Steel and Composite Structures, Vol. 12, No. 5 (2012) 413-426
DOI: http://dx.doi.org/10.12989/scs.2012.12.5.413

Construction stage analysis of three-dimensional cable-stayed bridges

Barbaros Atmaca and Sevket Ates*

Karadeniz Technical University, Department of Civil Engineering, 61080, Trabzon, Turkey

(Received December 11, 2011, Revised March 20, 2012, Accepted March 22, 2012)

Abstract. In this paper, nonlinear static analysis of three-dimensional cable stayed bridges is performed for the time dependent materials properties such as creep, shrinkage and aging of concrete and relaxation of cable. Manavgat Cable-Stayed Bridge is selected as an application. The bridge located in Antalya, Turkey, was constructed with balanced cantilever construction method. Total length of the bridge is 202 m. The bridge consists of one ë shape steel tower. The tower is at the middle of the bridge span. The construction stages and 3D finite element model of bridge are modeled with SAP2000. Large displacement occurs in these types of bridges so geometric nonlinearity is taken into consideration in the analysis by using P-Delta plus large displacement criterion. The time dependent material strength and geometric variations are included in the analysis. Two different finite element analyses carried out which are evaluated with and without construction stages and results are compared with each other. As a result of these analyses, variation of internal forces such as bending moment, axial forces and shear forces for bridge tower and displacement and bending moment for bridge deck are given with detailed. It is seen that construction stage analysis has a remarkable effect on the structural behavior of the bridge.

Keywords: manavgat cable-stayed bridge; construction stage analysis; time dependent material properties; 3d finite element model; balanced cantilever method.

1. Introduction

Structures are primarily used for the purpose of protection, and later building of different structures came into existence due to the different needs of human. One of the most attractive structures of these is bridges. In general bridges consist of towers, supports and deck. The bridges are used since the existence of human for beings to overcome obstacles such as rivers and deep valleys. Different types of bridges occurred after human knowledge has increased. These types of bridges can be grouped with the five main categories like girder, arch, truss, suspension bridges and cable-stayed bridges. The basic components of the cable-stayed bridges are deck, towers, main beams and cables. Unlike the suspended bridge and cable-stayed bridge's deck are directly connected to the tower with cable. Especially the number of cable-stayed bridges, increased rapidly after World War II. Cable-stayed bridges appear aesthetically pleasing, economical and easy construction that increased the area of application.

The constructions of large engineering structures such as bridges take a long time. During the construction of bridge, loads which structure is exposed continuously vary with time. But analysis of

* Corresponding author, Assistant Professor, E-mail: sates@ktu.edu.tr

structure with traditional method is considered structure constructed and loaded in a moment. But this situation does not reflect the actual behavior of structure because the properties of materials which used in structure changes with time. It could not be possible obtaining realistic results from non-anticipating real construction behavior settlement. The analysis of large engineering structures according to construction process like in the field will provide more realistic results to be obtained. Therefore, bridges and structure such as long-term building process, during the analysis of these structures time dependent material properties and construction stages must be taken into account. In the literature, some papers exist about the construction stage analysis of the bridges considering time dependent material properties. Nazmy (1987) studied on cable-stayed bridges and emphasized cable-stayed bridges required a non-linear analysis due to major changes in bridge geometry. In that study, it was reported that two-dimensional model of cable-stayed bridge could not reflect real behavior of bridges because two-dimensional model does not show the effects of the third dimension. Cluyet and Shepherd (1990) researched the effects of the time and external influences on the cable-stayed bridges. Abbas (1993) focused on the analysis of cable-stayed bridges built with balanced cantilever method by considering time-dependent effects such as shrinkage creep and relaxation while taking account of the bridge modeled as a two-dimensional. Somja and Goyet (2008) came up with an efficient numerical procedure for materially and geometrically nonlinear finite element analysis of segmentally erected structures including time dependent effects due to load history, creep, shrinkage and aging of concrete. In that study, it was observed that time have a strong influence, especially, on concrete type structures. Therefore, it was emphasized that these effects must be taken into account in the design process. Cho and Kim (2008) evaluated the risks in a suspension bridge by considering an ultimate limit state for the fracture of main cable wires. They examined the results compared with the conventional safety indices and allowable error for the control of deformations during construction. Altunisik *et al.* (2010) carried out the construction stage analysis of highway bridges constructed with balanced cantilever method using time dependent material properties. Ates (2010) examined the finite element analysis of long-span, concrete box girder highway bridges by balanced cantilever method. The time dependent material properties are also considered. Soyluk *et al.* (2010) studied non-linear analysis of cable bridges using balanced cantilever method. Malm and Sundquist (2010) studied the time-dependent analyses of segmentally constructed balanced cantilever bridges.

