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PARAMETRIC STUDY ON NONLINEAR STATIC ANALYSIS OF CABLE STAYED BRIDGES

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Abstract

This study is done to discuss nonlinear static behavior for cable-stayed bridges, hence develop a set of consistent design as well as a feasibility study of long span cable-stayed bridges over Nile River. In order to accomplish this goal, a thorough investigation of important key design parameters to determine the behavior of cable-stayed bridge and identify any gaps in current knowledge is done to be filled in order to enable the formation of a consistent set of design recommendations. Three span cable stayed bridge has been analyzed, the effects of the variation of different key design parameters: cross section of cables, cable layout either fan or harp pattern, pylon height to span ratio and mechanical properties of deck and pylon on the straining action of the bridge elements are investigated. The loads on the cable stayed bridge are a symmetrical load such as the own weight of all structural elements and live loads. The results related to the major factors to choose the ratio between the bending stiffness of a deck and axial stiffness of the cable to reduce bending moments and deflections in the deck and pylon are presented and discussed. Finally, some conclusions related to the cable stayed bridge's analysis/design are drawn.

Keywords: *Cable-stayed bridge, Pylon, Nonlinear static analysis, Finite element analysis, Design guidelines.*

1. INTRODUCTION

As demands for improved infrastructure increase around the world, civil engineers continue to be challenged to develop large bridges that must perform well even under extreme loading. An effective means of bridging large distances in both seismic and non-seismic regions is through the use of cable-stayed bridges. The decks of a cable-stayed bridge are supported using cables that climb diagonally to strong stiff towers, which act as the main load-bearing elements for the bridge. The orientation and construction methodology adopted for the bridges is such that under uniform loading the static horizontal forces imposed by the cables on the decks are typically balanced. Consequently, the towers will be designed to resist the vertical component of the gravity load and additional lateral loads associated with live loads, wind and seismic actions, impact from colliding objects, drag from water flow, and possibly others. The structural form and detailed behavior of cable-stayed bridges are well understood and there is a considerable amount of literature on cable-stayed bridges [1 ~ 4].

Cable-stayed bridges can be a very effective means of bridging large distances in both seismic and non-seismic regions. The rapid progress of this kind of bridges is mainly due to the development of computer technology, high strength steel cables, orthotropic steel decks and construction technology. Because of its aesthetic appeal, economic grounds and the ease of erection, the cable-stayed bridge is considered as most suitable for medium to long span bridges with spans ranging from 200 to about 1000 m [2, 5, 6]. As a matter of fact, the longer span length is, the more flexible the bridge structural system behaves. Because of their huge size and complicated nonlinear structural behaviors, the analysis of cable-stayed bridges is much more complicated than that of conventional bridges. Their design, analysis, and construction can be very challenging, and fortunately there is a considerable amount of literature that can assist engineers with both the analysis and detailed design of cable-stayed bridges. It is less common, however, to find simple recommendations for the conceptual design of cable-stayed bridges, the sources of nonlinearity in cable-stayed bridges mainly include the cable sag, beam-column and large deflection effects.

Gravity loading on the bridge will have a strong influence on the peak compression forces that develop in the longitudinal axis of the deck. Therefore, one might consider a strong heavy deck to resist the compression but of course the gravity loads themselves are a function of the weight of the deck. In addition, while a heavy deck may not be problematic for wind loading, it would certainly be an issue for seismic loading. As such, the ideal deck section is a strong but lightweight deck. One might also consider the benefits of a flexural stiff deck for non-seismic loads. An extensive parametric study undertaken by Walther [3] showed that for static loads, the use of a stiff deck section is not ideal for cable-stayed bridges, since it attracts significant bending moments at three critical zones: deck-to-pier, abutments, and mid spans [7]. However, this observation comes from static considerations, and a stiff deck can instead be quite beneficial when dynamic wind and earthquake loads are considered. Also note that vertical displacements of the deck may be most significantly affected by the stiffness of the cable-pier system. This can be appreciated simply considering the typical span-to-depth ratios of decks in cable-stayed bridges, which tend to be in the range of 100–200, well above normal ratios used to control deformations associated with beam flexure. As such, the deck stiffness will be more relevant for local deformations between cable support points and for dynamic vibrations associated with wind and seismic response.

The spacing of cables should be set with due regard to construction lifting and transport requirements for the deck, in addition to limiting static flexural demands. The gravity loads are also likely to impose the greatest axial loads on the piers and foundations. However, as both the piers and foundations of cable-stayed bridges tend to be massive structures in order to provide adequate lateral stiffness for wind, earthquake, and eccentric gravity loading, the vertical loading of these elements is not usually critical.

A long-span cable-stayed bridge exhibits nonlinear characteristics under loadings. It is well known that these long-span cable-supported structures are composed of complex structural components with high geometric nonlinearities: The nonlinear axial force-elongation behavior for the inclined cable stays under different tension load levels due to the sag initiated by their own weight (sag effect); The combined axial load and bending moment interaction for the girder and towers; Large displacement, which is produced by the geometry changes of the structure. In addition, nonlinear stress-strain behavior of each element including yielding should be included in the nonlinear analysis and overall safety evaluation.

The objective of this research is to study the static behavior of a cable stayed bridge as well as a feasibility study of a long span cable stayed bridge. A reference model is designed and used to investigate the influence of key design parameters on static behavior of a cable stayed bridge. This model is submitted as part of a feasibility study for a cable stayed bridge to cross over the Nile River, Egypt. The reference design is modeled in a finite element program to investigate and calculate the force distribution and deformations. With the help of the reference design, Three span cable stayed bridge has been analyzed, the effects of the variation of different key design parameters: cross section of cables, cable layout either fan or harp pattern, pylon height to span ratio and mechanical properties of deck and pylon on the straining action of the bridge elements are investigated. The loads on the cable stayed bridge are a symmetrical load such as the own weight of all structural elements and live loads. Finally, some conclusions related to the analysis/design of the cable stayed bridges are drawn. The results of the parameter study are used to determine an optimization of the reference design.

2. NONLINEAR CONSIDERATIONS IN ANLAYSIS

A cable-stayed bridge is a nonlinear structural system in which the girder is supported elastically at points along its length by inclined cable stays. Although the behavior of the material is linearly elastic, the overall load-displacement response may be nonlinear under normal design loads [6, 8, 9]. Geometric nonlinearities arise from the geometry changes that take place as the bridge deforms under loadings. As mentioned above, there are usually three sources of the geometric nonlinearity: cable sag effects; axial force and bending interactions; and large displacements. To account for the sagging of inclined cables, an equivalent straight chord member with an equivalent modulus of elasticity is considered [10].

$$E_{eq} = \frac{E}{1 + AE(wL)^2 / 12T^2} \tag{1}$$

Where E_{eq} = equivalent elastic modulus of inclined cables; E = cable material effective elastic modulus; L = horizontal projected length of the cable; w = weight per unit volume of the cable; and T = cable tensile stress. The nonlinear analysis of a long-span cable stayed bridge finally reduces to forming the nonlinear incremental equilibrium equations of the system and to solving these equations. Based on the characteristics of the geometric nonlinear sources, the sag effect of cables was accounted for by the Ernst's equivalent elastic modulus concept, the structural geometric change due to large displacement was included in updating each set of node coordinates, whereas the axial force-bending moment interaction effect was considered by the stability beam functions. Actually, by using the Ernst's equivalent elastic modulus concept, the secant stiffness matrix of an inclined cable is simply equal to the stiffness matrix of a truss element with length L and cross-sectional area A. Truss elements can therefore be sufficiently used to model the inclined cables. Both bending and axial forcing members such as the girder and towers are suitably modeled by general beam elements. It can be assumed that the geometrical deformations of structural members in a long-span cable-stayed bridge are characterized by large displacement and small strains.

