DYNAMIC NON-LINEAR BEHAVIOUR OF CABLE STAYED BRIDGES UNDER SEISMIC LOADINGS

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ABSTRACT

The cable stayed bridges represent key points in transport networks and their seismic behaviour need to be fully understood. This type of bridge, however is light and flexiable and has a low level of inherent damping. Consequenly, thery are susceptible to ambient excitation from seismic loads. Since the geometric and dynamic properities of the bridges as well as the characteristics of the excitations are complex, it is necessary to fully understand the mechanism of the interaction among the structural componenets with reasonable bridge shapes. This paper discuss the dynamic response of a cable stayed bridge under seismic loadings. All possible sources of nonlinearity, such cable sag, axial-force-bending moment interaction in bridge towers and girders and change of geometery of the whole bridge due to large displacement are based on the utilization of the tangent stiffness matrix of the bridge at the dead-load deformed state which is obtained from the geometry of the bridge under gravity load conditions ,iterative procedure is utilized to capture the non-linear seismic response and different step by step integration schemes are used for the integration of motion equations. In this study, three spans cable-stayed bridge with different cable systems has been analyzed by three dimensional nonlinearity finite element method. The three dimensional bridge model is prepared on SAP 2000 ver.14 software and time history analyses were performed to assess the conditions of the bridge structure under a postulated design earthquake of 0.5g. The results are demonstrated to fully understand the mechanism of the deck-stay interaction with the appropriate shapes of a cable stayed bridges.

Keywords: Deck, Pylon, Stay System, Dynamic Analysis, Nonlinear Analysis, Finite Element Analysis, SAP 2000, Time History, Frequency, Acceleration, Earthquake

1. Introduction

Due to their aesthetic appearance, efficient utilization of structural materials and other notable advantages, cable-stayed bridges have gained much popularity in recent decades. Bridges of this type are now entering a new era with main span lengths reaching 1000 m. This fact is due, on one hand to the relatively small size of the substructures required and on the other hand to the development of efficient construction techniques and to the rapid progress in the analysis and design of this type of bridges.

The recent developments in design technology, material qualities, and efficient construction techniques in bridge engineering enable the construction of not only longer but also lighter and more slender bridges. Thus nowadays, very long span slender cable stayed bridges are being built, and the ambition is to further increase the span length and use shallower and more slender girders for future bridges. To achieve this, accurate procedures need to be developed that can lead to a thorough understanding and a realistic prediction of the structural response due to earthquake loading.

Rapid progress has been made over the past twenty years in the design techniques for cable-stayed bridges; this progress is largely due to the use of electronic computers, the development of composite sections of decks, and manufacturing of high strength wires that can be used for cable. The behavior of cable-stayed bridges has been very interested by researchers due to their efficient use of materials and due to their pleasant aesthetics. Some of the researchers analyzed the behavior of cable-stayed bridges by using finite element method. This type of structures requires non-linear analysis, not only for dynamic actions but also for static loads. Modern cable-stayed bridges exhibit geometrically nonlinear behavior, they are very flexible and undergo large displacements before attaining their equilibrium configuration. Cable-stayed bridges consist of cables, pylons and girders (bridge decks) and are usually modeled using beam and bar elements for the analysis of the global structural response. To consider the nonlinear behavior of the cables, each cable is usually replaced by one bar element with equivalent cable stiffness. This approach is referred to as the equivalent modulus approach and has been used by several investigators, see e.g. [13, 9, 8]. It has been shown in [1]. that the equivalent modulus approach results in softer cable response as it accounts for the sag effect. Whereas, long span cable-stayed bridges built today or proposed for future bridges are very flexible, they undergo large displacements, and should therefore be analyzed taking into account all sources of geometric nonlinearity. Although several investigators studied the behavior of cable-stayed bridges, very few tackled the problem of using cable elements for modeling the cables. See ref. [10, 11] where different cable modeling techniques are discussed and references to literature dealing with the analysis and the behavior of cable structures are given.

In fact, the cable itself has a non-linear behavior, as its axial stiffness is a function of the sag and of the tension [1]. This structural synthesis provides a valuable environment for the nonlinear behavior due to material nonlinearity and geometrical nonlinearities of the relatively large deflections of the structures on the stresses and the forces [2,5,6,7,12,14]

Bridges are critical lifeline facilities which should remain functional without damage after an earthquake to facilitate the rescue and relief operations. This, in addition to the increase in the span lengths of these flexible structures raises many concerns about their behavior under dynamic loads such as earthquakes. Very long span cable stayed bridges are flexible structural systems. These flexible systems of cable stayed bridge were susceptible to the dynamic effects of earthquake. A reference model is designed and used to investigate the influence of key design parameters of dynamic 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 River Nile, Egypt. Three span cable stayed bridge has been analyzed, the effects of the variety of different key design parameters: cross section of cables, cable layout either fan, semi fan and harp pattern, pylon height to mid span ratio and mechanical properties of deck and pylon on the dynamic response of the bridge elements are investigated. The loads on the cable stayed bridge are seismic load, the reference design is modeled in a finite element program to investigate and calculate the acceleration, moment and shear force and deformations. 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 optimal of the reference design. In order to estimate the importance of the lateral and torsional modes as well as their coupled modes for dynamic analysis, three dimensional nonlinear analysis may not be ignored for the longer span of cable-stayed bridges. The finite element methods present the engineer with a powerful structural analysis technology reliant on modern digital computers. The dynamic analysis calls for the use of a computer once there are more than three degrees of freedom, very productive design program have been developed which permit simulation of ground movements in three directions simultaneously. The 3D bridge model is prepared on SAP 2000 ver.14 software [4].

2. Finite Element Analysis Procedures

Although the techniques for linear and nonlinear earthquake response analysis are well-established, a brief outline of the equations for the multiple-support seismic excitation analysis is presented to assure a complete, detailed understanding of the interpretation of the results. The future trend in the design of cable-stayed bridges with longer spans makes nonlinear analysis inevitable. Nonlinearity of this type of flexible long-span bridge is mainly of geometric type due to large displacements. Sources of nonlinearity are cable sag, axial force-bending moment interaction in the bridge towers and girders and change of geometry of the whole bridge due to large displacements. Nonlinear earthquake response analysis can be conducted using step by step integration procedures in which a tangent stiffness iterative procedure is utilized.

The equations that govern the dynamic response of the bridge structure can be derived by following the well-known fact that the work of external forces is absorbed by the work of internal inertial, and, in a general sense, damping forces for any small admissible motion that satisfies compatibility and essential boundary conditions.

The equation of motion can be written, at time t + Δt, in a finite element semi-discretized form as Eq.1 [2,3,14,15]

$$[M]\{u''\}^{t+\Delta t} + [C]\{u'\}^{t+\Delta t} + [k]^{t+\Delta t}\{\Delta u\}^{t+\Delta t} = \{F_{ext}\}^{t+\Delta t} - \{F_{int}\}^{t+\Delta t} \qquad \text{Eq.1}$$

Where [M], [C], and $[K]^{t+\Delta t}$ are the system mass, damping, and tangent stiffness matrices at time (t + Δt).

Accelerations, velocities, and incremental displacements are represented by u'', u', and Δu, respectively. The external forces term $\{F_{ext}\}^{t+\Delta t}$ includes the effect of concentrated forces, body forces, and earthquake excitations. The vector of internal forces is denoted by $\{F_{int}\}$. A damping ratio of 2% was used for all modes.

These structure matrices are constructed by the addition of overlapping coefficients of corresponding element matrices which will be discussed in the following sections.

It can be noticed that the equation of motion is general and can account for different sources of nonlinearities. Both geometric and material nonlinearities affect the calculations of tangent stiffness matrix and internal forces. Different step-by-step integration schemes can be used for the integration of equations of motion. For problems with complicated nonlinearities, direct integration methods are more expedient. Many methods of direct integration are popular and the choice of one method over another is strongly problem dependent. In this analysis the ***Newmark integration scheme*** is used.

3. Selected Input Ground Motion

In the dynamic response analysis, the seismic motion by an inland direct strike type earthquake that was recorded during Elcentro Earthquake 1940, To evaluate the effects of strong ground motions on the seismic response of long span cable-stayed bridges. Elcentro earthquake records of 0.5g are used in this analysis Fig.1. In the dynamic analysis of time-definition area 0.5g, load on north-south direction was applied.

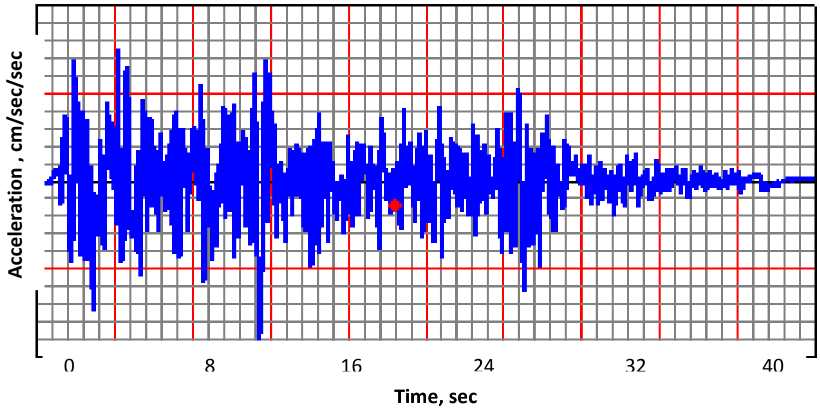


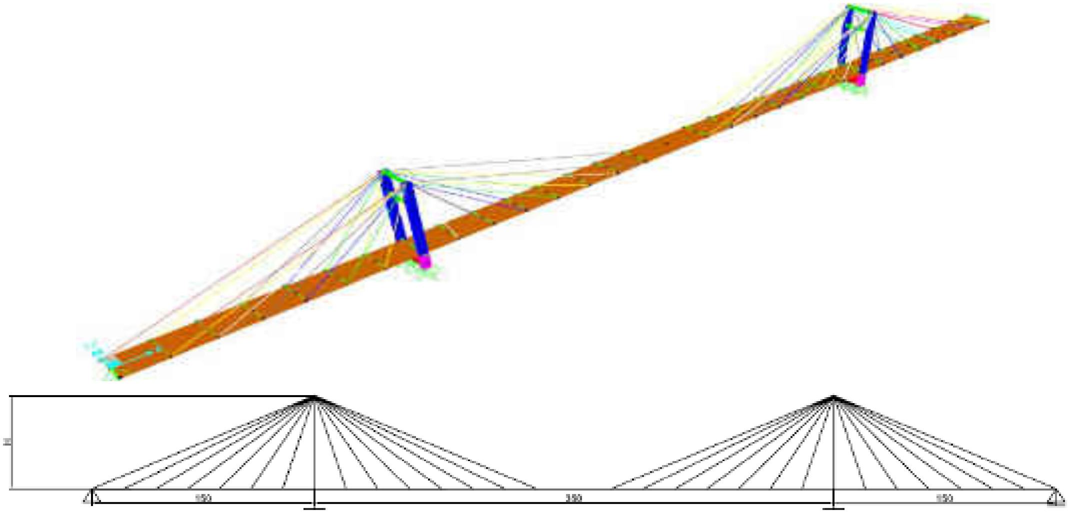
Fig. 1. The records of Elcentro Earthquake

4. Cable Stayed Bridge Model

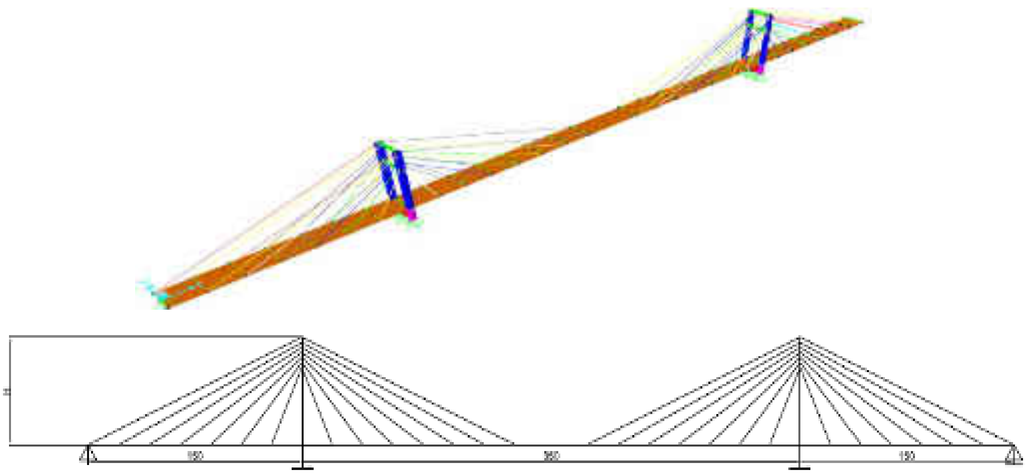
Three spans model are selected to describe the cable stayed bridge model with central span of 350 m, 150 m left/right side spans. The elevation view of the bridges are shown in Fig. 2. The precast concrete deck has a thickness of 0.23 m and a width of 18 m as illustrated in Fig 2.a. It also has two steel main girders that are located at the outer edge of the deck.

The pylons have two concrete legs as they are connected internally with struts. The lower legs of the pylon are connected by a 1.12 m thick wall. Other geometric and parameters of the bridge are given in Table 1. As one can see, the pylon has an H-shape with two concrete legs. The upper struts cross beams height are 45 m, 65m respectively and the lower strut cross beam supports the deck. The cross sections of the pylons are also given in Fig 2. The total height of the towers is 79 m (65 m over the girder, around 19 percent of the central span). Each tower is fixed to the ground and support 28 cables, 14 per side; the cables are connected to the girder with 21.42 m spacing one from each other and the properties of each structure elements are shown in Table 1.

