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CONSTRUCTION STAGE ANALYSIS OF SEGMENTAL BOX GIRDER BRIDGE

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ABSTRACT: Segmental bridge construction is very common in Pakistan. However, the impact of structural system changes before and after the completion of segmental bridge construction is a challenge and major concern for construction stakeholders. Therefore, in this study, construction stage analysis was performed for balanced cantilever construction technique to check the safety and serviceability of the bridge. Through time dependent analysis, considering the construction sequence and creep deformation of concrete, structural responses related to the member forces were reviewed. Moment reversal was observed in the cantilever segments near the face of the pier. Therefore, maximum compressive and tensile stresses were observed in the top and bottom fibers of the girder, respectively. These stresses were within the AASHTO LRFD limits for construction stage loads. Results demonstrated that the creep moment redistribution started when the structural system changed from statically determinate to indeterminate stage. Furthermore, it was concluded that the time dependent deformation of concrete was the governing factor in order to achieve the final design moments.

Keywords: Segmental bridge construction, balanced cantilever, time dependent analysis, creep deformation.

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INTRODUCTION

Segmental concrete bridge construction has become very essential for crossing deep valleys, wide waterways, highways and in urban areas without the use of costly and environmentally sensitive temporary false work (Islam and Habib, 2000). The most considerable difference from other bridges is the method of construction and flexibility to many applications (ASBI, 2005; Lucko and Garzade, 2003; James and Norman, 1993; and Heins and Lawrie, 1992). Every segmental bridge construction project has its own challenges in type and size selection, design, production, formation and construction (Bishara and Papakonstantinou, 1996; and Chiu *et al*., 1996). Consideration of all intermediate construction stages including changes in structural system, sequence of installing tendons, construction methods and load effects from erection equipment are governing factors in the analysis of segmental concrete bridges (Chung and Shuqing, 2014; and Tackas, 2002; Ketchum, 1986).

A bridge is said to be segmental, if all or some portion of the dead load of the bridge is applied to the structure in such configuration which is different from its final one (Ketchum, 1986). Segmentally erected box girder bridges are widely used in order to achieve economy and to obtain high longitudinal and torsional stiffness (Trivedi, 2014; Kwak and Seo, 2002; and 2004). Segmentally erected prestressed bridges are becoming very popular for moderate spans all over the world (Ketchum, 1986).

The designer must assess the structural system changes in each construction stage and each structural system needs to be analyzed throughout the construction process. Considerable work has been done in the field of segmental concrete construction all over the world. In Pakistan, although such type of bridge construction is becoming very popular, there is hardly any research work carried out in this area. Therefore, in this study, for analysis purpose, segmental bridges constructed by balanced cantilever construction were selected. The main objective was to evaluate internal moment variation before and after the construction of bridge.

MATERIALS AND METHODS

The super structure of selected bridge was modeled on two commercially available computer software's MIDAS 7.01 and STAAD Pro 2007. Three different types of analysis including construction stage analysis, completed stage analysis and time dependent analysis were carried out in order to check the structural response of the bridges.

Selected Bridge Description: A cast-in-place three span continuous box girder bridge was selected having a total length of 150 m. The length of the middle span was 75 m and the length of the two end spans was 37.5 m each. The width of the bridge was 9.6 m and each of its piers had a height of 10 m (Figure 1).

As cast-in-place bridge was assumed in the analysis, the cantilever span on either side of the pier was divided into small segments having a length of 4 m. The length of the closing segment was taken as 1 m. Generally, the width of the segment was selected equal to the width of the bridge. As in the selected bridge, the bridge width was 9.6 m less than 12 m; therefore, a single box cell option was selected instead of box section having multiple webs.

Thickness of top flange was taken as 250 mm between the webs and 225 mm for the overhang portion (Figure 2). Thickness of the bottom slab varied along the parabolic length (450 mm thick bottom slab was taken near the support and 225 mm at the mid span). The thickness for webs was taken as 400 mm in accordance with AASHTO (2007) criteria.

Cantilever length of 2.5 m from centre line of webs to the outer end of the top flange was selected for the bridge cross section. In top flange, 350 mm thick haunches having a length of 1300 mm were provided on both sides of the web to accommodate the longitudinal post tensioning tendons and 250×750 mm thick fillets were provided in the bottom slab. Internal cantilever tendons were installed to resist the maximum negative moment during construction near the pier.

Construction Sequence: It was assumed that total time required during construction process was 250 days. Figure-3 shows the construction sequence adopted for the analysis of balanced cantilever bridge.

Construction Stage Loads: The self weight of the supported structure was calculated on the basis of 2.48 T/m^3 as the unit weight of the concrete. At one part of the cantilever, 2% of the dead load of the total structure was applied.

During construction, due to miscellaneous items, a small proportion of construction live load was taken. For one side of cantilever, 0.050 T/m^2 of deck slab area was applied and on other side 0.025 T/m^2 was taken. A special construction equipment load (i.e., weight of gantry/form traveler) of 80 tons was taken.

