Passive Control of Bridges

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ABSTRACT

In the design of bridges with large spans, the significant values of the moments at the main deck-structure require very heavy members, either in the case of a beam or a truss deck-structure. In order to minimize the influence of bending moments, several applications of passive control of displacements using cable nets are herein proposed. The base for all the proposed systems is the cables supported beam nets with additional prestressing control on support cables to optimize the structural behavior of the system. The passive control design problem leads to an optimal control problem for structures governed by variational inequalities. In this presentation several bridge systems are proposed and studied as applications of this method.

Keywords: Bridges, Passive Control.

INTRODUCTION

The proposed displacement control systems for large span structures need parallel, external lines of flexible cables. The optimal shape of a prestressing support cable leads to a form-finding problem of the structure. The analysis is based on the two-nodes-curved-cable-element, where large displacements and large rotations appear. Therefore, a multi-span cable is a non-linear and flexible structure since every loading case defines a new equilibrium configuration for the system. Using a prestressing strategy, the configuration of the cable structures due to the permanent loading is fixed in its final form. The first proposed system (of type **MPSB** named after Michalopoulos, Panagiotopoulos, Stavroulakis, Baniotopoulos) is formed by the cable (a^u) which is actually a group of similar, parallel and loaded cables that support the simply supported concrete plate (deck), which in turn follows the curved shape of the loaded cables (Nikolaidis, 2003). The second proposed system (of type **MBN** named after Michalopoulos, Baniotopoulos, Nikolaidis) needs two parallel, external lines of flexible cables. The upper cable (a^u) is actually a group of similar, parallel and loaded cables that support the simply supported concrete plate (deck), which in turn follows the curved shape of the loaded cables. The lower cable (a^l) is also a group of similar, parallel and loaded external cables following also a curved shape form. The two (upper and lower) cable lines are connected with stanchions of appropriate length between the homologous nodes (Michalopoulos et al., 2006). Finally the third proposed system (of type **MPS** named after **M**ichalopoulos, **P**anagiotopoulos **S**tavroulakis) is formed by a stiffened cross-section of box-girder type (such as concrete) where the support element is an external prestressed flexible cable having a negative curvature (concave facing upwards). For many years, the simply supported beam system has been used for the case, of bridges with large spans, where the

Received on 1/2/2007 and Accepted for Publication on 1/4/2007.

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support element is either an arch having positive curvature or a chain with negative curvature. In this case the deck-structure is statically indeterminate having one degree of freedom ambiguous due to the support element. The use of prestressed concrete and in particular recently the use of box-girder cross-sections, where prestressed cables are incorporated in suitable positions in the interior of the cross-section, proved to be a very promising design solution. Evolution of this deck type constitutes the proposed system (Michalopoulos et al., 2005).

FORMULATION OF THE OPTIMAL PRESTRESS PROBLEM

The analysis of the proposed model has been performed by means of the finite element method within an optimal prestressing theoretical framework (Panagiotopoulos, 1982; Stavroulakis, 1995). Assuming that displacements and deformation are infinitesimal, the formulated structural analysis problem leads to the following potential energy minimization problem:

$$\min \left\{ \prod(\{u\} = \frac{1}{2}\cdot\{u^T\}\cdot[K]\cdot\{u\} - \{P^T\}\cdot\{u\} - \{z^T\}\cdot[C^T]\cdot\{u\} \middle| \{u\}\in U_{ad}(z) \right\} \quad (1)$$

Where the space of admissible displacements $U_{ad}(z)$ is defined by:

$$U_{ad}(z) = \left\{ \{u\} \in \Re^n \middle| [A]\cdot\{u\} \le b, [\Gamma]\cdot\{u\} = \{u_0\} \right\} \quad (2)$$

Solving problem (1), (2) can be explicitly written as follows:

$$[K]\cdot\{u\} - \{p\} - [C]\cdot\{z\} + [\Gamma^T]\cdot\{\mu\} + [A^T]\cdot\{\lambda\} = \{0\} \quad (3)$$

$$-[\Gamma]\cdot\{u\} + \{u_0\} = \{0\} \quad (4)$$

$$-[A]\cdot\{u\} + \{b\} + N_{R_+^S(\lambda)} \ni \{0\}. \quad (5)$$

Here *N*c denotes the Normal Cone of the set *C*. Thus, vector $\{\mu\}$ corresponds to the discrete nodal reactions at the boundary condition (5), vector $\{\lambda\}$ corresponds to the discrete interface tractions along the interface and relation (5) is more often written in structural analysis literature in the following complementary system of inequalities:

No penetration relation $\{y\} = -[A]\cdot\{u\} + \{b\} \ge 0$ (6)

No tensile tractions $\{\lambda\} \ge \{0\}$ (7)

Complementarity $\langle\{\lambda\},\{y\}\rangle = 0$ (8)

Relations (6), (7) and (8) are the well-known optimality conditions for the quadratic minimization problem. Moreover, this problem is also known as the generalized linear complementarity problem. All the aforementioned problems that describe the structural response of the subjected to the action of the variable control vector $\{z\}$ can be considered in the sequel as parametric state models for the analysis of the optimal control problem.