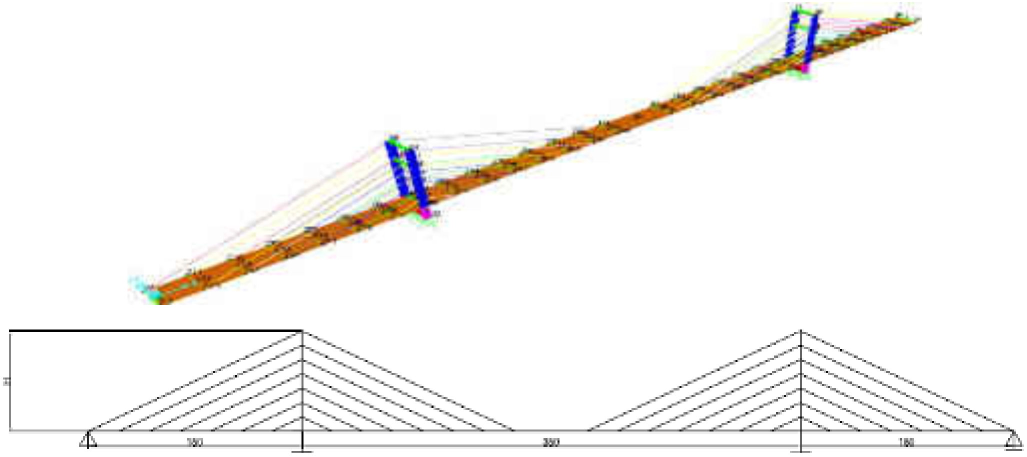
The bridge is composed of 56 stay cables. The stay cables are double arrangements. The type of cables adopted in the conceptual design is being parallel wire strands with an ultimate tensile strength of 1.600 MPa and a Young modulus of 200.000 MPa. The weight per unit volume of each cable depends on the number of wires in individual cables, the cross sections areas of the cables are various from 170.548 cm^2 to 582.692 cm^2, The detail of the cable cross section areas is listed in **Table 2.**



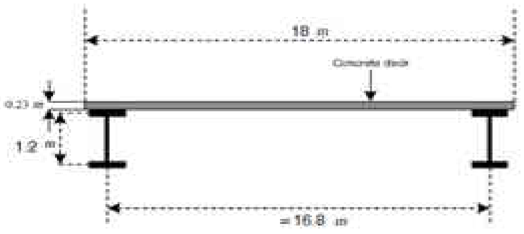
(a) Finite Element Model of Fan type



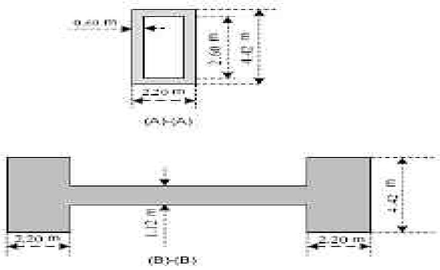
(b) Finite Element Model of Semi-Fan



(c) Finite Element Model of Harp type



(a) Cross section of the bridge deck



(b) Cross section of pylon

Fig.2. Longitudianl bridge arrangements and deck cross section

Table 1.
The material properties for cable stayed bridge

	Material Type	Unit weight; KN/m^3	Modulus of Elasticity; KN/m^2
1	The precast concrete deck	25	250.000.000
2	Pylon	25	250.000.000
3	Cable	76.973	2000.000.000

Table 2.
The cross section Area of the cable used in the cable stayed bridge

Left Tower		**Right Tower**	
Cable no	Cross section Area (cm^2)	Cable no	Cross section Area (cm^2)
1	582.692	15	442.586
2	582.692	16	426.106
3	479.615	17	374.651
4	426.106	18	333.470
5	284.070	19	172.900

Left Tower		Right Tower	
Cable no	Cross section Area (cm^2)	Cable no	Cross section Area (cm^2)
6	284.070	20	216.135
7	183.216	21	172.900
8	170.845	22	181.161
9	214.080	23	230.183
10	271.741	24	282.016
11	329.361	25	419.941
12	368.486	26	473.451
13	421.996	27	582.567
14	440.531	28	582.692

The properties of the reference design of cable stayed bridge are that: As one can see, the pylon has an H-shape with two concrete legs giving inertia of pylon cross section (section a-a as shown in Fig 2.b) $I_x = 3.705\ m^4$, $I_y = 14.366\ m^4$ and Area $A = 7.124\ m^2$. The precast concrete deck has a thickness of 0.23 m and a width of 18 m, it also has two steel main girders that are located at the outer edge of the deck and the properties of composite section of the deck $I_y = 1.0756\ m^4$, $I_z = 251.042\ m^4$ and Area $A = 5.979\ m^2$. The height of tower = 79 m and height of pylon above the deck equal to 65 m. The connection between the pylon and deck is roller, while the pylon base is fixed and other supports at the ends of bridge are roller and hinged. The cables used to study the behavior of cable stayed bridge have the properties shown as Table 2 and with modulus of elasticity $E=2.000 \times 10^9\ KN/m^2$.

4.1. Finite element modeling

The finite element model of the cable stayed Bridge has been modeled with three different types of elements, shell element, truss element and beam element. The cables are modeled as truss element with tangential modulus of elasticity. The deck and the tower are modeled as Bernoulli-Euler beam elements with axial forces. **Fig 2.** represents the over view of the three-dimensional finite element model of the cable-stayed bridge. The role of dynamic forces in cable stayed bridge is very important more than any other type of bridges; such forces can identify the very feasibility of the project. Three-dimensional nonlinear finite element model is developed for cable-stayed bridges under dynamic 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.

4.1.1 Stiffness matrix for cable

The cables of cable stayed Bridge have been modeled as truss elements with tangential modulus of elasticity. The truss element is tension-only member. The elements consist three degrees of freedom of translations in x, y and z-direction.

In the global analysis of cable stayed bridges, one common practice is to model each cable as single truss element with an equivalent modulus to allow for sag. The element stiffness matrix in local coordinates for such a cable element can be written as

$$K_c = \frac{A_c E_{eq}}{l_c} \begin{bmatrix} 1 & 0 & 0 & -1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ -1 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \end{bmatrix}$$

In term of the equivalent modulus of elasticity E_{eq} given by [1,16]

$$E_{eq} = \frac{E}{1 + \frac{AE(wL)^2}{12T^3}}$$

Where

E is the material effective modulus of elasticity

L is the horizontal projected length of the cable,

w is the weight per unit length of the cable,

A is the cross-sectional area of the cable,

T is the tension in the cable.

And l_c is the chord length

The overall behavior of cable-stayed bridges is highly complex; it depends on the interaction among different structural elements, the girder, the towers and the cables. The girder is supported by several inclined steel cables connected to towers. The cables carry only axial tension force, while the towers and the girder can resist bending as well as axial compression. The behavior of an inclined cable is non-linear since the sag of the cable due to the dead load effects the internal tension. This is a source of non-linear behavior of the whole system. Similarly, the effect of axial deformation on the bending stiffness of beam-column elements introduces additional geometric non-linearity. Other nonlinear effects on the system are introduced by material non-linearity typical both of steel and reinforced concrete elements.

In this paper; SAP 2000 Ver.14 program [4] 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 seismic load from which the effects like *displacement, acceleration, moment and shear force of the bridge are investigated.*

5. Results and Discussions

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 layout system of staying cables; height of the pylon to the middle span ration; the mechanical properties of the deck and the mechanical properties of the pylon. Layout system of cables either fan, semi fan and harp patterns are investigated, the area of cross section of cables for different cases is increased by μ factor multiple in cross section of cables in Table 2. Moreover; the effect of moment of inertia of deck variation as a ratio from 102% to 1214% to that of reference design, the moment of inertia variation of pylon from 40.87% to 332.08% to that of reference design, different values of pylon height to span ratio H/L from 0.186 to 0.24 are studied.

5.1 Effects of mechanical properties and layout system of stayed cables

The effects of variation of cross section μ from 1.0 to 1.9 where μ is the factor multiple in area cross section of cables in **Table 2** to indicate the effect of change cables cross section areas on the response of the cable stayed bridge.

The layout of cable system: Increasing of the stiffer of cables lead to slight differences in the level of acceleration on the deck at distance 64.286m (nearly half the side span) between harp and fan system and there are slight difference in the displacement at the middle point of the mid span of the deck between harp and fan system as shown in **Fig. 3. & Fig. 4**. The moment response at the midpoint of the middle span of the deck for fan layout system displays higher values than that for harp layout as shown in **Fig. 5.**

As the cable system gets stiffer, the acceleration at distance 64.286m (nearly half the side span) significantly decreases by **75.97%, 75.61%** and **65.89%** for fan, semi fan and harp systems respectively and the moment at the midpoint at the middle span decreases by about **10.4%** for all cable systems as shown in **Fig. 3 & Fig. 5**. And the shear force near the pylon decreased by **86.29%, 85.34% and 78.53%** for fan, semi fan and harp systems respectively as shown **Fig.6** but stiffer cables have small effect in reducing displacement at the midpoint of the middle span of the deck as shown in **Fig. 4**.

As the cable system gets stiffer, the acceleration at the top of the pylon decreases to value **53.92%** in the fan system, and **28.49%** in the semi fan system. **Fig. 7.** shows the values of acceleration at the top of the pylon. The variation of moment response at the fixed support of the pylon in the harp ,semi fan and fan system decreases around **47.09%, 27.51% and 11.12%** for fan, semi fan and harp layout system respectively as shown in **Fig.8**.

The influence of the stays layout is analyzed with the same properties in the previous case. While the cable layout system has slight effect on the deflection peak response at middle point in the mid span of the deck. **Fig.9** indicates the acceleration distributions along the harp, semi fan and fan system. *The acceleration in the fan system along the height of the pylon is bigger than in the harp system by* **175.78%** *at height 27.857m from the deck,* and indicates the maximum acceleration is occurring at near the mid of the pylon. **Fig.9** *indicates that the type of harp system is preferred than the fan system to reducing the acceleration along the pylon*. And **Fig.10** and **Fig.11** indicate the lateral bending moement alonge the pylon, and **Fig.12** indicates the values of lateral bending moment along the deck, From **Figs 10,11,12** it can be seen that the values of lateral bending moments along the pylon and the deck in the fan system are bigger than the harp system.

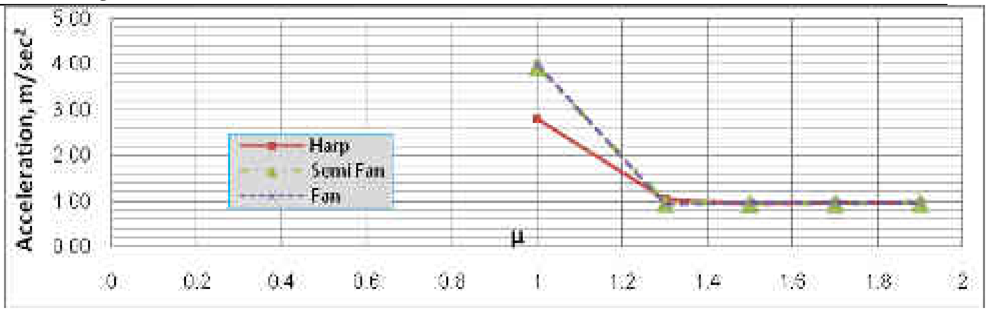


Fig. 3. The variation of acceleration (at distance 64.286m on the deck) for different μ

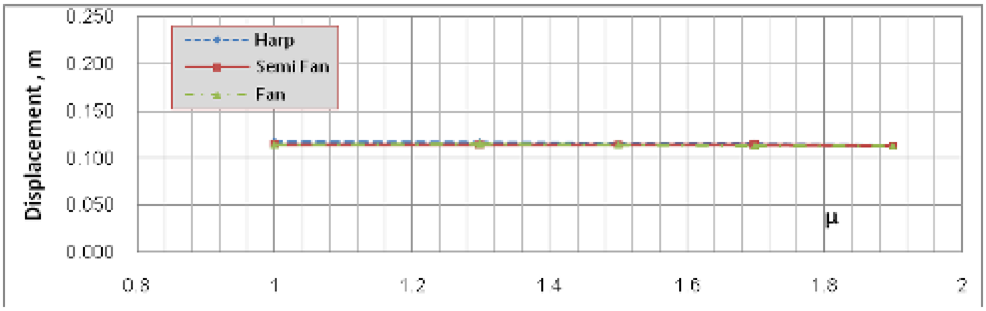


Fig. 4. The variation of the maximum displacement at the midpoint of the middle span of the deck with μ

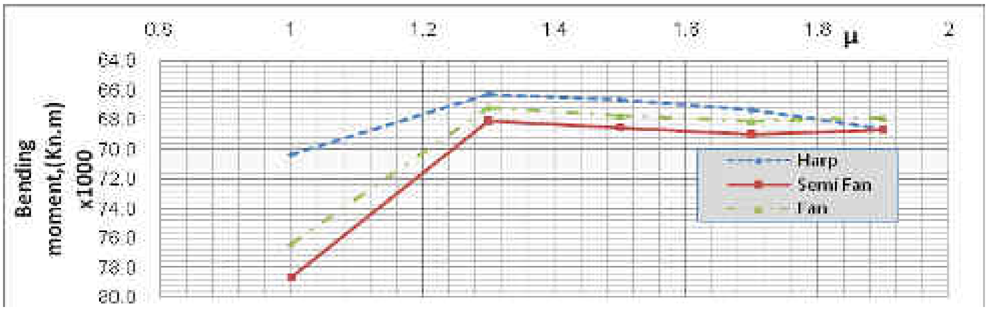


Fig. 5. The variation of the moment at the midpoint of the middle span of the deck with μ