As seen in literature, studies on construction stage analysis of three dimensional cable-stayed bridges not enough and need to be enlarged by inserting new studies. So, in this paper, Manavgat Cable-Stayed Bridge which constructed with balanced cantilever method is selected as an application. Construction stage analysis of three dimensional model of the bridge is performed with using time dependent material properties.

2. Description of manavgat cable-stayed bridge

In this study, Manavgat Cable-Stayed Bridge is preferred for a numerical application. The bridge is first example of the cable-stayed bridge of Turkey. The bridge, shown in Fig. 1, is 202 m long and 13.7 m in width, with equal spans of 101 m; and designed for two lanes of road traffic. The bridge have approximately 42 m λ shape steel pylon; which is 40 m in height above the steel deck. The pylon has a hollow hexagonal cross-section. The deck of bridge is composite and consists of 25 cm thick concrete, 10 cm thick asphalt and steel profiles. The main I cross section steel profiles which is used in the deck extends continuously from one end to the other end of the bridge.

Fig. 1 Manavgat Cable-Stayed Bridge

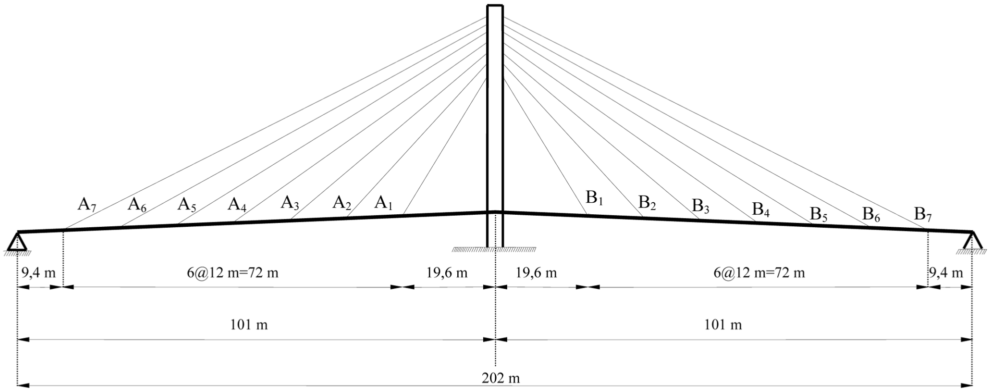

Fig. 2 The schematic form of Manavgat Cable-Stayed Bridge

Deck of the bridge is supported with 28 steel cables which is a link to tower. The distance between the pylon and the closest cable to the pylon is 19.6 m while the distance between cables is 12 m. Distance between supports which are on shore and last cable connection point on the deck is 9.4 m, as well. The schematic form of Manavgat Cable-Stayed Bridge is shown in Fig. 2 and stay cable properties are given in Table 1.

3. Finite element model of the bridge

Three-dimensional finite element model is formed in SAP2000 (2008) in order to determine the structural behavior of Manavgat Cable-Stayed Bridge. The bridge model consist of 2257 nodal points, 28 truss elements, 1102 beam elements and 1980 area elements. The deck and tower are represented with beam elements while cables are described by using truss elements. The computer model of the bridge is given in Fig. 3.

Table 1 Stay cable properties

Cable No	Number of strands	Diameter of strands (mm)	Total area of cable (mm^2)
A_1	15	15,2	2722
A_2	16	15,2	2903
A_3	19	15,2	3448
A_4	19	15,2	3448
A_5	22	15,2	3992
A_6	19	15,2	3448
A_7	24	15,2	4355
B_1	15	15,2	2722
B_2	16	15,2	2903
B_3	19	15,2	3448
B_4	19	15,2	3448
B_4	22	15,2	3992
B_6	19	15,2	3448
B_7	24	15,2	4355

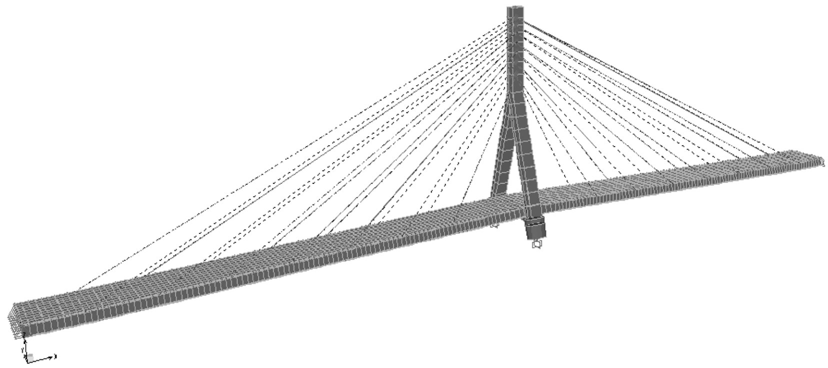

Fig. 3 Three-dimensional finite element model of the Bridge

4. The construction stages

The structural analysis methods excluding are assumed that the structures are built and loaded at one time. In addition, time dependent material properties are not taken into account in these methods. On the contrary, the constructions of large engineering structures take a long time and major changes in the properties of the material occur during this time. In construction stage analysis of the structure time-dependent material changes taken into accounting and construction stage of structure modelling same way in the computer how structure building in the field. The construction of Manavgat Cable-Stayed Bridge has 13 stages. The total construction period is taken as 243 days. Some of the stages are given in Fig. 4. The schedule related to the construction stages are shown in Table 2.

