tag{21}$$

The relation between the reduced state-arrays of the end supports can be expressed as:

$$\{S'\}_N = [T']_{N-1}^{N} \times [T']_{N-2}^{N-1} \times \ldots \times [T']_2^3 \times [T']_1^2 \times \{S'\}_1 = [T']_1^N \times \{S'\}_1 \tag{22}$$

which comprises of two equations with four unknowns. By knowing two end cantilever moments, the unknown terms viz., rotations at two ends can be calculated using Eq. (22) which relates state arrays of two end sections of the structure at any step of launching and further, the relative shear can be calculated using Eq. (16). Once the definition of the support section state array is complete, in terms of compatibility and equilibrium, structural behaviour parameters at any section can be determined by progressive multiplication of transfer matrices.

Normally, during launching, time dependent deformations due to creep may not occur in incrementally launched bridges. However, abnormal delay in construction or intermediate stoppage of launching work for many months may lead to additional long-term deformations, due to creep and shrinkage, in the bridge deck already launched. As a result, when the launch is resumed, the deformed continuous deck over fixed bearings would produce the same effect as the effect of support settlement on an undeformed deck during launching. In such cases, a settlement analysis can be carried out using the modified reduced transfer matrix as described above and the deformation magnitudes evaluated using the guidelines specified in BS-5400 Part 4 (1984) for estimation of deformation due to creep and shrinkage.

3. Computer program for analysis of incrementally launched continuous bridges

A computer program has been developed based on the formulations presented in the preceeding sections for analysis of incrementally launched continuous bridges. The flow chart of the computer program is given in Fig. 2.

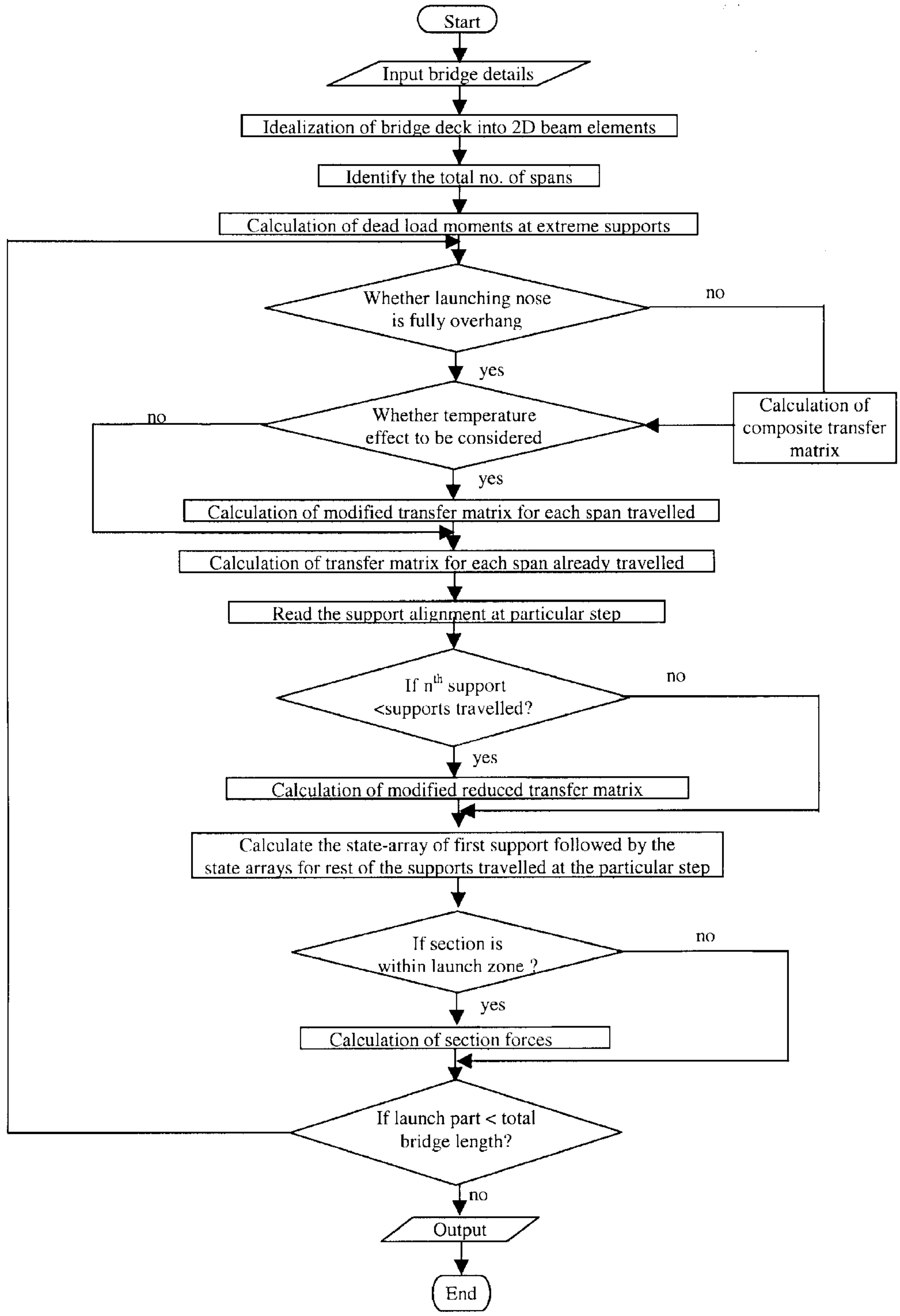

Fig. 2 Flow chart of computer program for analysis of incrementally launched continuous bridges

4. Validation study

Consider the case of bridge girder erected by incremental launching procedure, in which the steel nose has just touched the 10th pier (as shown in Fig. 3). The structure is composed of concrete deck and steel launching nose. The self weight of concrete deck (q) and that of launching nose (q_n) are 207 kN/m and 24 kN/m respectively. The material and sectional properties of deck and nose are: $E = 3.7 \times 10^7$ kN/m^2, $I = 6.8$ m^4, $q_n = 24$ kN/m, $E_n = 2.1 \times 10^8$ kN/m^2 and $I_n = 0.24$ m^4. Length of the concrete deck segment (l) and the steel launching nose (l_n) are taken as 30 m and 10 m respectively. The above mentioned problem has been analyzed using the computer program developed in this study and standard finite element analysis (FEA) package. Two-noded, three-dimensional, beam finite elements are used to model the bridge deck in FEA package. A convergence study is carried out by considering three different meshes viz., 10, 20 and 30 elements per span. The structural responses obtained corresponding to these discretisations are presented in Table 1. For the table, it is observed that the results obtained using 30 elements per span are well-converged. Hence, the results of the present study are compared with those obtained using standard FEA package using 30 elements per span. The comparison of results is shown in Table 2. From this table, it can be noted that the results of the present study are in good agreement with those obtained using standard finite element analysis package. On comparison of CPU time, it is found that standard package has taken 1 second for analysing the structural arrangement during a particular launching position. Whereas, in the same duration of time, the computer program developed in this study based on transfer matrix methodology could complete analysis of the bridge deck for entire launching considering launching steps of 1 m.

