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Construction stage analysis of fatih sultan mehmet suspension bridge

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Abstract. In this study, it is aim to perform the construction stage analysis of suspension bridges using time dependent material properties. Fatih Sultan Mehmet Suspension Bridge connecting the Europe and Asia in Istanbul is selected as an example. Finite element models of the bridge are modelled using SAP2000 program considering project drawing. Geometric nonlinearities are taken into consideration in the analysis using P-Delta large displacement criterion. The time dependent material strength variations and geometric variations are included in the analysis. Because of the fact that the bridge has steel structural system, only prestressing steel relaxation is considered as time dependent material properties. The structural behaviour of the bridge at different construction stages has been examined. Two different finite element analyses with and without construction stages are carried out and results are compared with each other. As analyses result, variation of the displacement and internal forces such as bending moment, axial forces and shear forces for bridge deck and towers are given with detail. It is seen that construction stage analysis has remarkable effect on the structural behaviour of the bridge.

Keywords: construction stage analysis Fatih Sultan Mehmet Suspension Bridge; finite element analysis; time dependent material properties

1. Introduction

Bridges are one of the most important engineering structures which are commonly used transportation. Over the last half century a large number of bridges has been built or are under construction all over the world. The use of suspension bridges which has been one of these structures has increased recently. They are built for both crossing the long spans (>550 m) and giving to rise to the usage of domains under the bridge. This kind of bridges has high cost and logistical importance. So, analysis of suspension bridges must be done on the best possible analytical model since structural elements such as deck, towers and cables show different structural behaviour.

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To obtain the structural behaviour of the suspension bridges under variable loads, finite element analysis is carried out. But, in the analytical solutions based on finite element models, it is assumed that the structure is built and loaded in a second. However, this type of analysis does not always give the reliable solutions. Because, construction period of this type of the structures continue along time and loads may be change during this period. So, construction stages and time dependent material properties should be considered in the analysis to obtain the more reliable results. Therefore, analysis of suspension bridges is carried out considering construction stages and time dependent material properties.

In the literature, there are some papers about the construction stage analysis of the bridges considering time dependent material properties. Ko *et al*. (1998) calculated the dynamic characteristics such as natural frequencies and mode shapes of suspension deck in construction stages. The Tsing Ma suspension bridge with a main span of 1377 m and an overall length of 2160 m is performed. Kwak and Seo (2002) determine the time dependent behaviour of precast prestressed concrete girder bridge. To analyze the long-term behaviour of bridges, the effects of creep, the shrinkage of concrete, and the cracking of concrete slabs in the moment regions is considered. Cheng *et al*. (2003) carried out the wind induced load capacity of a long span steel arch bridge during two construction stages. The Lupu Bridge which has 550 m central span length and 100 m side spans is selected as a case study. Wang *et al*. (2004) analyzed a cable stayed bridge during construction using the cantilever method. Two computational processes, one is a forward process analysis and the other is a backward process analysis are established. Pindado *et al*. (2005) investigated the influence of the section shape of box girder decks on the moments during construction stages experimentally. Karakaplan *et al*. (2007) performed the construction stage analysis of a cable supported pedestrian bridge considering time dependent material strength variations. Analysis results are compared with the conventional finite element analysis and the differences are determined. Cho and Kim (2008) carried out probabilistic risk assessment for the construction stages of the Hanbit suspension bridge. The bridge is under construction and will be one of the longest suspension bridges in Korea in 2010. The main span is designed to be 850 m with two side spans of 255 and 220 m each. Tensile forces for main cables and deflections for stiffening girders are controlled for each construction stages. Dost and Dedeoğlu (2008) studied about design and construction stages of Fatih Sultan Mehmet Bridge. Design criteria and basic technical data related to construction phase are explained. Somja and Goyet (2008) studied about nonlinear finite element analysis of segmentally constructed cable stayed bridge. Time dependent effects including load history, creep, shrinkage and aging of the concrete are considered in the analyses. Modification of the bridge topology has been carried out using an efficient procedure for creating/removing elements. Altunışık *et al*. (2009) performed the construction stage analysis of Kömürhan Highway Bridge. The bridge is a reinforced concrete box girder bridge and constructed with balanced cantilever method, located on the 51st km of Elazığ-Malatya highway. Adanur and Günaydın (2010) studied about construction stage analysis of Bosporus Suspension Bridge. Bosporus Suspension Bridge connecting the Europe and Asia in Istanbul is selected as an example. Two different finite element analyses with and without construction stages are carried out and results are compared with each other. Ates (2010) studied about analytical modelling of continuous concrete box girder bridges considering construction stages. Budan Bridge is selected as a numerical example. The Bridge constructed with balanced cantilever method and located on Artvin-Erzurum highway, Turkey, at 55+729-56+079.000 kilometers The structural behaviour of the bridge at different construction stages has been examined. As analyses result, variation of internal forces such as bending moment, shear forces and axial forces, and displacements for bridge deck and pier are given with detail. Soyluk *et al.* (2010) carried out time dependent nonlinear analysis of segmentally erected cable-stayed bridges. The analysis phase is divided into two phases: The construction phase and the service phase. In the analyses, while 33 stages which cover 970 days are considered for the construction phase, 3 stages which lasts up to 10 years are used for the service phase. The analytical models of the selected numerical example are solved by considering the self weight and the time-dependent nonlinear effects. The bridge responses are then compared with respect to the time-dependent effects. The results of the study show that time-dependent effects can have important effects on cable-stayed bridges. Beside these studies, more studies exist in the literature about the structural behaviour of suspension bridges (Li *et al.* 2010, Ubertini 2010, Wang *et al*. 2010, Nikitas *et al.* 2011, Zhang *et al.* 2011). As seen in literature, studies on construction stage analysis are meagre and need to be enlarged by inserting new studies. In the light of aforementioned researches, construction stage analysis of suspension bridges using time dependent material properties is performed in this paper. Time dependent material property is considered as relaxation for steel.