The material nonlinear analysis of a long-span cable-stayed bridge depends on the nonlinear stress-strain behavior of individual materials for individual structural elements. When some points (integration points) of an element exceed the yielding limit of individual materials, the stiffness matrix of the element should be revised to form the elastic plastic stiffness matrix.

For a long-span cable-stayed bridge, the dead loads always contribute the most to total bridge loads. After the bridge is completed and before the live loads is applied, the bridge has sustained large dead loads so that the large deformations and initial stresses that already exist in each member should be considered. Therefore, the geometrical nonlinear analysis of a long-span cable-stayed bridge under live loads should start from the nonlinear equilibrium configuration after dead loads are applied.

A simplified 2D numerical model of the bridge is used for obtaining their nonlinear responses and self-weight and live load distributions are considered. The bridges studied are defined by combining characteristics such as the layout of the stays, fan or harp pattern and the stiffness of the deck.

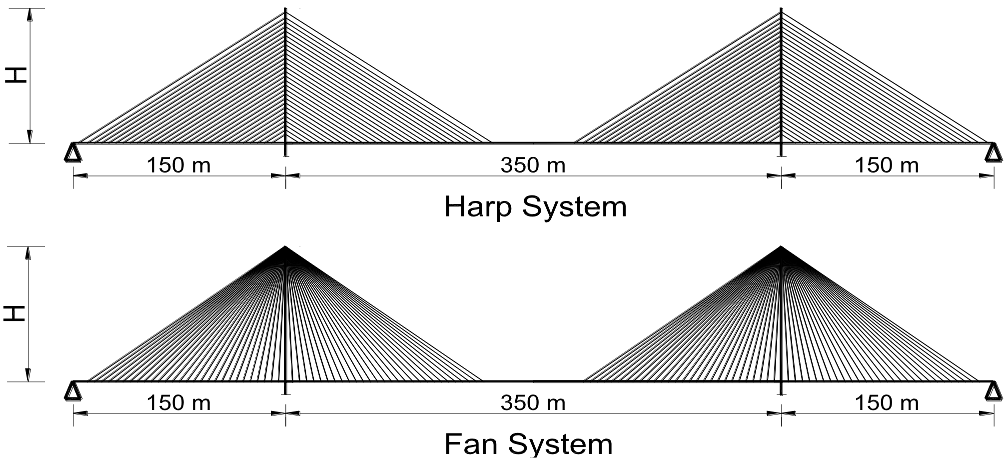
3. MATHEMATICAL MODEL OF THE STUDIED BRIDGES

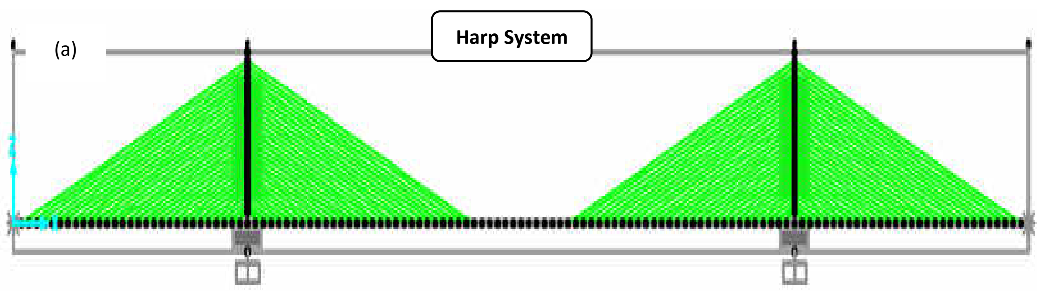
3.1 Description of Cable-Stayed Bridge

The example bridge studied here is Nile River long span cable-stayed bridge with 350 *m* central span length, 150 *m* left/right side spans. The elevation view of the bridge is shown in Figure 1. The deck cross section is an aerodynamically shaped closed box steel girder 18 *m* wide and 1.2 *m* high. The bridge towers are *H*-shaped hollow reinforced concrete towers 105 *m* high. The four groups of cables are composed of high strength steel wires 7 *mm* in diameter with from 294 to 1175 wires per cable. The stay cables are single arrangements. The three main bridge elements of the example bridge, namely, steel girder, cables, and reinforced concrete towers are composed of two different materials. The weight per unit volume of each cable depends on the number of wires in individual cables, the properties of each structure elements are shown in Tables 1 and 2.

3.2 Reference Design

A general assumption in cable supported bridge design is to have a reduced global bending moment to about zero under self-weight loading. This means that one wants to achieve that the self-weight load is completely supported by the cables. This can be approximately achieved by manipulating the initial tensile force in the stayed cables with minimum deck deflection and minimum bending stresses caused by the global bending moment in the stiffening girder. In the FEM program this initial tensile force on the main cable is done by applying a temperature load that causes the cables to become shorter which is just a modeling tool to apply a pretension on a structural member. The ideal state of a cable-stayed bridge can be defined as the minimized total bending energy accumulated along the girder. The dominant issue of the design and build of a cable-stayed bridge is to compute and achieve the ideal state. The dead and traffic loads including impact on the girders are transmitted to the pylons by inclined cable stays.





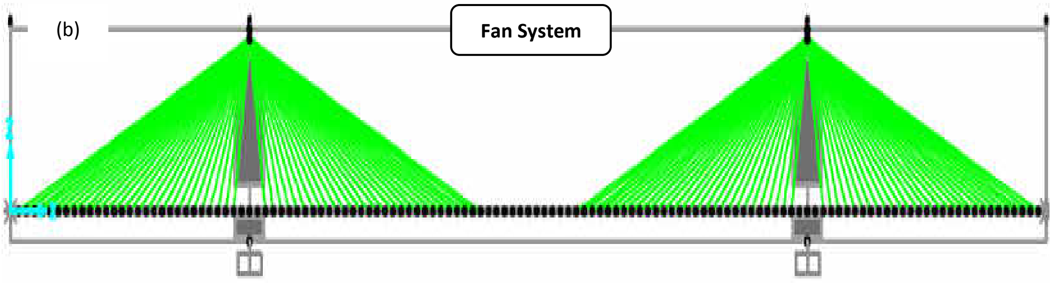


Figure 1 Layout of the cable stayed bridges

Table 1 The material properties for cable stayed bridge

	Material Type	Unit weight; KN/m^3	Modulus of Elasticity; KN/m^2
1	Box steel Girder	76.973	199947978.8
2	Pylon	23.563	33500000.0
3	Cable	76.973	199947978.8

Table 2 The cables properties used to study the behavior of Cable stayed bridge

Diameter, m	Area of cross section; cm^2	Weight; KN/m
0.12	113.04	0.8817
0.16	200.96	1.5675
0.20	314.00	2.4492
0.24	452.16	3.5269

The total width of the Nile River and river banks is succeeding 1000 *m* and the average width of the river is 500 m. Horizontal navigation clearance of 350 *m* and Vertical of 9.10 *m* are requested to be considered in design of the cable-stayed bridge. The bridge has overall width of 18.0 *m*. A main span length of 350 *m* and two 150 *m* side spans length with several approach spans are chosen as a starting point for the so-called reference model.

To get the best configurations of cables and the optimum pylon height, many parametric studied are taken into considerations. The major of these parameters are the arrangements of cables, height of pylon to span ratios (H/L), the inertia of deck and pylon. The design of a pier-to-deck connection should consider the control of both longitudinal and transverse response. The critical response direction will depend on the relative magnitude of seismic and eccentric gravity loads, as well as displacement limits for the different response directions. For standard cable-stayed bridge configurations, the connection in the longitudinal direction will need good stiffness to limit deck displacements due to eccentric gravity loads.

