Earthquakes and Structures, Vol. 2, No. 4 (2011) 337-356
DOI: http://dx.doi.org/10.12989/eas.2011.2.4.337

Investigations on seismic response of two span cable-stayed bridges

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(Received April 9, 2010, Revised February 10, 2011, Accepted June 2, 2011)

Abstract. In this paper, cable-stayed bridges with single pylon and two equal side spans, with variations in geometry and span ranging from 120 m to 240 m have been studied. 3D models of the bridges considered in this study have been analysed using ANSYS. As the first step towards a detailed seismic analysis, free vibration response of different geometries is studied for their mode shapes and frequencies. Typical pattern of free vibration responses in different frequencies with change in geometry is observed. Further, three different seismic loading histories are chosen with various characteristics to find the structural response of different geometries under seismic loading. Effect of variation in pylon shape, cable arrangement with variation in span is found to have typical characteristics with different structural response under seismic loading. From the study, it is observed that the structural response is very much dependent on the geometry of the cable-stayed bridge and the characteristics of the seismic loading as well. Further, structural responses obtained from the study would help the design engineers to take decisions on geometric shapes of the bridges to be constructed in seismic prone zones.

Keywords: cable-stayed bridge; free vibration; parametric study; seismic response; mode shapes; frequencies; bridge response; time history.

1. Introduction

Cable supported bridges, i.e. cable-stayed and suspension bridges, have gained much popularity in recent decades due to their aesthetic appearance, efficient utilization of structural materials and other notable advantages. Cable-stayed bridge is an innovative structure that is both old and new in concept. It is old in the sense that it has been evolving over a period of approximately 4000 years ago when the concept of cable-stayed bridge was used by Egyptians for sailing ships and new because its modern day implementation began in the 1950s in Germany. Bridges of this type are entering a new era with main span reaching 1000 m. This is possible because of the development of efficient construction techniques and the rapid progress in robust analysis tools and advanced design methodologies for this type of bridges. Further, a great advancement has taken place in developing high strength materials too. For small spans upto 50 m, cable-stayed bridges are being used as pedestrian bridges. Although cable-stayed bridges are economical in range from 150 m to 500 m,

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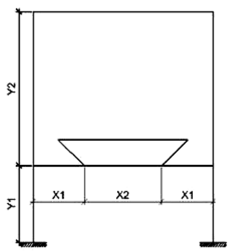
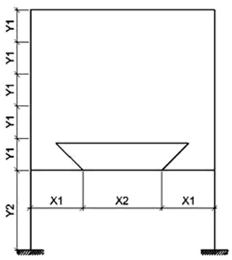
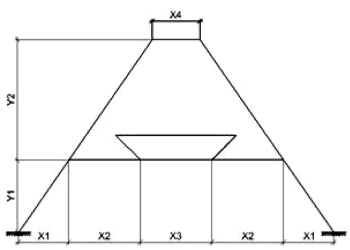
considerable efforts are being paid to longer span also.

Abdel-Ghaffar and Nazmy (1991) and Kanok-Nukulchai *et al*. (1992) discussed the issues of mathematical modeling of cable stayed bridges. Nonlinear seismic behavior of cable-stayed bridges was brought out by Ali and Abdel-Ghaffar (1995). Parametric study of cable-stayed bridges for their static response has been carried out to identify the best combination of geometry by Agarwal (1997). The effect of deck material on static response of cable-stayed bridges is studied by George (1999). The effect of number of cables on static behavior of cable-stayed bridges is studied by Wang (1999). With increase in number of cables, i.e. if cable spacing is reduced then local bending moments due to imposed load in deck increases, however if cable spacing is increased then local bending moments due to dead load increases (Bhagwat 2008). Further, it is important to state that global moment of the cable stayed bridge will not be greatly influenced by the spacing of the cables. Cable-stayed bridge is a highly indeterminate and nonlinear structure. Hence, its analysis is complex and very tedious. Types of nonlinearities to be taken into consideration and their effect on accuracy of result in dynamic analysis are studied by Fleming and Egeseli (1980). It is observed that response of the structure with nonlinear behaviour subjected to static loads (dead loads and superimposed dead loads) under linear dynamic and non- linear dynamic conditions is quite similar. There is no significant change in the structural response obtained from nonlinear dynamic analysis in comparison with that obtained from linear analysis. Similarity in response of the bridge obtained from the linear dynamic and non- linear dynamic conditions is due to the "dead-load condition", where the deformed shape of the bridge under dead load only (which is also the load condition for seismic analysis) matches the design geometry. Therefore, the nonlinear effects are not very significant. In the study of seismic behaviour of cable-stayed bridges by Allam and Datta (1999), it has been found out that, the response of the bridge is considerably influenced by pylon-deck inertia ratio. With increase in pylon-deck inertia ratio, both displacement and bending moment responses decrease in outer span, whereas the change in structural response (displacement and bending moment) in the inner span is not so significant. An investigation on non-linear static analysis of three-dimensional long-span cable-stayed bridges under the effect of their own dead weight and with cable tensions was presented by Nazmy and Ghaffar (1990) where all form of possible geometric non-linearities such as cable sag, large displacement of the bridge deck and axial force-moment interactions were dealt with.