2. Description of Fatih Sultan Mehmet Suspension Bridge

Fatih Sultan Mehmet Suspension Bridge (Fig. 1) connecting the Europe and Asia Continents in Istanbul, Turkey. Construction of the bridge started in 1985 and completed in 1988. The bridge has a box girder deck with 39.4 m wide overall and 1090 m long. There are no side spans and the steel towers rise 110 m above ground level. The hangers are vertical and connect to the deck and cable with singly hinged bearing. The horizontal distance between the cables is 33.8 m and the roadway is 28 m wide, accommodating two four-lane highways. The roadway at the mid-span of the bridge is approximately 64 m above the sea level. Schematic representation of Fatih Sultan Mehmet Bridge including dimension given in Fig. 2.

The deck was constituted considering aerodynamic form to reduce of the wind affect along the bridge deck. The aerodynamic steel box girder deck (Fig. 3) of the bridge consist of 62 box girder deck pieces: one piece of 18.92 - m unit, fifty five pieces of 17.92 - m units, two pieces of 16.92 - m units, two pieces of 13.44 - m units and two pieces of 8.88 m units. Total deck length is 1083 m.

Fig. 1 Fatih Sultan Mehmet Suspension Bridge

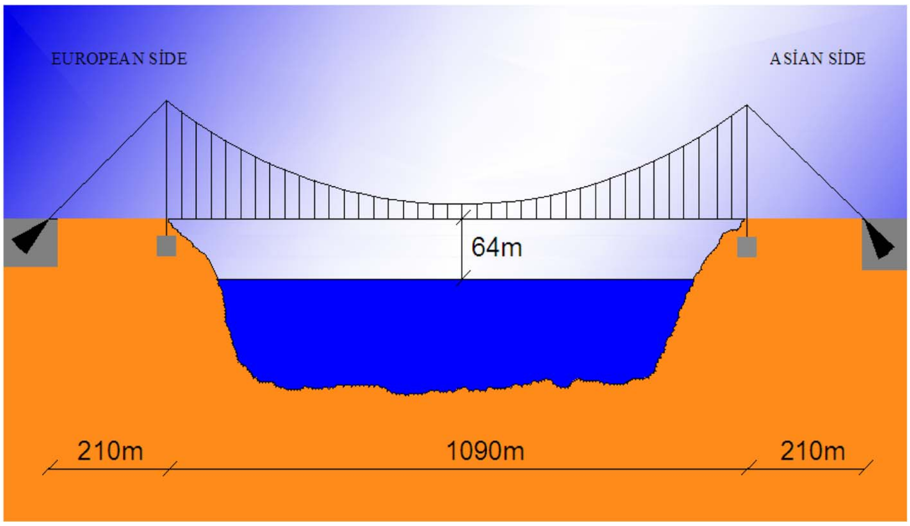


Fig. 2 Schematic representation including dimension

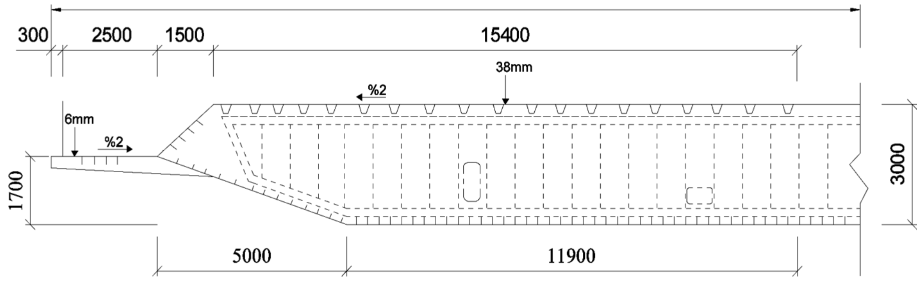


Fig. 3 Dimensions of aerodynamic steel box girder deck (dimensions as mm)

The deck has a 33.80 m × 3 m box section and two cantilever sidewalks of 2.80 m at each side. Total width of the deck is 39.40 m. The top of each box section constitutes an orthotropic plate on which 35 mm thickness mastic asphalt surfacing is laid.

The bridge has steel towers of 110 m above ground level. These towers consist of a variable section. The width of the section decreases from 5000 mm at the base to 3000 mm at the top of the tower. The tower length could be divided into eight similar section each 13100 mm long - a length suitable for fabrication. Vertical tower legs are connected by two horizontal portal beams. Dimension of towers are given in Fig. 4.

The main cables made up highly tensile and galvanized steel wire (Fig. 5). Each main cable has 32 strands, which extend from anchorage to anchorage with an addition of 4 thinner strands in the backstays between anchorages and main tower saddles. The main strands contain 504 wires each and the thinner strands 288 and 264 wires. The diameter of wires is 5.38 mm. Finally, the cross-section area of the cables on the main span is 0.7333 mm^2 and of the backstays is 0.7835 mm^2.

Hangers of the bridge have been formed at the vertical shape. The cable clamps have been erected along the main cables with 17.92 m intervals and tightened to the cable surface by mean of rods. Each hanger has 76 mm diameter and tensile strength of 370 tons.