The properties of the reference design of cable stayed bridge are that: the pylons are composed of reinforced concrete with hollow rectangular uniform section 18*m* x 2*m* giving inertia of $I_x = 562.583\ m^4$, $I_y = 10.583\ m^4$ and Area $A = 19\ m^2$. The deck is box girder with an equivalent thickness of 0.2 *m*, with a width of 18 *m* and $I_y = 37.903\ m^4$, $I_x = 0.3127\ m^4$ and Area $A = 1.3403\ m^2$. The height of tower = 70 *m* and height of pylon above the deck equal to 60 *m*. The connection between the pylon and deck is rigid, while the pylon base is fixed and other two supports are rollers. The cables used to study the behavior of cable stayed bridge have the properties shown as **Table** 1 and with modulus of elasticity $E=1.999 \times 10^8\ KN/m^2$.

3.3 Finite Element Modeling

This section introduces the finite element three-dimensional model of the studied bridges. **Figure 1** shows the configuration for the studied bridges and the number of nodes and of cable elements in the global coordinate system. The girders, which have the central span length of 350 m, are supported by a series of cables aligned in a fan-type, and a harp-type structure respectively. The girders, the cross beams and the pylons are modeled using a number of beam–column elements. Each cable stay consists of an equivalent straight truss element, and is connected with pins at the girders and the pylons. The behavior of cable-stayed bridge depends highly on the manner in which the girders are connected to the pylons. In finite element analysis, the girder and pylon are simulated by beam elements, while the cables by tension-only truss elements. Nonlinear factors, large deformation, initial internal force and sag of the cable are taken into account.

A three-dimensional nonlinear finite element model is developed for cable-stayed bridges under static loadings based on the total Lagrangian formulation. The model can account for the large displacements that are usually associated with extended in plane contemporary cable-supported structures. At each step the equilibrium of the structure is established taking into account displacements (geometric non-linearity) and laws of behavior of the materials. In this paper; SAP2000 Ver. 14 program [11] is used for nonlinear static analysis of the behavior of cable stayed bridge. This program enables the designer to model a structure and to apply certain loads and loading combinations from which the effects like member forces and deflections can be calculated. With beam elements, the 3D model is built up with one dimensional line elements. This enables to model the total bridge structure and calculate member forces due to certain load cases and combinations. The scope of the model is to be able to analyze the model statically in a three dimensional way. Also an assessment will be made with respect to the geometric nonlinear effects of a cable supported bridge, the so called second order effects.

For FEM modeling, the girder is divided into 130 beams-column elements and each tower are divided into 13 beam-column elements. Each cable is treated as a plane truss element. Because of the complex cross-section shape of the bridge, for simplicity, the equivalent thin-walled box section of the girder and towers are used [12, 13]. The equivalent sections are obtained by equalizing the cross-section areas and section inertia moments of the girder and towers. The connection between the pylon and deck is rigid, while the pylon base is fixed and other two supports are rollers as shown in **Figure 1**. Under normal design loads, the material in a cable-stayed bridge is considered to remain elastic; however, the overall load deformation relation can still nonlinear. The analysis is carried out on an elastic model frame, making it possible to take into account the influence of geometric non-linearity in members in compression (second order effect) which are produced by finite deformations coupled with changes in the stiffness of a structure under applied loadings.

3.4 Loading for Analysis

The static analysis for all considered cases is carried out with uniformly distributed dead; DL (self weight + super imposed load) and super imposed load; LL loads along all spans lengths with intensity of 20 and 100 KN /m, respectively. The ultimate load combination that is taken into consideration is W_u = 1.4 DL + 1.6 LL.

4. NUMERICAL ANALYSIS AND DISCUSSIONS

Parametric studies on cable-stayed bridges are performed for investigating the individual influence of different key design parameters in such bridges. The geometry of the bridges is defined on the basis of the parametric study; each one of the bridges has a total length of 650 *m* spaced out in one principal central span of 350 *m* and two side spans of 150m. The length of the spans, the layout of stays and the distance between stays are detailed for both fan and harp patterns in **Figure 1**. The main mechanical properties are detailed in **Tables 1** and **2**. Numerical models of the bridges are developed and studied by means of linear static analysis using the finite element code SAP2000 [11]. The deck and pylon of the bridge is modeled using beam-column elements while truss elements are considered for the stays. The nonlinear behavior of the stays is taken into account with Ernst's modulus of elasticity, the influence of geometric non-linearity in members in compression (second order effect) which are produced by finite deformations coupled with changes in stiffness of a structure under applied loadings.

In this paper, different key design parameters of cable stayed bridge are studied to get their influence on the principal characteristics of the target bridge, which are: the mechanical properties and layout system of stayed cables; the mechanical properties and height to span ration of the pylon; the mechanical properties of the deck. Layout system of cables either fan or harp patterns and a wide range of cables' cross section diameter from 012 *m* to 0.24 *m* are investigated. Moreover; the effect of moment of inertia of deck variation as a ratio from 114.07% to 131.66% to that of reference design, the moment of inertia variation of pylon from 113.61% to 127.32% to that of reference design with different values of pylon height to span ratio *H/L* from 0.17 to 0.5 are studied. The static analysis is carried out taking into account the symmetrical gravity load only and the effect of cables' cross section diameter is studied for *H/L* equal to 0.3 (H = 105 *m*, and L main span = 350 *m*, $L_{\text{left}} = L_{\text{right}}$ = 150 *m*).

4.1 Effects of Mechanical Properties and Layout System of Stayed cables

It can be seen that there is slight difference in the maximum deflection at mid span of the deck between harp and fan cable layout system, moreover, it is obviously that with cable diameter increases, the deflection for harp and fan layout system decreases for stiff cable, and the difference in deflection between harp and fan is reduced as shown in **Figure 2**. The variation of diameter of cables from 0.12 *m* to 0.24 *m* could result in the bridge deck's deformation response decrease in the harp system by percentage from 26.3% to 51.4%, and in the fan system from 25.6% to 50.3%.

The bending moment peak response at the deck's mid span for fan layout system displays bigger values than that for harp layout, this effect decreases with the stiff cable system, could reach 7.3% for low stiffness of cable system. The bending moment peak response decreases 20% with increasing the diameter of cables from 0.12 *m* to 0.24 *m* as shown in **Figure 3**.

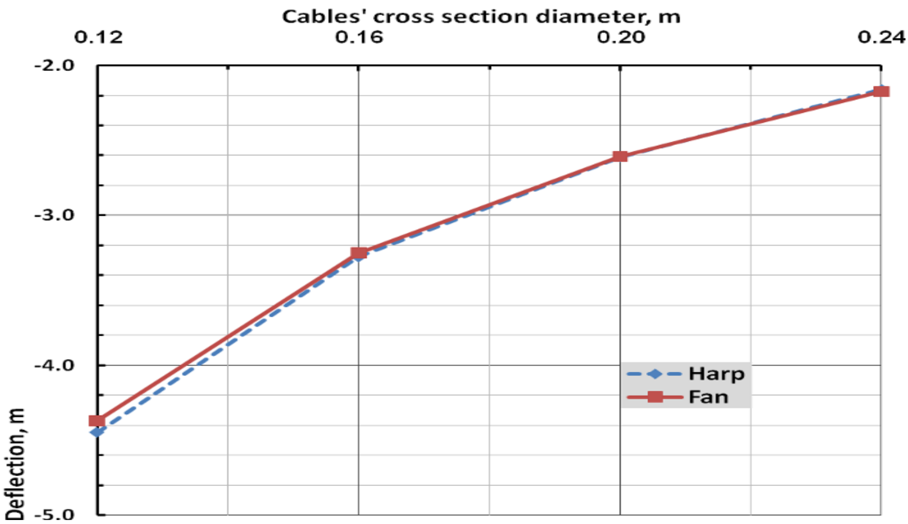


Figure 2 Maximum deflection variation with cables' diameter at the deck's mid span