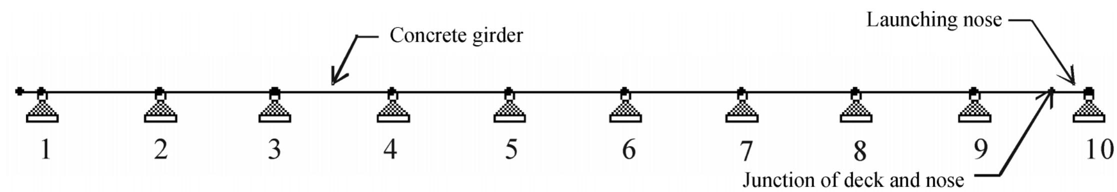

Fig. 3 Modelling of incrementally launched continuous concrete deck and with launching nose just touching the 10th support

Table 1 Convergence study using mesh refinement in each span of the bridge deck

Number of elements per span	Bending Moment (MNm) at				Shear force (MN) at			
	2nd support	9th support	Mid of first span (1-2)	Mid of 9th span (9-10)	2nd support	9th support	Mid of first span (1-2)	Mid of 9th span (9-10)
10	18.68618	19.37564	12.08141	9.02466	3.60367	3.26771	0.49867	0.34085
20	18.68618	19.37565	12.08141	9.02466	3.60367	3.26771	0.49867	0.34086
30	18.68618	19.37565	12.08141	9.02466	3.60367	3.26771	0.49867	0.34086

Table 2 Comparison of results of present study with results of FEA using 30 elements per span

Item	Bending Moment (MNm) at				Shear force (MN) at			
	2nd support	9th support	Mid of first span (1-2)	Mid of 9th span (9-10)	2nd support	9th support	Mid of first span (1-2)	Mid of 9th span (9-10)
Results of the present study	18.686	19.369	12.081	9.021	3.604	3.268	0.499	0.341
Finite element analysis result	18.686	19.376	12.081	9.025	3.604	3.268	0.499	0.341
Difference in percentage	0.000	0.0361	0.000	0.0443	0.000	0.000	0.000	0.000

5. Parametric studies on the behaviour of incrementally launched continuous bridges

Using the computer program developed in the present study, important parametric studies are carried out to obtain nose-deck interaction and to evaluate the influence of geometric and mechanical parameters of girder and launching nose, settlement and thermal gradient effects, on the state of stress in incrementally launched continuous bridge girder. The studies are carried out on bridge girder with launching nose which is described in the validation study.

5.1 Variation of bending moment during launching

The course of launching procedure is repetitive and launching of each span can be achieved in two subsequent phases as shown in Fig. 4. At first, the deck-nose joint is just on a particular support (shown as ST-1 in Fig. 4). During first phase of launching, the steel nose comes in contact with the

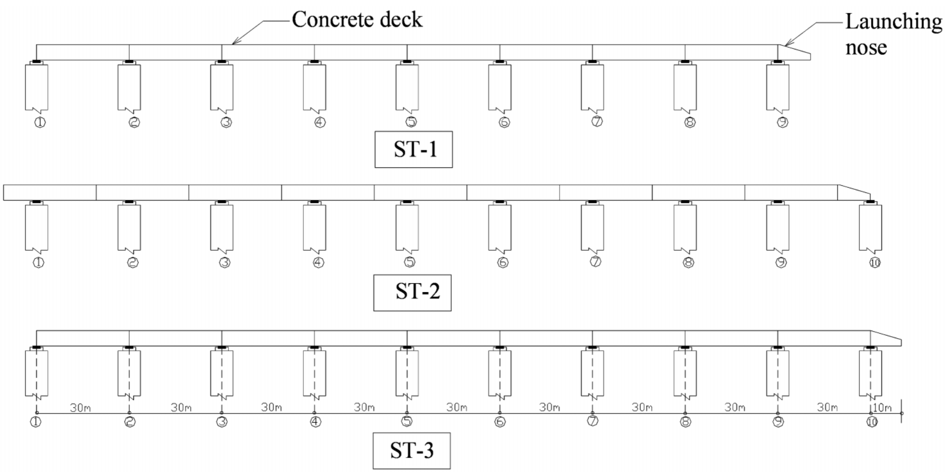

Fig. 4 Two phases (ST-1 to ST-2 and ST-2 to ST-3) of launching of bridge deck from one support to another

next support (shown as ST-2 in Fig. 4). The launching process goes on from the stage ST-2 till deck-nose joint reaches the next pier (shown as ST-3 in Fig. 4). This is the second phase. Bending moment profile obtained when the structural system is launched from ST-1 position to ST-2 position in steps of 1 m, is shown in Fig. 5. Bending moment profile during launching of the deck from ST-2 to ST-3 is as shown in Fig. 6. For both the phases of launching, the bending moment profiles are

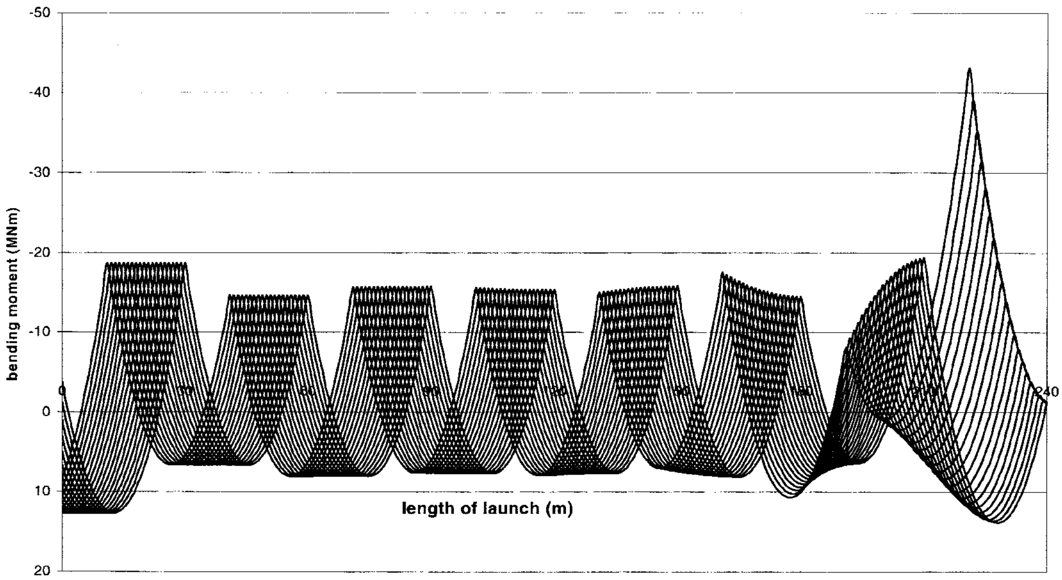

Fig. 5 Bending moment profile during first phase of launching (From ST-1 to ST-2)

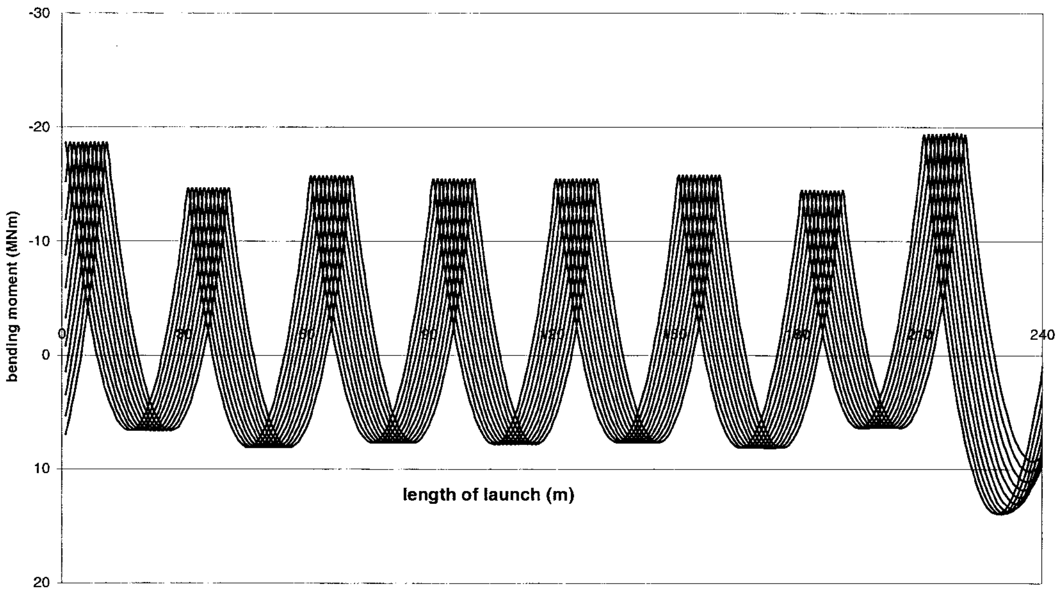

Fig. 6 Bending moment profile during second phase of launching (From ST-2 to ST-3)