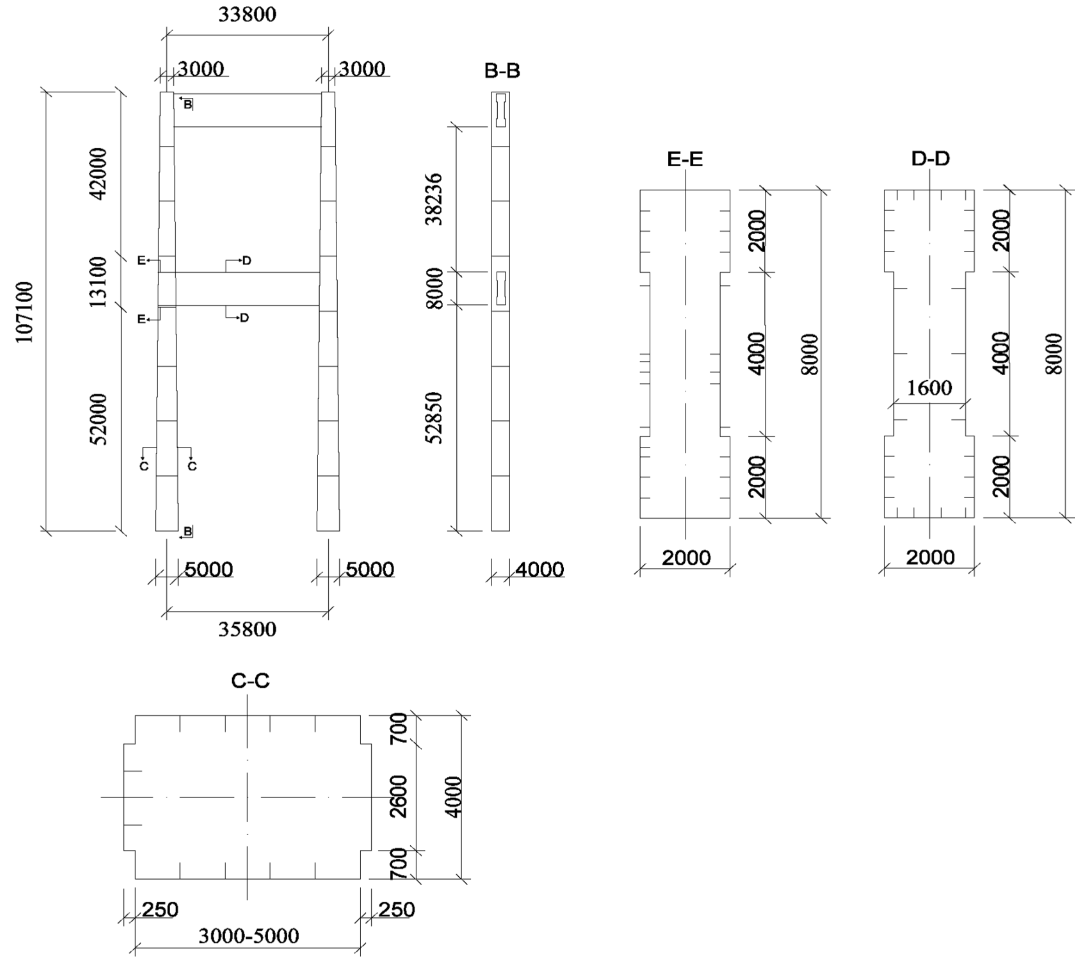
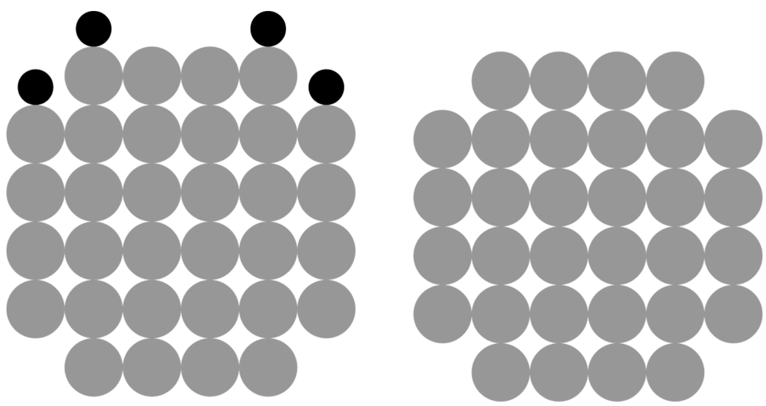


Fig. 4 Dimensions of towers (dimensions as mm)



(a) (b)

Fig. 5 Cross-section of (a) backstay and (b) between towers of the cables

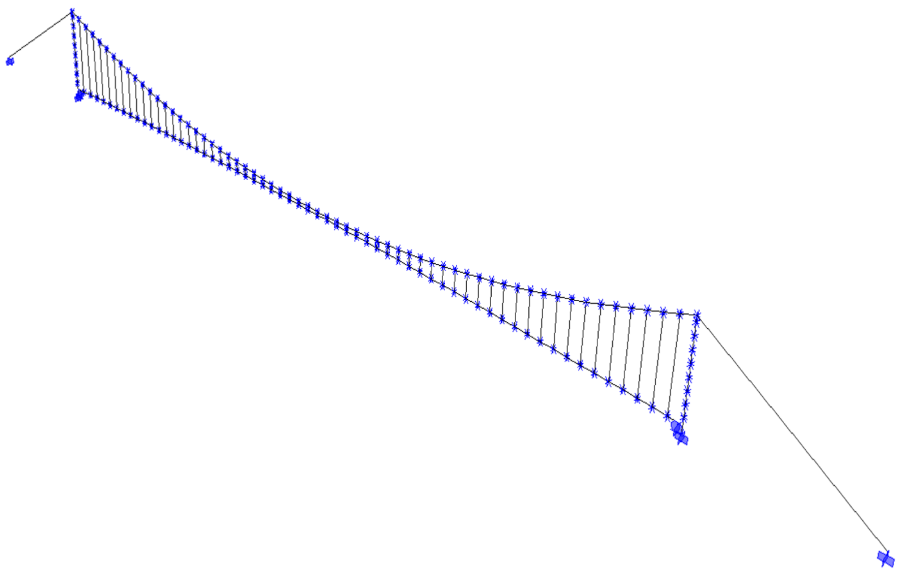


Fig. 6 Two-dimensional finite element model of Fatih Sultan Mehmet Suspension Bridge

3. Finite element analysis

Finite element models are commonly considered in the design and project phase of the important engineering structures such as bridges using some special software. In this study, SAP2000 finite element program (SAP2000 2008) which is used for linear and non-linear, static and dynamic analyses of 3D models of structures is used in the analysis. To investigate the construction stage response of the Fatih Sultan Mehmet Suspension Bridge, two-dimensional finite element model are used for calculations. The finite element models of Fatih Sultan Mehmet Suspension Bridge are shown in Fig. 6. As the deck, towers and cables are represented by beam elements, the hangers are represented by truss elements in the model. Finite element model of the bridge with vertical hangers has 149 nodal points, 142 beam elements and 60 truss elements and the model is represented by 418 degrees of freedom.