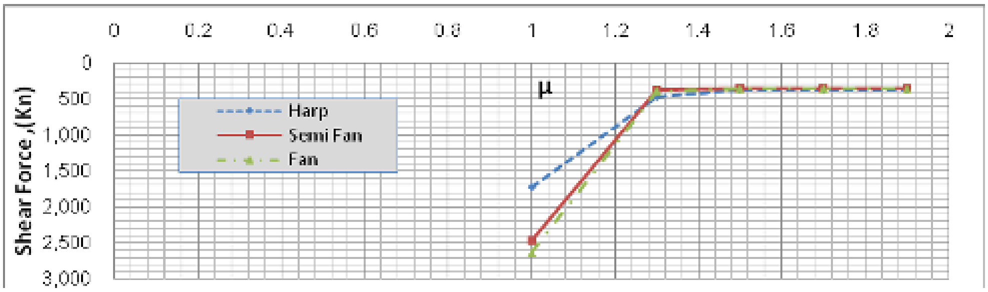


Fig. 6. The variation of the shear force on the deck (Near the pylon) with μ

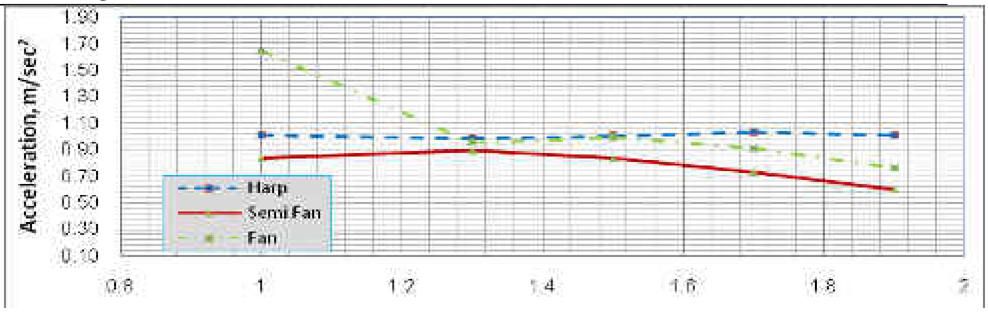


Fig. 7. The variation of acceleration at the top of pylon for different cases of μ

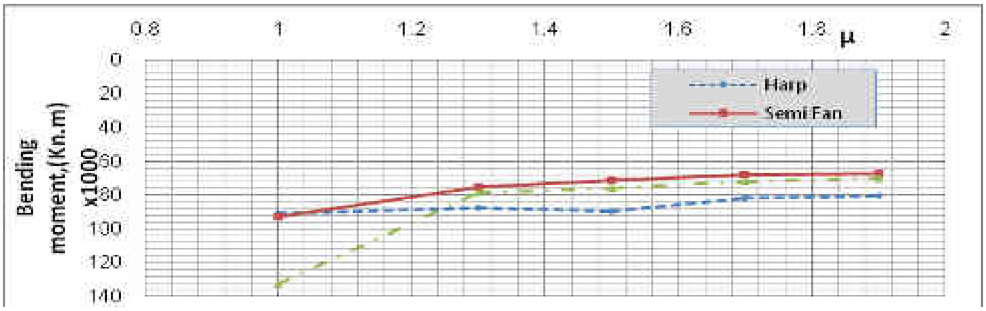


Fig. 8. The variation of moment at the fixed support of tower for different cases of μ

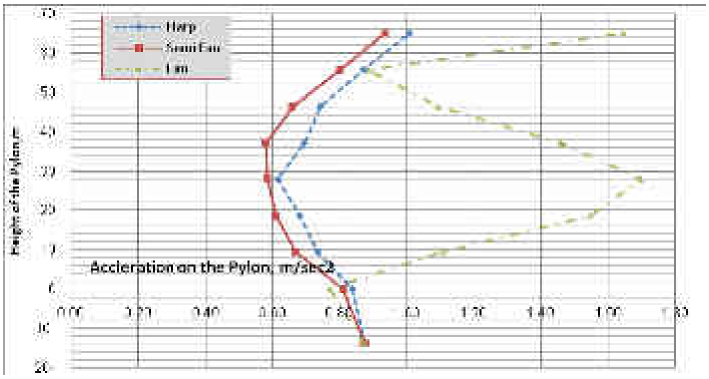


Fig.9 The values of acceleration along the height of pylon for harp, semi fan and fan pattern

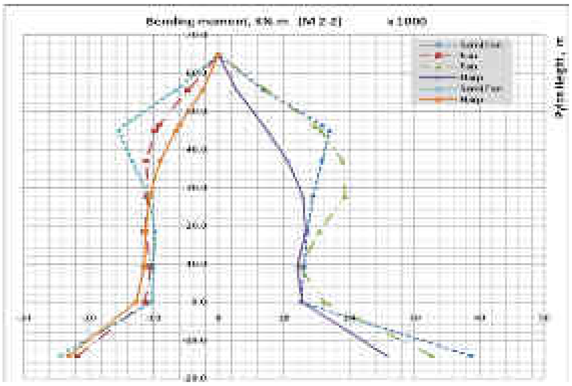


Fig.10. Tower lateral bending moment extreme values (beside hinged support)

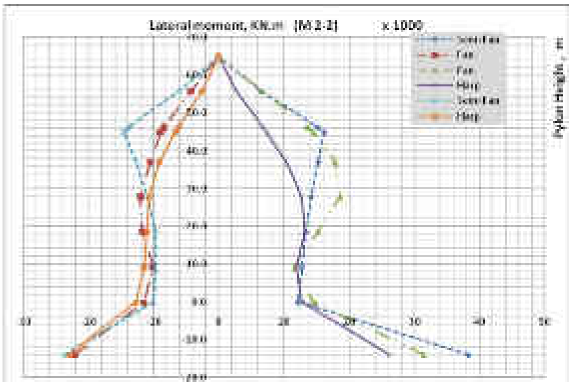


Fig.11. Tower lateral bending moment extreme values (beside roller support)

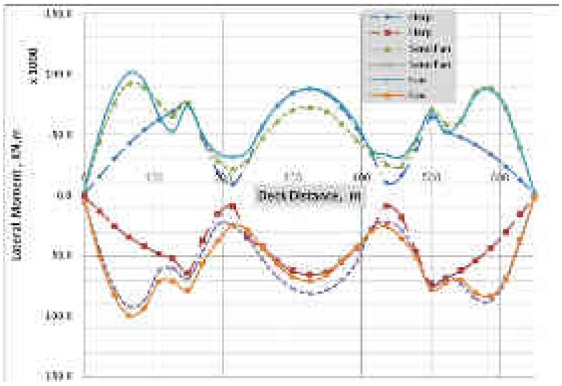


Fig.12. Deck lateral bending moment extreme values

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 0.186 to 0.24 on bridge response are studied:

Increasing of H/L could result in the bridge deck's acceleration at distance 64.286m (nearly half the side span) response decreases in the harp, semi fan and fan system by percentages **66.79%, 76.49%, 76.73%** respectively as shown **Fig.13**, and the decreasing in shear force (near the pylon) reaches around **85.85%, 84.79% and 76.87**% for fan, semi fan and harp systems, respectively as shown **Fig.16**. The slight increase in displacement and moment response at the midpoint of the middle span of the deck is obtained as shown in **Fig.14, Fig.15**.

And increasing H/L has small effect on the acceleration and displacement at the top of the pylon as shown in **Fig.17, Fig.18**. On the other hand increasing H/L lead to decrease the moment at the fixed support of the pylon around **67.27%, 46.46%** and **37.96%** for fan, semi fan and harp layout system respectively as shown in **Fig.19** and the shear force at the fixed support decreases also by **64.26%, 43.65%** and **35.44%** for fan, semi fan and harp layout system respectively as shown **Fig. 20**.

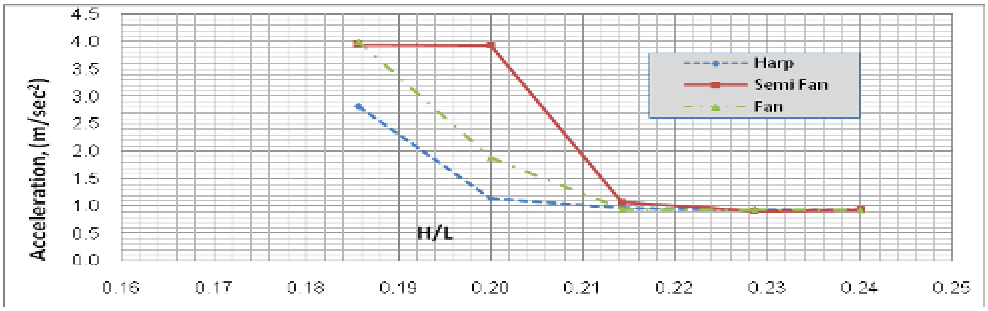


Fig. 13. The variation of the acceleration (at the distance 64.286m in the deck) with H/L

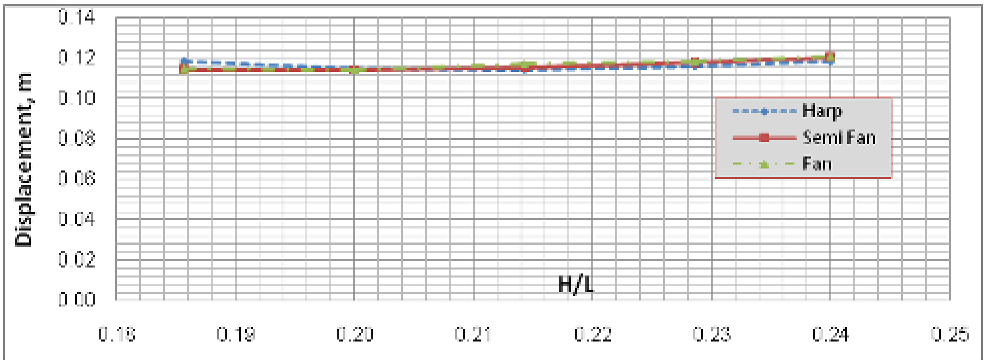


Fig. 14. The variation of the displacement at the midpoint of the middle span of the deck with H/L

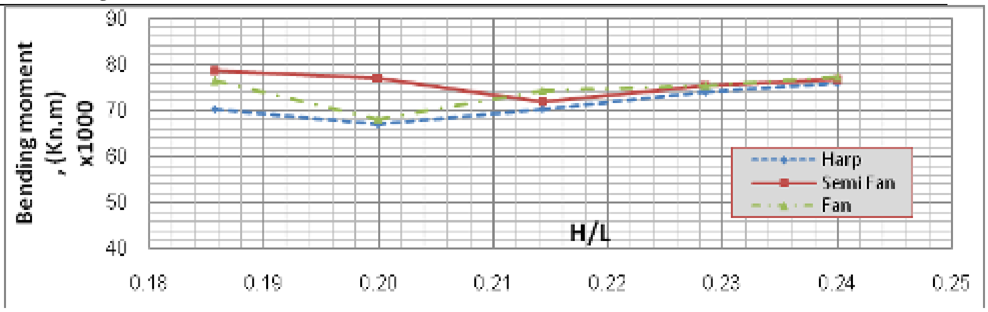


Fig. 15. The variation of the moment at the midpoint of the middle span of the deck with H/L

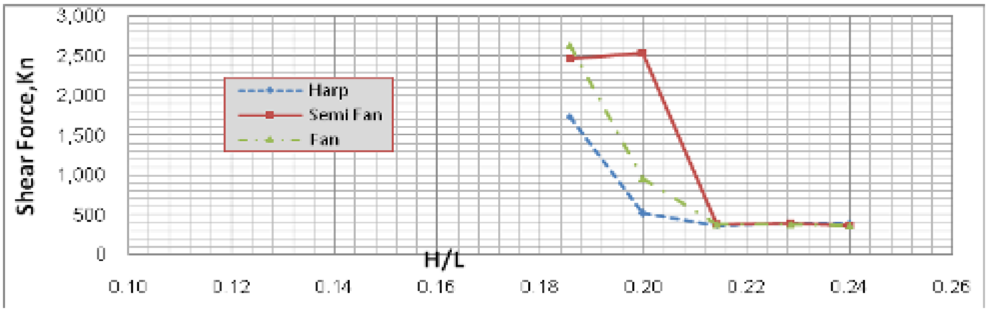


Fig. 16. The variation of the shear near the pylon with H/L

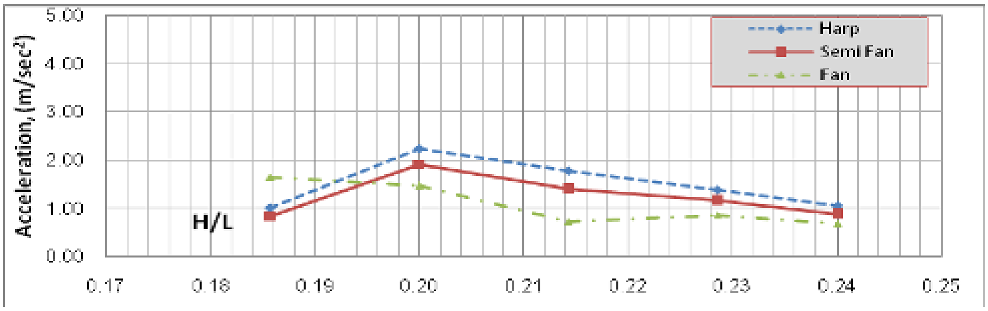


Fig. 17. The variation of acceleration at the top of pylon with change of H/L

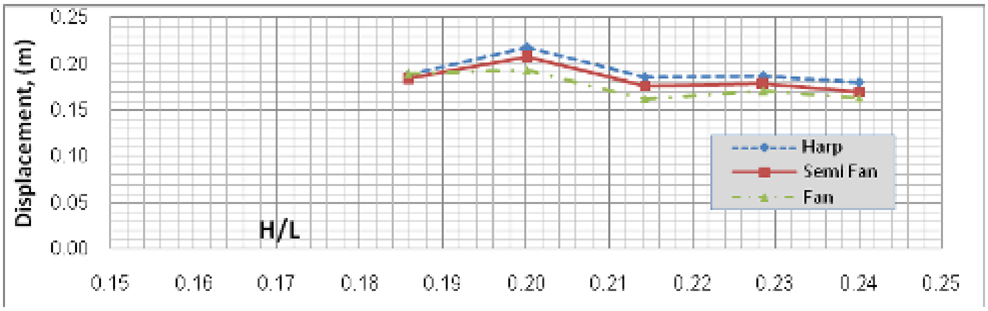


Fig. 18. The variation of the displacement at the top of pylon with change of H/L

