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THE FEASIBILITY OF LONGER SPANS IN CABLE-STAYED BRIDGES

إمكانية الحصول على بحور أطول للكباري الملجمة

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خلاصة :

إن زيادة البحور القصوى للكباري الملحمة التقليدية يعتبر دائماً من المواضيع المهمة للباحثين. للتغلب على تلك المشكلة فانه لابد من الأخذ في الاعتبار نظام استاتيكي مختلف. في هذا البحث تمت دراسة التحليل الإنشائي لنظام استاتيكى معدل و مقارنته بالنظام التقليدي. تم استبدال الأبراج الرأسية التقليدية في الكباري الملحمة بنظام استاتيكى آخر يعتمد على أبراج ذات أرجل مفتوحة متصلة بشدادات من أعلى. و تمت دراسة نوعين من تلك الكباري. و قد تم اخذ تأثير عدد الكبلات في الاعتبار (١٢، ٢٤، ٣٦). و أيضاً تمت مقارنة النتائج مع نتائج النظام الاستاتيكى التقليدي للكباري الملحمة. وبناء على النتائج المستخلصة فانه يمكن زيادة البحور القصوى للكباري الملحمة باستخدام النظام الاستاتيكى المعدل حيث إن كل الاستجابات الناتجة اقل بكثير من مثيلاتها في النظام التقليدي.

ABSTRACT

Increasing the maximum span length for cable-stayed bridges has been always a challenging target. To overcome restrictions on maximum span length, associated with the classical systems, a modified statical system should be considered. In this paper, a comparative analysis for a modified statical system is presented. The classical vertical pylons are replaced by pairs of inclined pylon legs (V-shaped) spreading out longitudinally and connected at the top by horizontal ties. Two different V-shaped systems are investigated. The effects of different cables configurations and numbers are considered (12, 24 &36). The results are compared with a classical fan-type cable-stayed bridge system. The numerical results indicate the feasibility of longer spans, since the modified statical system gives much less responses than the classical system.

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INTRODUCTION

A considerable amount of literature in the past few years has been devoted to cable-stayed bridge analysis and design, [1-11]. The different types of cable supported bridges can be characterized by the configuration of the cable system. The suspension system contains a parabolic main cable and vertical hanger cables connecting the stiffening girder to the main cable. The Cable-stayed system contains straight inclined cables connecting the stiffening girder to the tower (pylon). In recent years the span of cable supported bridges has reached 2 Km (Akashi – kaikyo, Japan 1998). For complete and comprehensive review of cable stayed bridges the reader is referred to Gimsing [1].

The behavior of a bridge girder supported by cables and roller supports at the towers simulates the behavior of a continuous beam on elastic supports. Due to the cable inclination, the girder and towers are subjected to significant axial forces. These axial forces differ according to the number of cables and their connection points with the girder and the tower, which result in different horizontal cable reactions along the bridge girder. The analytical analysis is based on a continuous model of the bridge that agrees with the physical behavior of the long-span. This is similar to that of a large truss structure where the main state of stress is given by axial forces in the stays and in the girder, while the deck bending is of secondary importance.

Two critical factors often determine the maximum span of classical cable-stayed bridges; (1) the magnitude of compressive stresses in the deck girder due to the horizontal components of the stay loads; and (2) the girder deflection at midspan. To overcome the span-length restriction, Gimsing [2] has proposed anchoring a part of the backstay cables (cables at the beginning and end of the bridge) in earth abutments (anchor blocks) to reduce the compressive forces in the deck. On the other hand, the maximum span length of a classical earth-anchored cable-stayed bridge is limited by the cable capacity to carry loads.

To increase the maximum span and reduce the main span deflection, Starossek [3] has proposed a modified statical system. The basic idea of this system is to replace the vertical pylon (tower) with pairs of inclined pylon legs (V-shaped) that spread out longitudinally from the girder level. Horizontal ties are used to balance the horizontal force in each pair of pylon legs. The study of Starosşek [3] was mainly concerned with the economical point of view (reduction of steel amount) and not with the overall structural behavior.

In this paper, the spread leg pylon system for cable stayed bridges considering their overall behavior have been investigated. Two types have been studied; (1) the fan type with spread leg pylon system (V-fan system), and (2) the harp type with spread leg pylon system (V-harp system). The results have been compared with the classical fan type system with vertical pylon. Different cable arrangements were used in the study with the number of cables ,n , equal to, 12, 24, and 36 respectively.