Completed Stage Loads: When both cantilevered portions were joined together with closing segment, the superimposed loads were applied on the structure. Superimposed dead loads included weight of new jersey barriers, footpath and wearing surfaces.

The live load over the bridge includes the weight of the moving vehicles and pedestrians. Vehicle live load will be either class “A” (standard truck loading) or class “AA” (tank loading) live load, whichever results in the more critical force effect, will be considered for design.

Analytical Modeling of Balanced Cantilever Bridge: The analysis for the study was carried out using two commercially available computer software's MIDAS 7.01 and STAAD Pro 2007. Two main models were considered with varying structural system. First model was prepared for construction stage analysis in which the structure was in determinate stage (cantilever spans on both sides of pier). Second model was made for completed stage analysis when the structure changed its state from determinate to indeterminate state (closure segment was cast between cantilever segments). Time dependent analysis was carried out on second model by varying time dependent parameters for creep and shrinkage at different ages (t = 1000 days and t = 10,000 days). The deck was modeled as a line element in the form of grillage on STAAD Pro 2007 and bridge wizard was used for modeling in MIDAS 7.01. The moving live loads (truck-train and tank) were generated over the bridge deck along the girder. Description of models are explained in Table 1.

The concrete compressive strength for approach slab and sidewalks, piers and box girder was 21 MPa, 28 MPa and 35 MPa, respectively. The yield strength for reinforcement was considered as 420 MPa.

RESULTS AND DISCUSSION

Creep Coefficient and Creep Moment: Creep coefficient of concrete was determined in an appropriate way by using analytical equations considering the humidity around the structural member, shape and dimensions of member sections, age of concrete when the load acts on it. Based on CEB-FIP model code (1990), influence of creep on concrete (creep coefficient) was computed as follows (Eq. 1):

$$\varphi\,(t,t_0) = \varphi d_0 \beta_d(t,t_0) + \varphi f_0\{\beta_f(t) - \beta_f(t_0)\} \quad \text{Eq. 1}$$

Where, $\varphi\ (t,t_0)$ was the creep coefficient of concrete at an age of t-th day, with a sustained load imposed on the concrete at an age of t_0-th day. t_0 and t are the ages of concrete (days) when the sustained load was imposed and when the value of creep coefficient is calculated. The age t_0 and t was calculated by the following equation (Eq. 2) according to the concrete temperature and cement type.

$$t\ or\ t_0 = \frac{\alpha \sum (T+10)\Delta t}{30} \qquad \text{Eq. 2}$$

Where, α was 2.0 for high early strength portland cement and 1.0 for normal portland cement; T was the concrete temperature (^oC) and Δt was the number of days during which the concrete temperature is T ^oC. φd_0 was the creep coefficient for the strain that would vanish with the passage of time (delayed elastic strain, if the sustained load was removed then 0.4 was taken in general. $\beta_d\,(t - t_0)$ was the function of the time ($t - t_0$) days since the imposition of the sustained load. If the time ($t - t_0$) days since the imposition of the sustained load exceeds 3 month, $\beta_d\ (t - t_0) = 1$ might be taken. φf_0 was the creep coefficient for the creep strain that would not vanish. $\beta_f\ (t)$ was the function of the age t (days) of concrete and the equivalent member thickness t_{th} can be calculated as follows (Eq. 3):

$$t_{th} = \frac{\lambda A_c}{u} \qquad \text{Eq. 3}$$

Where, λ is the coefficient related to environmental conditions; A_c is the cross-sectional area of member (mm^2) and u is the peripheral length of member exposed to the open air (mm).

Based on above equations (Eqs. 1-3) creep coefficient was calculated as 0.890 and 1.250 for 1000 days and 10,000 days, respectively.

Creep moment redistribution was done by using Dischinger's formulation. Moments at end span and mid span for 1000 days and 10,000 days are shown in Table 2.

Construction Stage Moment Diagram: Figure 4 illustrated the maximum cantilever moment due to construction stage load at the centre line of pier. The behavior of moment variation was the same at other piers because both cantilevers were constructed simultaneously. Maximum tensile stresses were developed at top fibers of box girder section and bottom fibers were subjected to compressive stresses. Figure 5 showed the moment values at different sections due to top cantilever tendons and this diagram showed entirely opposite behavior from Fig- 4. Due to maximum prestressing moment at the centre line of the pier, moment reversal was observed at construction stage. Similar results were reported in previous studies carried out by (Kwak and Seo, 2002; and 2004). Ultimately, tensile stresses were developed at top fibers of box girder section and compressive stresses were developed at bottom fibers.