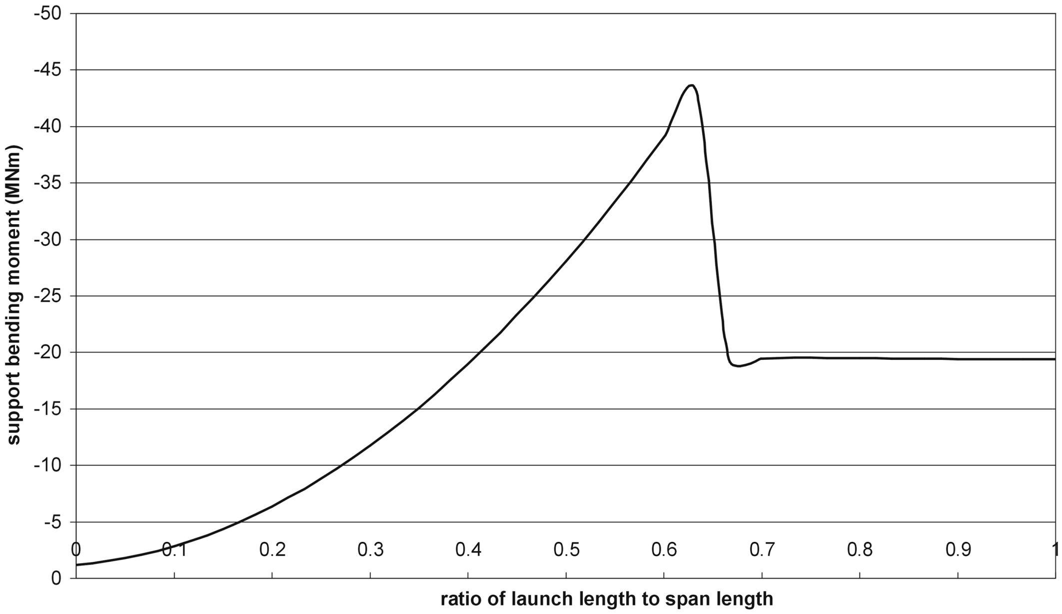

Fig. 7 Variation of support moment during launching

shown till the concrete deck-launching nose joint reaches the 9th support. From Figs. 5 and 6, it may be noted that the magnitude and the distribution of bending moments varies considerably with the launching steps. So, launching of particular concrete deck segment, for example, 9th deck segment (i.e. between 9th support and 10th support), imposes set of responses on the already launched deck as shown in Figs. 5 and 6. Hence, the superposition of responses corresponding to these two steps, ST-1 to ST-2 and ST-2 to ST-3, will complete the launching analysis for the launching of 9th deck segment.

During launching, the deck nearer to the foremost support experiences maximum shear or bending moment. A typical variation of support moment during launching is shown in Fig. 7. From the variation of load effect at each cross section (shown in Fig. 7), it may be noted that the maximum cantilever moment is much higher than the end of launch (E.O.L) moment. It means that, for particular load value and geometric properties of concrete deck and the launching nose, the reduction of support moment is huge once the launching nose touches the next support and thereafter, the moment remains constant until entire length of the nose becomes overhang from the support. But, from design point of view, the deck section has to be designed for the maximum moment. So detailed parametric studies are required to evaluate the influence of the load ratio and the geometric properties of the both components (deck, launching nose) at each step of launch to avoid undesirable section requirement and to carry out optimisation of the parameters.

5.2 Optimisation of deck behaviour during launching

One of the ways to optimise the section is through matching the values of maximum cantilever moment and the E.O.L moment. For carrying out the optimization studies, the parameters considered are: i) Load ratio i.e. ratio of weight of launching nose (q_n) to deck load (q), ii) Length ratio i.e. ratio of length of launching nose (l_n) to span length (l), and iii) Stiffness ratio i.e. ratio of flexural stiffness of launching nose (E_nI_n) to deck stiffness (EI)

5.2.1 Effect of load ratio (q_n/q)

The effect of load ratio on variation of moment has been studied while keeping the length ratio (l_n/l) and stiffness ratio (E_nI_n/EI) as constant parameters. Analyses have been carried out for load ratios (q_n/q) varying from 0.02875 to 0.2. For short launching nose (length ratio less than 0.5, for example 0.33), the support moment is almost non-responsive with the change of load ratio. Once the launching nose reaches the next support, the reduction of moment is huge and almost constant for different load ratios (as shown in Fig. 8). The behaviour has improved with the increase in length ratio. For length ratio equal to 0.5, though the variation of support moment with the variation of load ratio is not so large, the difference between the maximum cantilever moment and the E.O.L moment is reduced. For long launching nose (length ratio greater than 0.5), the variation shows substantial improvement. The effect of load ratio has been studied for a particular length ratio of 0.67 and the stiffness ratio of 0.20 and the variation of support moment for different load ratios is shown in Fig. 9. From this figure, it can be noted that as the load ratio decreases, the E.O.L moment increases consistently and beyond a particular load ratio, the E.O.L moment exceeds the maximum cantilever moment which shows the sign of optimisation. For this particular case, optimization is achieved for the load ratio of 0.11. It is needless to adopt the load ratio lesser than 0.11 because the section has to be designed to resist highest of cantilever and E.O.L moments.

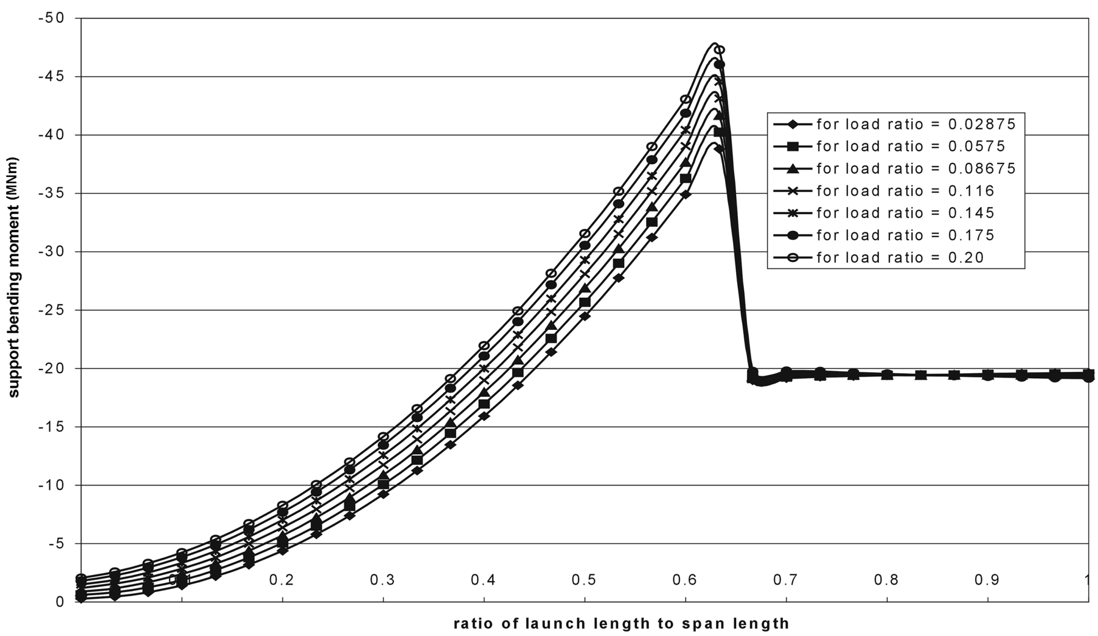

Fig. 8 Variation of support moment for different load ratios (q_n/q) when length ratio (0.33) and stiffness ratio (0.20) are the fixed parameters