3.1 Modelling of the construction stages

In the construction stage analyses of Fatih Sultan Mehmet Bridge, a total of 33 construction stages are considered. Total duration from the beginning of construction to ending of construction is considered as 604 days. Maximum total step and maximum iteration for each step are selected as 200 and 100, respectively. Some construction stages using SAP2000 finite element analysis program is shown in Fig. 7.

In the construction stage analysis, some special points given in below should be considered;

- All construction stages and their details should be determined from design to opening the traffic of the bridge,
- Working plan including construction durations of main structural elements (tower, deck and cable) of the bridge should be prepared,

- Added and removed loads for each construction stages should be determined,
- To obtain the reliable solution, each stage results should be added to end of the each stage and next stage analysis is done,
- Non-linear solution parameters should be selected depending on the literature.

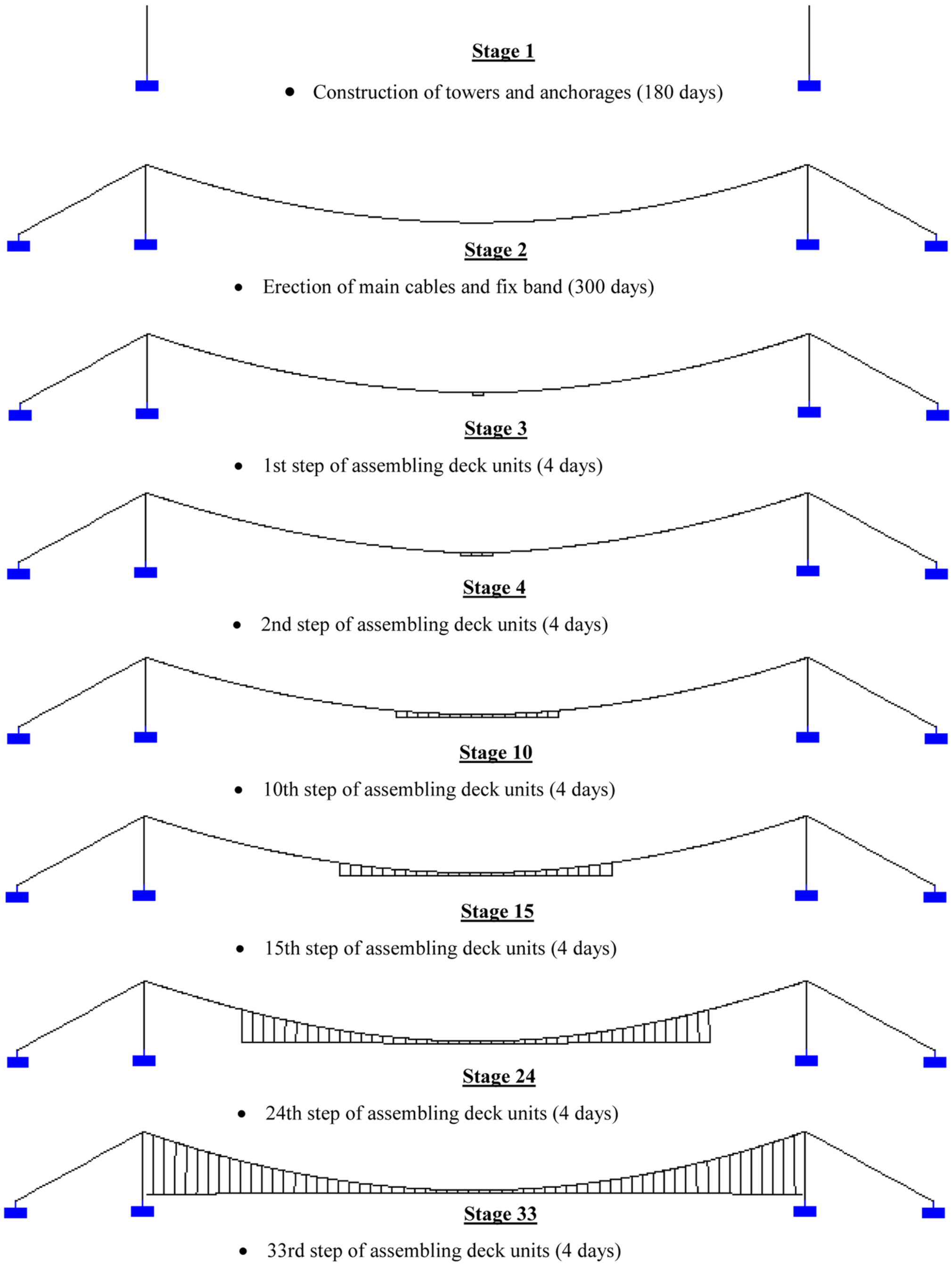


Fig. 7 Some construction stages of Fatih Sultan Mehmet Suspension Bridge

3.2 Time dependent material properties

In the construction stage analysis of bridges, time dependent material properties such as elasticity modulus, creep and shrinkage for concrete and relaxation for the prestressed steel should be considered, because they are variable due to the climate during construction (Altunışık 2010, Altunışık *et al*. 2010). For example, strength of the concrete increase continuously at 7th, 28th and 1000th days of concreting. If these properties are not considered in the analysis, analysis of the bridges may not give the reliable results. In this study, Fatih Sultan Mehmet Suspension Bridge has steel structural system, so only prestressing steel relaxation is considered as time dependent material properties.

The iterative calculations at each construction stage considering added stiffness from the initial equilibrium state. The matrix form of finite element method is given the following equation

$$\{F\} = [K]\{U\} \tag{1}$$

where $[K]$ is the stiffness matrix including elastic stiffness matrix and geometric stiffness matrix. The finite element analysis is performed at each construction stages of the bridge by using SAP2000.