PASSIVE CONTROL OF THE DISPLACEMENTS

The herein proposed models concern the introduction of a net of cables (β) that acts as an effective passive control system of displacement against the deformations caused by moving loads (Michalopoulos, 1982). Thus, apart from the cables type (a) being the main cables of suspension, a second set of cables called cables (β) being inactive in the situation of equilibrium under permanent loads is herein proposed. Before the selection of the actual cross-section of cable (β) the notion of an "ideal" cross-section of the cable (β^*) is introduced. Hence, due to the form finding, the design cable (β^*) undertakes 90% of moving stresses $S^{mov.}$. This cable has a cross-section that is analogously multiplied by that of the main cables type (a). These cables are anchored in the same positions of suspension as the cables of type (a) and are activated as soon as additional moving loads start acting on the deck. By analyzing the structure with a finite element program taking into account the additional cross sections, if the obtained stresses and displacements are within the allowable limits, the cable (β^*) is replaced by a statically equivalent (β). The real cable (β) consists of a sequence of prestressed segments with significant length, being interconnected by non-prestressed segments of small length. The cables are placed along the central axis of the stiffness plate embedded to a priori constructed channels within the mass of the stiffness plate. The cable (β) has the same geometry with respect to (β^*). It is also hinged and anchored at the same positions of the structure. The real cables (β), as it was reported above, are the statically equivalent cables of (β^*). Note that the whole cable (β) exhibits small deformability subjected to the same load with the ideal cable (β^*) and undertakes the same part of stress $S^{mov.}$. The cables (β) along with the additional diagonal cables (after the initial condition) are used as passive control system for the suppression of the displacements of the large span structure. This method leads to very satisfactory results for both displacements and stresses. It is included in all the following displacement control systems.

Table 1: MPSB model, load combination (COMB1).

A/A	Load case	Remarks
1	$P_i^{perm.} = -2160kN$	Permanent load at all the nodes of the plate
2	$A_i = 2160kN$	Reaction load due to the prestressing at the nodes of the cable set (a)

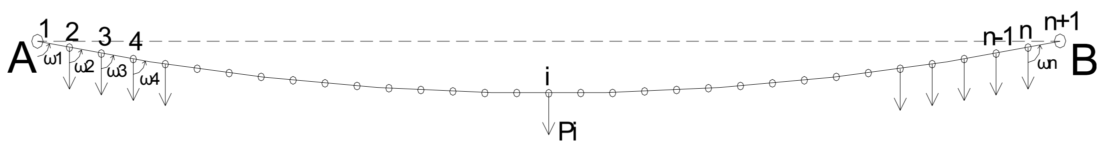

Fig 1: On the form of the cable.

DISPLACEMENT CONTROL SYSTEM OF TYPE MPSB

First, the displacement control system named **MPSB**, for large span structures, use a curved line of cable nets herein proposed. The novel passive control system is based on the usage of a prestressing cable mechanism, where permanent loads and a part of the moving loads are relieved by the line of prestressing cables net named (a). The moving loads together with excessive displacements are taken by an additional cable net (β) that depends on the form and the use of the structure.

The flexible cable connected at the two extreme nodes of the structure exhibits a (stress-unilateral) structural behavior and therefore, is loaded by the vertical loads that correspond to cable tension. It is a structure where

Table 2: MPSB model, load combination (COMB2).

A/A	Load case	Remarks
1	$q_i = -375kN$	Distributed live load – common at all the nodes of the upper plate

Table 3: MPSB model, load combinations (COMB3-COMB4).

A/A	Load case	Remarks
1	$q_i = -375kN$	Distributed live load – common at all the nodes of the upper plate
2	$Q_i = -378kN$	Concentrated live loads (due to heavy vehicle) at two consecutive nodes of the deck, a) COMB3 at the nodes 8 & 9, b) COMB4 at the nodes 16 &17