STRUCTURAL MODELS

The three-span bridges shown in Figure 1 (a, b & c) are geometrically symmetric. For each one the overall dimensions are as follows: the total length Lt=365m, the main span Lm=203 m, and the two equal side spans, Ls=81 m each. The pylon height, H, is 45 m. The points of attachment of the stay cables radiating from each pylon to the main span are equidistant (i.e., distance between cables = Ls /(n/4)), leaving a central unsupported length Lc=41 m. The inclination angle for the spread leg pylon $\alpha = 60^{\circ}$. Different cables configurations were considered with 12, 24, and 36 cables, respectively. It should be noted that the total cross-sectional area of the cables remains constant for all cases (0.32 m²), regardless of the number of cables. The deck girder for each bridge is modeled as a continuous beam with a hinged support at the left end while the other supports are of the roller type. Moreover, all towers Vertical and V-Shaped, have the same cross sectional area (0.81 m²). The material properties used in this paper are shown in Table 1:

Table 1: Material Properties

Material Properties	Area (m^2)	E (pa)	γ (KN/m^3)	υ
Deck (Plate Girder)	1.64	$21X10^{10}$	78.33	0.3
Towers	0.81	$21X10^{10}$	78.33	0.3
Cables	0.32	$21X10^{10}$	78.33	----

Where; E = Modulus of elasticity, γ = Material unit density, and υ = Poisson's ratio.

For long-span cable-stayed bridge, the dead loads always contribute the most to the total bridge loads. The live load is assumed uniformly distributed with load intensity of 40 KN/m on the main span only. This load is calculated according to the recommended loads for bridges, Committee on Loads and Forces on Bridges 1981 [4] and the British Department of Transport (BD37/88) Standard [5]. Figure 1(d), shows the live load configuration considered.

The cable-stayed bridges have been treated as 2D finite element models. The bridge girders and towers were modeled as 2D frame elements, while the cables were modeled as 2D truss elements. The formulation was made within the following assumptions:

1. The structure is assumed to remain elastic (elastic behavior).
2. The initial state for linear analysis model is the equilibrium configuration of the bridge under the effect of dead loads and prestress forces.
3. The effect of catenary action due to the self-weight of cables is neglected.

Regarding assumption 3, Podolny [6] has shown that the effect of catenary action for moderate sag to span ratio is not large. However, in this study the modulus of elasticity is modified using Ernst formula [7] to incorporate for this effect. Prestress forces are shown in Table 2, where cables numbers are starting from the bridge left side.

Table 2: Prestress Forces in Stay Cables

Cable Number	Initial Tension (MN)	Cable Number	Initial Tension (MN)	Cable Number	Initial Tension (MN)
12 Cables Configurations		24 Cables Configurations		36 Cables Configurations	
1,6,7,12	6.8	1,12,13,2	3.5	1,18,19,3	2.4
2,5,8,11	6.0	2,11,14,2	3.3	2,17,20,3	2.3
3,4,9,10	5.2	3,10,15,2	3.1	3,16,21,3	2.2
		4,9,16,21	2.9	4,15,22,3	2.1
		5,8,17,20	2.7	5,14,23,3	2.0
		6,7,18,19	2.5	6,13,24,3	1.9
				7,12,25,3	1.8
				8,11,26,2	1.7
				9,10,27,2	1.6

RESULTS AND DISCUSSION

A numerical analysis was carried out. All three analyses were performed for the following sequence: (1) Dead load (structure own weight) and prestress cable forces are applied at one step. (2) A live load of 40 KN/m is applied on the middle span only in the next step. Such case of loading was reported, Abou-Rayan and Seleemah [8], to be the most critical loading condition. Results for maximum central span moment, maximum bridge deck deflection, pylon tip deflection, distribution of tension in cables, cumulative compressive axial force on bridge deck, and tension in pylon ties are shown in Figures 2,3,4,5,6&7 respectively. It should be noted that, all the shown results are due to dead load + prestress force + live load.

The maximum deck moments at the middle of central span with the effect of different cables configurations are shown in Figure 2. It can be seen that the fan system gives the highest moment values for all cables configurations. Where, for the 12 cables configuration the V-harp system gives less moment values than the V-fan system. For the 24 cables configurations both V systems give almost the same moment values. The trend for the 36 cables configurations is opposite to that of the12 cables configurations for the V-shaped pylon systems. The V-fan system gives the least moment values. From Figure 2, it can be observed that when the number of cables is reduced while the total area of cables is kept constant, the maximum central span deck moments are decreased. This can be explained by the fact that for bridge girder with small number of cables (12) the girder can be considered as a continuous beam on rigid supports. For the case of large numbers of cables (24&36), the bridge girder can be considered as a continuous beam on elastic supports, as stated by Agrawal [9].