3.2.1 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)

$$f_{cm}(t) = \beta_{cc}(t) f_{cm} \tag{2}$$

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

$$\beta_{cc}(t) = \exp\left\{ s\left[1 - \left(\frac{28}{t/t_1}\right)^{1/2}\right]\right\} \tag{3}$$

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

3.2.2 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

$$E_{ci}(t) = E_{ci}\sqrt{\beta_{cc}(t)} \tag{4}$$

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.

3.2.3 Shrinkage of concrete

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

$$\varepsilon_{cs}(t,t_s) = \varepsilon_{cso}\beta_s(t-t_s) \tag{5}$$

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

$$\varepsilon_{cso} = \varepsilon_s(f_{cm})\beta_{RH} \tag{6.a}$$

$$\varepsilon_s(f_{cm}) = \left[160 + 10\beta_{sc}\left(9 - \frac{f_{cm}}{f_{cmo}}\right)\right] \tag{6.b}$$

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.

$$\begin{aligned} \beta_{RH} &= -1.55\beta_{sRH} \quad 40\% \le RH < 99\% \\ R_{RH} &= 0.25 \qquad RH \ge 99\% \end{aligned} \tag{7}$$

where

$$\beta_{sRH} = 1 - \left(\frac{RH}{RH_o}\right)^3 \tag{8}$$

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

$$\beta_s(t-t_s) = \sqrt{\frac{(t-t_s)/t_1}{350(h/h_o) + (t-t_s)/t_1}} \tag{9}$$

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.

3.2.4 Creep

The effect is calculated using CEB-FIP Model Code (1990) creep model. For a constant stress applied at time t_o, this leads to

$$\varepsilon_{cc}(t,t_o) = \frac{\sigma_c(t_o)}{E_{ci}}\phi(t,t_o) \tag{10}$$

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 from

$$\phi(t,t_o) = \beta_c(t-t_o)\phi_o \tag{11}$$

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

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

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

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

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

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

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

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

where t_1 = 1 day; RH_o = 100 and h_o = 100 mm.

3.2.5 Relaxation of steel

According to CEB-FIB Model Code (1990), 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 which k to be 0.12 for relaxation class1, and 0.19 relaxation class2; 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.

Selected analysis parameters to consider time dependent material properties are given in Table 1.

Variation of time dependent material properties used for prestressed steel is given in Fig. 8. These parameters are selected from CEB-FIP design code (CEB-FIP 1990) in SAP2000. According to the parameters given in Table 1, these graphics may be changed automatically. Total duration from the beginning of construction to ending of construction is considered as 604 days.

Table 1 Selection of analysis parameters to consider time dependent material properties in SAP2000

Parameters		Main Structural Elements		
		Deck	Tower	Prestress Steel
Material Properties		Tendon Isotropic	Tendon Isotropic	Tendon Isotropic
Nonlinear Material Data	Hysteresis type	Kinematic	Kinematic	Kinematic
	Stress-Strain diagram	User defined	User defined	User defined
Time Dependent Properties	Elasticity modulus	√	√	√
	Creep	-	-	-
	Shrinkage	-	-	-
	Creep analysis type	-	-	-
	Cement type coefficient	-	-	-
	Relative humidity %	-	-	-
	Notional size	-	-	-
	Shrinkage coefficient	-	-	-
	Shrinkage start age	-	-	-
	Steel relaxation	√	√	√
	Relaxation analysis type	Full	Full	Full integration
	CEB-FIP class	1	1	1

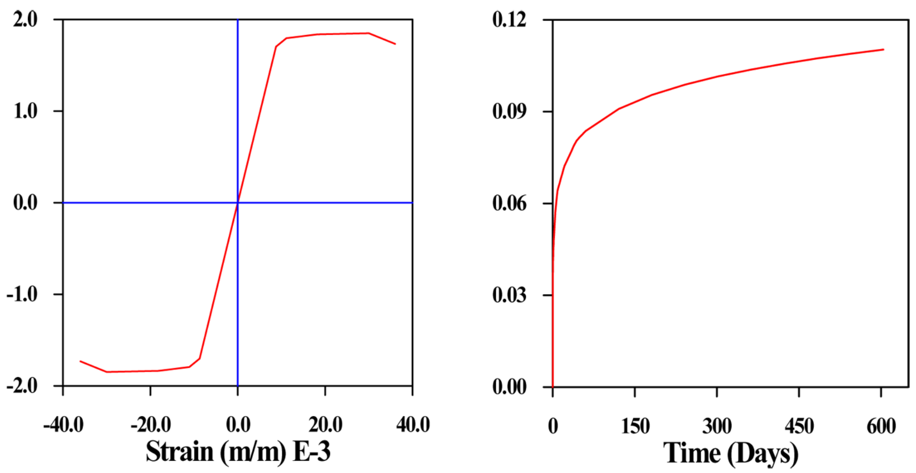


Fig. 8 Variation of time dependent material properties for prestressed steel

3.3 Construction stage analysis

For the construction stage analysis of suspension bridges considering time-dependent material properties, Fatih Sultan Mehmet Suspension Bridge connecting the Europe and Asia in Istanbul is selected as an example. This bridge has a main span of 1090 m and there are no side spans. The bridge has steel towers of 110 m high above ground level, a steel box-deck and vertical hangers.

The roadway at the mid-span of the bridge is approximately 64 m above the sea level. Analysis is performed using SAP2000 program. Nonlinear staged construction and P-Delta plus large displacements options are selected as analysis type and geometric nonlinearity parameters, respectively.