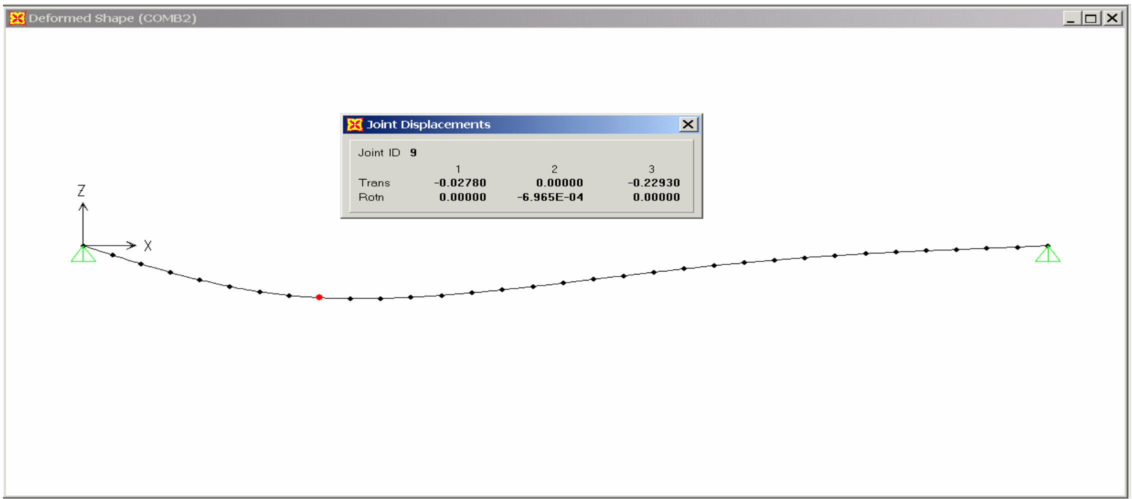

Fig. 2: MPSB model – Load case COMB4- joint displacements.

unknown is the stress field (S_i) caused by any vertical loading P_i, and also unkown is the form (ω_i) influenced by the respective loading. Therefore, the first step is to formulate the general system of equations of a cable hinged from the two extreme nodes and loaded by a general loading corresponding to a set of nodal vertical forces. Using the equations of the cable, the system [1] of $2n$ non-linear equations is first formulated. The solution of $2n$ non-linear equations of system with $n+1$ cable nodes gives the unknown stresses S_i and unknown angles ω_i (hence also the geometry of cable) before deformation. In this presentation a bridge model of open length L=200.0m is proposed and studied as a numerical application of the method. The starting configurations are the calculated lengths by the system [1] for the cable (a) hinged by the common ends of suspension A and B.

Taking into consideration the solution of the cable, we determine the corresponding cross-section $F^{(a)} = 2200cm^2$. Concerning the cable set (β), an idealized cable (β^*) with cross-section $F^{(\beta^*)} = 9xF^{(a)}$ is first taken and the sum of the cross sections for the upper cable becomes $F^{(a)} + F^{(\beta^*)} = 22000cm^2 = 2.2m^2$. The final form of the supporting system constitutes the initial condition of balance against the deflection of the structure due to the live loading.

The cables (β) along with the additional diagonal cables (after the initial condition) are used as passive control system for the suppression of the displacements of the bridge. The complete design of the bridge has been based on a three-dimensional finite element (3-D) model. The loading cases that have been considered in the structural analysis problem are the following:

COMB1: Solution for the initial deck structure where only the cable set (a) is present.

For this combination (loading case), the composite structure remained at the initial condition of equilibrium (without displacements for the bridge).

COMB2: Load combination including only the distributed live load.

The solution for the COMB2 gives as results:

$$\max\ \Delta u = -0.06571\ m < \frac{L}{750} = -0.266\ m$$

COMB3-COMB4: Load combination including distributed live load and additional concentrated live loads (due to heavy vehicle) at two consecutive nodes of the deck.

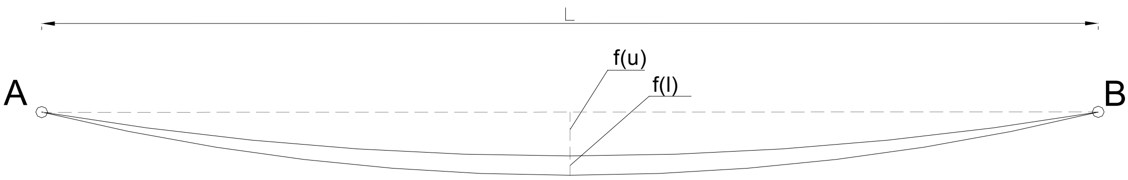

Fig. 3: The two line cable net system – MBN model.

The displacements for each case become:

a) $\max\ \Delta u^{COMB\ 3} = -0.2293\ m < \frac{L}{750} = -0.266\ m$,

b) $\max\ \Delta u^{COMB\ 4} = -0.13912\ m < \frac{L}{750} = -0.266\ m$

(the design criterion for deflection).

The maximum stress of the cable nets $(a)\,\&\,(\beta)$ is $S^{(a+(\beta))} = 40042kN$ and the cross section of the cable (β) becomes $F(\beta) = 1320cm^2$, equal to 1/15 of the ideal cross section (β^*) of the model. In the proposed analysis of the bridge with this model, the action of the wind as well as the action of the earthquake has been considered and the conclusion is that this method leads to very satisfactory results for the displacements and the stresses.

The results are similar with that of the following system of type MPS where they are shown with more detail.

DISPLACEMENT CONTROL SYSTEM OF TYPE MBN

The second displacement control system of type **MBN** for large span is herein proposed. The novel passive control system is based on the usage of a prestressing cable mechanism, where permanent loads and a part of the moving loads are relieved by the two lines of prestressing cables nets named (a^u) the upper and (a^l) the lower, respectively and the rest of the moving loads together with excessive displacements are taken by an additional cable net (β), that depends on the form and the use of the structure. In this presentation a bridge model of open length L=200.0m is proposed and studied as a numerical application of the method. It must be noted here that this system gives very satisfactory results and for very large openings (of length L=1000.0m and more), but here we present only the case of the same open length for comparison with the other two methods. The upper cable (a^u) is actually a group of similar, parallel and loaded cables that support the simply supported concrete plate (deck), which in turn follows the curved shape of the loaded cables.