Figure 3 shows the maximum deflection at the middle of the central bridge span for all cables configurations and numbers (12, 24, &36). The deflection at the center of the main span may be considered as one of the most important criterion on which the bridge superstructure design depends. The maximum deflections of the central span for the 12 cables configurations are 25.1, 10 , and 24.8 cm for the fan, V-fan, and V-harp systems, respectively. This represents a difference of 250% between the V-fan system and the other systems. For the 36 cables configurations the maximum deflection of the fan system is twice that of the V-fan system. The V-harp system gives the highest deflection value. Also, it is seen that when the number of cables increases while the total area of cables remains constant, the maximum deflection values increase. When considering the deflection criterion from the design point of view, the V-fan system is superior to the other systems.

Figure 4 represents the maximum deflection at the tip of the pylon. The bold line represents the deflection at the tip of the vertical pylon for the fan system, lines with □ marks, and o marks represent the defection at the tip of the spread leg pylon for the V-harp and V-fan systems, respectively (continuous lines represent right leg deflection and dashed lines represent left leg deflection). For all cables configurations it is clear that the V-fan system gives the least pylon tip deflection, which in turn gives the least central span deflection, see Figure 3. The V-harp system gives the maximum tip deflection. Again, for all systems, the tip deflection increases as the number of cables increases. As can be seen due to the case of loading, the tip deflection for the pylon inner leg is larger than that for the pylon outer leg.

Figure 5 shows the tension in the ties for both V-shaped systems for different cables configurations. It is obvious that the tie in the V-fan system develops larger tensile force than that for the V-harp system. In V-fan system all cables are lumped in one point developing high tensile force in the tie, while they are distributed along the pylon legs for V-harp system. Also, it is seen from Figure 5 for the V-harp system, that the tensile forces in the ties are inversely proportional to the number of cables. On the other hand, for the V-fan system, changing the number of cables has a negligible effect on the tensile force in the ties.

The variations of cables tension forces with different cables configurations for all systems are shown in Figure 6. For all systems the 36 cables configuration develops less tensile forces than the 12&24 cables configurations. Since the total cables area remains constant, increasing the number of cables decreases the axial stiffness of each cable. This in turn develops smaller cables tensile forces. It can be seen that, for all systems, the cables tension decreases rapidly with the increase of number of cables. Also, the tension in cables is directly proportional to the distance from the tower. It should be noted that, for the V-harp system the cable away from the pylon (next to the middle central span or the end support) develops very high tensile force. This tensile force decreases rapidly in the next cables towards the pylon. Finally, the force approaches a zero value at the cables very near to the pylon.

Although the maximum cables tension decreases rapidly with the increase of number of cables, yet the total cumulative horizontal cable reactions acting on the bridge deck is increased. Figure (7) represents the cumulative axial force created by the cable reactions acting on the deck for all systems and for different cables configurations. Increasing the number of cables will cause different cable reactions and this varies the axial forces acting at different locations along the bridge deck. As can be seen, the maximum cumulative axial force occurring in the side spans is frequently higher than that occurring in the main span. Also, the maximum axial force always occurs when 12 cables support the bridge deck. From Figure 7, it is clear that the least compressive axial forces in the deck occur for the V-fan system for all numbers of cables (12, 24 & 36).

Finally From Figures 6 and 7, for the V-harp system, it is obvious that cables near the pylon develop zero axial forces. The corresponding accumulative axial forces along the deck are constant contrary to the general trend for other cases. Therefor, these cables can be considered as inactive members.

CONCLUSIONS AND RECOMMENDATIONS

A comparative analysis between the spread leg pylon system for cable-stayed bridge systems and the classical systems has been presented. In general, deck moments and deck deflections increase as the number of cables increases, even though the total cross sectional area of the cables remains constant. This trend is reversed for the deck cumulative axial forces. The tension in cables decreases rapidly with the increase of number of cables. Based on the previous discussions, the following conclusions concerning the spread leg pylon statical system can be drawn:

1- The compressive stresses in the deck are reduced and are more equally distributed, thus a larger maximum span is possible.
2- The horizontal cable force component introduced into the deck is smaller.
3- Deck moments, deck deflections, and tensions in cables are reduced.