Construction Stage Stress Check: The maximum allowable compressive and tensile stresses can be considered as $0.6f'_{ci}$ and $3\sqrt{f'_{ci}}$, respectively. Where, f'_{ci} is the compressive strength of concrete at the time of initial prestress. Stresses at top fiber (σ_1) and bottom fiber (σ_2) of box girder section due to construction stage loads could be calculated as follows (Eqs. 4 and 5):

$$\sigma_1 = \frac{M_T}{Z_t} \;\; \text{(positive for tension)} \qquad \text{Eq. 4}$$

$$\sigma_2 = \frac{M_T}{Z_b} \;\; \text{(negative for compression)} \qquad \text{Eq. 5}$$

Where, M_T is the total moment, Z_t and Z_b are the section modulus for top and bottom fiber, respectively. Furthermore, stresses due to prestressing force at transfer can be estimated using Eqs. 6 and 7. At Construction stage only top tendons were included.

$$\sigma_1 = \left(\frac{-P_i}{A}\right) + \left(\frac{-P_i e C_t}{I_x}\right); \;\; \text{stresses at top fiber} \quad \text{Eq. 6}$$

$$\sigma_2 = \left(\frac{-P_i}{A}\right) - \left(\frac{-P_i e C_b}{I_x}\right); \;\; \text{stresses at bottom fiber} \quad \text{Eq. 7}$$

Where, σ_1 and σ_2 were the top and bottom fiber stresses, P_i was the initial prestressing force; A was the cross sectional area; e was the eccentricity of centroid of prestressing steel with respect to centroid of the gross section; I_x was the moment of inertia; C_t and C_b were the distance from the centroid of gross section to top and bottom fibers, respectively. The resultant stresses at construction stage were the sum of the stresses due to construction load and prestressing force as shown in Table 3. The resultant stresses satisfied the allowable stresses limitations (Table 3).

Completed Stage Moment Diagram: Figure 6 illustrates the moment envelope at service stage. The moment diagram included the impact of self-weight (DL), superimposed dead load (SDL), live load (LL), top prestressing tendons (TP) and bottom prestressing tendons (BTP).

Completed Stage Stresses Check: The maximum allowable compressive and tensile stresses for completed stage can be calculated as $0.6fc'$ and $3\sqrt{f'_c}$, respectively. Where, f'_c was the compressive strength of concrete at 28 days. Table 4 showed the service stage stress values, indicating the values within the allowable limits.

Thermal Stress Analysis: Thermal stress analysis was carried out for three main reasons: Temperature rise or fall in the mean temperature of the body of structure, effect of non-linear distribution of temperature across the deck depth and continuity effect.

Temperature rise/fall in the mean temperature of the body of structure: The mean temperature was considered 50 ^oC. Range (ΔT) was 25 ^oC. The coefficient of thermal expansion (α) was assumed to be 0.000012/^oC Therefore, total change in length of bridge was calculated as $L\alpha\Delta T$ = 150×0.000012×25 = 45 mm.

Effect of non linear distribution of temperature across the deck depth: The total stress (f_c) was the sum of f_{c1}, f_{c2} and f_{c3}. Assuming E equals to 2.78E+07 kN/m^2 and the difference in temperature between top and bottom deck slab was 10 ^oC. The value of f_{c1} was calculated as: f_{c1} = $E\alpha\Delta T$ = -2.78E+07 × 2.78E +07 × 10 = -3334.8 kN/m^2. The value for f_{c2} was calculated as f_{c2} = $E\alpha\Delta T$ × A/B. Where, A was the area of top slab/flange of box girder and B was the cross sectional area of box girder. The value for fc_{3b} was calculated as: $fc_{3b} = E\alpha\Delta TA(C_t\text{-}t_f).(C_b/I_x)$. The effect of non-linear distribution of temperature was calculated at two critical sections (support and mid span). For supports, the stress at top fiber (f_t) was calculated as f_t = f_{c1} + f_{c2} + f_{c3t} = -3334.8 + 922.6 +1116.13 = -1265.9 kN/m^2. Similarly, bottom fiber stress (f_b) at support was: f_b = f_{c2} + f_{c3b} = 922.6 -1416.13 = -493.53 kN/m^2. For section at mid span, the top and bottom fiber stresses were -1082.48 KN/m^2 and -274.45 KN/m^2, respectively.

Stress Due To Continuity Moment: The effect of intermediate support restraint on the free hogging (or sagging). Desire of the structure caused by unequal extreme fiber temperature produces the continuity effect. The moment can be calculated as:

$$M = \frac{EI\alpha\Delta T}{h}$$

Where, h was the depth (at support section = 4.5 m and at mid span = 2.25 m), ΔT was the temperature difference (10 ^{o}C). At support, the total stresses at top and bottom fiber were -2735.44 kN/m^2 and 1370.97 kN/m^2, respectively. Top and bottom fiber stresses at mid span were -2264.78 kN/m^2 and 1749.55 kN/m^2, respectively.

In order to satisfy the tensile stress check, all the bottom fiber stresses should be less than $7\sqrt{f_c'} = 7\sqrt{5000}$ = 495 psi = 3414 kN/m^2. Therefore, all the bottom fiber stresses were within the allowable limits.

Time Dependant Analysis Results: Figure 7 showed that the time dependent moment redistribution initiated after the continuity of the cantilever parts. With the passage of time, negative moment near the support decreased and positive moment increased at the mid span. Figure 8 illustrated the behavior of tendon moment redistribution at different time intervals (t = 1000 days, t = 10,000 days). Analysis of results showed that with the passage of time, tendon moment decreased at support and at mid span.