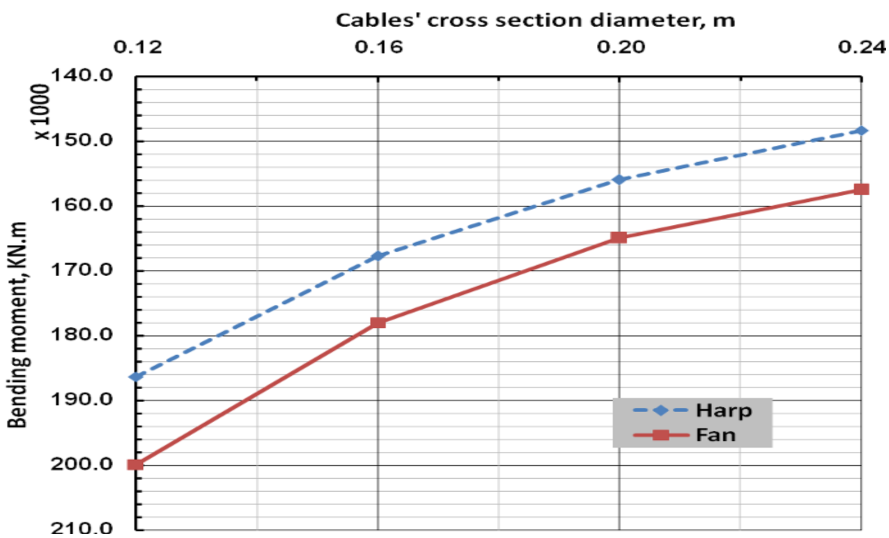


Figure 3 Maximum bending moment variation with cables' diameter at deck's mid span

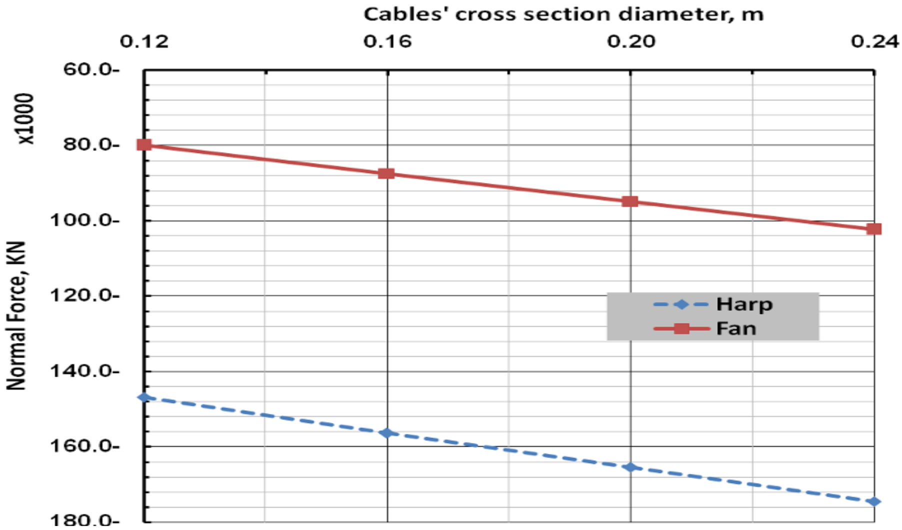


Figure 4 Normal force variation with cables' diameter at 150 *m* distance on the deck

The normal force near the pylon (150 *m* distance on the deck) gets higher values with stiff cable system, this effect reach around 19% and 28% for fan and harp systems, respectively, as shown **Figure 4**. The influence of the stays layout is analyzed with the same properties in the previous except of the height H above the deck is equal to 105 *m* and diameter of stay equal to 0.2 *m*. while the cable layout system has no significant effect on the deflection peak response at mid span of the deck, the deflection response is significantly depends on the cable layout system in the side/middle span around pylon, it can be seen that there is very little difference in the level of deformation at the mid span but there are a gap between the response of the deflection between the fan and harp system in the side span as shown in **Figure 5**. The bending moment response of the deck is affected by the cable layout system in the region around the pylon deck connection only as shown in **Figure 6.**