Accordingly, the spread leg system has advantages over the classical cable-stayed bridge system. Also, the V-fan system is superior to either the classical or the V-harp systems. It is obvious that the spread leg pylon system makes it feasible for larger maximum span since it gives the least response values. Moreover, to minimize pylon moments, and to enhance overall stiffness, both the stiffness of the horizontal ties and the stiffness of backstay cables must be increased.

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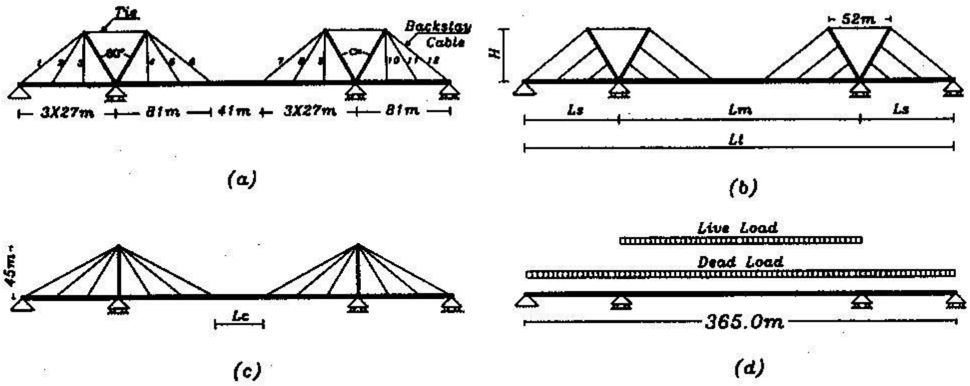