Figure 9 described the variation of moments at different stages of construction. Analysis results showed that moment diagrams including creep effect which lies somewhat between the construction stage moment diagram and completed stage moment diagram. Similar findings regarding moment redistribution at various construction stages were reported in previous studies (Ketchum, 1986; Tackas, 2002; Kwak and Seo, 2002; 2004; and Trivedi, 2014).

Table 1: Model description.

Model	Description
Model 1	Bridge was modeled in order to get maximum cantilever moments without placing closing segment.
CNSA-M1	Construction stage analysis is performed to investigate the stability at construction stage.
Model 2 CMSA-M2	Complete bridge was modeled after placing of closing segment. Analysis was performed to investigate the performance of bridge at service stage.
Model 3 TDA-M3	Time dependent analysis was performed to investigate the moment variation along the span after construction at t = 1000 days. TDA-M3 was made by using time dependent parameters in model 2.
Model 4 TDA-M4	Time dependent Analysis was performed to investigate the moment variation along the span after construction at t = 10000 days. TDA-M4 was made by using time dependent parameters in model 2.

Table 2: Moment redistribution for end and middle span

Span	Time (days)	M_L (T-m)	M_B (T-m)	M_B-M_L (T-m)	K=1-e$^{-\emptyset}$	(M_B-M_L)K (T-m)	M_L+(M_B-M_L)K (T-m)
End	1000	8510	3360	-5150	0.75	-3841	4669
	10000	8510	3360	-5150	0.88	-4513	3997
Middle	1000	8510	6370	-2140	0.75	-1605	6905
	10000	8510	6370	-2140	0.88	-1875	6635

Table 3: Resultant construction stage stresses

Stresses		Values
Construction stage load	M_T (T-m)	11361
	σ_1 (T/m^2)	729
	σ_2 (T/m^2)	-949
Prestressing force	Pi (T)	6680
	σ_1 (T/m^2)	-1354
	σ_2 (T/m^2)	74
Total stresses	σ_1 (T/m^2)	-626
	σ_2 (T/m^2)	-875
Allowable stresses	σ_1 (T/m^2)	134
	σ_2 (T/m^2)	-1687
Stress check	Ok	

Table 4: Stresses at service stage

Stresses		Values
Service stage	M_T (T-m)	4274
	σ_1 (T/m^2)	274
	σ_2 (T/m^2)	-357
Allowable stresses	σ_1 (T/m^2)	298
	σ_2 (T/m^2)	-2109
Stress check	Ok	

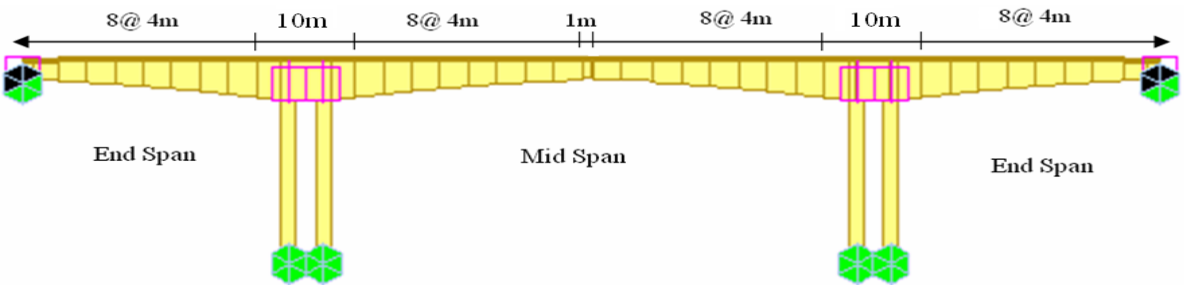

Figure 1: Graphical representation of bridge model

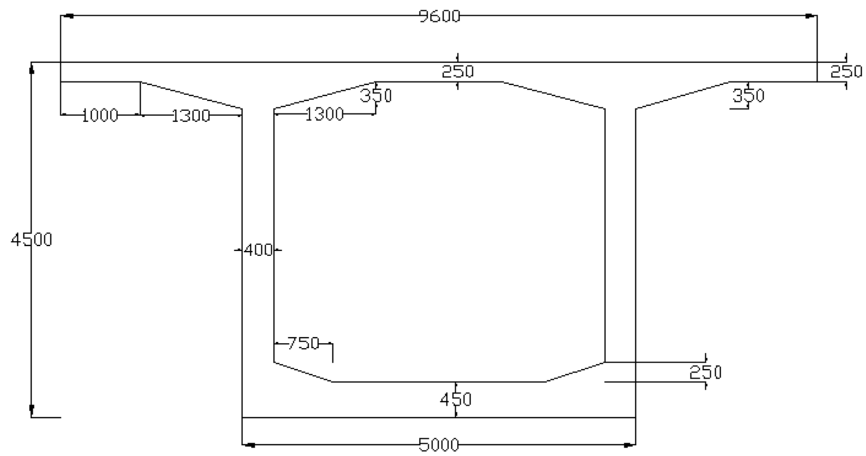

Figure 2: Typical cross-sectional properties (Units are in millimeters (mm))