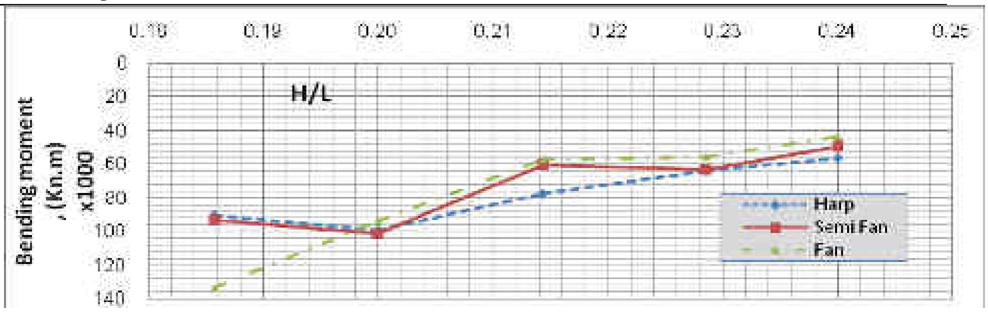


Fig. 19. The variation of the moment at the support of pylon with H/L

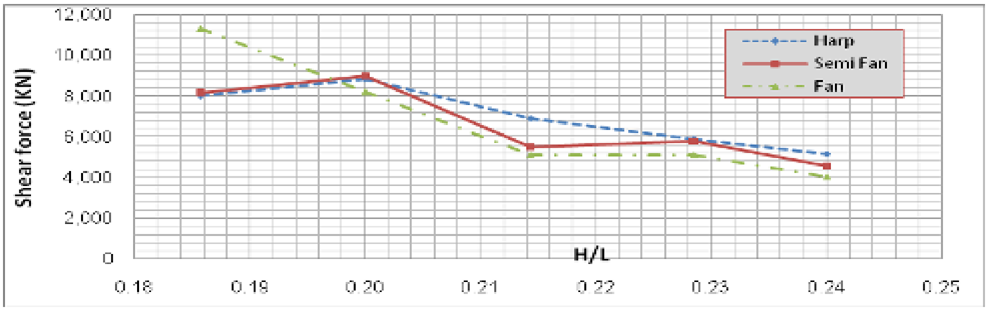


Fig. 20. The variation of the shear force at the support of pylon with H/L

4.3. Effects of mechanical properties bridge deck

The pylon inertia of 14.366 m^4; the height of pylon above the deck remains of 65 m (H/L = 0.186); cable stay cross section area as shown in Table 2 and number of cables of 7 x 8 are used as a reference values, while the deck inertia is varied from 1.076 m^4 to 14.135m^4.

Increasing the deck stiffness have a slight effect on the acceleration at distance 64.286m and displacement on the midpoint of the middle span of the deck as shown in **Fig.21& Fig.22**. On the other hand increasing the deck stiffness can lead to a big increase in a moment at midpoint of the middle span of the deck **275.19%, 274.53%, 296.10%** in fan, semi fan and the harp system respectively as shown in **Fig.23**. For the shear force (near the pylon) increasing the deck stiffness can lead to an increase by **143.77%, 217%, 326%** in fan, semi fan and the harp system respectively as shown in **Fig.24**.

And increasing the deck stiffness lead to increasing the acceleration on top of a pylon by **270.47% and 211.37%** in the semi fan system and harp system respectively as shown in **Fig.25**.but increasing the deck stiffness has small increasing on the displacement at the top of the pylon as shown **Fig.26.** On the other hand increasing the deck stiffness lead to increasing in a moment in the fixed support of the pylon by **32.40%, 173.94%** and **116.35%** for fan, semi fan and the harp system respectively as shown in **Fig.27** and shear force in the fixed support increase by **29.73%, 151.05%, 103.19%** fan, semi fan and the harp system respectively as shown in **Fig.28.**

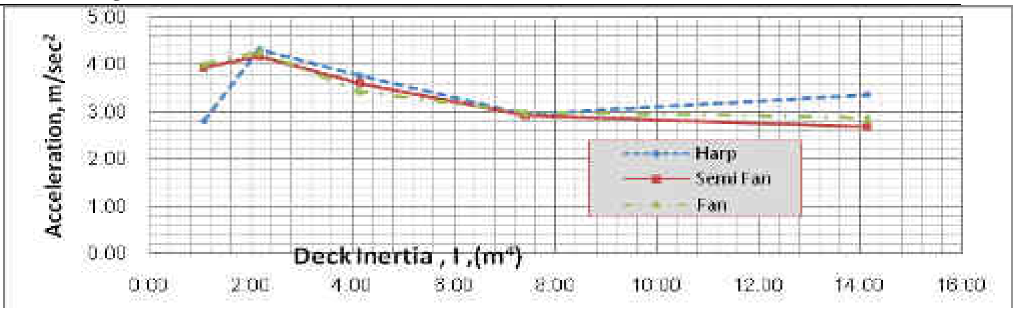


Fig. 21. The variation of the acceleration at distance 64.286m in the deck with I_{deck}

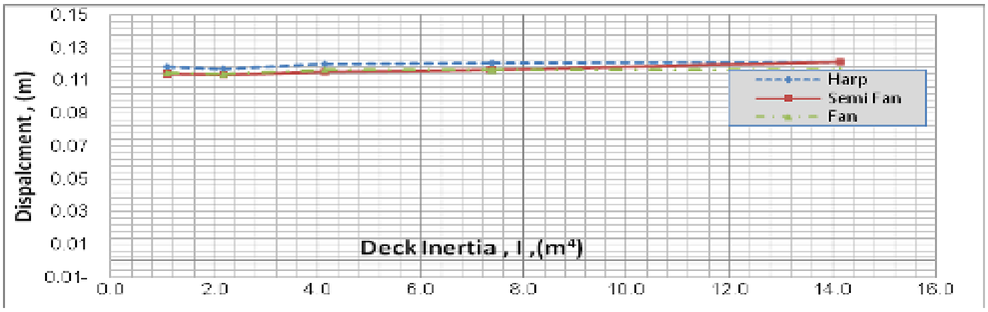


Fig. 22. Maximum displacement at the midpoint of the middle span of the deck with I_{deck}

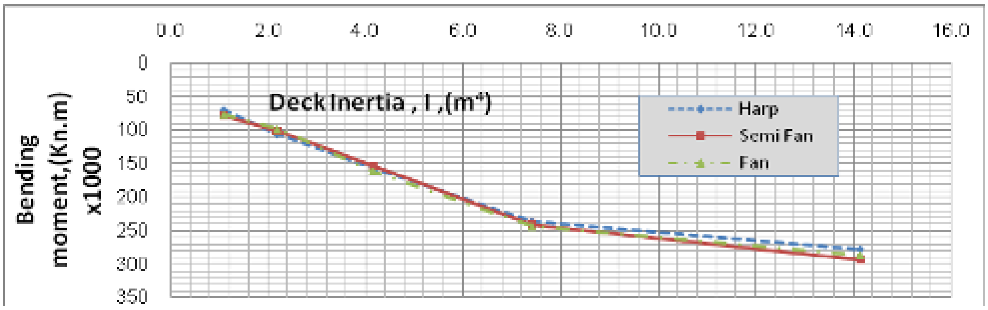


Fig. 23. The variation of moment at the midpoint of the middle span of the deck with I_{deck}

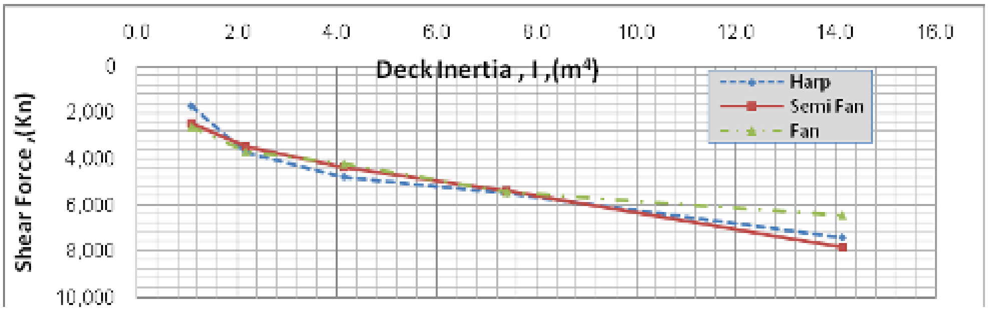


Fig.24. The variation of shear force at a distance near the pylon with I_{deck}

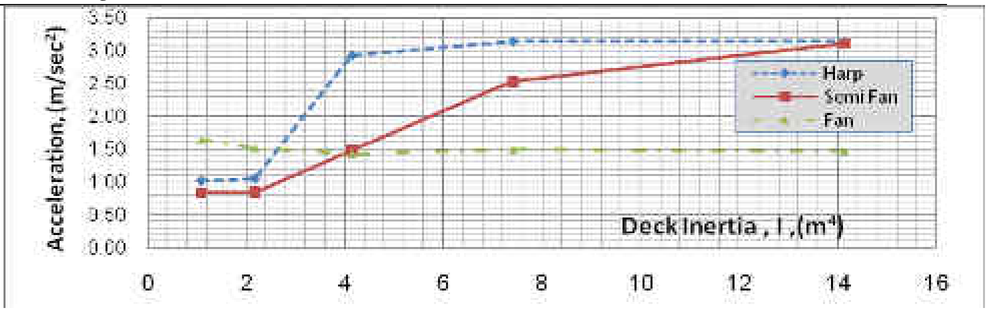


Fig. 25. The variation of acceleration at the top of the pylon with I_{deck}

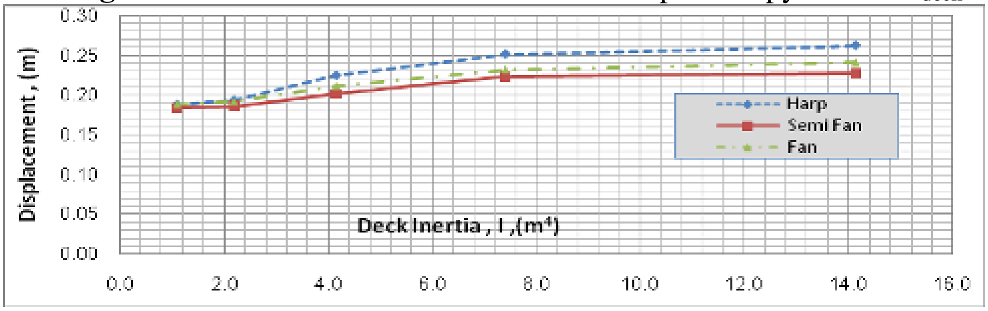


Fig. 26. Maximum displacement at the top of the pylon with I_{deck}

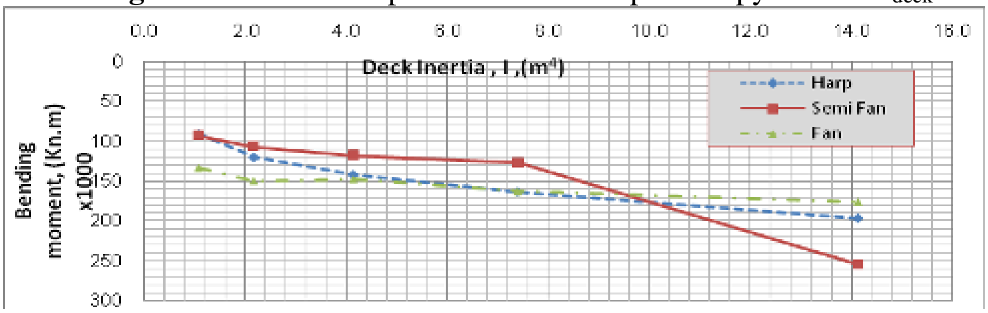


Fig. 27. The variation of moment at the fixed support of the pylon with I_{deck}

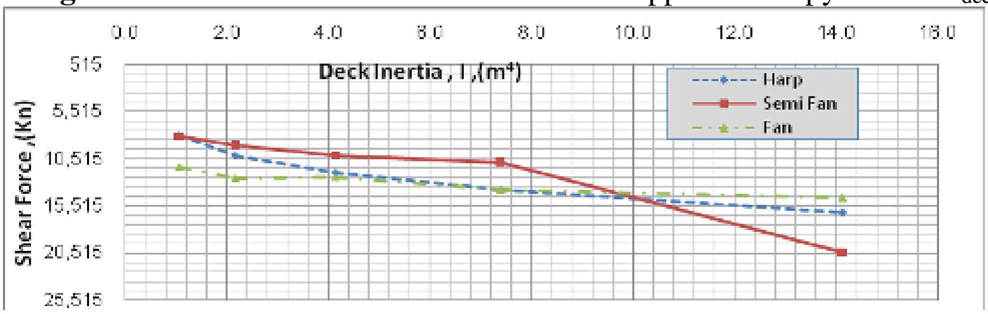


Fig. 28. The variation of shear force at the fixed support of the pylon with I_{deck}

4.4. Effects of mechanical properties bridge pylon

The deck inertia of 1.076 m^4 is used as a reference value, while the pylon inertia is varied from 14.366 m^4 to 62.073 m^4.