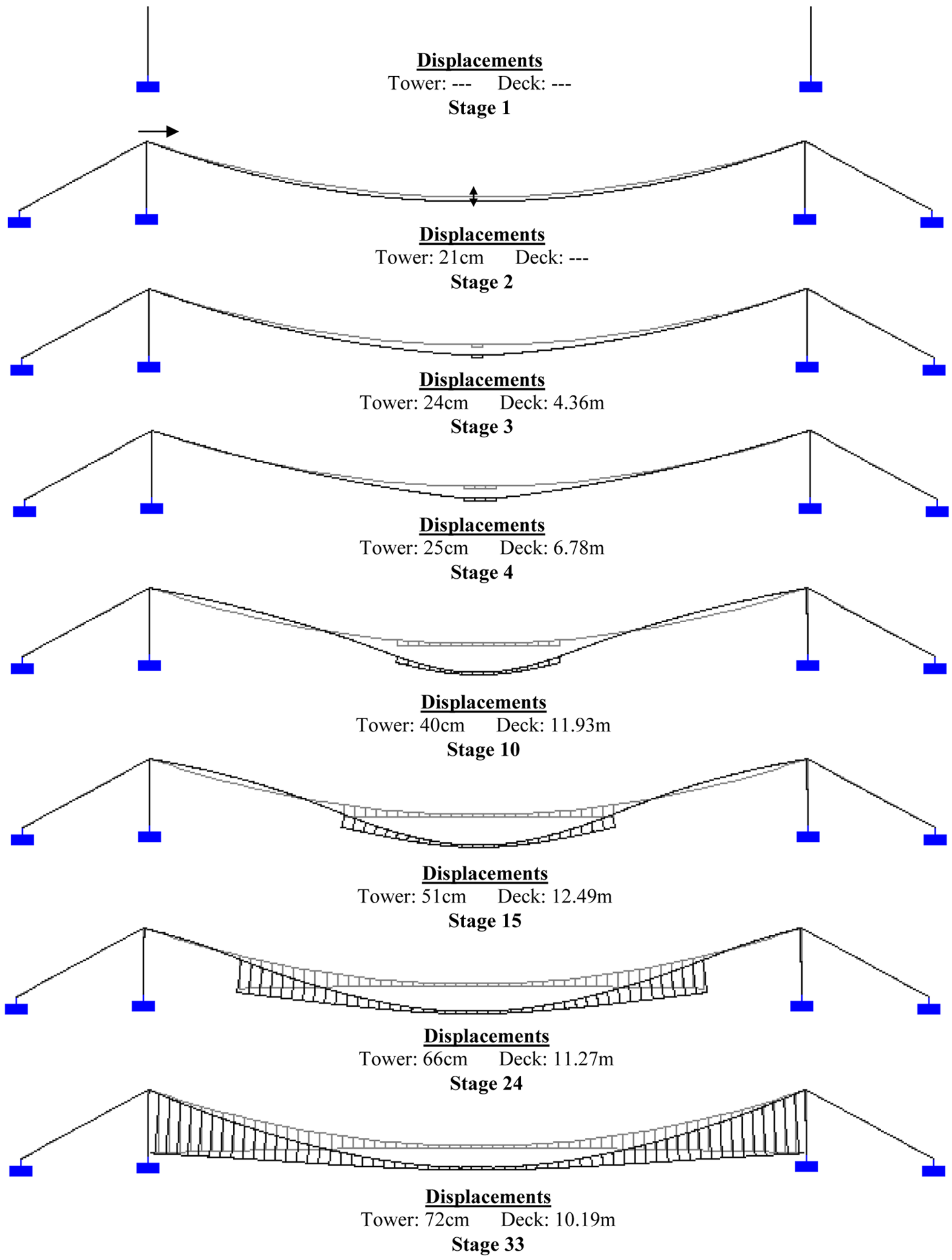


Fig. 9 Deformation of Fatih Sultan Mehmet Suspension Bridge during some construction stages

3.3.1 Load cases of analyses

In the analyses of the bridge, the following load cases are considered;

- Dead Load: Weight of all elements. They are calculated from the finite element software directly.
- Additional Mass: Weight of the asphalt, cobble, pipeline and its supports, scarecrow. 40 kN/m distributed load is added to each segment.

3.3.2 Deformation shapes

The deformations of the bridge at some construction stages are plotted and the maximum vertical displacements of the bridge deck and maximum horizontal displacements of the bridge tower are also given in Fig. 9. It is seen that displacements increase along the middle of the bridge deck and reach a maximum of 12.59 m at the 15th stage for the analysis including the construction stage. When the construction of the bridge is completed at the 33th stage, maximum displacement is obtained as 10.19 m at the middle point of the bridge deck. Variation of the displacement increases along the height of the bridge towers and reach a maximum of 72 cm at the 33th stage.

3.3.3 Deck response

Distributions of vertical displacements and bending moments along the bridge deck are given in

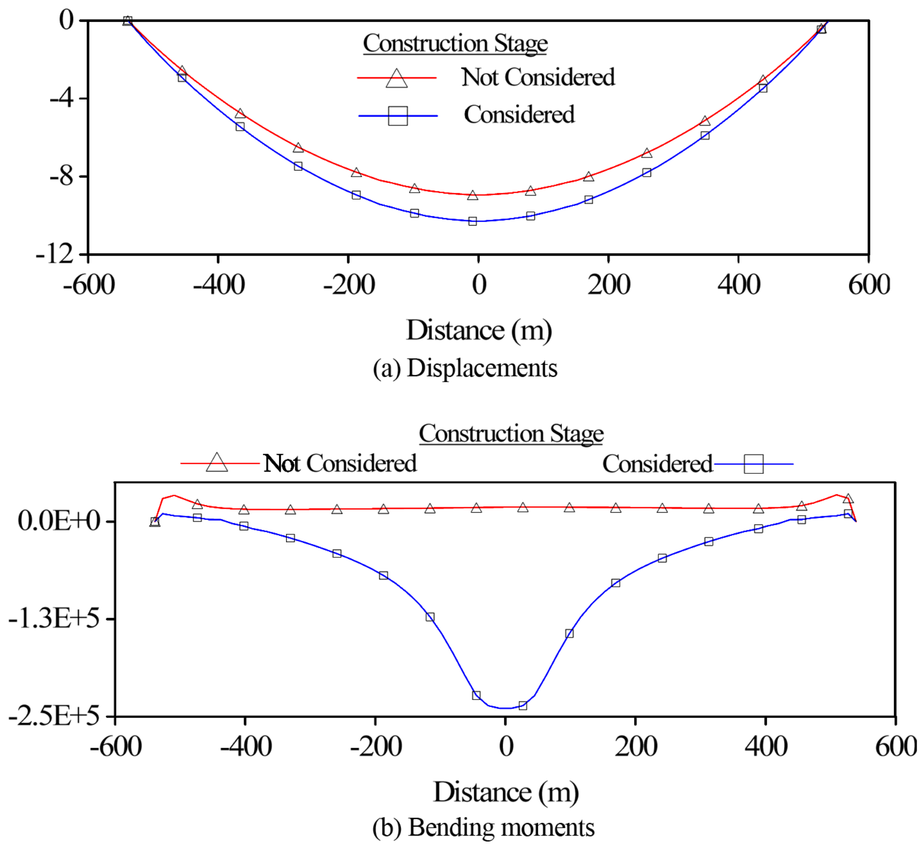


Fig. 10 Changing of maximum displacements and (a) bending moments (b) along the bridge deck