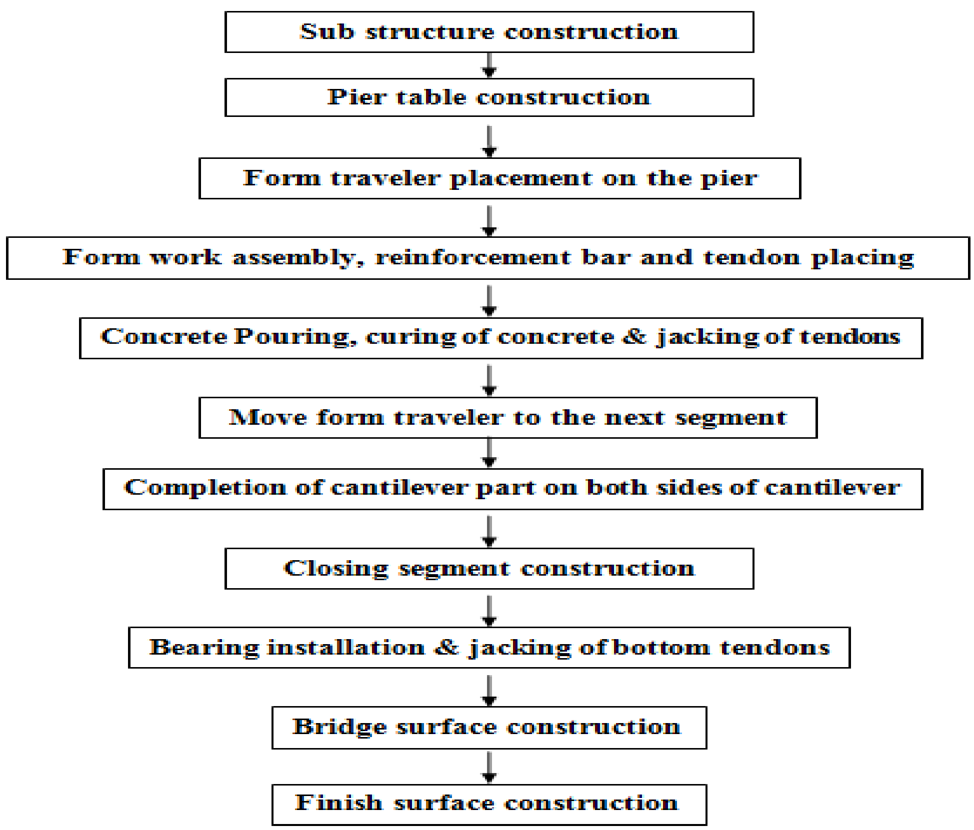

Figure 3: Construction process sequence

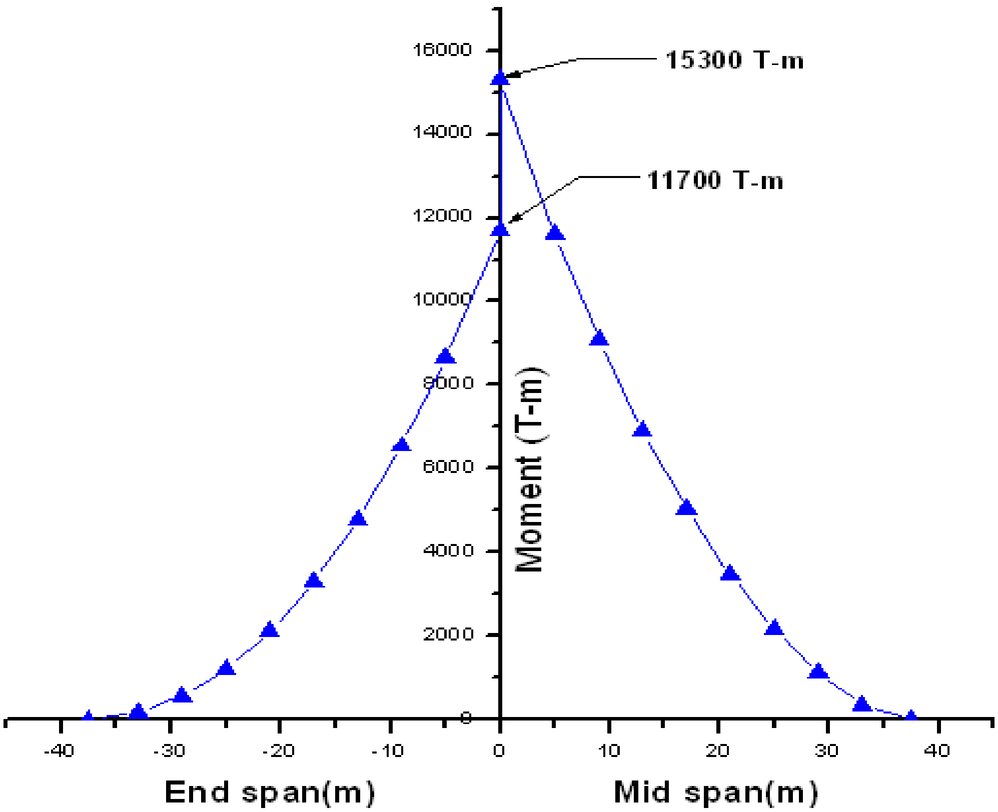

Figure 4: Moment due to dead loads

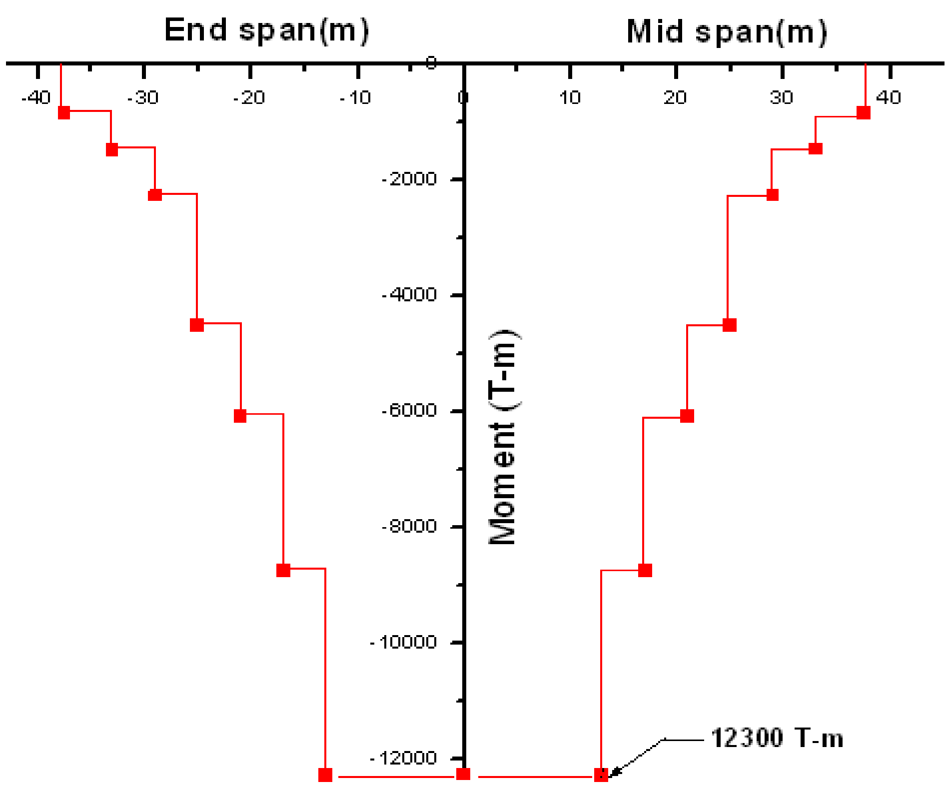

Figure 5: Moment due to top cantilever tendons

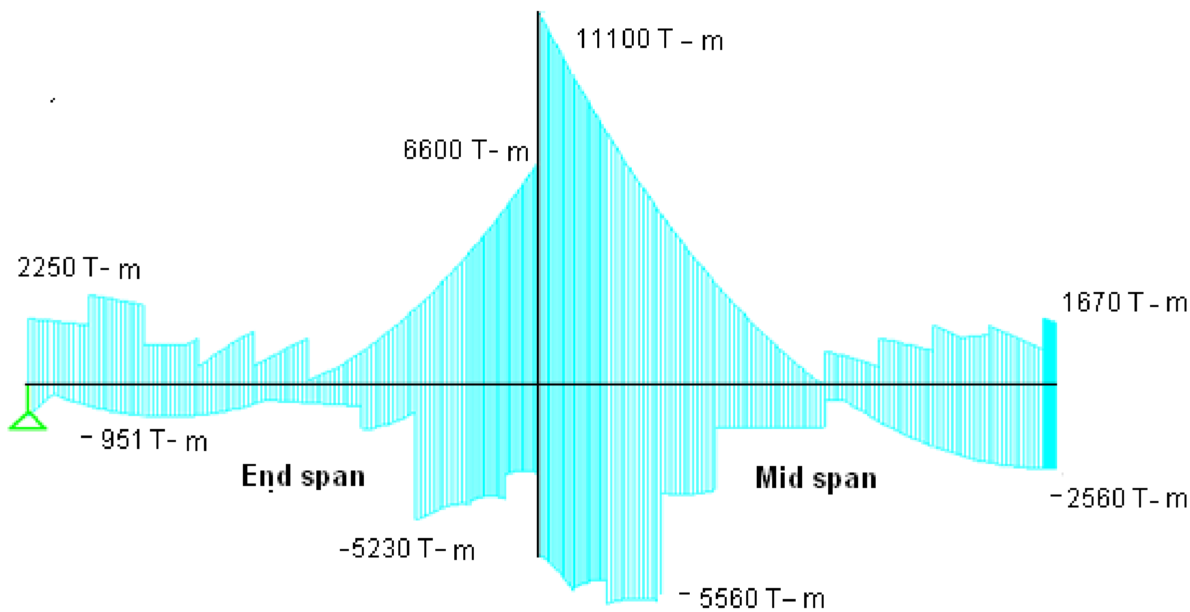

Figure 6: Moment envelope

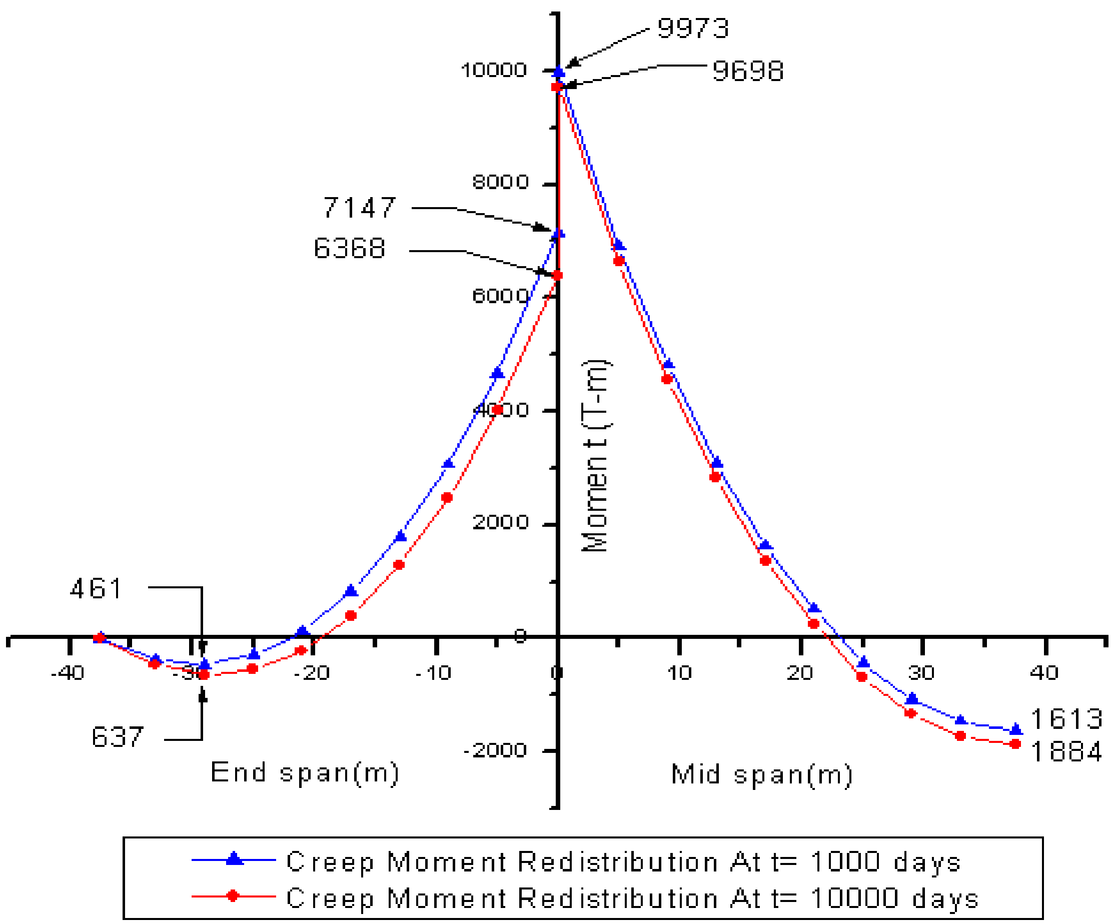

Figure 7: Creep moment redistribution at different time interval

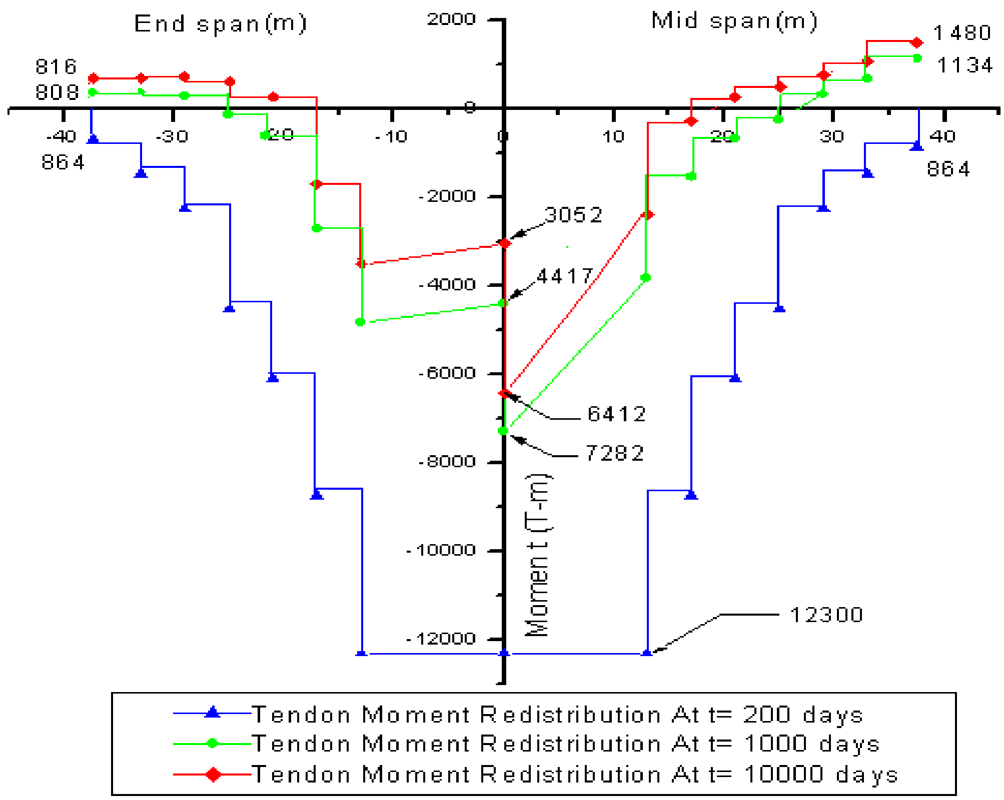

Figure 8: Tendon moment redistribution at different time interval