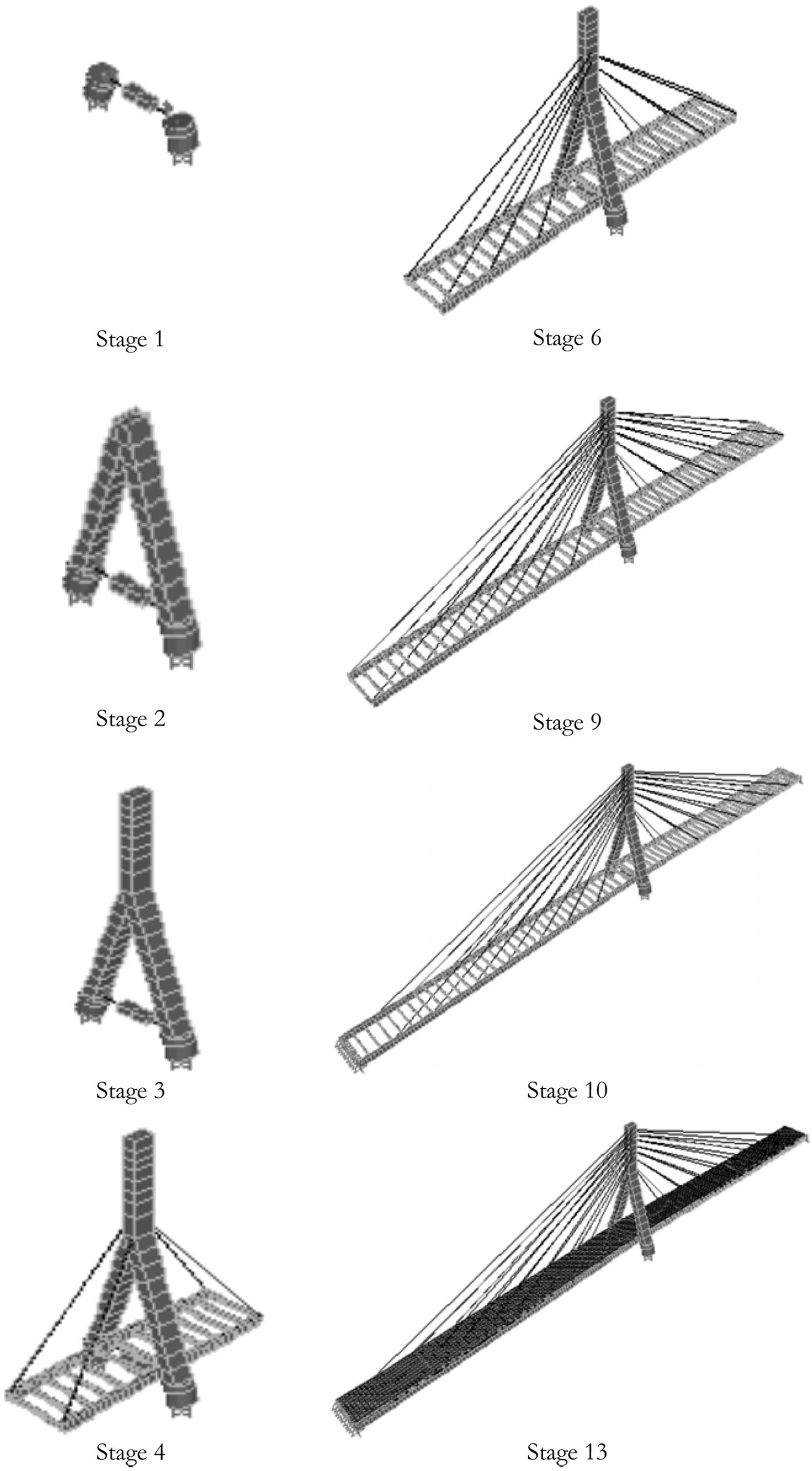

Fig. 4 Some construction stages of the bridge

Table 2 Construction schedule of Manavgat Cable-Stayed Bridge

Stage Number	Construction phase	Time (Day)
1	Completion of the concrete pylon base	30
2	The construction of the steel pylon base	20
3	Completion of the construction of the tower	20
4	The construction of the first deck segment	30
5 to 11	The construction of steel deck segments	7×20
12	The construction of reinforced concrete slab	3
13	Completion of the asphalt layer	1
	Total	243

5. Time dependent material properties

In order to determine the effect of material properties for structural behavior of Manavgat Cable-Stayed Bridge, relaxation of steel material, creep, shrinkage and aging of concrete are taken into account. Selected analysis parameters to consider time dependent material properties are given in Table 3.