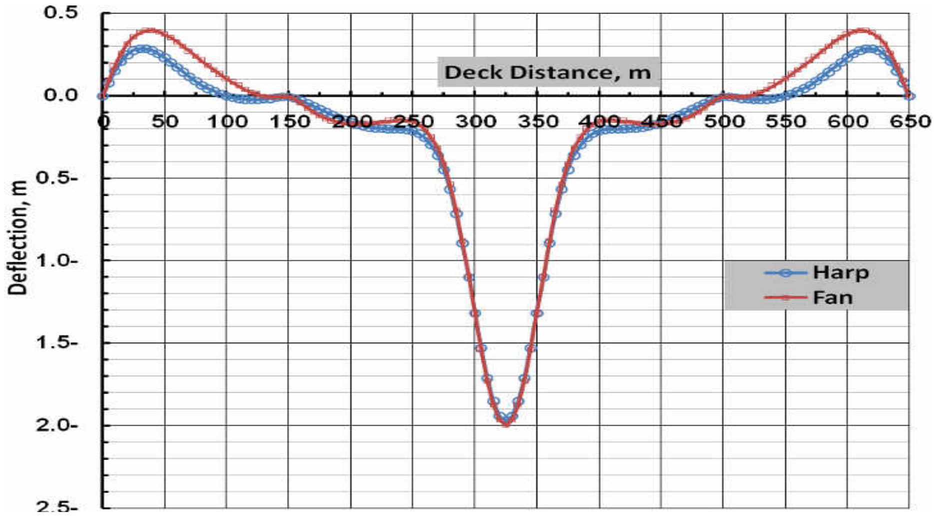


Figure 5 Deflection response of the deck

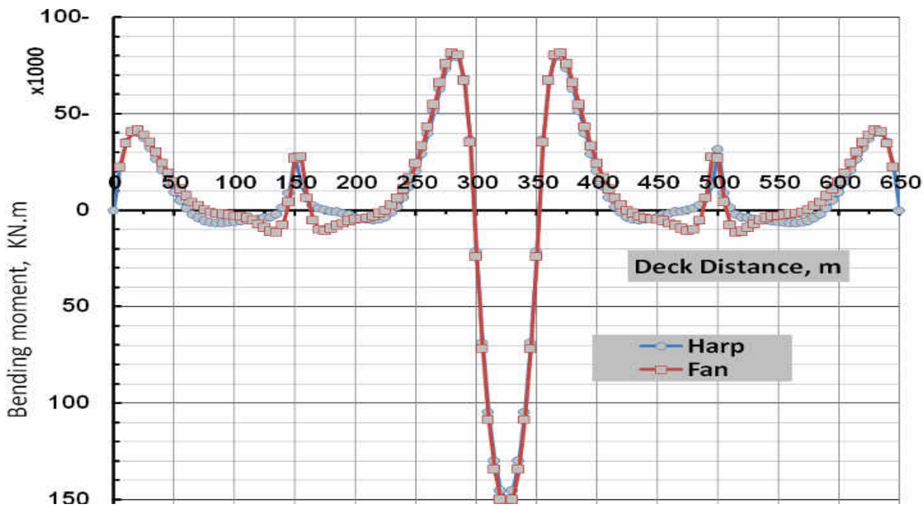


Figure 6 Bending moment response distribution over the deck

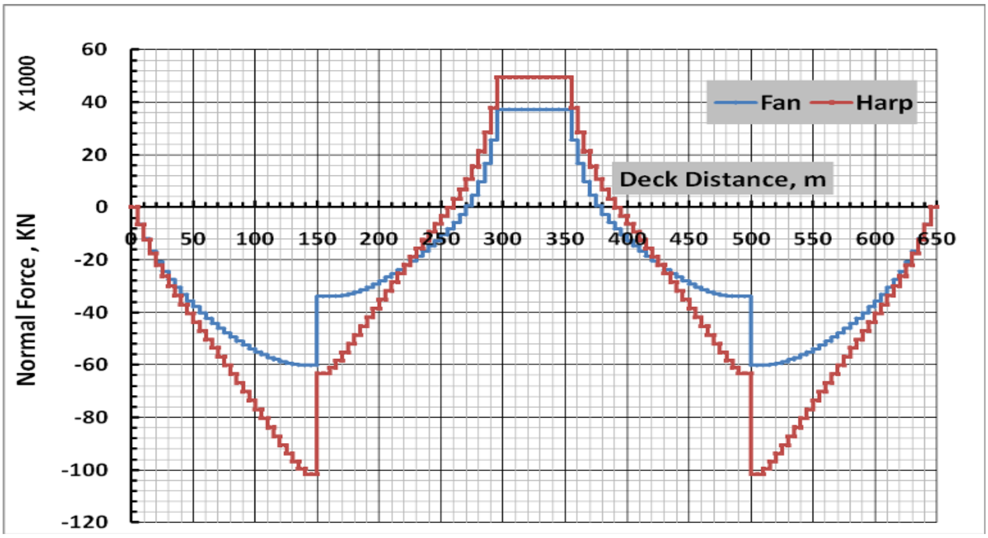


Figure 7 Normal force response distribution over the deck

It should be said that the deck is appreciably more heavily stresses in a bridge of harp pattern than in fan pattern. In the particular case of this study, the difference is about 50% under combination load. As a result, the harp layout appears to be less suitable for large-span bridges, since the value of the normal force calls for considerable strengthening of the cross-section, to provide both strength and stability in addition, the total quantity of cables required is higher as shown in **Figure 7**.

The variation of bending moment response along the pylon height with cable cross section's diameter from 0.12 *m* to 0.2 *m* in the harp and fan system is shown in **Figure 8**, as the cable system gets stiffer as cable's diameter increases, the bending moment response decreases around 17% and 23.5% at the pylon base for harp and fan layout system, respectively. The harp layout system has higher bending moment demands at pylon base compared to that of fan layout system irrespective of stiffness level of cable system.

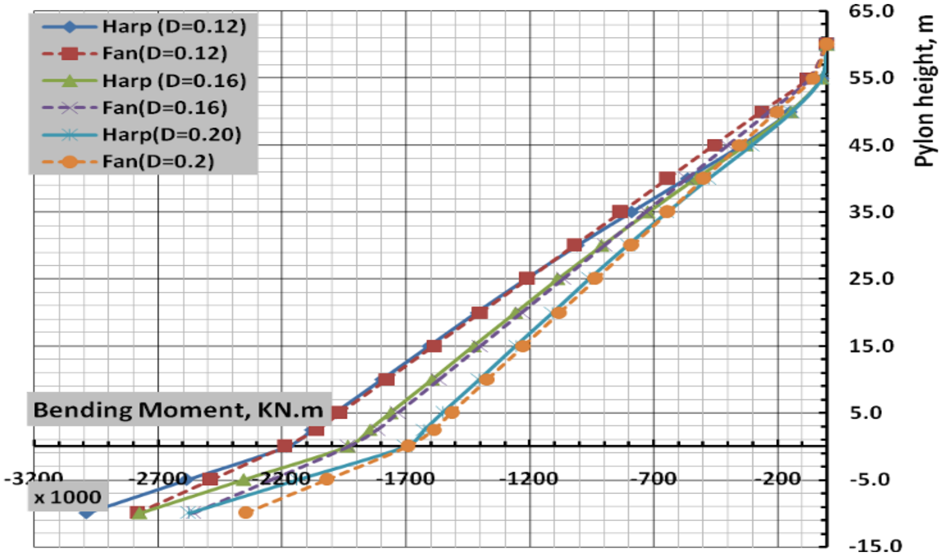


Figure 8 Bending moment response along the pylon height

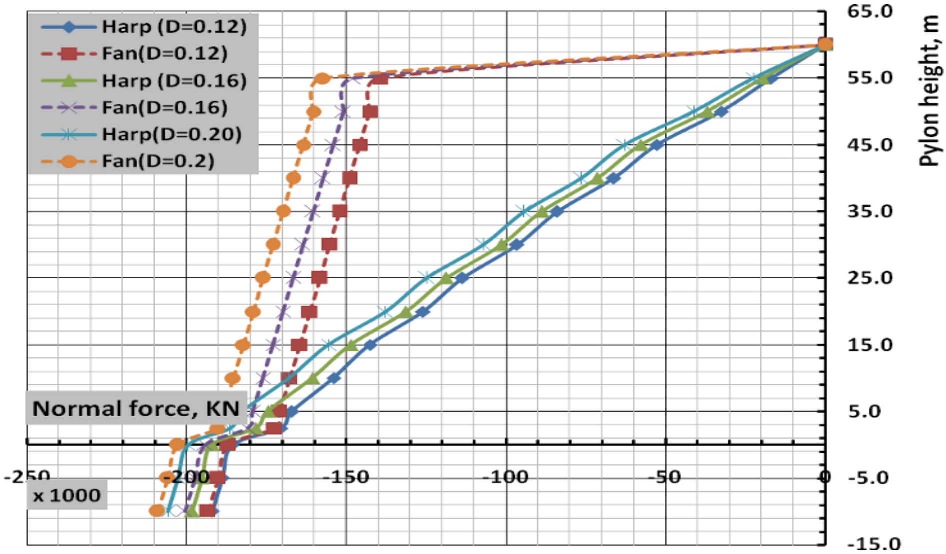


Figure 9 Normal force response along the pylon height

The variation of normal force response along the pylon height with cable cross section's diameter from 0.12 *m* to 0.2 *m* in the harp and fan system is shown in **Figure 9**, as the cable system gets stiffer as diameter's increase, the axial force response decreases around 12.5% and 10.5% at the pylon base for harp and fan layout system, respectively. The fan layout system has higher axial force demands along the pylon height compared to that of harp layout system irrespective of stiffness level of the cable system. The normal force distribution along the pylon height for fan layout system has almost constant uniform distribution with values approach the axial force demands at pylon in deck level that could affect the pylon stability due to buckling.

The cables of the single connection point solution are all relatively long in comparison to the multiple connection point solution, in which the cables close to the base of the piers are short. As such, a uniform longitudinal displacement of the deck imposes much larger strains on the short cables, which implies that the longitudinal displacement capacity of the bridge is much lower than an identical bridge with a fan-type cable arrangement. While the increased displacement capacity offered by the fan-arrangement may be attractive, there are also reasons for which the distributed cable arrangement of may be preferred. For example, the shorter cables will provide a stiffer solution, which may be particularly useful in limiting the lateral displacement of the deck, thereby limiting demands on dampers and expansion joints. Kawashima *et al.* [14] also pointed out that greater damping could be expected in the longitudinal mode when a distributed cable arrangement is adopted. As such, in deciding on a cable arrangement, the designer should weigh the benefits of reducing deformation demands on dampers and joints against the increased cable diameters that are likely to be required to sustain the larger design forces associated with the stiffer system.

*4.2*Effects of Mechanical Properties and Height to span ratio of Pylon

The effects of pylon height to span ratio; *H/L* range from 0.17 to 0.50 on bridge response are studied, it is obviously that the increasing of *H/L* leads to significant decrease of deflection response of the deck. The deflection response decreases as *H/L* increases, these reductions in case of harp system reach 25.5 % and 32.2 % for *H/L* equal to 0.30 and 0.50 compared to the response for *H/L*= 0.17; respectively, while these reductions in case of fan system reach 24.5% and 30.4% as shown in **Figure 10.**

Figure 11 shows that the bending moment response demands display slight decrease as pylon height to span ratio; *H/L* increases. The bending moment response of the deck at mid span decreases as *H/L* increases, these reductions in case of harp system reach 3.5 % and 2.3 % for *H/L* equal to 0.30 and 0.50 compared to the response for *H/L*= 0.17; respectively, while these reductions in case of fan system reach 5.8% . In brief, the increase of the height of tower is beneficial to decrease the response, but larger cross-section of tower and higher cost are needed.