Increasing the pylon stiffness lead to increasing in acceleration at the distance 64.286m by **4.20%, 2.09% and 52.78%** for fan, semi fan and the harp system respectively as shown in **Fig. 29.** but increasing the pylon stiffness has slight effect on displacement at the midpoint of the middle span of the deck as shown in **Fig. 30**. On the other hand increasing the pylon stiffness can lead to increase in a moment at midpoint of the middle span of the deck by values about **11.95%** for the harp system as shown in **Fig. 31**. For the shear force (near the pylon) increasing the pylon stiffness can lead to an increase by **4.62%, 66.88%** in the fan and the harp system respectively as shown in **Fig. 32**. Increasing the pylon stiffness has increasing the acceleration at the top of the pylon by about **12.56%, 10.59% and 5.78%** for fan, semi fan and the harp system and lead to increasing moment at the fixed support on the pylon by **29.72%, 18.44% and 65.44%** for fan, semi fan and the harp system respectively as shown **Fig. 33.** & **Fig. 34**. On the other hand increasing the pylon stiffness lead to increasing in the shear force at the fixed support of the pylon by **31.06%, 21.94% and 62.48%** for fan, semi fan and harp system respectively as shown in **Fig. 35**.

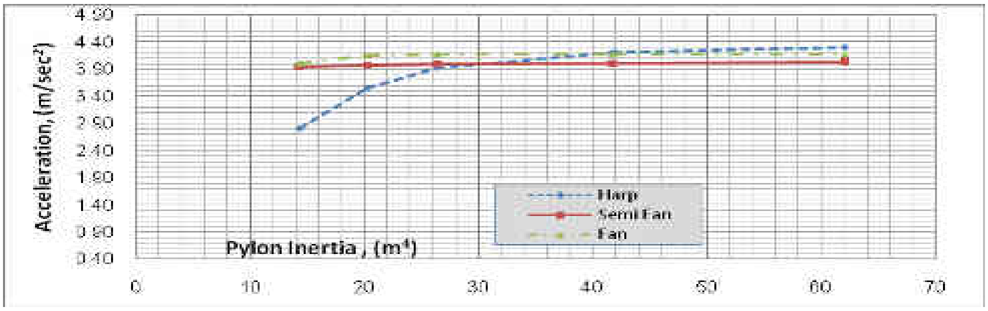


Fig. 29. The variation of acceleration at distance 64.286 of the deck with I_{pylon}

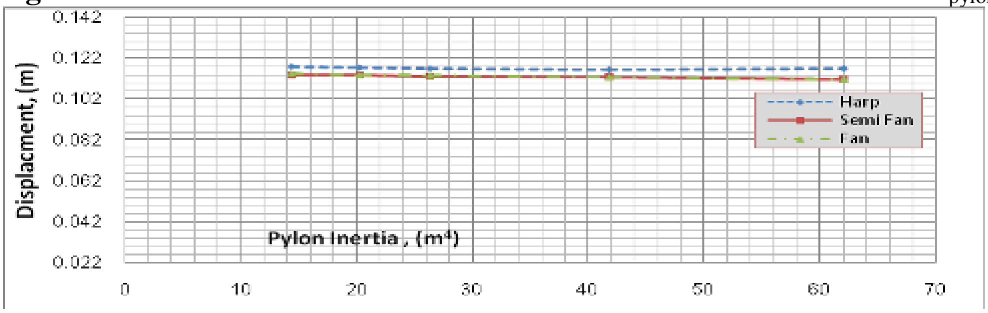


Fig. 30. Maximum displacement at the midpoint of the middle span of the deck with I_{pylon}

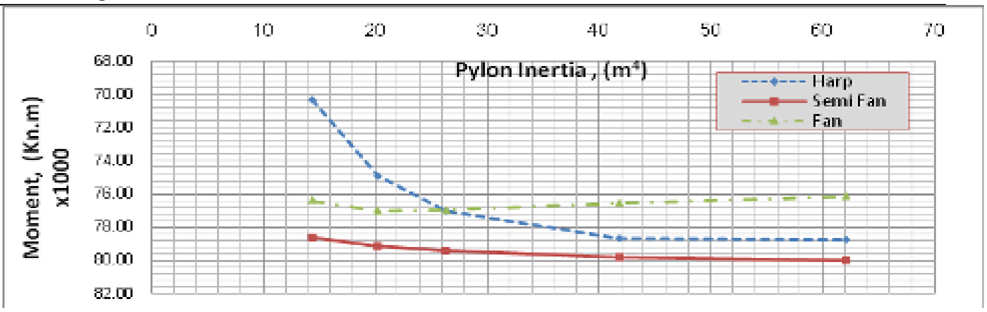


Fig. 31. Moment at the midpoint of the middle span of the deck with I_{pylon}

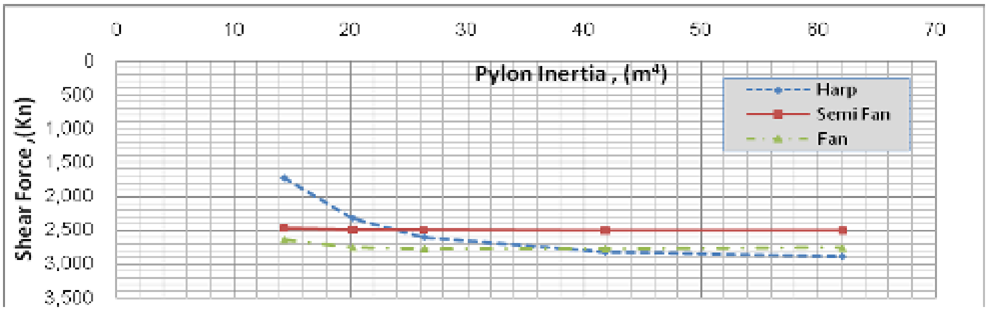


Fig. 32. The variation of shear force (Near the pylon) with I_{pylon}

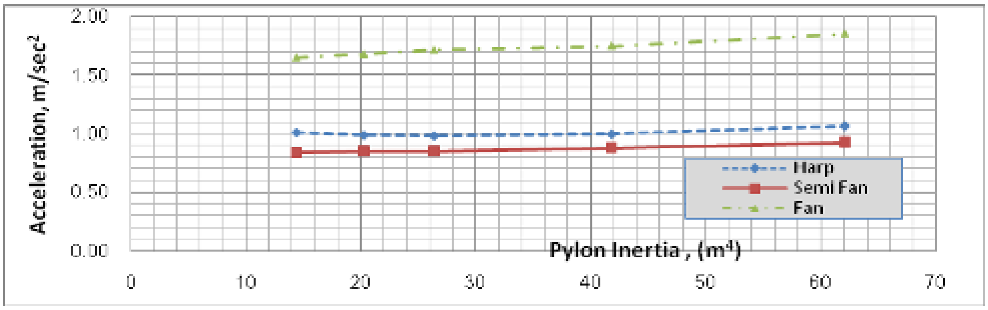


Fig. 33. The variation of acceleration at top of the pylon with I_{pylon}

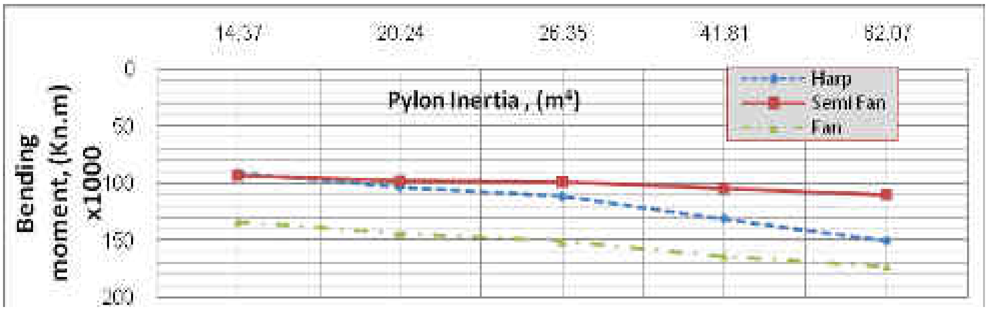


Fig. 34. The variation of moment at fixed support of the pylon with I_{pylon}

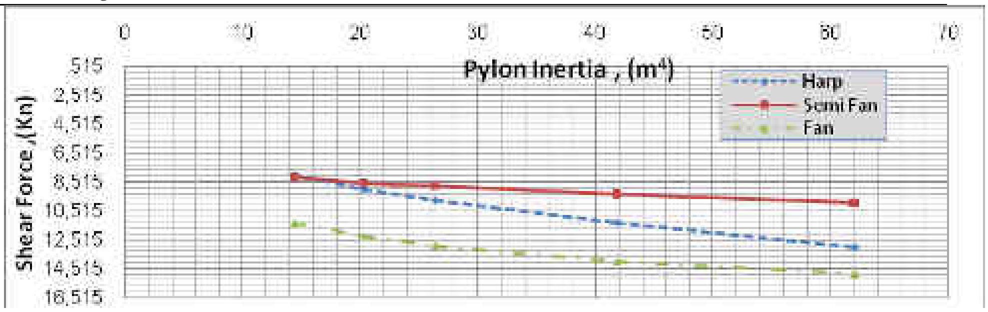


Fig. 35. The variation of shear force at fixed support of the pylon with I_{pylon}

4.5. Time period vs. mode number graphs for harp and fan system

The dynamic analysis with total time of 40 second and time step as 0.01second (4000 time steps) and damping ratio as 2% for studied cases is carried out. In the dynamic analysis, the energy method based on the minimization of the total potential energy of structural elements, via conjugate gradient technique is used. The procedure is carried out using a computer program based on the iterative scheme taking geometric nonlinearity into account. The dynamic behavior of a structure can be well characterized by a modal analysis. The linear response of the structure to any dynamic excitation can be expressed as superposition of its mode shapes. The contribution of each mode depends on the frequency content of the excitation and on the natural frequencies of the modes of the structure.

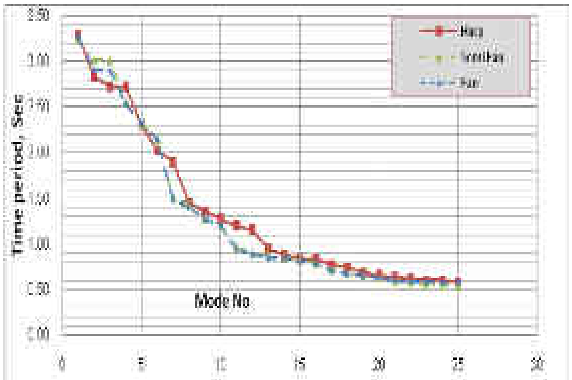


Fig. 36. Time Period vs. Mode Number Graphs

The first modes of vibrations are dominant having very long period of several seconds and are mainly deck modes, these are followed by cable modes which are coupled with the deck modes, Tower modes are usually later modes and their coupling with the deck depends on the support conditions.

From the above graph **Fig.36** it can be seen clearly that time period of vibration of cable-stayed bridges in the fan system is nearly the same as for the harp system

4.6. Frequency versus mode number graphs for harp ,semi fan and fan system

The frequency of vibration of cable-stayed bridges under seismic load has slight difference between harp and fan system and the values of frequency increases with increasing the time and mode number as shown in **Fig.37**.

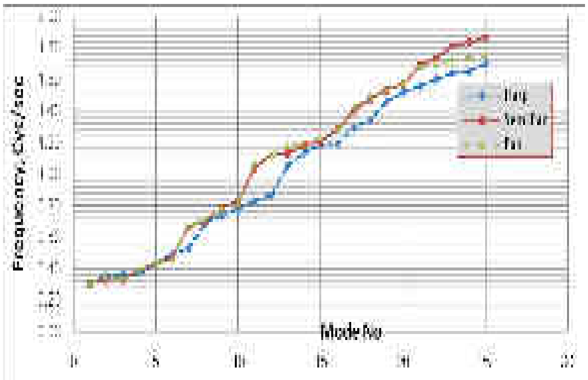


Fig. 37. Frequency vs. Mode Number Graphs for Harp and Fan System

4.7. Dynamic analysis in time domain

4.7.1. Acceleration vs. time at the top of pylon for harp and fan system

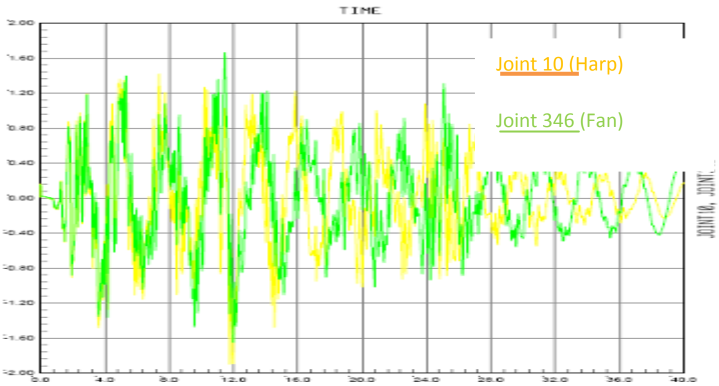


Fig. 38. Acceleration vs. time at the top of pylon for harp and fan system

From **Fig. 38.** the change of the stay layout has an effect on the shape of the relation *between the values of acceleration vs. time at the top of the pylon*

4.7.2. Velocity and displacement vs. time at the top of pylon for harp and fan system

In the **Fig. 39., Fig. 40.** the velocity values and displacement vs. time has change due to changing of the stay layout.