5.1 Time dependent properties for concrete

5.1.1 Compressive strength

The compressive strength of concrete at an age t depends on the type of cement, temperature and curing conditions. The relative compressive strength of concrete at various ages may be estimated by the following formula (CEB-FIP 1990)

Table 3 Selected parameters for time dependent material properties

<table>
<tr><th colspan="2" rowspan="2">Parameters</th><th colspan="2">Materials</th></tr>
<tr><th>Concrete</th><th>Prestress steel</th></tr>
<tr><td rowspan="2">Nonlinear material data</td><td>Hysteresis type</td><td>Kinematic</td><td>Kinematic</td></tr>
<tr><td>Stress-Strain diagram</td><td>User defined</td><td>User defined</td></tr>
<tr><td rowspan="12">Time dependent properties</td><td>Elasticity modulus</td><td>Yes</td><td>YES</td></tr>
<tr><td>Creep</td><td>Yes</td><td>N/A</td></tr>
<tr><td>Shrinkage</td><td>Yes</td><td>N/A</td></tr>
<tr><td>Creep analysis type</td><td>Full</td><td>N/A</td></tr>
<tr><td>Cement type coefficient</td><td>0.25</td><td>N/A</td></tr>
<tr><td>Relative humidity %</td><td>50</td><td>N/A</td></tr>
<tr><td>Notional size</td><td>0.1</td><td>N/A</td></tr>
<tr><td>Shrinkage coefficient</td><td>5</td><td>N/A</td></tr>
<tr><td>Shrinkage start age</td><td>0</td><td>N/A</td></tr>
<tr><td>Steel relaxation</td><td>N/A</td><td>Yes</td></tr>
<tr><td>Relaxation analysis type</td><td>N/A</td><td>Full integration</td></tr>
<tr><td>CEB-FIP class</td><td>N/A</td><td>1</td></tr>
</table>

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \tag{1}$$

in which $\beta_{cc}(t)$ is a coefficient with depends on the age of concrete and is calculated by

$$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\frac{28}{t/t_1}\right)^{1/2}\right]\right\} \tag{2}$$

$f_{cm}(t)$ is the mean concrete compressive strength at an age of t days, f_{cm} is the mean compressive strength after 28 days, t is the age of concrete in days and s is a cement type coefficient. The mean concrete compressive strength is given in Fig. 5

5.1.2 Aging of concrete

The modulus of elasticity of concrete changes with time. For this reason, the modulus at an age $t \neq 28$ days may be estimated as below equation

$$E_{ci}(t) = E_{ci}\sqrt{\beta_{cc}(t)} \tag{3}$$

where $E_{ci}(t)$ is the modulus of elasticity at age of t days; E_{ci} is the modulus of elasticity at an age of 28 days; $\beta_{cc}(t)$ is a coefficient which depends on the age of concrete. For the deck and the piers of the example bridge, the aging of concrete is plotted in Fig. 6

5.1.3 Shrinkage of concrete

The CEB-FIP Model Code (1990) gives the following equation of total shrinkage strain of concrete

$$\varepsilon_{cs}(t, t_s) = \varepsilon_{cso}\beta_s(t - t_s) \tag{4}$$

where ε_{cso} is notional shrinkage coefficient; β_s is the coefficient to describe the development of shrinkage with time; t is the age of concrete in days and t_s is the age of concrete in days at the beginning of shrinkage. The notional shrinkage coefficient may be obtained from