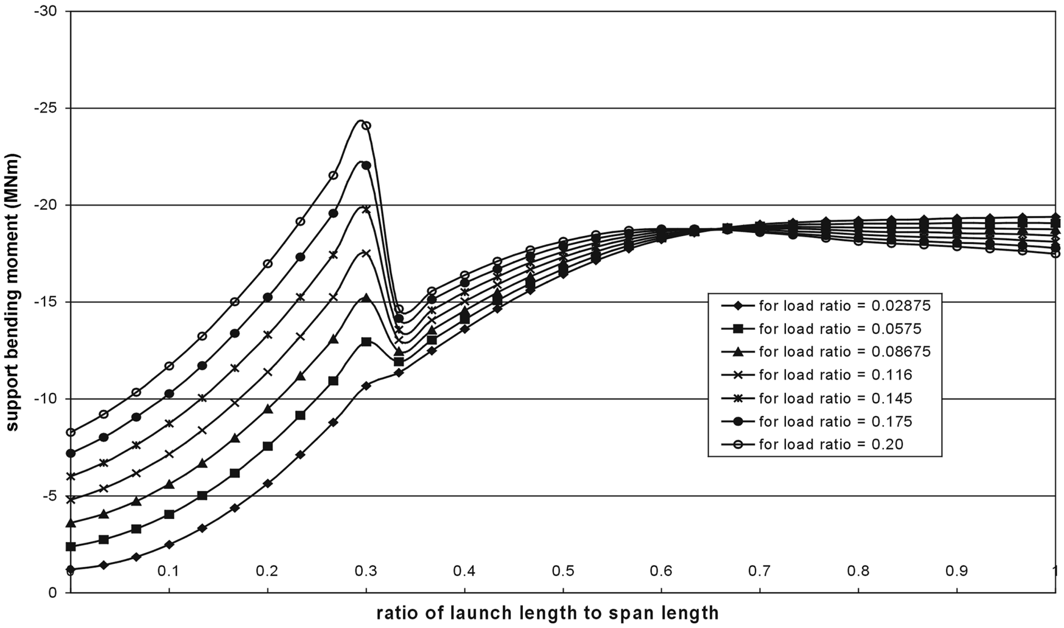

Fig. 9 Variation of support moment for different load ratios (q_n/q) when length ratio (0.67) and stiffness ratio (0.20) are the fixed parameters

5.2.2 Effect of length ratio (l_n/l)

The effect of length ratio on support moments has been studied while keeping the load ratio and stiffness ratio as constant parameters. The load ratio and stiffness ratio are taken as 0.10 and 0.20 respectively. The variation of the support moment for different length ratios is shown in Fig. 10. It is interesting to note that for launching cases with short launching nose, the design values are mainly governed by the maximum cantilever moment and behaviour has improved with the increase in length ratio. In the case of long launching nose, E.O.L moment governs the design. But the variation is limited for a particular range of length ratio. From Fig. 10, it can be seen that, for a length ratio of 1.0, the maximum moment has increased consistently even after E.O.L and process is further extended for the next launching also. So, the length ratio within a particular range may give the scope for optimization and it is limited to range of l_n/l values from 0.5 to 1.0.

5.2.3 Effect of stiffness ratio (E_nI_n/EI)

For a launching with short nose, the effect of stiffness ratio on support moment is quite insignificant because the maximum cantilever moment is much higher than the E.O.L moment. From the structural behaviour consideration, it is certain that the stiffness will play a role only after the nose touches the next support i.e. only the E.O.L moment will be affected. Lower the stiffness ratio, higher is the E.O.L moment and the rate of recovery of E.O.L moment compared to maximum cantilever moment has increased monotonically with the reduction of stiffness ratio. So, higher stiffness ratio is not required as it can not reduce the maximum design moment whereas lower stiffness increases the E.O.L moment considerably and the maximum cantilever moment is much less than the maximum design moment. Variation of support moment for different stiffness ratios is shown in Fig. 11. From Fig. 11, it is clear that the optimum stiffness ratio is 0.2.

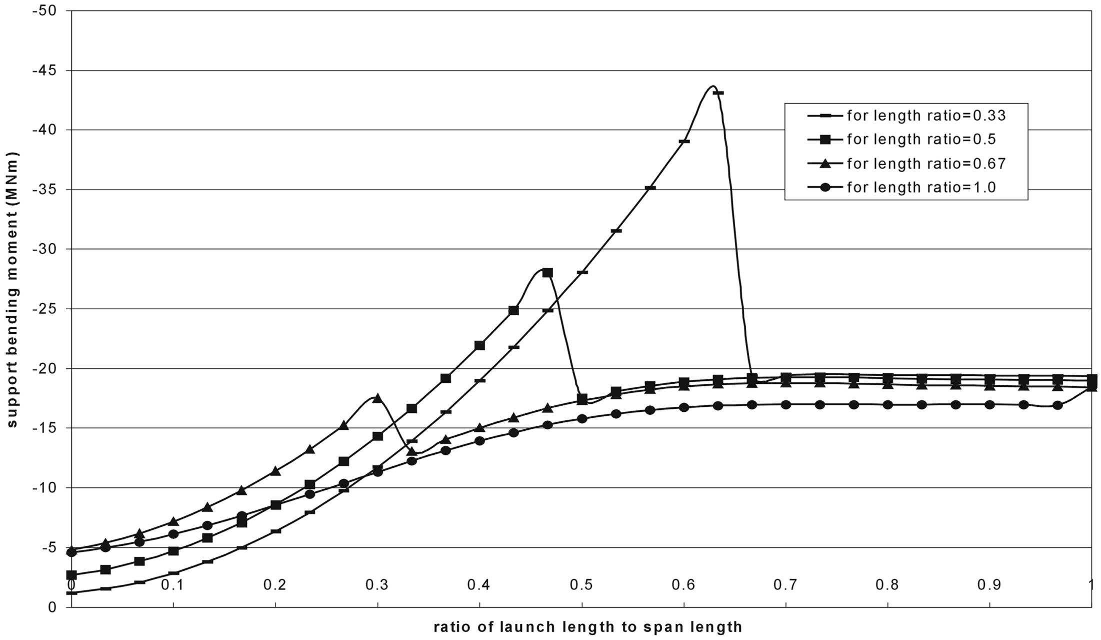

Fig. 10 Variation of support moment for different length ratios (l_n/l) when load ratio (0.1) and stiffness ratio (0.2) are fixed parameters