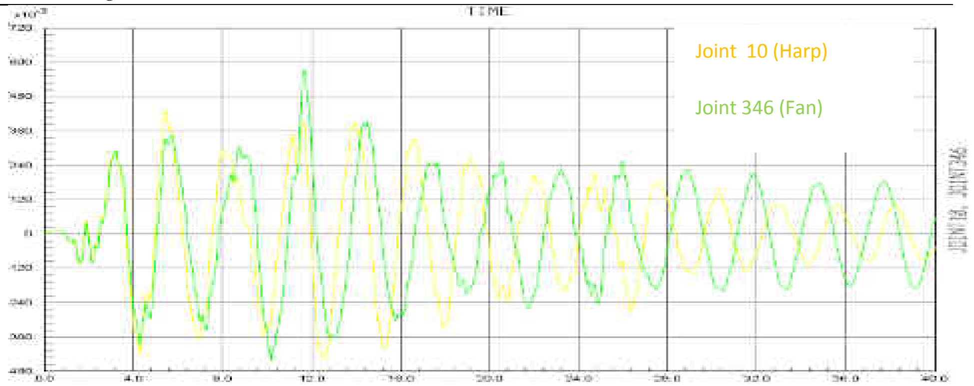


Fig. 39. Velocity vs. time at the top of pylon for harp and fan system

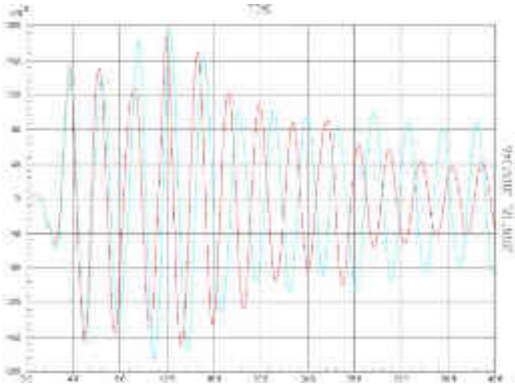


Fig. 40. Displacment vs. time at the top of pylon for harp and fan system

4.7.3. Acceleration vs. time at the mid span of the deck for harp and fan system

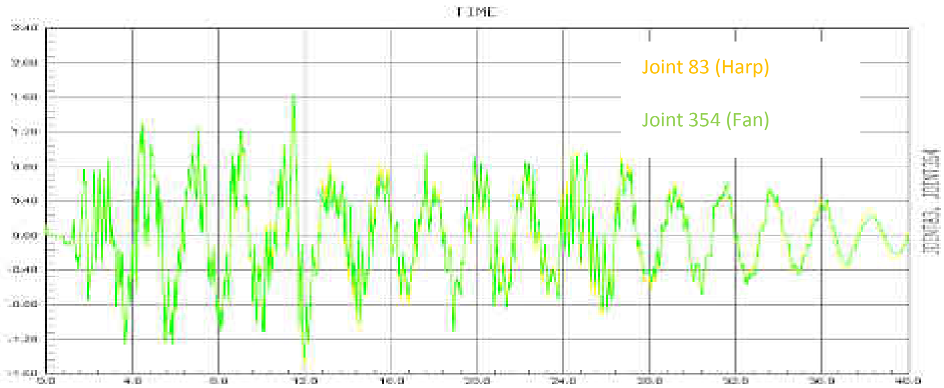


Fig. 41. Acceleration vs. time at the midpoint of the middle spans of the deck for harp and fan system

From **Fig. 41.** acceleration vs. time at the mid span of the deck has nearly the same values for fan and harp system. In the **Fig. 42., Fig. 43.** the velocity values and displacement vs. time has nearly the same values for the harp and fan system.

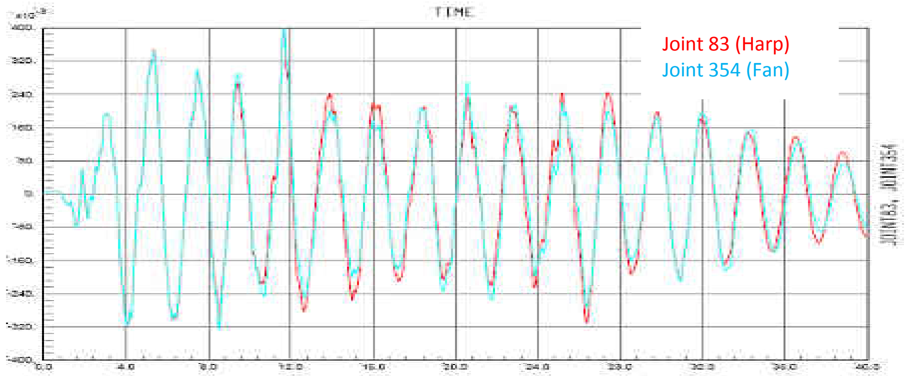


Fig. 42. Velocity vs. time at the midpoint of the middle spans of the deck for harp and fan system

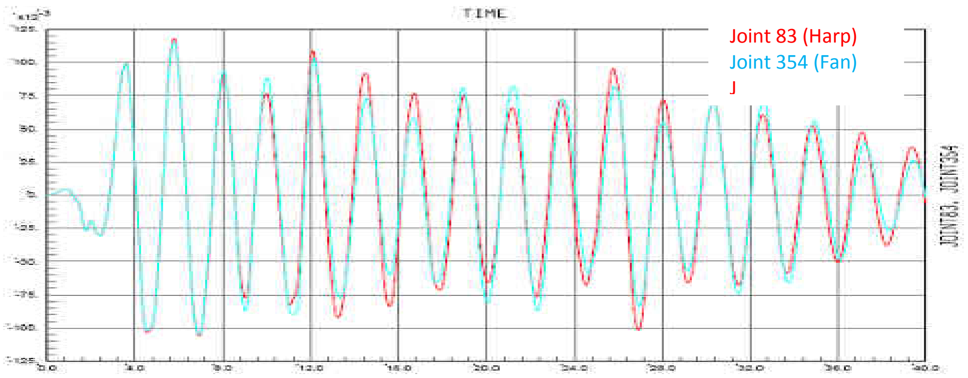
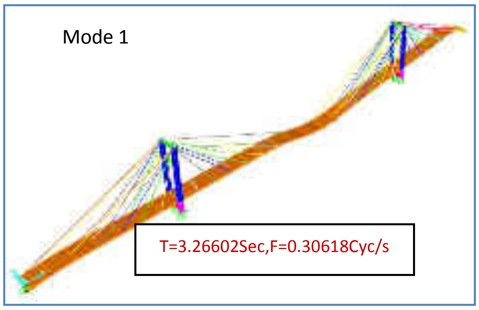
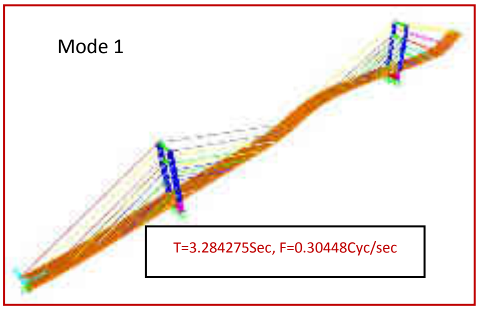


Fig. 43. Displacement vs. time at the midpoint of the middle spans of the deck for harp and fan system

Usually the modes obtained are classified in their directional properties. Thus, vertical, longitudinal, horizontal and torsional modes are distinguished.

As it was seen in **Fig. 44.**, Fan type model has less lateral deformation under dynamic effect, and in the 25 modes examined in the research and it was observed that there is a noticed difference between fan and harp concerning the mode shapes as the fan type model suffered from torsional deformation at the earlier modes (Mode 2).


Mode 1
T=3.26602Sec,F=0.30618Cyc/s


Mode 1
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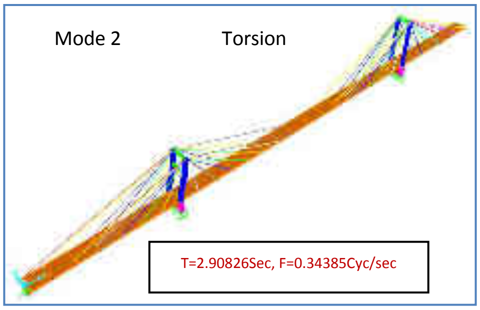
Mode 2
Torsion
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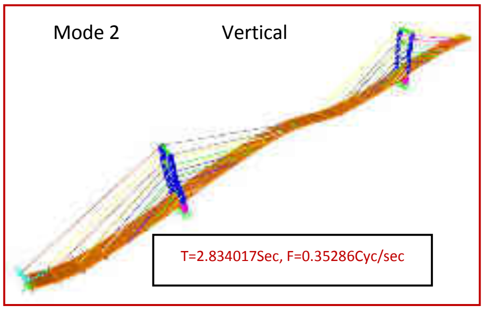
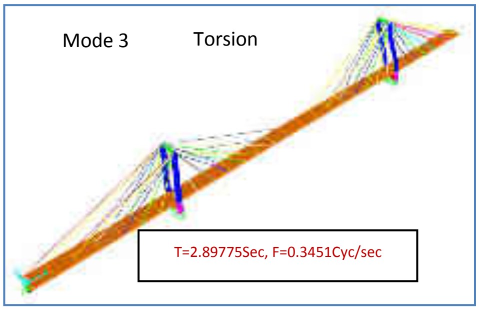

Mode 2
Vertical
T=2.834017Sec, F=0.35286Cyc/sec


Mode 3
Torsion
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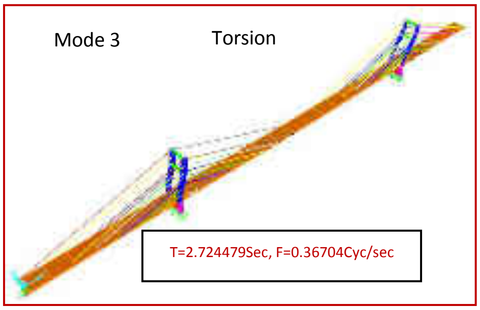
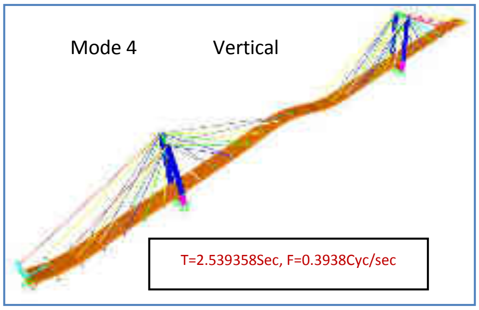