Fig. 10. It is seen that displacements have an increasing trend towards to the middle of the bridge deck and reach a maximum of 10.19 m at the middle for the analysis including the construction stages. The values of bending moments are nearly equal along the bridge deck as 1.8E4 for the analysis not including the construction stage. On the other hand, the values of bending moments increase along to the middle of the bridge deck and reach a maximum of –2.4E5 for the analysis including the construction stage. Both displacements and bending moments are obtained symmetrically according to the middle point of the bridge deck. It is seen from Fig. 10 that the displacements and bending moments obtained from the analyses including construction stages are significantly bigger than those of not including the construction stages.

3.3.4 Tower response

Variation of maximum displacements along the height of the tower is shown in Fig. 11. It can easily be seen that the horizontal displacements increase with the height of bridge tower and reach a maximum of 72 cm at the top for the analysis including the construction stage. But, the value of the horizontal displacement at the bridge tower top is 67 cm for the analysis not including the construction stage.

Fig. 12 points out the internal forces such as shear and axial forces of the bridge tower corresponding to the two analyses. The values of the axial forces are nearly equal along the height of the bridge tower as –3.3E5 kN for both analyses. The values of the shear forces are nearly equal along the height of the bridge tower as –0.4E4 kN for the analysis not including the construction stage, but the values of the shear forces increase along the middle of the bridge tower and degrease from the middle point to top of the bridge tower for the analysis including the construction stage. Shear forces increase non-linearly from the base (–0.4E4 kN) to middle point (–0.55E4 kN) and decrease non-linearly from the middle point (–0.55E4 kN) to top point (–0.4E4 kN). It can be easily seen from Fig. 12 that construction stage analysis is more effective than the other for both internal forces.

A variation of bending moment with height of the bridge tower is shown in Fig. 13. It can be easily seen from Fig. 13 that bending moments changes linearly from the base (-4E5 kNm) to top

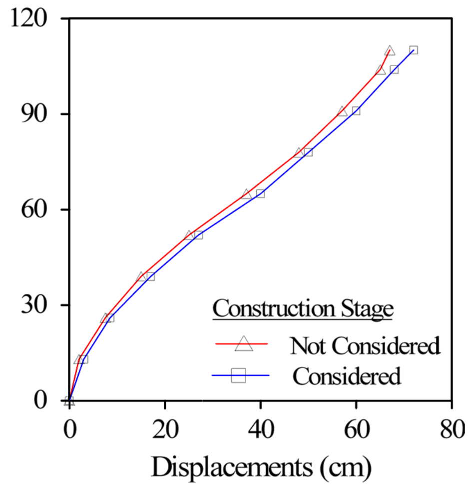


Fig. 11 Changing of displacements along to the height of the bridge tower

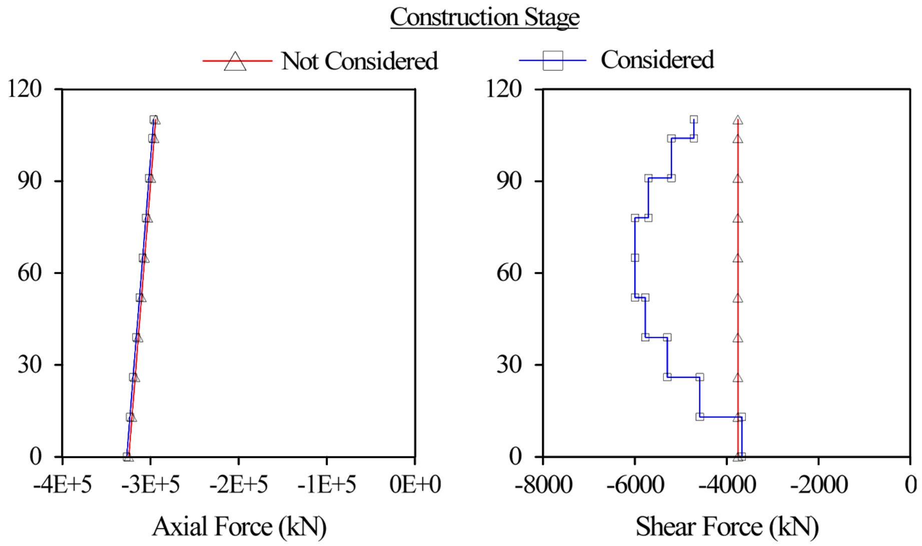


Fig. 12 Changing of internal forces along the height of the bridge tower

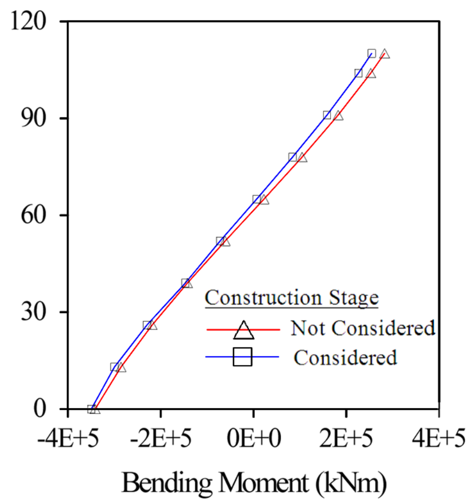


Fig. 13 Changing of bending moment along the height of the bridge tower

point (4E5 kNm) for both analyses. The bending moments are obtained nearly zero at the middle of the bridge tower.