Fig. 1: Bridges Geometry and Loading

(a) V-fan System, (b) V-harp System, (c) Fan System & (d) Loading Pattern

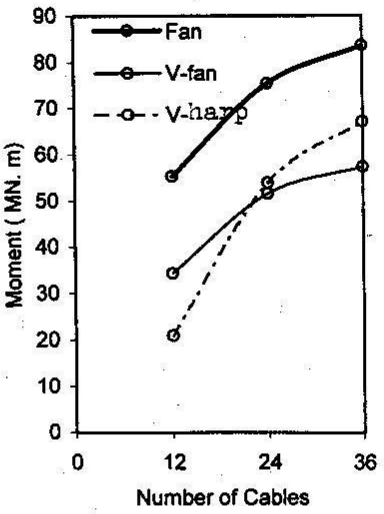

Fig.2: Maximum Deck Moments at Middle of Center Span

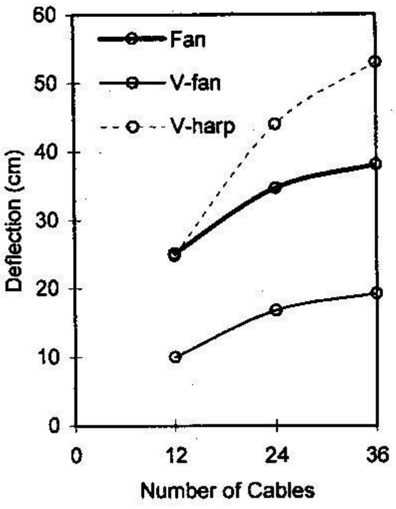

Fig.3: Maximum Deck Deflection

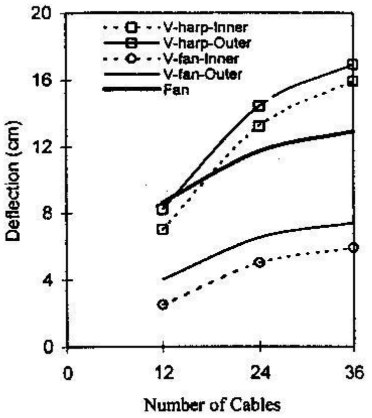

Fig.4: Pylon Maximum Tip Deflection

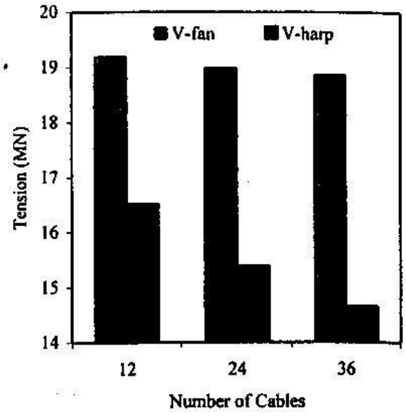

Fig.5: Tension in the Ties for V-Shaped systems

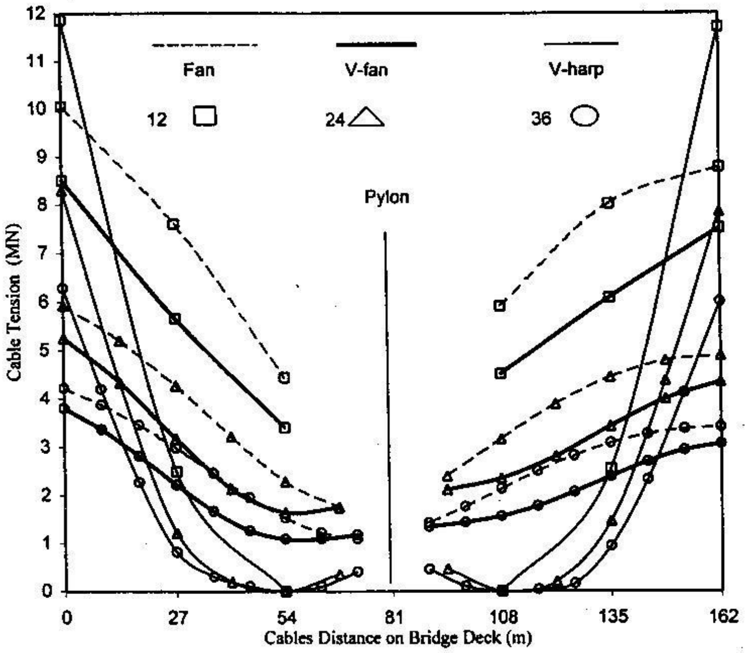

Fig.6: Distribution of Tension in Cables

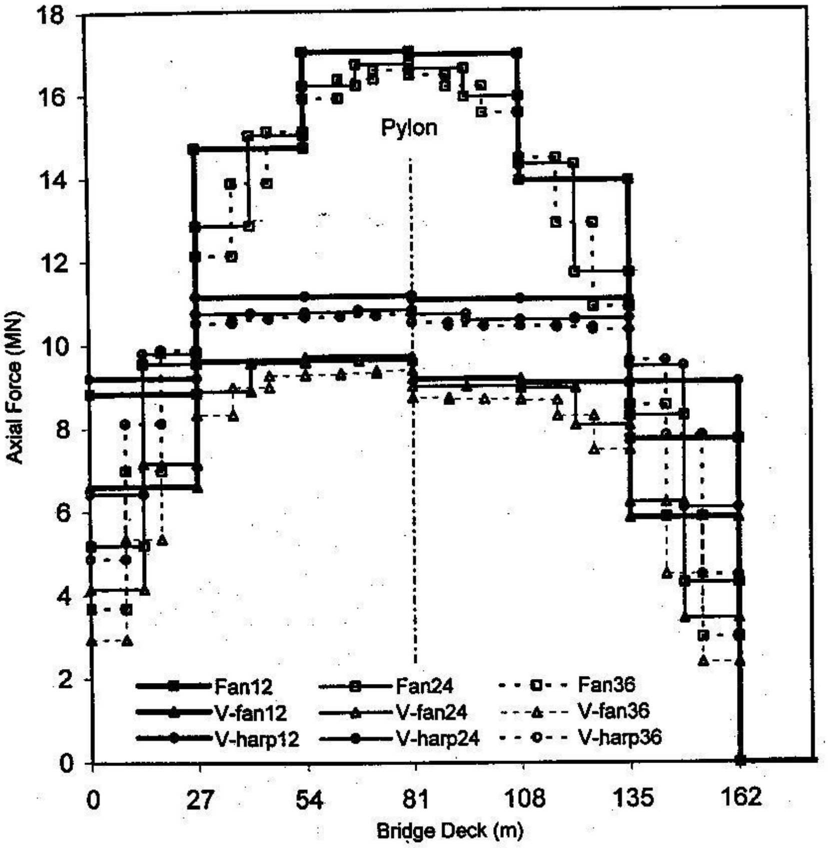

Fig.7: Cumulative Axial Force on Bridge Deck