Mode 3
Torsion
T=2.724479Sec, F=0.36704Cyc/sec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Vertical
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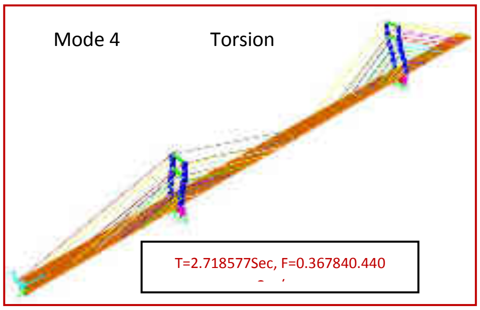
Mode 4
Torsion
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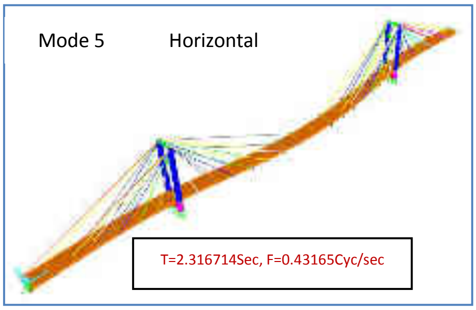
Mode 5
Horizontal
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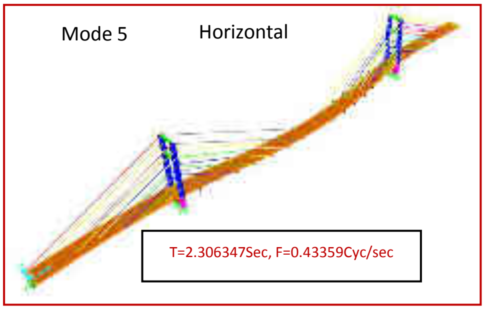
Mode 5
Horizontal
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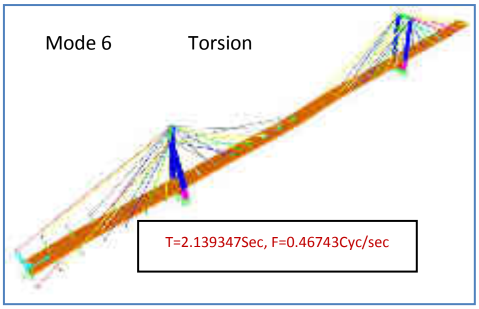
Mode 6
Torsion
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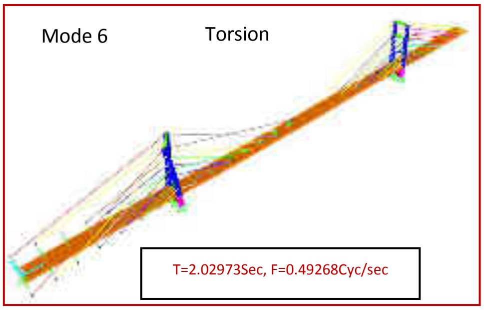
Mode 6
Torsion
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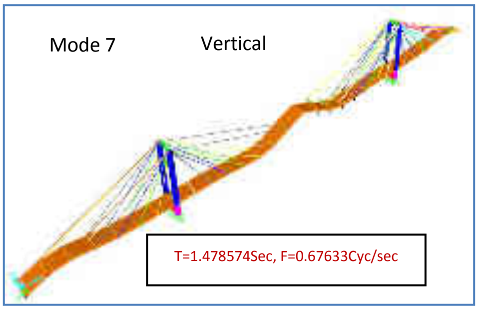
Mode 7
Vertical
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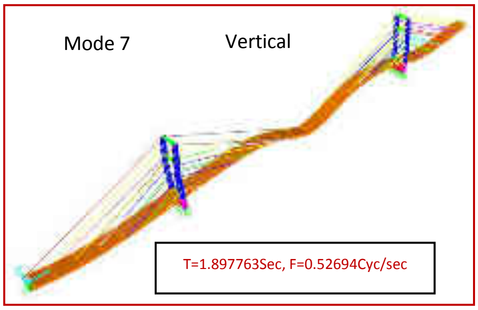
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Vertical
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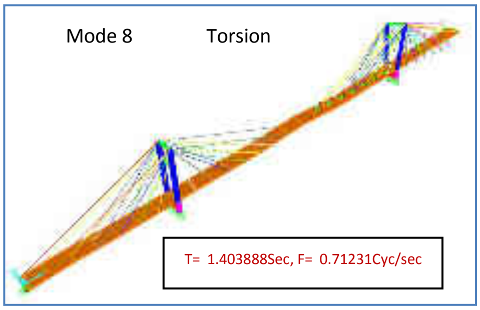
Mode 8
Torsion
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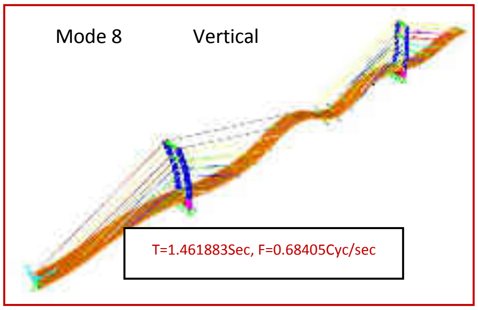
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Vertical
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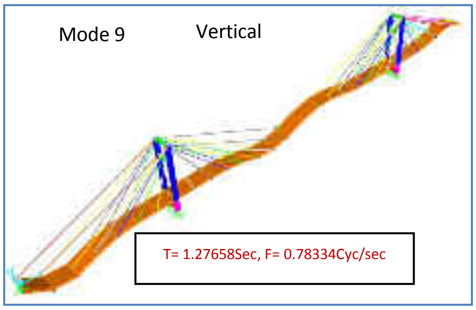
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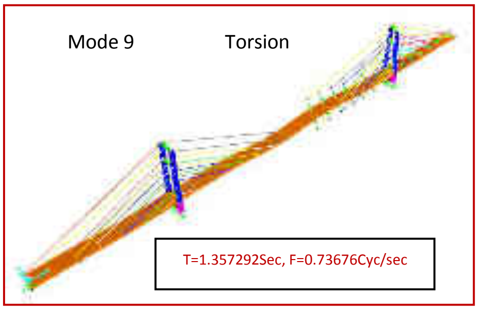
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Torsion
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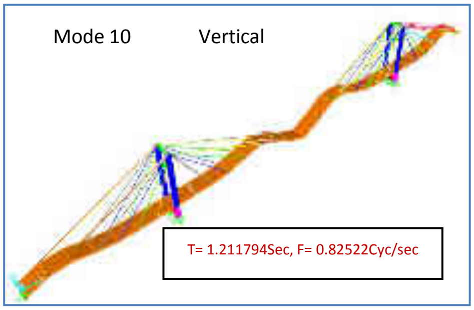
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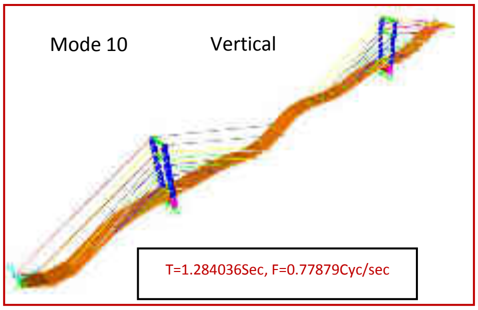
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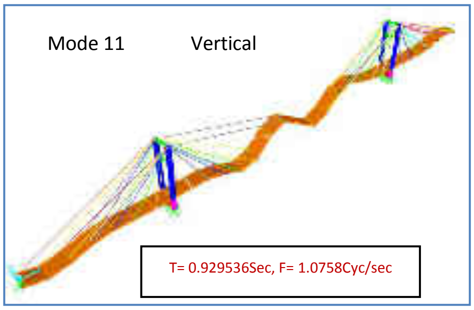
Mode 11
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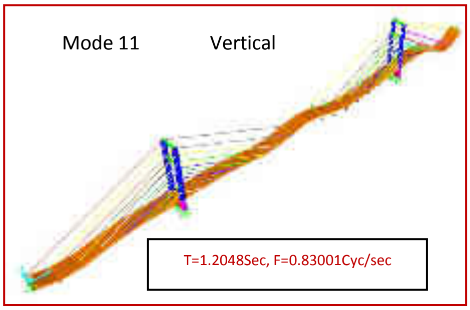
Mode 11
Vertical
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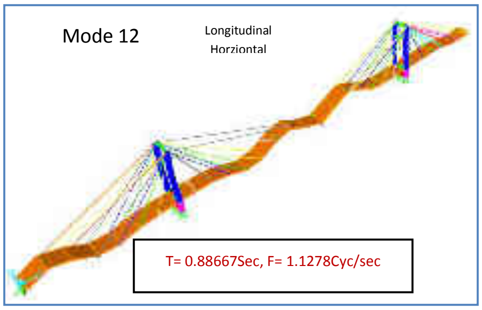
Mode 12
Longitudinal
Horziontal
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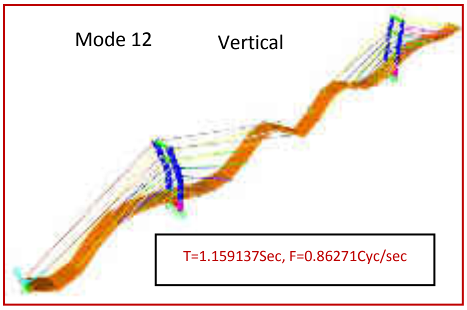
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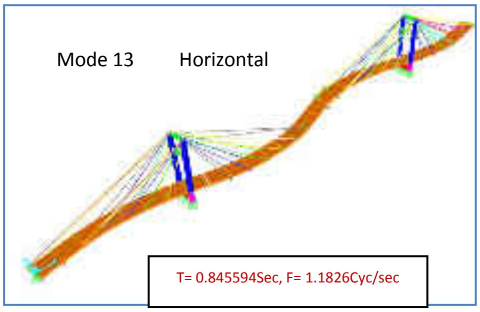
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Horizontal
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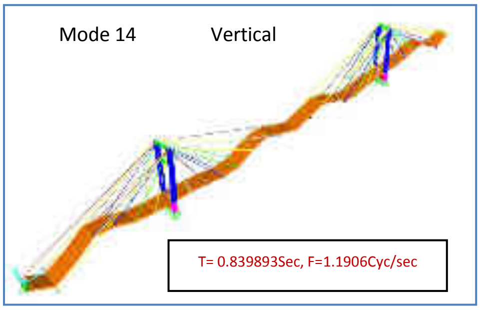
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Vertical
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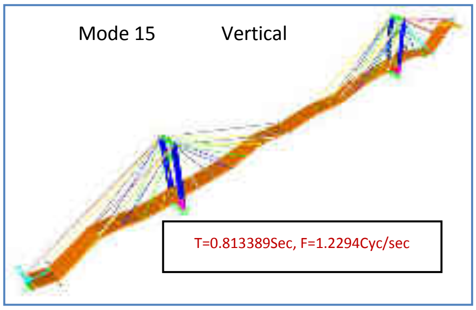
Mode 15
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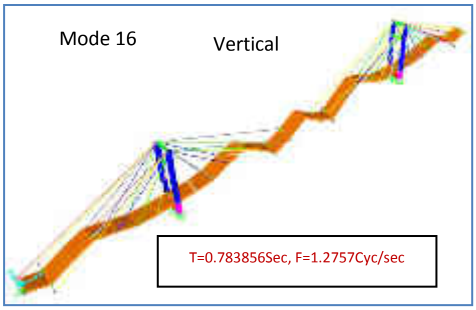
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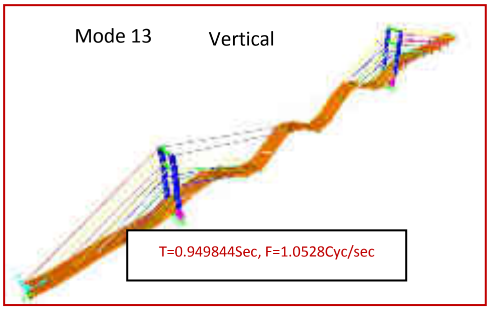
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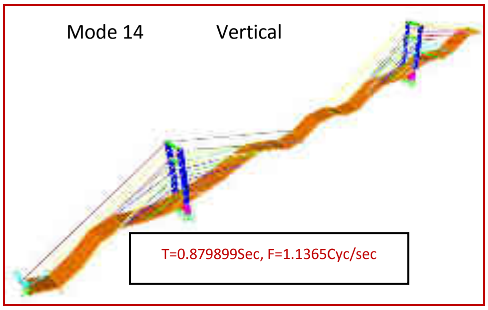
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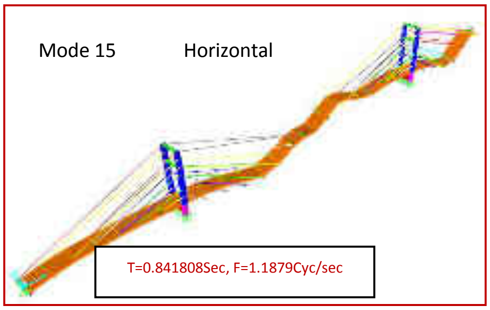
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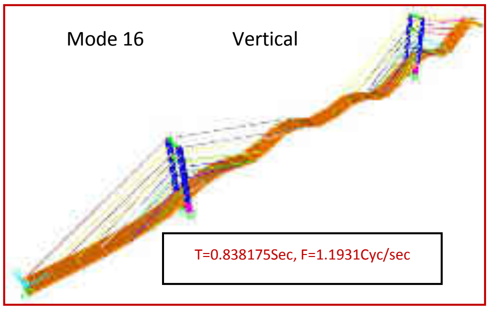
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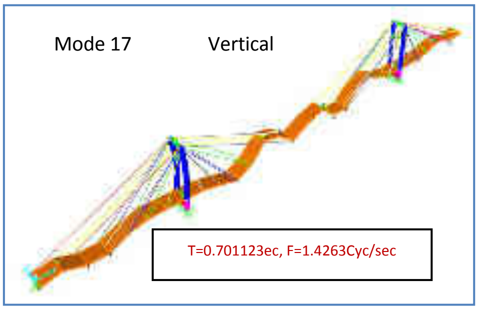
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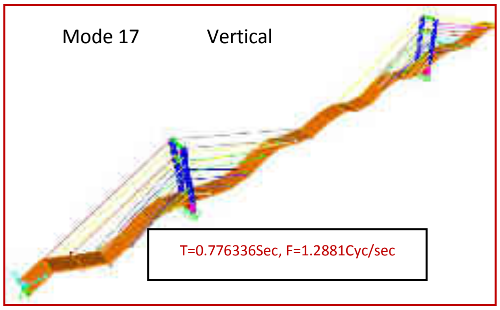
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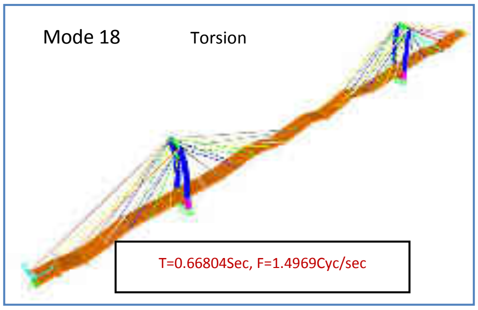
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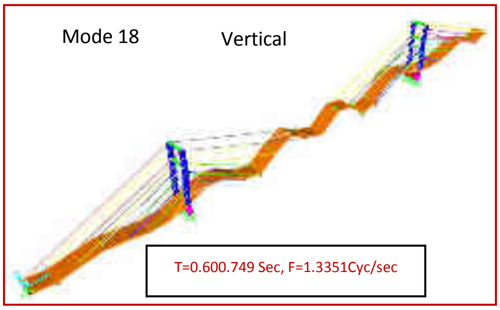
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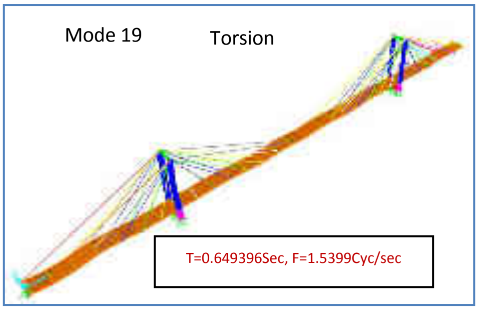
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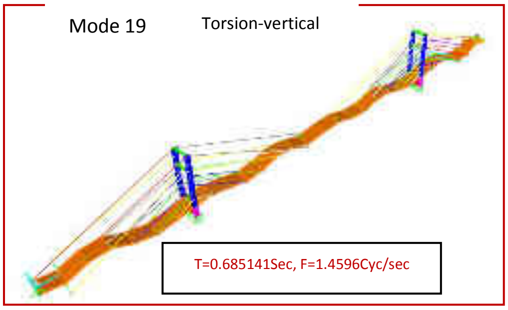
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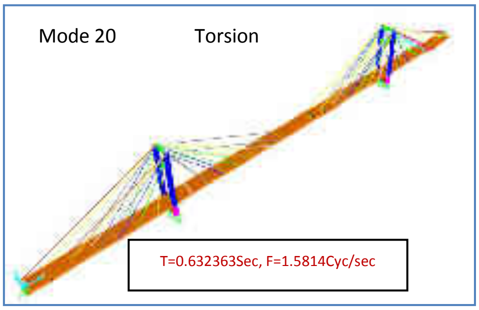
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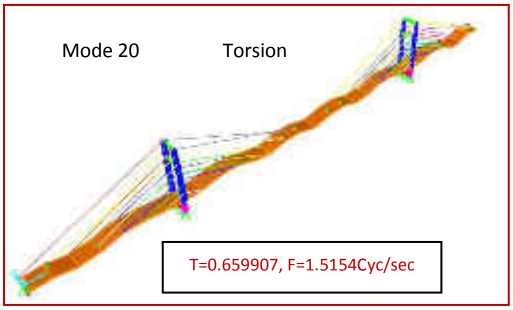
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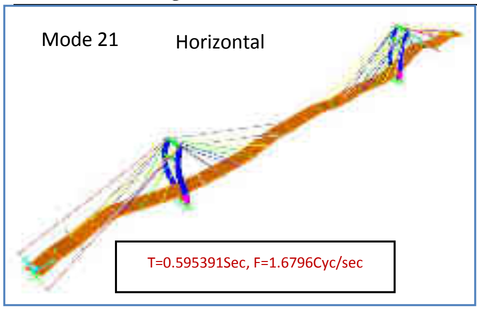
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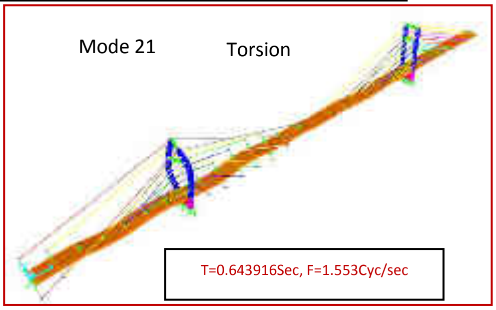
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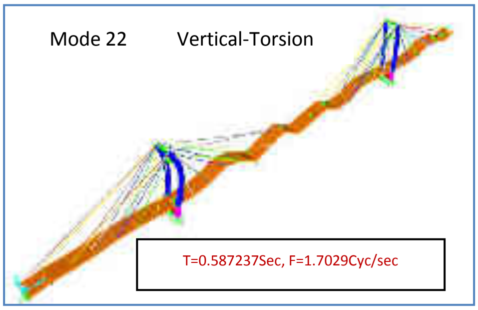
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Vertical-Torsion
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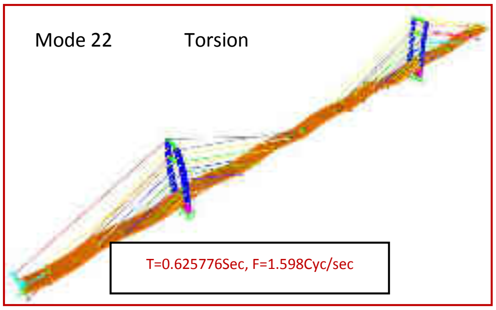
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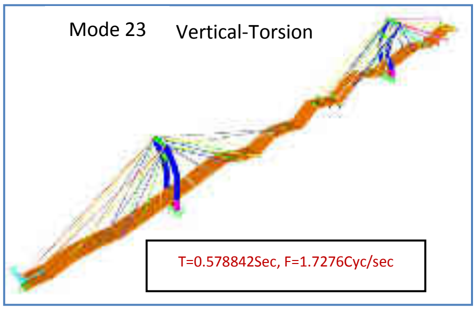
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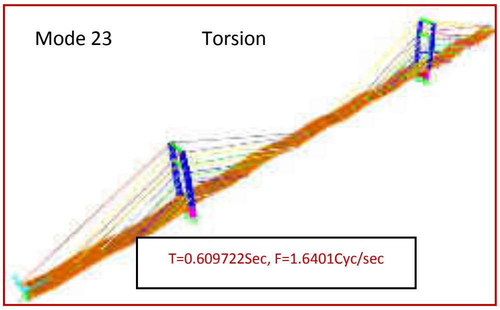
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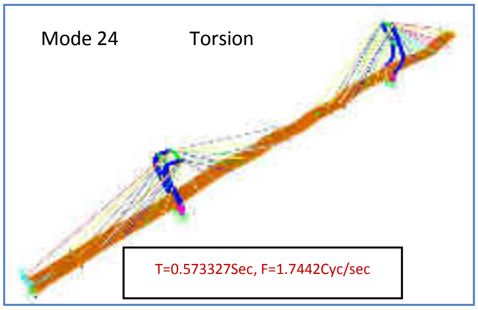
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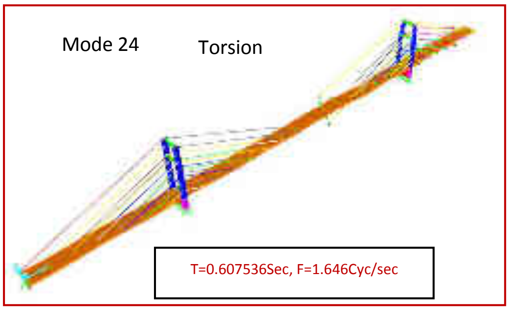
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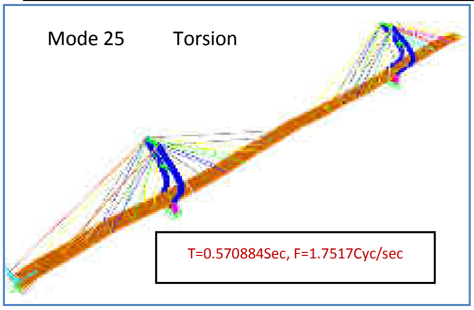