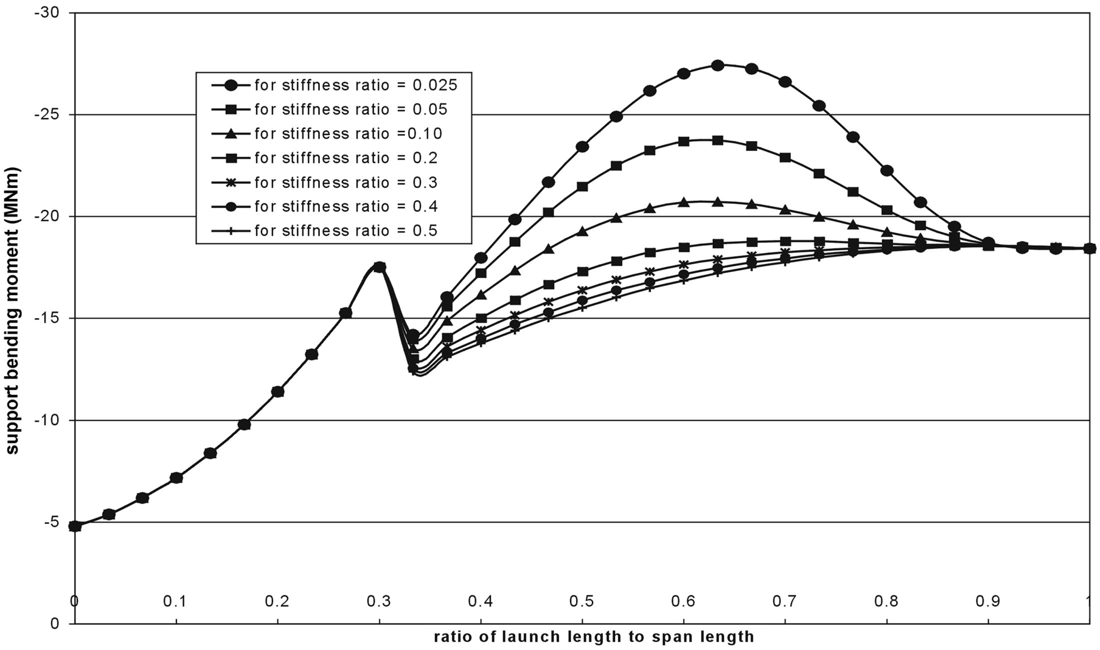

Fig. 11 Variation of support moment for different stiffness ratios (E_nI_n/EI) when length ratio (0.67) and load ratio (0.1) are fixed parameters

5.2.4 Optimum design parameters

As discussed in the previous section, the effect of stiffness ratio on support moment is quite insignificant. The load ratio and the length ratio are the key parameters for obtaining an optimum

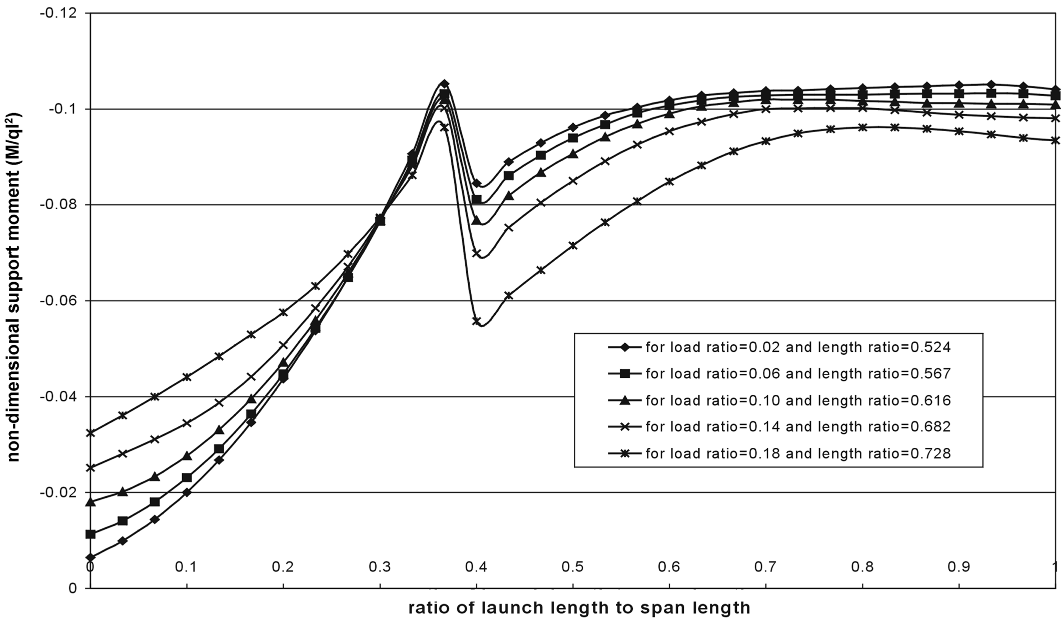

Fig. 12 Equalisation of support moment for different load ratios by adjusting the length ratio

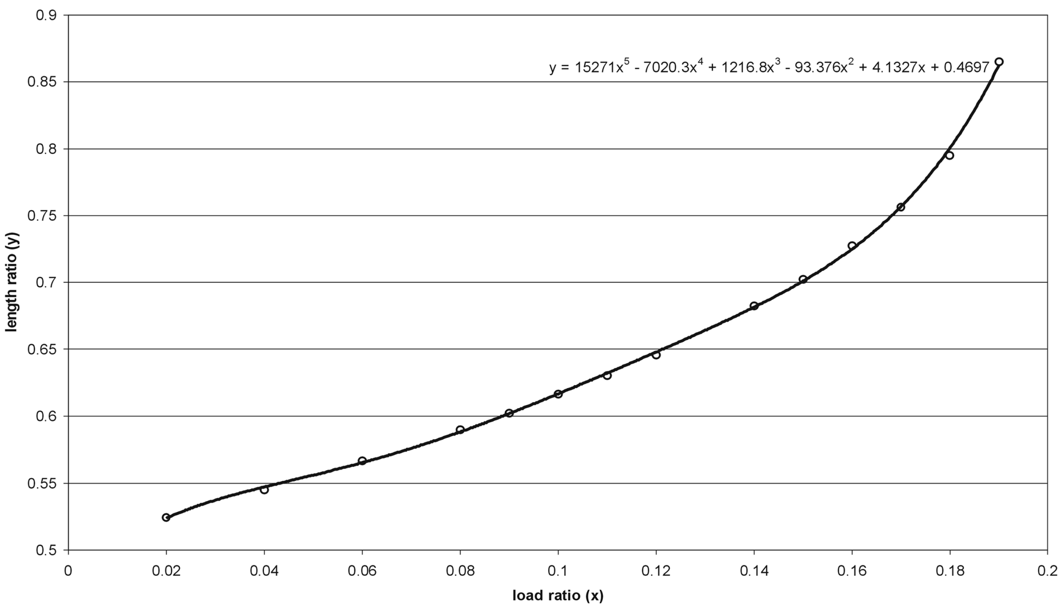

Fig. 13 Load ratio Versus length ratio for equalising cantilever moment and maximum E.O.L moment when stiffness ratio (0.2) is fixed parameter

bending moment value (equalising the maximum cantilever moment with the E.O.L moment value). A detailed iterative study, by changing load and length ratios, has been carried out to get the optimum bending moment. Variation of support bending moment obtained for different load ratios by adjusting length ratio is shown in Fig. 12. From this figure, it can be noted that the increase in load ratio requires greater length ratio to compensate the bending moments. Investigations have also been carried out to formulate an expression which would help the designer to get design parameters for optimum bending moments. From the best-fit curve (as shown in Fig. 13) which is drawn based on the results of parametric studies carried out for equalisation of support moment, an expression co-relating load ratio and length ratio, which is valid for $0.01 \le q_n/q \le 0.195$, is obtained as

$$\frac{l_n}{l} = 15271\times\left(\frac{q_n}{q}\right)^5 - 7020.3\times\left(\frac{q_n}{q}\right)^4 + 1216.8\times\left(\frac{q_n}{q}\right)^3 - 93.376\times\left(\frac{q_n}{q}\right)^2 + 4.1327\times\left(\frac{q_n}{q}\right) + 0.4697 \quad (24)$$