4. Conclusions

In this study, it is aimed to perform the construction stage analysis of suspension bridges using time dependent material properties. Fatih Sultan Mehmet Suspension Bridge connecting the Europe and Asia in Istanbul is selected as an example. The time dependent material strength variations and geometric variations are included in the analysis. In the analyses, total duration from the beginning of construction to ending of construction is considered as 604 days. Comparing the results of the study, the following observations can be made:

- The vertical displacements increase towards to the middle of the bridge deck and reach a maximum of 10.19 m at the middle for the analysis including the construction stages. On the other hand, maximum displacement is 8.95 m at the middle for the analysis not including construction stage. The difference is reached to 1.24 m at the middle of the bridge. The horizontal displacements increase with the height of bridge tower and reach a maximum of 72 cm at the top for the analysis including the construction stage. But, maximum displacement is 67 cm at the top for the analysis not including construction stage.
- The values of bending moments are nearly equal along the bridge deck as 1.8E4 for the analysis not including the construction stage. On the other hand, the values of bending moments increase along to the middle of the bridge deck and reach a maximum of −2.4E5 for the analysis including the construction stage. The values of bending moments obtained from the analyses including construction stages are significantly bigger than those of not including the construction stages.
- The values of the axial forces are nearly equal along the height of the bridge tower as −3.3E5 kN for both analyses. The values of the shear forces are nearly equal along the height of the bridge tower as−0.4E4 kN for the analysis not including the construction stage, but the values of the shear forces increase along the middle of the bridge tower and degrease from the middle point to top of the bridge tower for the analysis including the construction stage. Shear forces increase non-linearly from the base (−0.4E4 kN) to middle point (−0.55E4 kN) and decrease non-linearly from the middle point (−0.55E4 kN) to top point (−0.4E4 kN).
- There are some differences between the results with and without the construction stages. It can be stated that the analysis without construction stages cannot give the reliable solutions.
- To obtain real behaviour of engineering structures, construction stage analysis using time dependent material strength variations and geometric variations should be done. Especially it is very important for suspension bridges, because construction period continue along time and loads may be change during this period.

References

Adanur, S. and Günaydın, M. (2010), "Construction stage analysis of bosporus suspension bridge", *Proceeding of 9th International Congress on Advances in Civil Engineering*, Trabzon, September.

Altunışık, A.C., Bayraktar, A., Sevim, B., Domaniç, A. and Adanur, S. (2009), "Construction stage analysis of bridges using time dependent material properties", *Proceeding of International Symposium on Earthquake, Sakarya*, September.

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

Altunışık, A.C. (2010), "Karayolu köprülerinin yapýsal davranýþlarýnýn analitik ve deneysel yöntemlerle belirlenmesi", PhD Thesis, Karadeniz Technical University, Trabzon, Turkey. (In Turkish)

Ateş, Ş. (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*, ISBN: 0727716964.

Cheng, J., Jiang, J.J., Xiao, R.C. and Xia, M. (2003), "Wind-induced load capacity analysis and parametric study of a long-span steel arch bridge under construction", *Comput. Struct.,* **81**, 2513-2524.

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. D.,* **44**, 383-400.

Dost, Y. and Dedeoğlu, E. (2008), "Design and construction stages of Fatih Sultan Mehmet Bridge", *Proceeding of 1st Collaborative Symposium of Turk-Japan Civil Engineers,* Istanbul, June.

Karakaplan, A., Caner, A., Kurç, Ö., Domaniç, A. and Lüleç, A. (2007), "New strategy in the structural analysis: construction stage", *Proceeding of 1st Symposium of Bridges and Viaducts,* Antalya, November.

Ko, J.M., Xue, S.D. and Xu, Y.L. (1998), "Modal analysis of suspension bridge deck units in erection stage", *Eng. Struct.,* **20**, 1102-1112.

Kwak, H.G. and Seo, Y.J. (2002), "Numerical analysis of time-dependent behaviour of pre- cast pre-stressed concrete girder bridges", *Constr. Buil. Mater.,* **16**, 49-63.

Li, Z., Li, A. and Zhang, J. (2010), "Effect of boundary conditions on modal parameters of the Run Yang Suspension Bridge", *Smart Struct. Syst.,* **6**(8), 905-920.

Nikitas, N., Macdonald, J.H.G. and Jakobsen, J.B. (2011), "Identification of flutter derivatives from full-scale ambient vibration measurements of the Clifton Suspension Bridge", *Wind Struct.,* **14**(3), 221-238.

Pindado, S., Meseguer, J. and Franchini, S. (2005), "The influence of the section shape of box-girder decks on the steady aerodynamic yawing moment of double cantilever bridges under construction", *J. Wind Eng. Indus. Aerod.,* **93**, 547-555.

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**, 2871-2883.

Soyluk, K., Diri, T.G. and Sıcacık, E.A. (2010), "Time dependent nonlinear analysis o segmentally erected cable-stayed bridges", *Proceeding of 9th International Congress on Advances in Civil Engineering*, Trabzon, September.

Ubertini, F. (2010), "Prevention of Suspension Bridge flutter using multiple tuned mass dampers", *Wind Struct.,* **13**(3), 235-256.

Wang, P.H., Tang, T.Y. and Zheng, H.N. (2004), "Analysis of cable-stayed bridges during construction by cantilever methods", *Comput. Struct.,* **82**, 329-346.

Wang, S.Q., Xia, H., Guo, W.W. and Zhang, N. (2010), "Nonlinear dynamic response analysis of a long-span Suspension Bridge under running train and turbulent wind", *Interact. Multis. Mech.,* **3**(4), 309-320.

Zhang, W.M., Ge, Y.J. and Levitan, M.L. (2011), "Aerodynamic flutter analysis of a new Suspension Bridge with double main spans", *Wind Struct.,* **14**(3), 187-208.