The lower cable (a^l) is also a group of similar, parallel and loaded external cables following also a curved shape form. The two (upper and lower) cable lines are connected with stanchions of appropriate length between the homologous nodes. The curved shape form of each one of the cable lines depends on the load distribution between the upper and the lower cable. By this method, one is able to obtain the best mechanical properties of the structure by using the deep curve of the lower cable line (a^l) and at the same time having the low curve form (of the upper cable line (a^u)) for the deck which is necessary by the design procedures. After the achievement of this regulation and the distribution of the permanent loads between the two cables (upper and lower) creating balance between them, the further regulation due to the action of the moving loads should become passively from the system without external prestressing. This is necessary because the deflection at the mid-span for the two cables (upper and lower) after the action of the moving loads is different due to the different total length, curve and cross-section between them. In order to regulate the system of the two cables (a) and for the passive control of the expected displacements due to the moving loads, the previously mentioned system of cable net called cable (β) is introduced. The latter has the same length and follows the same curve as the upper cable (a^u), but under the permanent loads they are inactive, whereas they are activated only upon the introduction of moving loading on the structure. The prestressed cables require the determination of their suitable form so that the response of the system as a whole is optimized. The two flexible cables connected at the two extreme nodes of the structure exhibit a stress unilateral structural behaviour and, therefore, are subjected to vertical loads that correspond to cable tension. Therefore, the first step is to formulate the general system of equations of each one of the two cables hinged from the two extreme nodes and loaded by a general loading corresponding to a set of vertical nodal forces. This formulation begins by introducing the desired displacement of the cables. Using the equations of each cable, the system (9) of $2n\,(S_{(l)i}, \omega_{(l)i})$ non-linear equations is first formulated.

$$\begin{cases} \dots\dots\dots\dots\dots \\ S_{0i}\sin\omega_{0i} = S_{0i+1}\sin\omega_{0i+1} \\ S_{0i}\cos\omega_{0i} - S_{0i+1}\cos\omega_{0I+1} = P_{0i+1} \\ \dots\dots\dots\dots\dots \\ \dots\dots\dots\dots\dots \\ \sum_{i=1}^{n} l_i \sin\omega_{0i} = L_{A\overline{B}} \\ \sum_{i=1}^{n} l_i \cos\omega_{0i} = 0 \end{cases} \tag{9}$$

Table 4: MBN model, load combination (COMB1).

A/A	Load case	Remarks
1	$P_i^{perm.} = -2350kN$	Permanent load at all the nodes of the upper plate
2	$A_i^{upper} = 470kN$	Reaction load due to the prestressing at the nods of the cable set (a^u)
3	$A_i^{lower} = 1900kN$	Reaction load due to the prestressing at the nods of the cable set (a^l)

Table 5: MBN model, load combination (COMB2).

A/A	Load case	Remarks
1	$q_i = -375kN$	Distributed live load – common at all the nodes of the upper plate

Table 6: MBN model, load combination (COMB3-COMB4).

A/A	Load case	Remarks
1	$q_i = -375kN$	Distributed live load – common at all the nodes of the upper plate
2	$Q_i = -378kN$	Concentrated live loads (due to heavy vehicle) at two consecutive nodes of the deck, a) COMB3 at the nodes 8 & 9, b) COMB4 at the nodes 16&17

Table 7: MPS model, load combination (COMB1).

	Load case	Remarks
1	$P_i^d = -3350kN$	Permanent load at all the nodes of the upper plate
2	$A_i = 3350kN$	Reaction load due to prestress at the nodes of the cable set (a)
3	$S_x^{c1} = 190750kN$	Horizontal compressive load at the straight deck due to prestress
4	$\Delta t = 37.8^O C$	Thermal loading at the bridge straight deck

Table 8: MPS model, load combination (COMB2).

	Load case	Remarks
1	$P_i^d = -3350kN$	Permanent load at all the nodes of the upper plate
2	$A_i = 3740kN$	Reaction load due to prestress at the nodes of the cable
3	$S_x^{c1} = 212940kN$	Horizontal compress load at the straight deck due to prestress
4	$\Delta t = 37.8^O C$	Thermal loading at the bridge straight deck

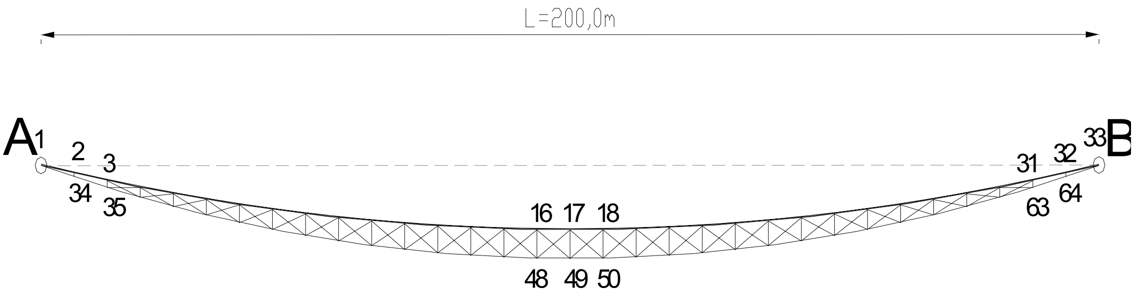

Fig. 4: MBN model - form of the controlled structure.