5.3 Effect of support settlement

In incremental launched process, it is usual to replace the neo-flon plates or the launching bearings just after launching. It may induce change of support alignment during launching. So, it is important to carry out a detailed study to investigate the influence of possible support settlements, both positive (upward) and negative (downward), during launching on the bending moment profile of the deck. A study has been carried out for different support settlements (10 mm up or 10 mm down) at 5th support (at 120 m from first support). Bending moment profiles due to support settlements at certain stage of launching are shown in Fig. 14. From this figure, it may be noted that

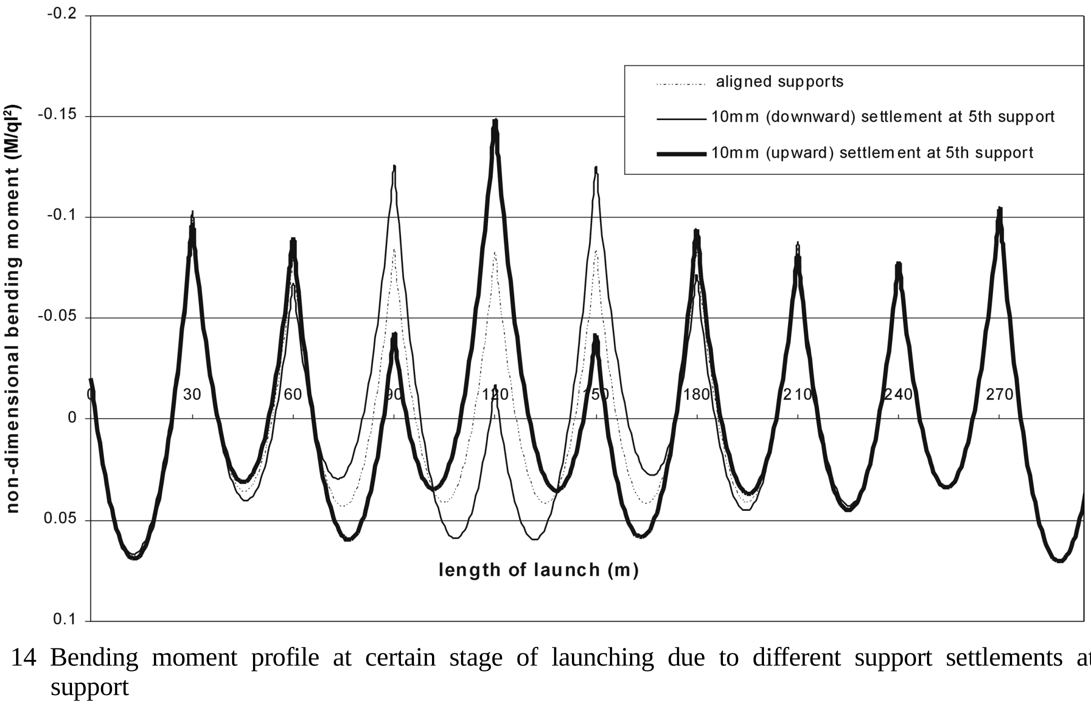

Fig. 14 Bending moment profile at certain stage of launching due to different support settlements at 5th support

both positive and negative moments are considerably affected by the mangitude and direction of support settlements. It can be noted that the requirement of the geometric properties is enhanced considerably at a particular section due to support settlement during the launching process.

Eventhough the effect of settlement is localised, there may be a case where different supports along the length of the bridge may suffer from different types of support settlements both in magnitude and direction simultaneously during launching. So, before evaluating the design values, it is necessary to assume the behaviour of different supports depending on site conditions, type of launching and previous experience. The computer program developed in this study is capable of taking into account of support settlements at any stage of launching.

Consider a case in which the 5th support (at 120 m from first support) of the bridge deck is under constant downward settlement of 10 mm and when the front edge of the nose touches the 9th support (at 240 m from first support), due to some lack of fit of the neo-flon plate, it is aligned at 10 mm above the level of the supports. The bending moment profile of the bridge deck due to simultaneous support settlements, at different supports, is shown in Fig. 15. From this figure, it is clear that simultaneous action of the support settlements produces the maximum bending moment, which has to be considered in the design.

Maximum support moments for different types of support settlements change considerably depending on the geometric properties of concrete deck and launching nose. Effect of support settlement on support moment for length ratios of 0.33 and 0.67 are shown in Figs. 16 and 17 respectively. For the length ratio of 0.33, the design moment is maximum cantilever moment which does not change with the support settlement that occurs only after the nose touches the next support. But, in the case of length ratio of 0.67, the maximum design moment will be the E.O.L moment due to upward settlement. So, it is necessary to study all the possible geometric properties of the concrete deck and the launching nose with the probable support settlement values.

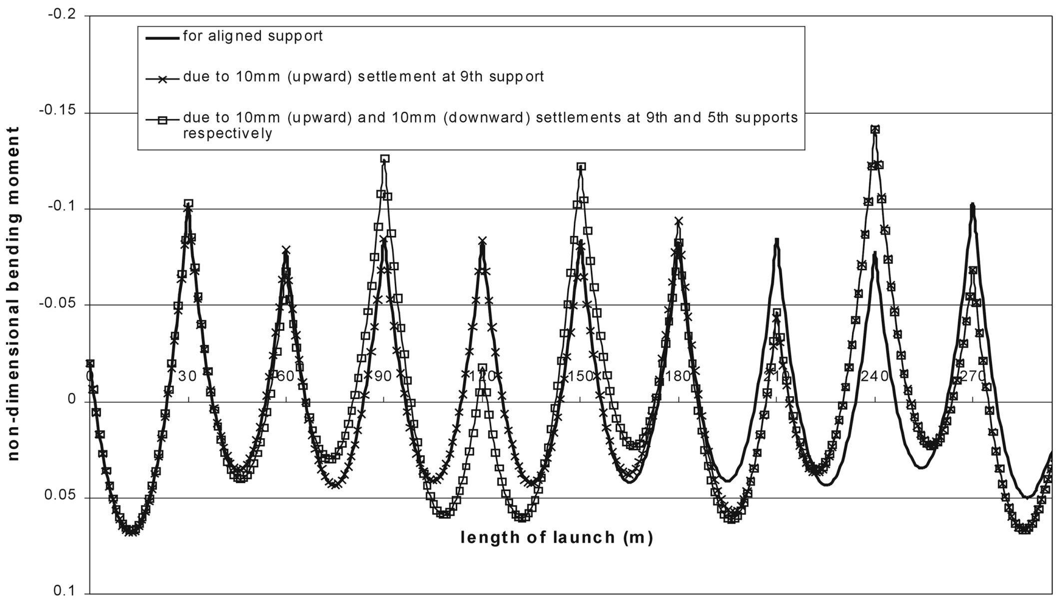

Fig. 15 Bending moment profile at certain stage of launching due to simultaneous support settlements