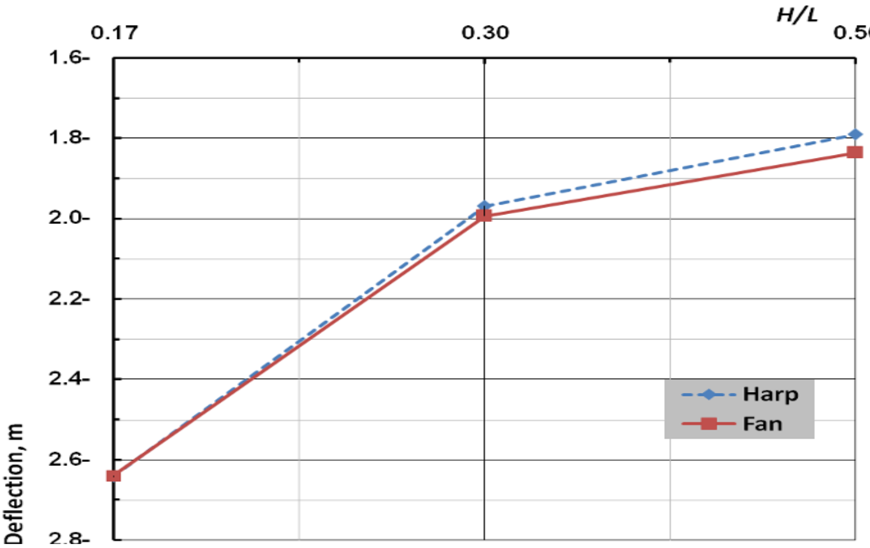


Figure 10 Deflection response of the deck at mid span for different *H/L* values

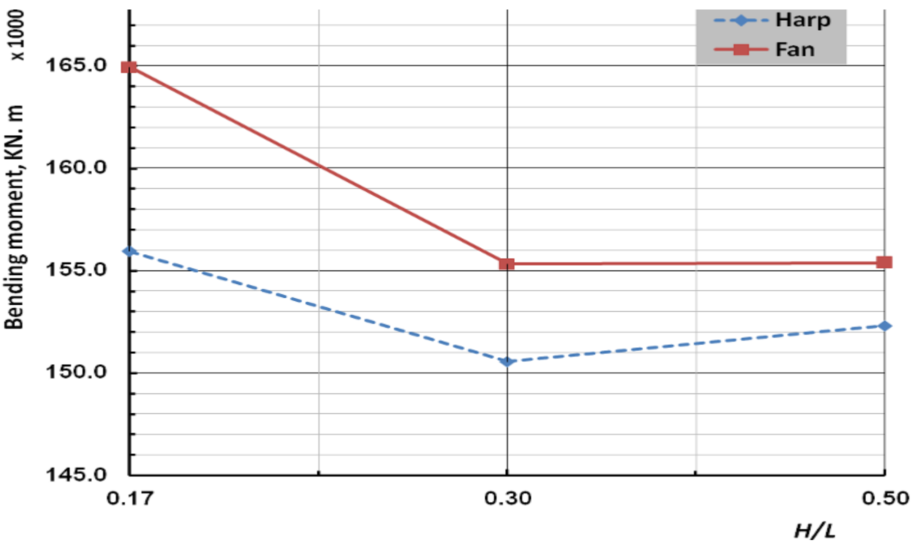


Figure 11 Bending moment response of the deck at mid span for different *H/L* values

Figure 12 shows that the normal force response demands display significant decrease as pylon height to span ratio; *H/L* increases. The normal force response of the deck at pylon position (deck distance 150 *m*) decreases as *H/L* increases, these reductions in case of harp system reach 39.4 % and 61.7 % for *H/L* equal to 0.30 and 0.50 compared to the response for *H/L*= 0.17; respectively, while this reductions in case of fan system reach 36.6 % and 59.2% . The deck inertia of 37.9 m^4 is used as a reference value, while the pylon inertia is varied from 562.58 m^4 to 715.75 m^4. The pylon rigidity has slight effects on the deformation demands of the deck, the reduction percentage is not more that 3.5 % either of harp or fan layout system for an increase of 1.5 times of the pylon reference inertia as shown in **Fig .13.**

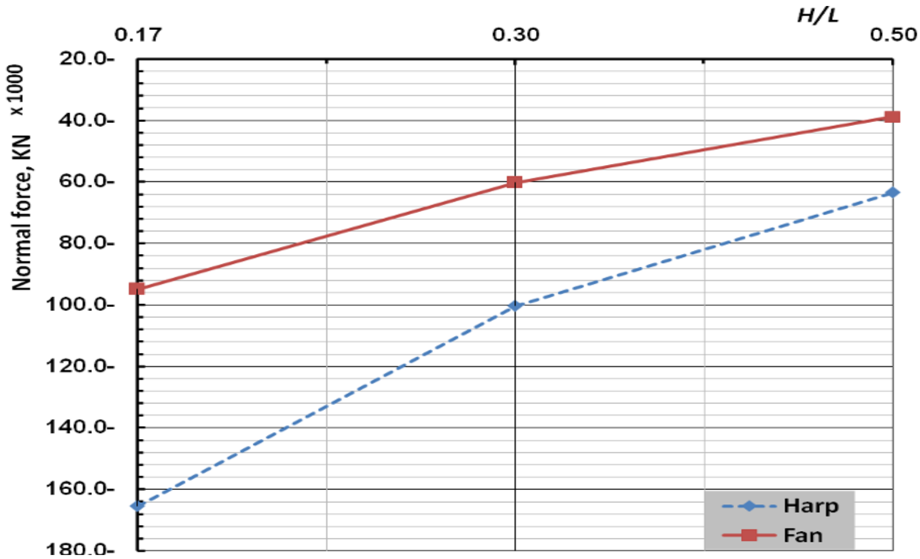


Figure 12 Normal force response of the deck at pylon (150 *m* deck distance) for different *H/L* values

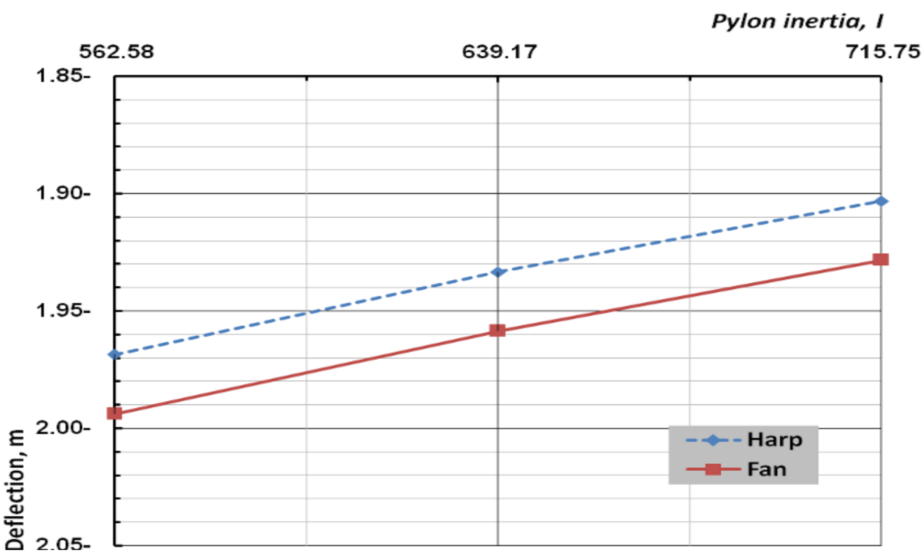


Figure 13 Deflection response of the deck at mid span for different pylon inertia