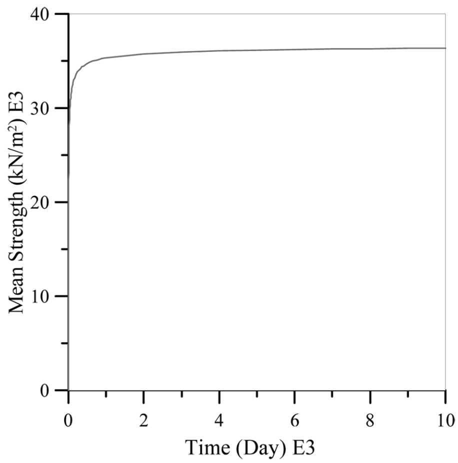

Fig. 5 Variation of the mean concrete compressive strength with days

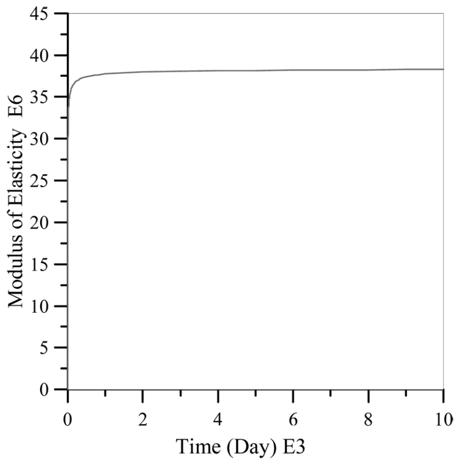

Fig. 6 Aging of concrete in days

$$\varepsilon_{cso} = \varepsilon_s(f_{cm})\beta_{RH} \tag{5a}$$

$$\varepsilon_s(f_{cm}) = \left[160 + 10\beta_{sc}\left(9 - \frac{f_{cm}}{f_{cmo}}\right)\right] \tag{5b}$$

where f_{cm} is the mean compressive strength of concrete at the age of 28 days in MPa; f_{cmo} is taken as 10 MPa; β_{sc} is a coefficient ranging from 4 to 8 which depends on the type of cement.

$$\begin{aligned} \beta_{RH} &= -1.55\beta_{sRH} && 40\% \le RH < 99\% \\ \beta_{RH} &= 0.25 && RH \ge 99\% \end{aligned} \tag{6}$$

where

$$\beta_{sRH} = 1 - \left(\frac{RH}{RH_o}\right)^3 \tag{7}$$

with RH is the relative humidity of the ambient atmosphere (%) and RH_o is 100%. The development of shrinkage with time is given by

$$\beta_s(t - t_s) = \sqrt{\frac{(t - t_s)/t_1}{350(h/h_o) + (t - t_s)/t_1}} \tag{8}$$

where h is the notional size of member (mm) and is calculated by $h = 2A_c/u$ in which A_c is the cross-section and u is the perimeter of the member in contact with the atmosphere; h_o = 100 mm and t_1 = 1 day. For the deck and the piers of the example bridge, the shrinkage strain of concrete depending on relative humidity, notional size and shrinkage coefficient is depicted in Fig. 7.

5.1.4 Creep

The effect is calculated using creep model [17]. For a constant stress applied at time t_o, this leads to

$$\varepsilon_{cc}(t, t_o) = \frac{\sigma_c(t_o)}{E_{ci}}\phi(t, t_o) \tag{9}$$

in which $\sigma_c(t_o)$ is the stress at an age of loading t_o; $\phi(t, t_o)$ is the creep coefficient and is calculated

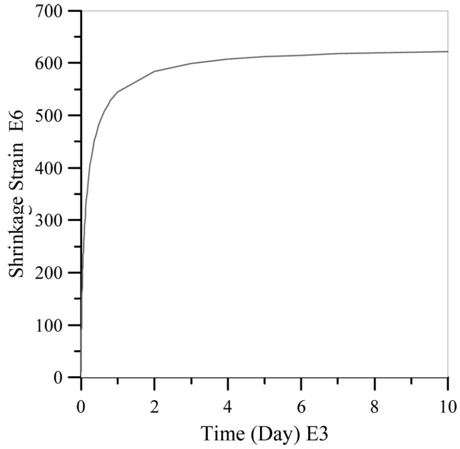

Fig. 7 Time dependent shrinkage strain of concrete

from

$$\phi(t, t_o) = \beta_c(t - t_o)\phi_o \tag{10}$$

where β_c is the coefficient to describe the development of creep with time after loading; t is the age of concrete in days at the moment considered; to is the age of concrete at loading in days. The creep coefficient is explained by

$$\phi_o = \phi_{RH}\beta(f_{cm})\beta(t_o) \tag{11a}$$

$$\phi_{RH} = 1 + \frac{1 - \left(\frac{RH}{RH_0}\right)}{0.46\left(\frac{h}{h_o}\right)^{1/3}} \tag{11b}$$

$$\beta(f_{cm}) = \frac{5.3}{\sqrt{\frac{f_{cm}}{f_{cmo}}}} \tag{11c}$$

$$\beta(t_o) = \frac{1}{0.1 + \left(\frac{t_o}{t_1}\right)^{0.2}} \tag{11d}$$

All parameter is defined above. The development of creep with time is given by

$$\beta_c(t - t_o) = \left[\frac{(t - t_o)/t_1}{\beta_H + (t - t_o)/t_1}\right] \tag{12a}$$

$$\beta_H = 150\left\{1 + \left(1.2\frac{RH}{RH_o}\right)^{18}\right\}\frac{h}{h_o} + 250 \le 1500 \tag{12b}$$

where $t_1 = 1$ day; $RH_o = 100$ and $h_o = 100$ mm. In the analysis, the creep coefficient of concrete is given in Fig. 8 for the deck and the piers having different notional size

5.2 Time dependent properties for steel

According to CEB-FIP Model Code, relaxation classes referring to the relaxation at 1000 hours are divided into three groups for prestressing steels. The first relaxation class is defined as the normal relaxation characteristics for wires and strands, the second class is defined as improved relaxation characteristics for wires and strands, and the last one is defined as relaxation characteristics for bars.