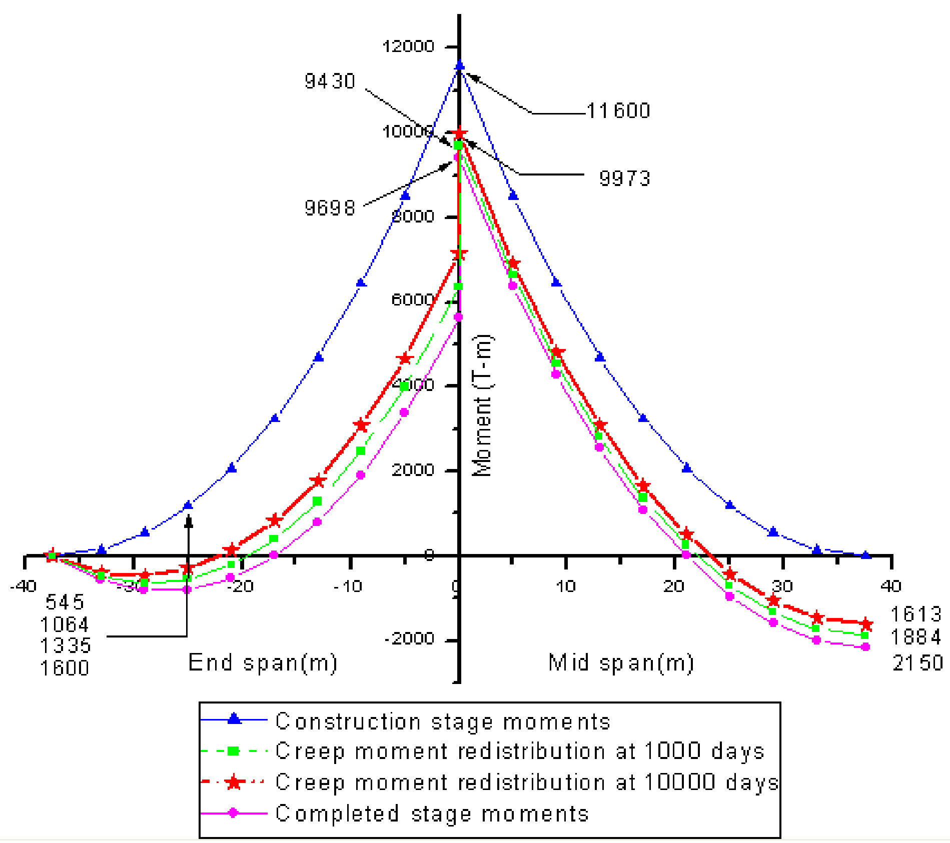

Figure 9: Creep moment redistribution at different time interval due to dead loads

Conclusions: Analysis for construction loads indicated that first segment near the face of the pier was over prestressed due to cantilever tendons during construction to resist the total dead load of the girder and other construction loads. Due to these tendons, moment reversal occured in the cantilever segments near the face of the pier. Hence, maximum compressive stresses were observed in the top fibers of the girder and bottom fibers were subjected to tensile stresses. These stresses were checked against the limits given in AASHTO LRFD for construction stage loads and found to be satisfactory. It was observed that the time dependent internal moment redistribution due to creep deformation of concrete started after the completion of construction when the bridge configuration changes from statically determinate to indeterminate stage. Due to creep moment redistribution, the negative moment reduces near the pier and positive moment increases at mid span. The construction stage negative moments due to self weight at pier were reduced to about 17% by creep moment redistribution at 1000 days, and about 20% after 10,000 days. The positive moments at mid span were developed after continuity of the bridge. With the passage of time, cantilever tendon moment decreases at support section and at mid span.

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