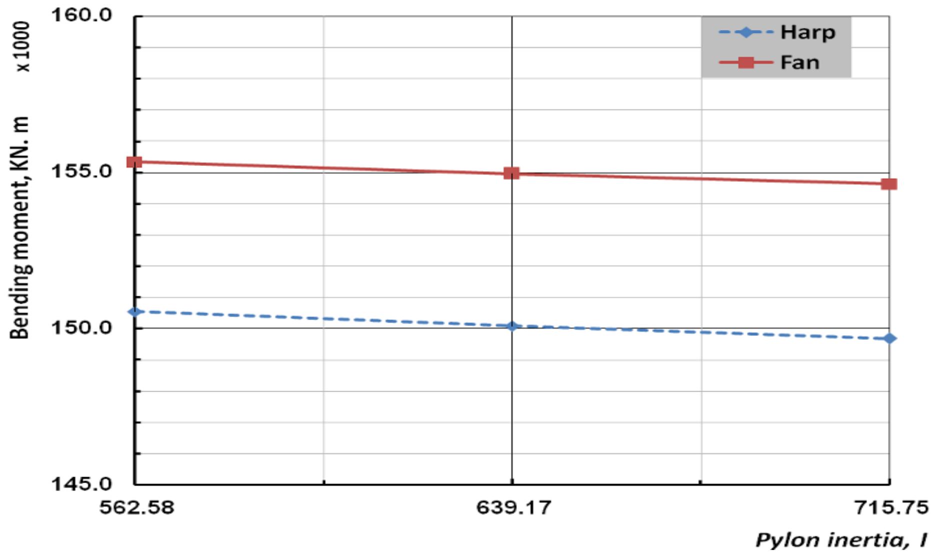


Figure 14 Bending moment response of the deck at mid span for different pylon inertia

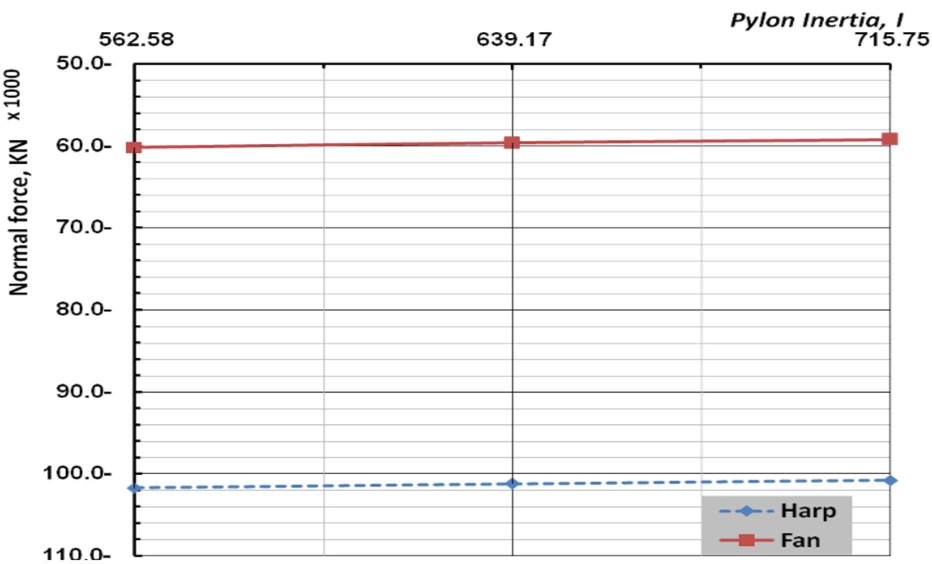


Figure 15 Normal force response of the deck at pylon for different pylon inertia

Figs. 14 and **15** show that the pylon rigidity has slight effects on the bending moment/axial force demands of the deck, the reduction percentage is not more than 1.6 % either of harp or fan layout system for an increase of 1.5 times of the pylon reference inertia**.** Under the dead load, with the increase of the pylon height, the horizontal component of cable force decreases gradually. In the meantime, the axial force and stress in girder and the anchorage force also reduce gradually, with the increase of the tower height, the axial force in girder, the stresses in cable and tower decrease. At the same time, the horizontal component of anchorage force increases, while the vertical component increases. In the case of deflection response, the value of deck decreases, while that the value of the pylon increases a little. These results indicate that the vertical rigidity increases while the longitudinal rigidity decreases a little.

4.3 Effects of Mechanical Properties Bridge Deck

The pylon inertia of 562.58 m^4; the height of pylon above the deck remains of 105 *m* (*H/L* = 0.3); cable stay diameter of 0.2 *m* and number of cables of 28 x 4 are used as a reference values, while the deck inertia is varied from 37.9 m^4 to 49.9 m^4. The deck rigidity has significant effects on the deformation demands of the deck, **Figure 16** shows this tendency of a decreasing deflection of the deck, the reduction percentage reaches more that 47% and 50% for harp and fan layout system; respectively. So it can concluded that the increasing of deck inertia plays a big role in decreasing deflection of the bridge this underlines the importance of the role played by the stays.

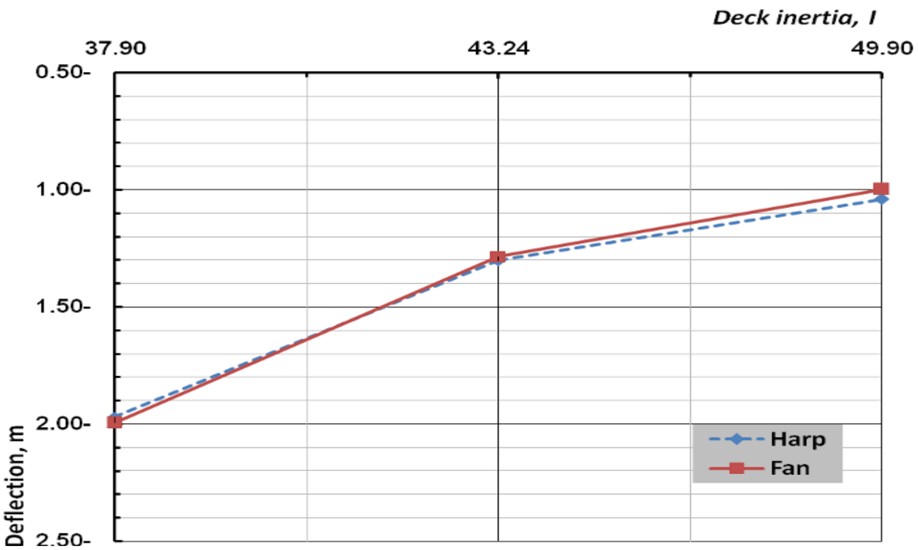


Figure 16 Deflection response of the deck at mid span for different deck inertia

Figure 17 shows the bending moment response with the variation of deck rigidity, where, the peak response of the bending moment demands at mid span of deck get higher values as deck rigidity increases, and reach more than 85% and 89% for harp and fan cable system, respectively. The bending response is accompanied by an extension of the highly stressed zone of the deck. **Figure 17** clearly shows that with an increasing deck inertia/stiffness, the deck tends to carry a larger part of total bending moment and smaller participating by the cable system and the deflection reduces significantly. Increasing the bending stiffness has a significant effect on the moment distribution. A larger stiffness of the deck means the bending moments increase significantly. These observations lead to the conclusion that a deck with a high inertia in the longitudinal direction is not basically favorable. It attracts considerable bending moments, without appreciably reducing the forces in the pylons and the cables, and it must be dimensioned in an appropriate manner.