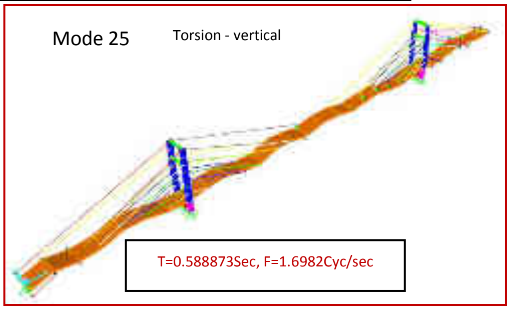

Fig. 44. The mode shapes

5. Conclusions

In the present study , an attempt has been to analyze the seismic response of cable-stayed bridges with two pylons and two equal side spans. This study has made an effort to analyze the effect of both static [11] and dynamic loadings on cable-stayed bridges and corresponding response of the bridge with variation of cable system. This paper investigated the seismic behaviour of cable stayed bridge through three dimensional finite element model. The geometric nonlinearity is involved in the analysis. The geometric nonlinearity comes from the cable sag effect, axial force – bending moment interaction and large displacements. The results have been made for different configurations of bridges, time period, frequency pylon top displacement , maximum deck displacement ad bending moment on the pylon. Parameters affecting the seismic response of these contemporary bridges are discussed. The following important findings have been drawn out and can be summarized as follows:

- The acceleration on the deck is slight dependent on the layout of cable system either harp or fan system
- The cross sections of cable system and H/L are the most important parameters affected on reducing the dynamic response on the cable stayed bridges.
- The deck with a high inertia in the longitudinal direction and high pylon inertia are not basically favorable, It attracts considerable moments; shear force without appreciably reducing acceleration and it must be dimensioned in an appropriate manner.
- The acceleration in the fan system along the height of the pylon is bigger than that in the harp system by a percentage reached to **175.78%** % at height 27.857m from the deck. So that the harp system is preferable to reducing acceleration response on the pylon.
- The first modes of vibrations are dominant having very long period of several seconds and are mainly deck modes, these are followed by cable modes which are coupled with the deck modes, Tower modes are usually later modes and their coupling with the deck depends on the support conditions.
- Fan type model has less lateral displacement under dynamic effect, and in the 25 modes examined in the research and it was observed that there is a noticed

difference between fan and harp concerning the mode shapes as the fan type model suffered from torsional deformation at the earlier modes (Mode 2).

5. References

[1] Abdel-Ghaffar, A.M., "Cable-stayed bridges under seismic action",Cable-Stayed Bridges - Recent Developments and Their Future, Ito, M. (ed.), Elsevier Science Publishers, 1991, pp. 171-192

[2] Abdel-Ghaffar, Ahmed M. and Rubin , Lawrence I, "Multiple-Support Excitations of Suspension Bridges", J. of Eng. Mech., ASCE, Vol. 108, No. EM2, April 1982, pp. 420-435.

[3] Baron, F., Arikan, M., and Hamati, E.," The effects of seismic Disturbances on the golden Gate Bridge", J. of Eng. Mech., ASCE, Vol. 108, No. EM2, April 1982, pp. 420-435.

[4] Computer and Structures (CSI), Inc. (2011) "SAP2000 Advanced 14.0.0 Software. Structural Analysis Program", Berkeley, California.

[5] Gimsing, N.J., "Cable supported bridges - concept & design", John Wiley, 2nd edition, 1998.

[6] Gupta, S.P., Kumar, A., "A study on dynamics of cable stayed bridge including foundation interaction", Proceedings of the 8th European Conference on Earthquake Engineering, Lisbon, 1986, Vol. 5, pp. 8.3/9-8.3/16.

[7] H. M. Ali and a. M. Abdel-ghaffarf , "Modeling the nonlinear seismic behavior of cable-stayed bridges with passive control bearings".

[8] Kanok-Nukulchai W., Yiu P.K.A., Brotton D.M., "Mathamatical Modelling of Cable-Stayed Bridges", Struct. Eng. Int., 2, pp. 108-113, 1992.

[9] Karoumi R., "Dynamic Response of Cable-Stayed Bridges Subjected to Moving Vehicles", IABSE 15th Congress, Denmark, pp. 87-92, 1996.

[10] Karoumi R., "Response of Cable-Stayed and Suspension Bridges to Moving Vehicles – Analysis methods and practical modeling techniques", Doctoral Thesis, TRITABKN Bulletin 44, Dept. of Struct. Eng., Royal Institute of Technology, Stockholm, 1998.

[11] Karoumi R., "Some Modeling Aspects in the Nonlinear Finite Element Analysis of Cable Supported Bridges", Computers and Structures, Vol. 71, No. 4, pp. 397-412, 1999.

[12] Nazmy, A.S., A.M. Abdel-Ghaffar, "Effects of ground motion spatial variability on the response of cable-stayed bridges", Earthquake Engineering and Structural Dynamics, Vol. 21, pp. 1-20, 1992.

[13] Nazmy A.S., Abdel-Ghaffar A.M., "Three-Dimensional Nonlinear Static Analysis of Cable-Stayed Bridges", Computers and Structures, 34, pp. 257-271, 1990.

[14] Nazmy, Aly S. and Abdel-Ghaffar, Ahmed M., "Nonlinear Earthquake-Response Analysis of long-Span Cable-Stayed Bridge" ,Theory, " the Int. Journal of Earthquack and Struct. Dyn., Vol. 19, 45-62, No. 1, Jan. 1990.

[15] Sayed Abdel-Salam, Hany O.Soliman, Osman Shalan, And Alaa M.Saad, "Sensitivity Analysis of Geometeric nonlinear parameters on cable stayed bridge".

[16] Shehata E. Abdel Raheem, Yasser Abdel Shafy, Fayez K. Abdel Seed and Hamdy H. Ahmed , "parametric study on nonlinear static analysis of cable stayed bridges"

التحليل الديناميكي اللاخطي للكباري الملجمة تحت تاثير الزلزال

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شحاته الضبع عبد الرحيم **ياسر عبد الشافي جمال علي**

الملخص العربى

لقد اصبحت الكباري الملجمة شائعة الاستخدام خلال العقود القديمة الماضية نتيجة لمظهرها الجمالي وكفائتها الانشائية وسهولة الانشاء والافضلية الاقتصادية علاوة على الخفة والمرونة ومستوى الاخماد الضئيل لذلك كان من الضروري فهم وادراك ميكانيكية (آليه) للتفاعل والتداخل بين المكونات الانشائية للاشكال المختلفة للكوبري واستخدامها في تزويد المعلومات الضرورية للحساب الدقيق للاستجابة الديناميكية المعقدة لهذه الجسور تحت الاحمال السيزمية .

وفي هذا البحث تم عمل محاولة لدراسة التحليل الديناميكي اللاخطي للكباري الملجمه ذات البحور الثلاثة وبعدد 2 برج هيكلي كركيزة من خلال استخدام شبكة العناصر المحددة كنموذج رياضي لهذه الكباري ثلاثية الابعاد وتم ادخال التحليل اللاخطى من خلال الاخذ في الاعتبار ارتخاء الكابلات والعلاقة التبعية للعزوم مع القوى المحورية والازاحات الكبيرة وتم اعداد النتائج لمختلف الانظمة للكابلات المشدودة وتبين الاتي:-

- مساحة قطاع الكابلات ونسبة ارتفاع البرج الحامل الى البحر الاوسط للكوبري اكثر العوامل اهمية في التأثير على الاستجابة الديناميكية للكباري الملجمه.
- عجلة ظهر سطح الكوبري تتوقف توقفا زهيدا او سطحيا على نظام الكابلات سواء كان قيثاري او مروحي.
- زيادة عزوم القصور لظهر الكوبري والبرج الحامل غير مفيد ويؤدي الى زيادة عزوم الانحناء وقوى القص بدون التاثير على نقص العجلة لظهر الكوبري والبرج الحامل وحساب التصميم لابعاد الكوبري يجب ان يتم بطرية ملائمة.
- عجلة البرج الحامل على ارتفاع 27.857 م في حالة النظام المروحي للكابلات يزيد بنسبة 1.75 مرة نظيرتها في الحالة القيثارية لذا يفضل النظام القيثاري لتقليل الاستجابة الديناميكية اثناء الهزات الارضية.