The herein proposed model concerns the introduction of the already mentioned net of cables (β) that acts as an effective passive control system of displacement against the deformations caused by moving loads. Thus, apart from the cables (a^u) and (a^l), respectively, being the basic cables of suspension, a third set of cables (β), being inactive in the situation of equilibrium under permanent loads is introduced. The latter cables are anchored in the same positions of suspension as the cables (a^u) and are activated as soon as additional moving loads start acting on the deck. As a numerical application of the method a road bridge of length L=200.0m is considered; the starting configurations are the calculated lengths by the system (9) for each one of the two cables (a^u) and (a^l), each one separately hinged by the common ends of suspension A and B. The formation of the supporting system begins with an arbitrary choice of dead nodal loads $P_i^{perm} = -2370kN$ between the two lines of cables. By taking into account a 20% of the total nodal load for the lower cable (a^l) and an 80% for the upper cable (a^u), one has $P_i^{perm(u)} = 20\% \cdot (-2370) = -474kN$ and $P_i^{perm(l)} = 80\% \cdot (-2370) = -1900kN$. Namely, the two cables system is regulated in such a way that the permanent loads are distributed by an optimal distribution according to the design procedures (i.e., a desirable value for the cable deflection at the mid-span). Taking into consideration the solution of the cables and the distribution of the permanent nodal loads in particular, we determine the corresponding cross section $F(a^{upper}) = 700cm^2$, for the upper cable and $F(a^{lower}) = 2000cm^2$ for the lower cable. Concerning the cable set (β) an idealized cable (β^*) with cross-section $F(\beta^*) = 14000cm^2$ is first taken and the sum of the cross sections for the upper cable becomes $F(a^u) + F(\beta^*) = 14700cm^2$. The final form of the supporting system constitutes the initial condition of balance against the deflection of the structure due to the live loading. The cables (β) along with the additional diagonal cables (after the initial condition) are used as passive control system for the suppression of the displacements of the bridge. The complete design of the bridge is based on a three-dimensional finite element (3-D) model. The loading cases that have been considered in the structural analysis problem are the following:

Table 9: MPS model, load combination (COMB3).

A/A	Load case	Remarks
1	$P_i^d = -3350kN$	Permanent load at all the nodes of the upper plate
2	$A_i = 3740kN$	Reaction load due to prestress at the nodes of the cable
3	$q_i^l = -390kN$	Moving load at all the nodes of the upper plate
4	$S_x^{c1} = 212940kN$	Horizontal compressive load at the straight deck due to prestress
5	$\Delta t = 37.8^O C$	Thermal loading at the bridge straight deck

Table 10: MPS model, load combination (COMB4-COMB5).

	Load case	Remarks
1	$P_i^d = -3350kN$	Permanent load at all the nodes of the upper plate
2	$A_i = 3740kN$	Reaction load due to prestress at the nodes of the cable
3	$q_i^l = -390kN$	Moving load at all the nodes of the upper plate
4	$Q_i^l = -378kN$	Heavy vehicle at two consecutive nodes of the upper plate (COMB4 at the nodes 8,9 – COMB5 at the nodes 16,17)
5	$S_x^{c1} = 212940kN$	Horizontal compressive load at the straight deck due to prestress
6	$\Delta t = 37.8^O C$	Thermal loading at the bridge straight deck