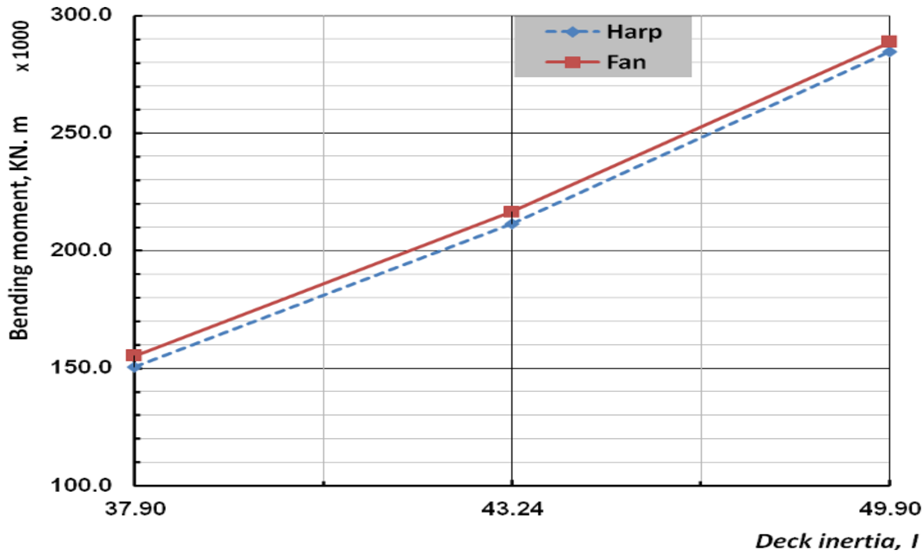


Figure 17 Bending moment response of the deck at mid span for different deck inertia

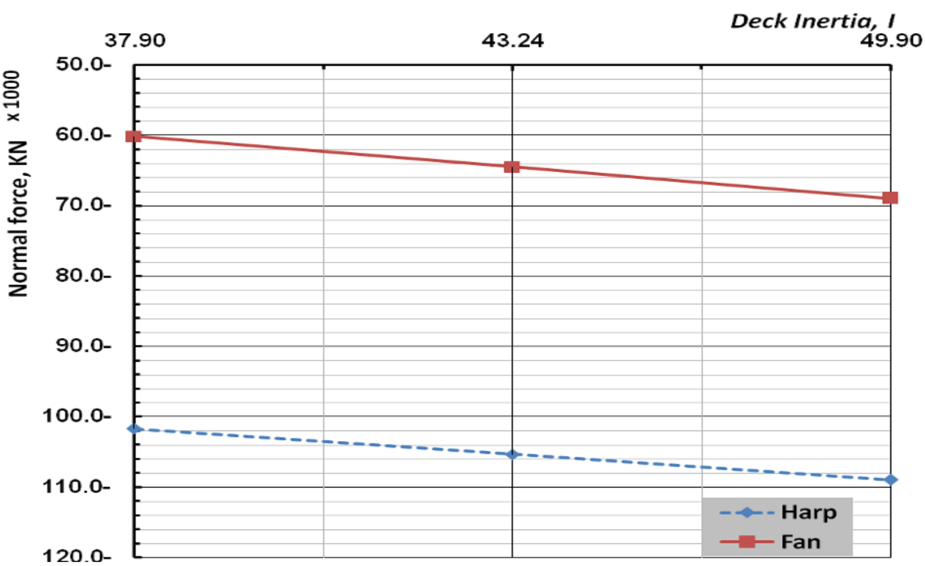


Figure 18 Normal force response of the deck at pylon for different deck inertia

The development of normal force peak response in the deck near the pylon (deck distance 150 *m* and 500 *m)* as a function of the deck rigidity is shown in **Figure 18**. It can be noticed that the significant increase of bending moment response is accompanied by slight increase in the normal force response in the deck. The fan layout cable system display higher normal force response compared to that of harp system, the normal force response increases for deck inertia of 49.9 m^4 by more than 7% and 14% of that of reference case, respectively.

5. CONCLUSIONS

Cable-stayed bridges have several advantages such as a wide range of span lengths and arrangements, the possibility of limiting and eliminating piers, slender decks, flexibility in the construction schedule, reduction of environmental impact (especially for river crossings), increased traffic safety, and enhanced appearance. One of the main advantages of these bridges is that they can be progressively erected on self-anchored cantilevers without interfering with roads or rivers underneath the structure

This paper discussed some of the important key design considerations for cable-stayed bridges under gravity loads. The mathematical model of a cable-stayed bridge is formulated for single plane of cables with a global system of coordinates for bridges having harp and fan shapes. All bridges have three spans of 150, 350, 150 m, each side have twenty and eight cables in each side of the pylon. Different parameters studied to get the influence of the principal characteristics, which are the layout of the stays, the inertia of the deck and the pylons, and the change of diameter of the cross section of cables. The recommendations of different cable-stayed bridge solutions have been highlighted, with review of deck sections, pylon sections and height to span ratio and cable arrangements and stiffness.

The investigations built on the factors that affect the behavior of the three span cable stayed bridge in this research have led to the following conclusions: The deflection of the deck is slight depends on the layout of cable system either harp or fan system. The bending moment demands in the deck using the fan cable system are higher than that in harp system. As the cable system gets stiffer, the normal force demand in the deck gets higher, which could lead to instability due to buckling. The deflection demands of the deck significantly decrease as the pylon height to span ratio; *H/L* increases, while the normal force demands decreases. A deck with a high inertia in the longitudinal direction is not basically favorable. It attracts considerable bending moments, without appreciably reducing the forces in the pylons and the cables, and it must be dimensioned in an appropriate manner. The pylon is appreciably more heavily stressed in a bridge of the fan pattern than in one of harp pattern. In the particular case of this study, the difference between normal in fan and harp is increasing when near to the top of pylon 85.75% from the value of fan system at height 55 m from the deck and reduce when near to the deck and there are a reversely effect on the change of stays on the deck. As a result, the fan layout appears to be less suitable for large-span bridges.

For three span bridges the optimum side span length is 0.40 - 0.45 of the length of the main span. The height of the tower above the deck is usually 0.2 times the length of the main span. Close spacing of cables can reduce longitudinal bending moments in the deck and the deck thickness. Regarding the force distribution and deflection, it is favorable to consider a stiff cable system, to increase the global stiffness of the bridge and to reduce the maximum bending moment in the girder. It is more profitable to increase cable system axial stiffness in order to increase the stiffness of the bridge and to reduce the bending moments in the girder. Designing and dimensioning structural bridge members under bending is always less effective and more material consuming than that of members under tensile loading, such as the cable system.

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دراسة بارمترية للسلوك الاستاتيكي اللاخطي للكباري المشدودة بكابلات

يهدف هذا البحث لدراسة السلوك الاستاتيكي اللاخطي للكباري المشدودة بكابلات (الملجمة) تحت تأثير كلا من الأحمال الميتة والأحمال الحية وبالتالي تطوير وتقديم تصميم متناسق ودراسة جدوى لنموذج كوبري فوق نهر النيل. من أجل تحقيق هذا الهدف، تم عمل دراسة شاملة لمعاملات التصميم الرئيسية والتي تلعب دورا مهما في سلوك الكوبري وذلك لتقديم وصياغة مجموعة من التوصيات لتصميم نموذج يمكن تنفيذه. حيث تم عمل تصميم مرجعي لكوبري مشدود بكابلات ثم عمل تمثيل رياضي لإجراء تحليل استاتيكي لاخطي باستخدام نظرية العناصر المحددة. تم دراسة تأثير تغيير قيم معاملات تصميم رئيسية علي سلوك الكوبري للوصول إلي أفضل تصميم. ومن المعاملات التي تم دراستها: قطاع الكابلات، وشكل/ترتيب الكابلات إما مروحة أو نمط القيثارة، نسبة ارتفاع البرج بحر/اتساع الكوبري، والاتصالات بين البرج ظهر الكوبري والخواص الميكانيكية لقطاع ظهر الكوبري والبرج الحامل. تم عرض ومناقشة نتائج تأثير العوامل السابقة علي على قيم الشد النهائي في الكابلات وكذلك القوى المحورية وقوى العزوم المختلفة وقيم الترخيم والإزاحات المختلفة في العناصر الإنشائية المكونة لهذه الكباري وأخيرا، تم صياغة بعض الاستنتاجات المتعلقة بتحليل وتصميم الكباري المشدودة بكابلات.