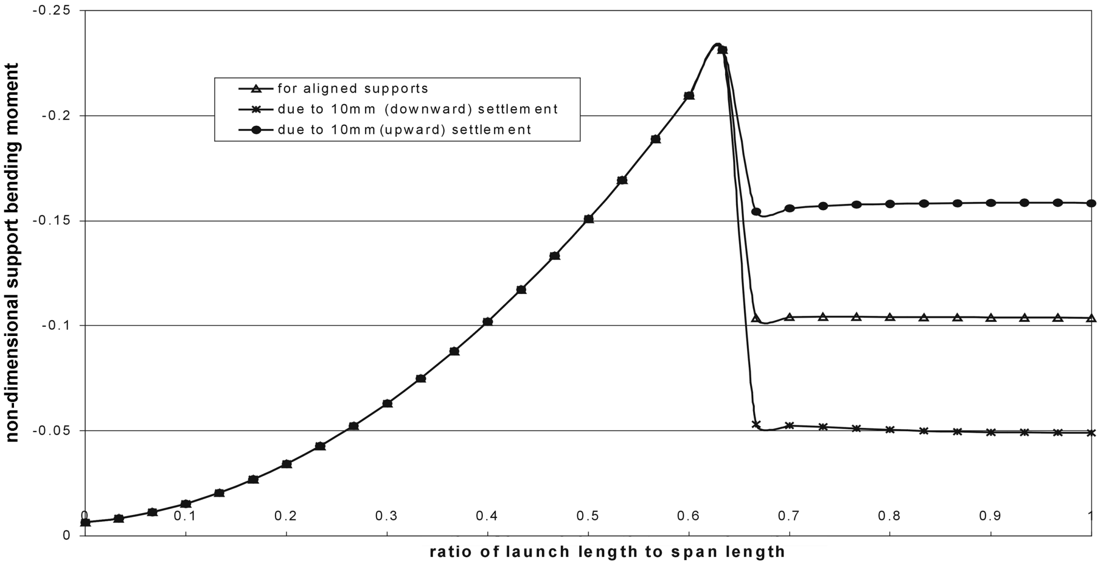

Fig. 16 Effect of settlement on support moment for length ratio of 0.33

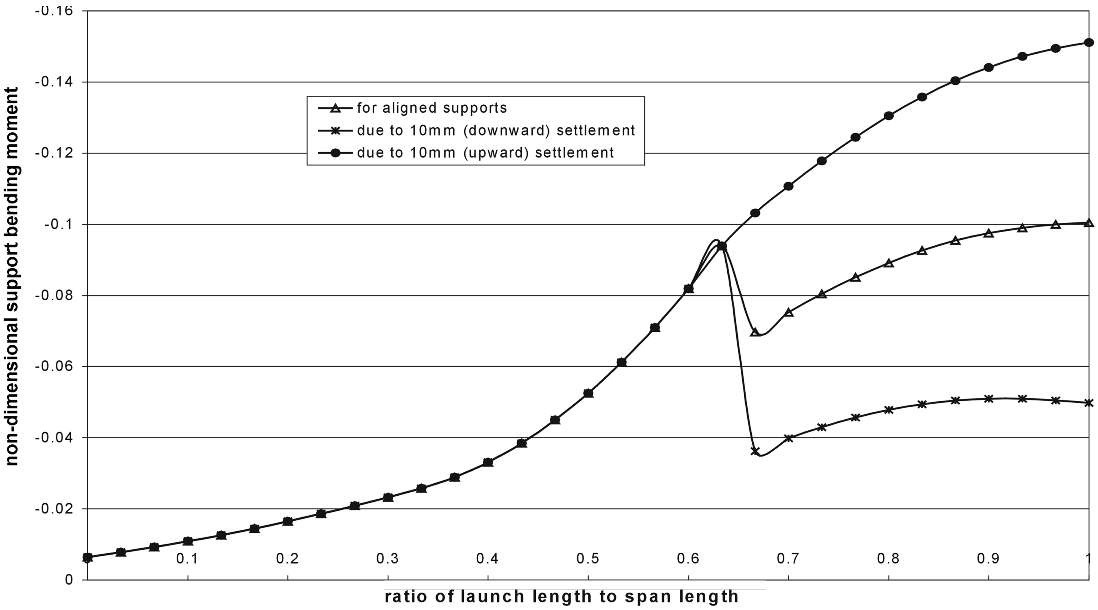

Fig. 17 Effect of settlement on support moment for length ratio of 0.67

5.4 Effect of temperature gradient

Bridges may undergo significant temperature changes under the combined influence of solar radiation, daily air temperature variation and wind speed. In some circumstances, these temperature changes can induce thermal stresses that are comparable to the stresses induced by dead and live loads. Therefore, investigations on the thermal behaviour of incrementally launched continuous

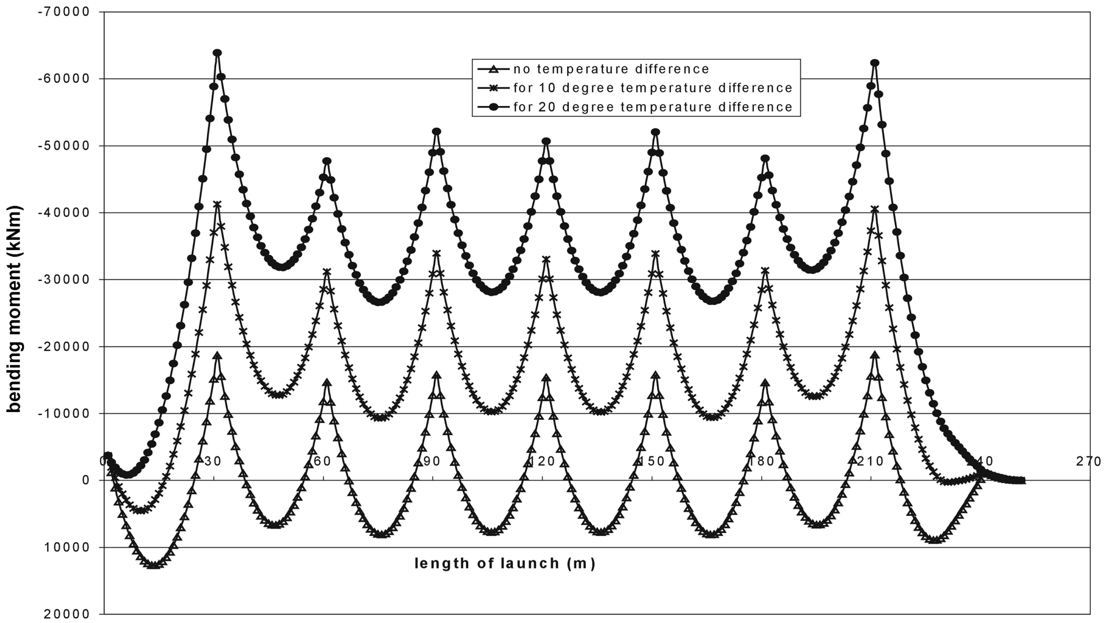

Fig. 18 Bending moment profile at a certain stage of launching due to self weight of deck and temperature gradient