For an estimate of relaxation up to 30 years the following formula may be applied

$$\rho_t = \rho_{1000}\left(\frac{t}{1000}\right)^k \tag{13}$$

where ρ_t is the relaxation after t hours; ρ_{1000} is the relaxation after 1000 hours; $k \approx \log(\rho_{1000}/\rho_{100})$ in

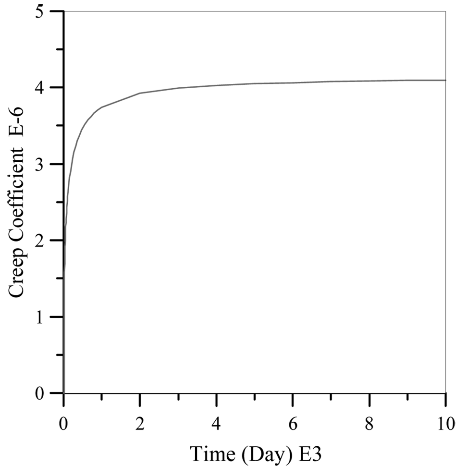

Fig. 8 Time dependent creep coefficient

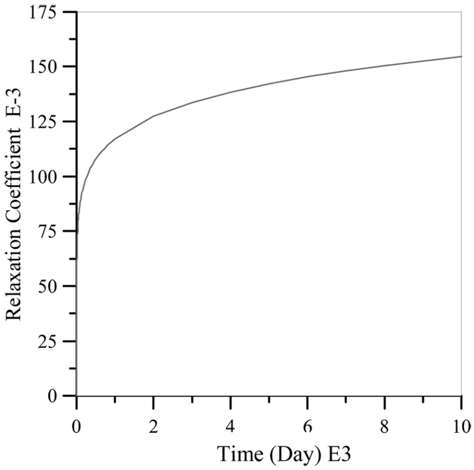

Fig. 9 Time dependent relaxation coefficient of prestressing steel

which k to be 0.12 for relaxation class 1, and 0.19 relaxation class 2; ρ_{100} is the relaxation after 100 hours. Normally, the long-term values of the relaxation are taken from long-term tests. However, it may be assumed that the relaxation after 50 years and more is three times the relaxation after 1000 hours. In the example bridge, all tendons compose of 19 strands with ϕ15.2 mm ant they are categorized as the normal relaxation characteristics for wires and strands, so the relaxation class is taken as class1. The relaxation coefficient as per mentioned properties is given in Fig. 9.

6. Numerical results

The construction stage analysis is performed using SAP2000 (2008) program. P-Delta plus large displacements and geometric nonlinearity options are considered due to large displacements occurred in the bridge body and this cause geometric nonlinearity, such as cable sag, axial force-bending moment interaction in the bridge deck and tower, and change of the bridge geometry. Dead and additional loads are taken due to weight of all elements and 10 cm thickness asphalt on the deck.

6.1 Deck response

Distribution of bending moments and vertical displacements along the deck obtained from the analysis with and without construction stage are given in Figs. 10 and 11. The maximum values of bending moment are reached to at the junction of tower and deck. The maximum values of bending moment achieve 6105 kNm and 5295 kNm for the analysis with and without construction stages, respectively.

The maximum value of vertical displacement occurs at the far from the pylon and close to the anchors on shore. The maximum vertical displacement occurs 33 cm and 21.4 cm for the analysis with and without construction stages, respectively.