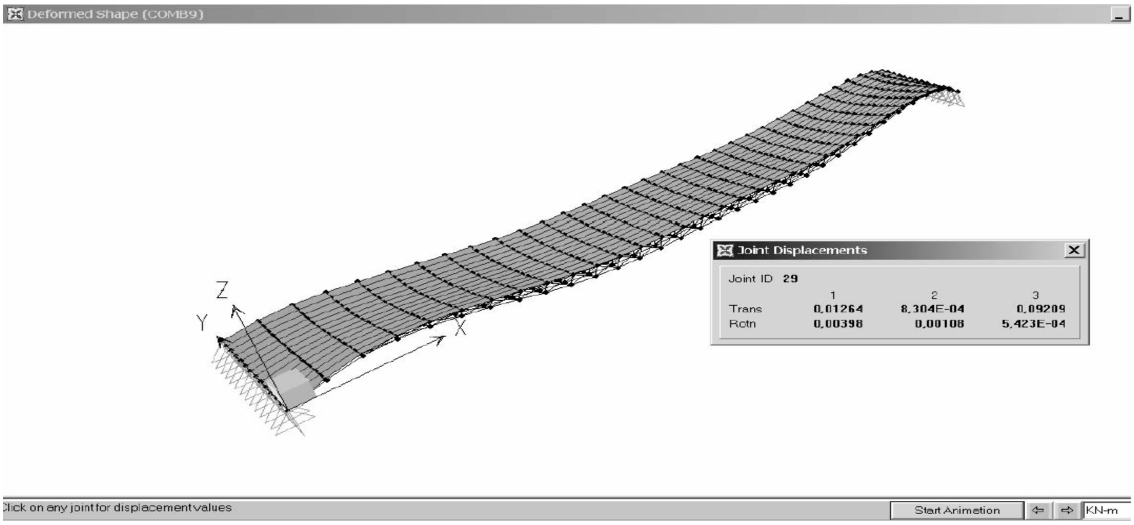

Fig. 5: MBN model – Earhquake action (COMB9)- joint displacements.

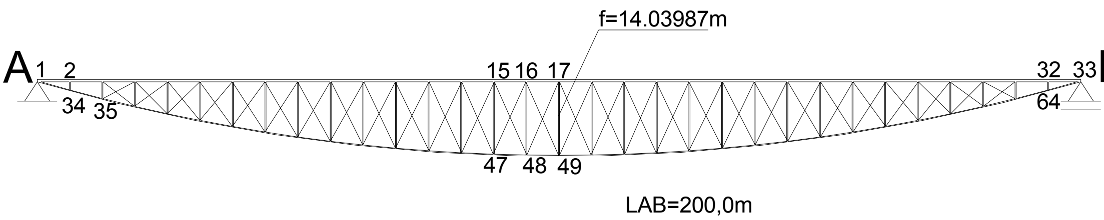

Fig. 6: Form of the structure including the diagonal cable element.

COMB1: Solution for the initial deck structure where only the cable set (a^u), (a^l) is present:

For this combination (loading case), the composite structure remained at the initial condition of equilibrium (without displacements for the bridge).

COMB2: Load combination including distributed live load.

The solution for the COMB2 gives:

$$\max \Delta u = -0.0353m < \frac{L}{750} = -0.266m \,.$$

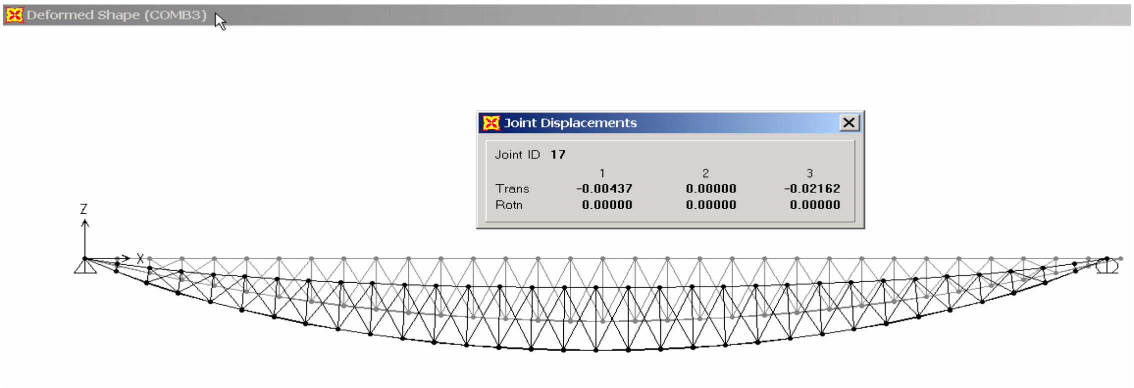

Fig. 7: MPS model – Load case COMB3- joint displacements.