bridges is important. In incremental launching technique, the induced stress due to thermal variation may cause unique criteria for designing the structure. In this paper, studies on influence of thermal gradient on the behaviour of bridge deck during launching have been carried out. The temperature gradient across the depth is assumed to be linear. The co-efficient of thermal expansion for concrete deck and steel nose are assumed as 8.5×10^{-6} and 11.3×10^{-6} per degree Celsius respectively. The bridge span moment due to dead load alone and bending moment due to dead load and temperature gradient are shown in Fig. 18. It is important to note that due to temperature gradient, the nature of bending moment diagram changes. From design point of view, the site location and the physical parameters influencing atmospheric temperature should be studied. Further, the geometric properties also influence design moment. Figs. 19 and 20 show the variation of support moment during launching due to different thermal gradients for length ratios of 0.165 and 0.67 respectively. For length ratio of 0.165, the moment corresponding to the cantilever moment governs the design and is too high with respect to E.O.L moment when there is no temperature difference (as shown in Fig. 19). But, for the same arrangement, for a temperature difference of 20 degree Celsius, the design moment will be E.O.L moment which is slightly higher than the maximum cantilever moment. In Fig. 20, the support moment variation is shown for the length ratio of 0.67. Maximum design moment is E.O.L moment when there is no temperature difference. From Fig. 20, it can be noted that bending moment profile changes rapidly with temperature difference and for a temperature difference of 20 degree Celsius, the design moment will be E.O.L moment which is considerably higher than the maximum cantilever moment. So, the design of the section should be carried out considering the geometric parameters along with the environmental factors such as temperature gradient. It is important to note that the bending moment induced due to temperature gradient, in some cases, is much greater than the dead load moment.

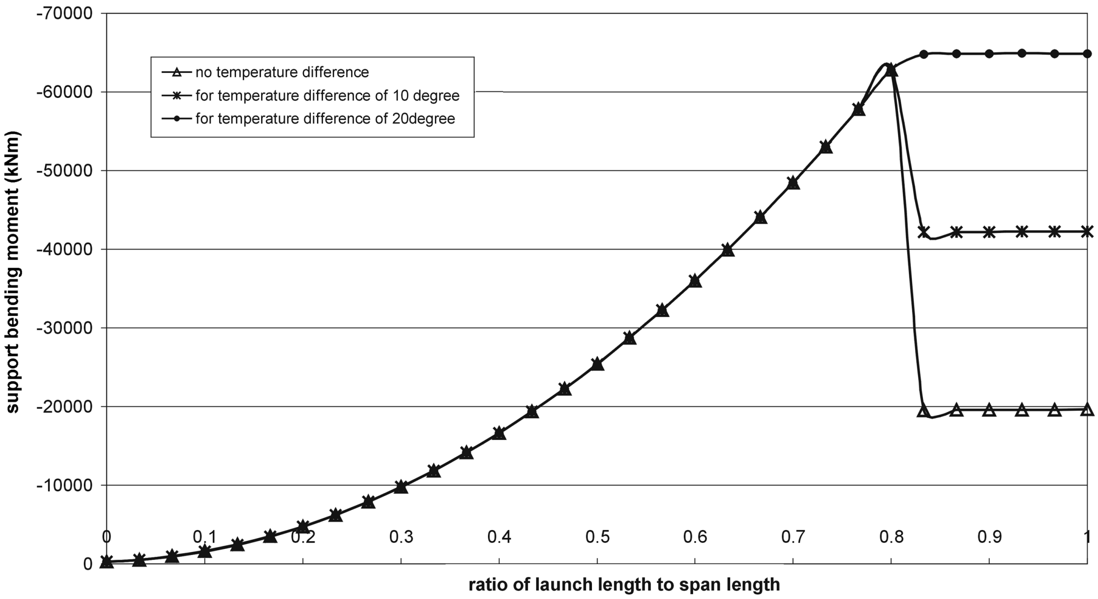

Fig. 19 Effect of temperature variation on support moment for length ratio of 0.165

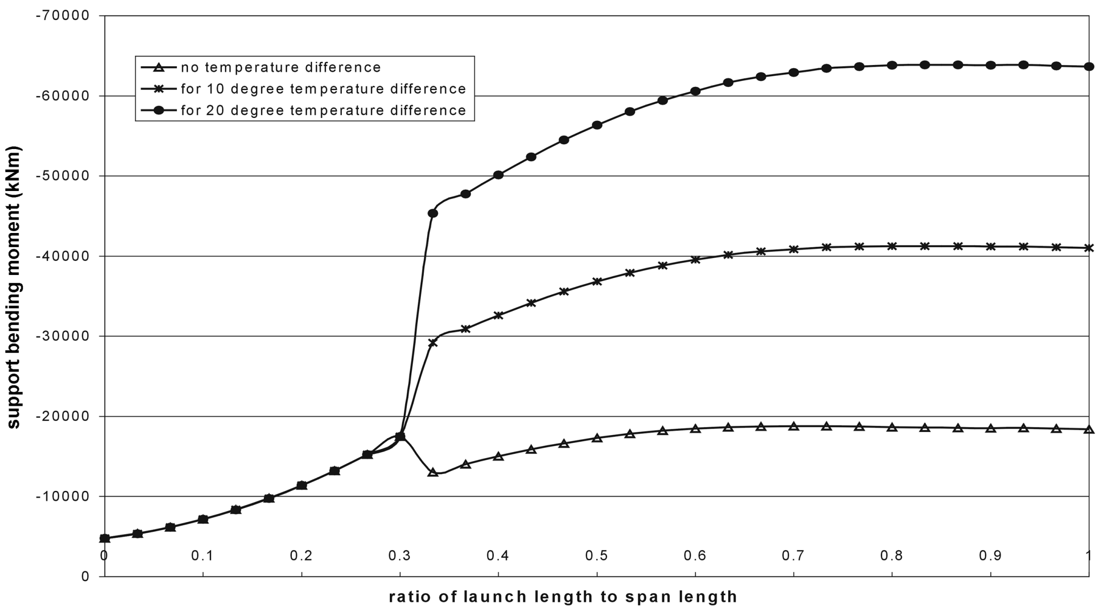

Fig. 20 Effect of temperature variation on support moment for length ratio of 0.67

6. Conclusions

The present paper addresses the construction phase analysis of incrementally launched continuous bridge based on the formulations made using transfer matrix technique. Different parametric studies have been carried out with the help of the computer program developed based on the formulations presented in the paper. The formulations and the computer program have been validated by comparing the results of this study with those obtained using Finite Element Analysis (FEA). The results of this study are in good agreement with those obtained using satndard FEA package. Using this simple and efficient technique, investigations have been carried out to study the influence of load ratio, length ratio and stiffness ratio on the behaviour of deck during incremental launching. It is important to note that for launching cases with short launching nose, the design values are mainly governed by the maximum cantilever moment and behaviour has improved with the increase of length ratio and E.O.L moment governs the design value as the length ratio increases. It is observed that with the change of load ratio the variation of support moment is almost non-responsive when short launching nose is used but with long launching nose, load ratio improves the support moment variation substantially. The load ratio and length ratio are found to be important for optimisation of behaviour of incrementally launched continuous bridges. From this study, it is also clear that these parameters can improve the structural responses only in a particular range which changes with the variation of other parameters. The expression proposed for the optimum geometric properties and load ratio would help the designer to get the optimal section. The method presented in this paper is capable of handling the settlement analysis to evaluate the influence of settlements, that occur during construction, on the stress state of incrementally launched bridge deck. The settlements may produce structural responses which are very different from that of aligned supports. The temperature analysis may be carried out easily with the help of the formulation presented in this paper. The results of parametric studies would help the structural engineer to get insight into the importance of different parameters to be considered during launching of continuous bridges using launching nose. Further, the algorithm presented in this paper provides a simple and efficient tool for construction phase analysis of incrementally launched continuous bridges.

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