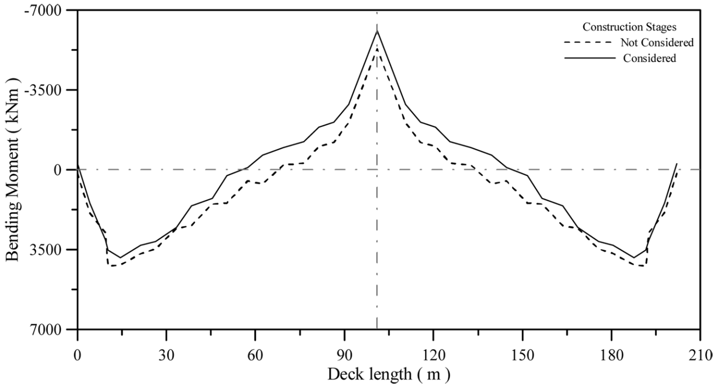

Fig. 10 Variation of deck bending moments

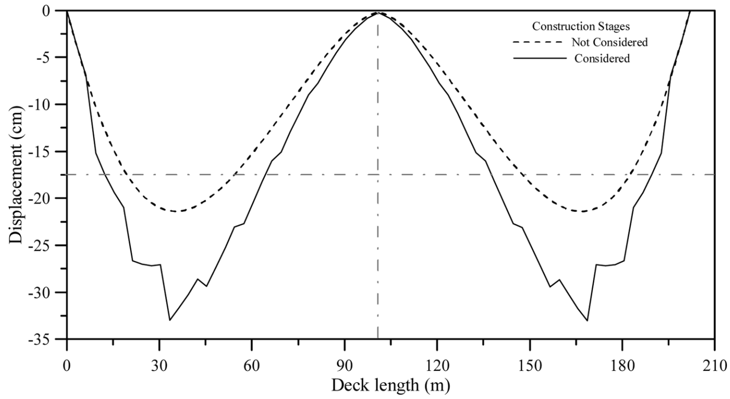

Fig. 11 Variation of vertical deck displacements

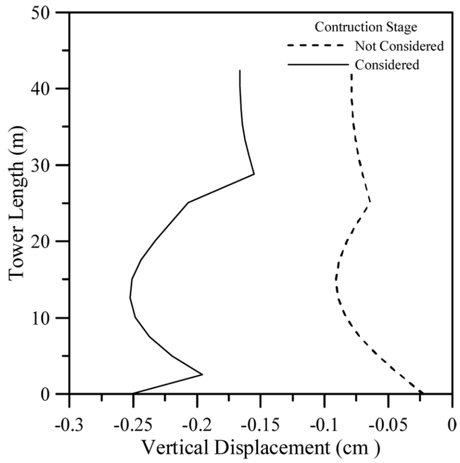

Fig. 12 Vertical displacements along the height of the bridge tower

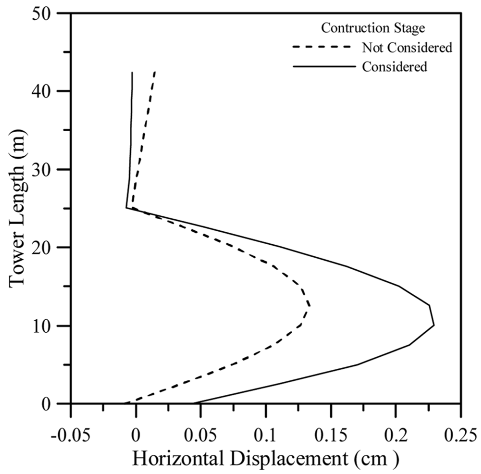

Fig. 13 Horizontal displacements along the height of the bridge tower

6.2 Tower response

The vertical and horizontal displacements shown in Figs. 12 and 13 on the pylon are very small. The maximum vertical displacement is 0.09 cm while the maximum horizontal displacement was 0.13 cm when the construction stage analysis is not considered. However, the construction stage analysis is considered, the maximum vertical displacement is 0.25 cm while the maximum horizontal displacement is 0.22 cm.

Axial force and shear forces that occurred along the height of the pylon are presented in Figs. 14 and 15. The result of both analyses show that the axial forces increase from top to bottom of the pylon. The maximum value of the axial force is obtained as 28512 kN and 25483 kN from the analysis with and without construction stage, respectively.

The bending moment along the height of the tower is given in Fig. 16. The bending moment reached maximum value at the junction point of the pylon.