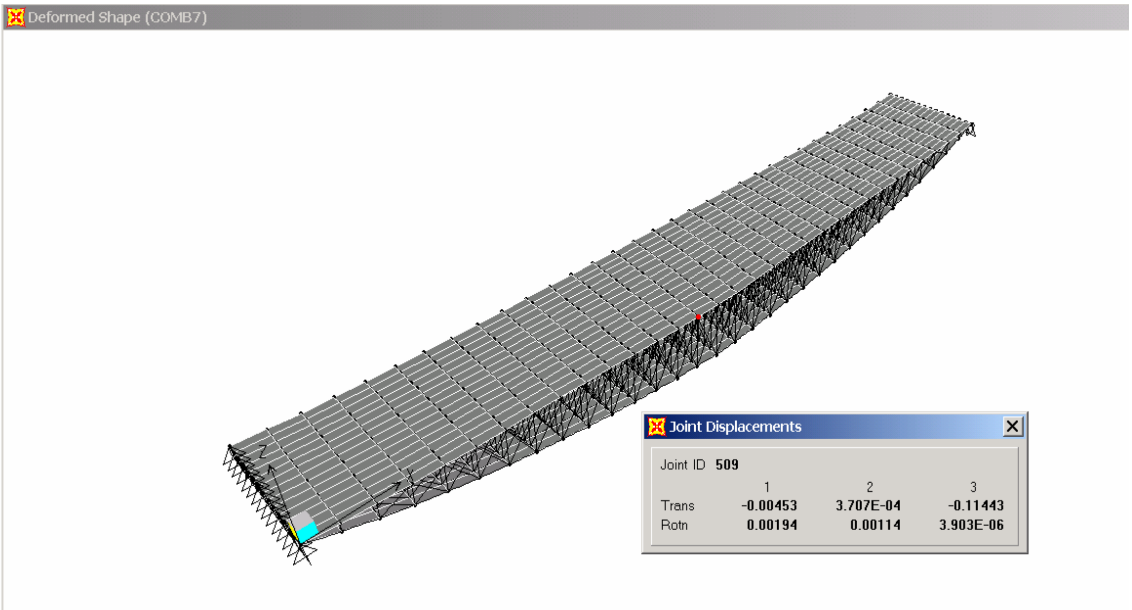

Fig. 8: MPS model – Load case COMB7- joint displacements.

COMB3-COMB4: The load combination includes the distributed live load and additional concentrated live loads (due to heavy vehicles) at two consecutive nodes of the deck. The displacements for each case become:

a) $\max\Delta u^{COMB3} = -0.11811m < \frac{L}{750} = -0.266m$,

b) $\max\Delta u^{COMB4} = -0.06107m < \frac{L}{750} = -0.266m$ (the design criterion for deflection).

The maximum stress of the upper cable nets $(a^u)\,\&\,(\beta)$ is $S^{(a^u+(\beta))} = 32017kN$ and the cross section of the cable (β) becomes $F(\beta) = 933cm^2$, equal to 1/15 of the ideal cross section (β^*) of the model. In the analysis of the bridge with the three-dimensional model (3-D), the wind action at the front side of the bridge has been taken into account as a horizontal live load. Therefore, the following additional load combinations have been assumed:

COMB5-COMB6: The bridge is considered without vehicles (COMB5) but with wind load $W_1 = 2.5kN/m^2$, where the height of influence is equal to $h_w = 3.00m$ (including the height of influence which corresponds to the amenable elements) leading to a load $q_w = 7.50kN/m$ or with vehicles in addition to the action of wind load $W_1 = 1.25kN/m^2$, where the height of influence is again equal to $h_w = 3.00m$ leading to a load $q_w = 3.75kN/m$. Taking into account the previous analyses, it becomes obvious that in the case of COMB6, only the cross-section of the bridge is moved $\max \Delta\upsilon^{w1} = 0.00227\,m$, whereas it is not deformed within the level of cross section. The results in the case of COMB6 are equivalent leading to a $\max \Delta\upsilon^{w2} = 0.00179m$. Especially in the case of turbulent wind flow and specifically for an upward loading being a percentage of the main wind load, no significant motion of the structure arises, due to the fact that this loading does not exceed the much bigger self-weight of the bridge deck.

COMB8: $1.0 \times E_x + 0.3 \times E_z$, & **COMB9:** $0.3 \times E_x + 1.0 \times E_z$, where E_x and E_z are the spectra of the reference earthquake along the x & z directions according to the Greek Aseismic Design Code (EAK), which is compatible to the European Codes and Norms EC8. Note that moving vehicle loads have not been taken into account due to the fact that the latter act favourably by limiting the displacement caused by the seismic action.

COMB8: $\max \Delta u = 0.03909 < L/750 = 0.26m$,COMB9, $\max \Delta u = 0.09209 < 0.26m$.

This can be obtained by means of a self-hinged system of the bridge that uses external prestressed cable lines (a^u) & (a^l) which directly neutralize the permanent loads of the structure without creating significant bending moments. By applying the method proposed here, the value for the necessary prestressing force of the cables depends only on the neutralization of the permanent loads and does not depend on the length of the span of the bridge.

DISPLACEMENT CONTROL SYSTEM OF TYPE MPS

This system of type **MPS**, corresponds to a complex deck-structure constituted by a straight upper part (a plate of concrete) and a lower one being a completely exterior steel flexible prestressed cable, anchored at the extreme cross-sections of the plate. The two (upper and lower) parts of the deck-structure are connected with stanchions of appropriate length between the homologous nodes. The operation between the two parts of the deck is based on the advantageous collaboration created between the upper stiffened plate and the bottom prestressed cable; the latter having a polygonal form develops tensions causing alleviation at the deck-structure undertaking not only the permanent loads but also a part of the moving ones. The prestressed cables used for the neutralization of the permanent loads are called cables (a). For the meticulous control of the expected displacements due to the moving loads, a system of cables (β) is also introduced. The latter have the same length and the same curve as cables (a), but under the permanent loads they are inactive, whereas they are activated only upon the introduction of moving loading on the bridge. The connection and the collaboration between the plain deck structure and the prestressed external cable impose the finding of a suitable form for the cable so that the response of the system as a whole is optimal. The flexible cable connected at the two extreme nodes of the structure exhibits a stress unilateral structural behavior and therefore, is loaded by the vertical loads that correspond to cable tension. It is a structure where unknown is the stress field (S_i) caused by any vertical loading P_i, and unknown is also the form (ω_i) influenced by the respective loading. Therefore, the first step is to formulate the already mentioned for the system of type MPS general system (1) of equations of the cable (a).