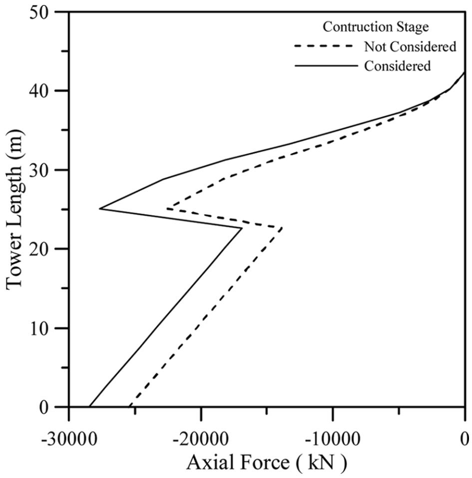

Fig. 14 Axial force along the height of the tower

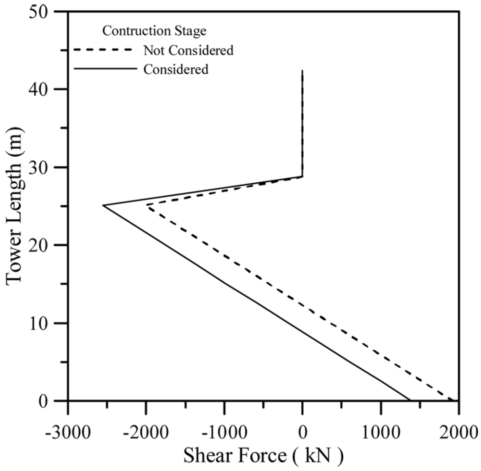

Fig. 15 Shear force along the height of the tower

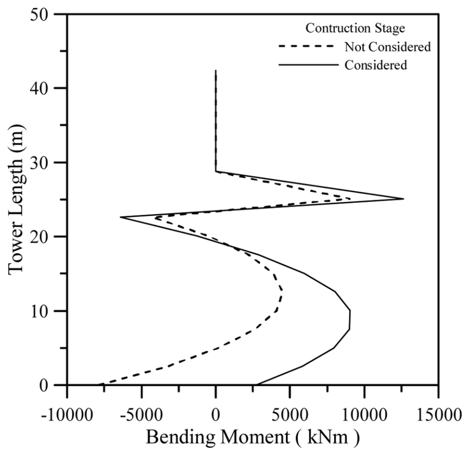

Fig. 16 Changing of bending moment along the height of the tower

7. Conclusions

The aim of this study is to determine the effect of the construction stages and time-dependent material properties on structural behavior of cable-stayed bridges. Manavgat Cable-Stayed Bridge was selected as an example. The assumed material and cross sectional properties are taken from a real bridge. The maximum and minimum response values of the bridges are compared with each other. From the point of view of the investigation carried out, the following conclusions are reached:

The bending moments, while the construction stage is considered, are significantly greater than those of inconsideration of the construction stage. The deck bending moments are increased around 15% in case of consideration of construction stages in the analysis. Similarly, the deck displacements are also increased around 54% in case of consideration of construction stages in the analysis. The results clearly show that consideration of construction stage in the analysis increases the deck bending moments and displacement.

When the results of the construction stage analysis are compared to inconsideration of the construction stage analysis, it is seen that there are differences between internal forces and displacements for the deck and the pylon. It means that the analysis in case of inconsideration of construction stages may not give the reliable solution.

Large differences observed between the results with and without considering construction stages. It can be stated that the analysis with construction stages may give more reliable solutions.

Construction stage analysis using time dependent material properties and geometric nonlinearity should be considered. It specifically is very important for bridges, because construction period continue along time and loads may change during the construction period and after.

References

Abbas, S. (1993), Nonlinear Geometric Material and Time Dependent Analysis of Segmentally Erected, Three Dimensional Cable Stayed Bridges, PhD Thesis, Department of Civil Engineering, University of California, Berkeley, USA.

Altunisik, A.C., Bayraktar, A., Sevim, B., Adanur, S. and Domanic, A. (2010), "Construction stage analysis of kömürhan highway bridge using time dependent material properties", *Struct. Eng. Mech.*, **36**(2), 207-244.

Ates, S. (2010), "Numerical modelling of continuous concrete box girder bridges considering construction stages", *Appl. Math. Model.*, **35**(8), 3809-3820.

CEB-FIP. Model Code. (1990), *Thomas Telford*, London.

Cho, T. and Kim, T.S. (2008), "Probabilistic risk assessment for the construction phases of a bridge construction based on finite element analysis", *Finite Elem. Analy. Des.*, **44**(6-7), 383-400.

Cluyet, N.C. and Shepherd, R. (1994), Analysis of Concrete Cable-Stayed Bridges for Creep, Shrinkage and Relaxtion Effects, Department of Civil Engineering, Calofornia State Polytechnic University, Pomona, USA, 110-134.

Karakaplan, A., Caner, A., Kurç, Ö., Domaniç, A. and Lüleç, A. (2007), "A new era in structural analysis: construction staging", *Technical Journal*, Digest 2009, **20**, December 2009.

Malm, R. and Sundquist H. (2010), "Time-dependent analyses of segmentally constructed balanced cantilever bridges", *Eng. Struct.*, **32**(4), 1038-1045.

Nazmy, A.S. (1987), Nonlinear Earthquake-Response Analysis of Cable-Stayed Bridges Subjected to Multiple-Support Excitations, PhD Thesis, Princeton University, New Jersey, USA.

SAP2000. (2008), Integrated Finite Element Analysis and Design of Structures, Computers and Structures Inc, Berkeley, California, USA.

Somja, H. and Goyet, V.V. (2008), "A new strategy for analysis of erection stages including an efficient method

for creep analysis", *Eng. Struct.*, **30**(10), 2871-2883.
Soyluk, K., Diri, T.G. and Sicacik, E.A. (2010), "Time Dependent Nonlinear Analysis of Segmentally Erected Cable-Stayed Bridges", *9th International Congress on Advances in Civil Engineering*, 27-30 September, Karadeniz Technical University, Trabzon, Turkey.

CC