A road bridge of length L=200.0m is first considered; it is created by the connection of a simply supported concrete deck with a horizontal upper plate where the moving loads are acting, with the flexible external prestressed steel cable (a) that constitutes a system hinged by itself. The cable (a) is actually a group of similar, parallel and loaded cables that support the simply supported concrete structure (lower plate) that follows the curved shape of the loaded cables (a). The 33 nodes of the upper plate are separated in 32 segments of equal length $a = [200/32] = 6.25m$. The prestress cable is anchored at the extreme cross-sections of the upper plate of the deck structure. The loading cases considered in the structural analysis problem are as follows.

COMB1: The solution for the primal deck structure only with cable set (a).

For this combination (loading case) the composite structure remained horizontal with zero bending moments.

COMB2: The solution for the deck structure with cable set $(a)+(\beta^*)$ and diagonal cables between consecutive nodes of upper and lower plates, respectively.

The solution shows an upward vertical deformation:

$$\max\ \Delta u = 0.01162\ m < \frac{L}{750} = 0.266\ m\ ,$$

$$u(B) = 0.0002404m\ M_{\max}^{deck} = 41.09tm\ .$$

COMB3: The solution for the deck structure with cable set $(a)+(\beta^*)$ and diagonal cables between consecutive nodes of upper and lower plates, respectively including distributed live load.

The solution for the COMB3 gives a downward vertical deformation:

$$\max\ \Delta u = -0.02162\ m < \frac{L}{750} = -0.266\ m\ .$$

COMB4-COMB5: A combination including distributed live load and additional concentrated live loads (due to heavy vehicles) at two consecutive nodes of the deck.

The solution for the COMB4 gives biggest downward vertical deformation than COMB3: $\max\Delta u = -0.030m < \frac{L}{750} = -0.266m$, $M_{\max}^{deck} = 470.10kNm$.

Finally the solution for the COMB5 gives biggest downward vertical deformation than COMB4**:**

$$\max\Delta u = -0.03242\,m < \frac{L}{750} = -0.266\,m\ , \upsilon\mathrm{B} = -0.00949\ m\ .$$

COMB6: The solution for the case of the bridge without vehicles but with wind load $W_1 = 2.5\,kN/m^2$, where the height of influence is equal to $h_w = 3.00\,m$, (including the height of influence which corresponds to the amenable elements) and finally load) $q_w = 7.50\,kN/m$.

COMB7: The solution for the case of the bridge with vehicles, in which all the moving loads of COMB4 or COMB5 combinations participate together with the action of wind load $W_1 = 1.25\,kN\ /\ m^2$, where the height of influence is again $h_w = 3.00\,m$ with load $q_w = 3.75kN\ /\ m$. From the solutions one concludes that in the case of COMB6, only the cross-section of the bridge is moved (that is to say it is not deformed into the level of cross section) with $\max\ \Delta\upsilon^{w1} = 0.0006961\ m$. Equivalent are the results in the case of COMB7, where $\max\Delta\upsilon^{w2} = 0.003707m$. Especially in the case of turbulent wind flow and specifically for an upward loading (as a percentage of the main wind load), no significant motion arises, since this loading, does not exceed the much bigger self-weight of the bridge deck.

COMB8: $1.0\times E_x + 0.3\times E_z$, where E_x and E_z are the spectra of earthquake according the Greek Aseismic Design Code (EAK-2000).

COMB9: $0.3\times E_x + 1.0\times E_z$, where also E_x and E_z are the spectra of earthquake according EAK-2000 in the x and z directions.

COMB8: $\max\Delta u = 0.0303 < L/750 = 0.26m$

COMB9: $\max\Delta u = 0.04033 < 0.26m$

At the same time the horizontal displacements of the moving bearing is also inside the acceptable limits for such type of bearings as well as for the corresponding joint of the deck:

COMB8: $\max\Delta Bx = -0.00741\,m$,

COMB9: $\max\Delta Bx = 0.00169\,m$.

CONCLUSIONS

The displacements control systems that use external prestressed cable lines of type (a) directly neutralize the permanent loads without creating significant bending moments for the structure. By applying the methods proposed here, the value for the necessary prestressing force of the cables depend only on the neutralization of the permanent loads and does not depend on the length of the span of the structure. For that reason, the proposed systems are available for large and very large openings (of length L=1000.0m and more) with very satisfactory results. As passive control system for the bridge displacements a novel design is proposed which uses beta cables and additional diagonal cables for same cases. This design leads to very satisfactory results for both the displacements and the stresses. The proposed forms of the structures are very beneficial, they can be produced with lower cost and have high security and design factors, especially for countries with high earthquake and wind loads. This system of analysis gives also new ideas for the design of large span structures in the future.

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