Wyatt (1991) proposed the following the criteria for ground motions (irrespective of uniform or non uniform) to be treated as input ground motion for theoretical studies on response evaluation bridges under seismic loading. These are: (i) Three or more sets of appropriate ground motion time histories should be used; (ii) They should contain at least 20 seconds of strong ground shaking or have a strong shaking duration of 6 times the fundamental period of the bridge, whichever is greater; (iii) The ordinates of the input ground spectra should not be less than 90 percent of the design spectrum over the range of the first five periods of vibration of the bridge in the direction being considered; and (iv) Dynamic response analysis should then be carried out for bridges which are tentatively designed by the pseudo dynamic method, in order to verify more accurately their seismic performance in terms of maximum bearing capabilities. Further, Youliang *et al*. (2008) brought out a methodology for static and dynamic simulation of box girder cable-stayed bridge where shell elements with equivalent orthotropic material property was adopted. Alexander (2008) used recorded ground acceleration data for evaluating the effect of multi-support excitation on single span bridges.

From the review of literature, it has been found that though a considerable amount of work has been carried out on seismic performance of cable-stayed bridges (Herzog 1987, Walther 1999, Krishna 2007), studies on seismic behaviour of comparatively short to medium span cable-stayed bridges are not adequate. In this study, cable-stayed bridges with single pylon and two equal side spans along with different pylon geometry and cable distributions under seismic loading have been investigated in details. These include 3 types of cable and pylon arrangements (A-shape pylon with Fan shaped cable; H-shape pylon with Fan shaped cable; and H-shape pylon with Harp shaped cable) with length of 120 m, 180 m and 240 m. Different types of cable- and pylon- arrangements and their parametric variables considered in the present study are presented in Table 1. Deck and pylon of the cable-stayed bridge are assumed to be made of concrete. Cross sectional dimension of the pylon (rectangular) at base is considered as 3 m × 5.75 m and it is uniformly reduced to 3 m × 3 m at top. The cross section of the deck of the bridge is considered to be box girder (deck area = 14.49 m^2,

Table 1 Types of bridges considered with parametric dimensions (in meter)

Type of the bridge	Length of the bridge	*y*1	*y*2	*x*1	*x*2
 Fan shaped cable with H-shaped pylon	120 180 240	8 8 8	22 32 42	5.44 5.44 5.44	16.62 16.62 16.62
 Harp shaped cable with H-shaped pylon	120 180 240	6.67 10 13.34	8 8 8	5.44 5.44 5.44	16.62 16.62 16.62
 Fan shaped cable with A-shaped pylon	120 180 240	8 8 8	22 32 42	4.63 3.19 2.42	5.44 5.44 5.44

Note: ×3 and ×4 are assumed to be 16.62 m and 2 m, respectively

Ixx = 9.02 m^4, Iyy = 541.25 m^4, J = 45.2 m^4). Here, Ixx and Iyy represent moment of inertia of the deck about the vertical and horizontal (along the transverse direction) axis respectively. For determining the geometry of the structural elements of the bridge, loadings are considered as prescribed in Indian road congress (IRC). Dimensions of the bridges considered in this study are based on dead load, live (traffic) and seismic load analysis. For evaluating seismic load, response spectrum analysis has been performed and the spectrum has been considered as that is stipulated in Indian standard (IS1893-2002) for structures in soft soil condition. Since the evaluation of initial size of the bridge geometry is not the central aim of the present study, for brevity, it is not discussed in the present paper. Further, analysis and preliminary design of the cable stayed bridge has been carried out by using public domain software SOFiSTiK. Three load conditions, i.e. dead load including superimposed dead load, traffic load and seismic load without any factor, as specified in IRC, are considered. In the software, material properties and the probable available sections have to be assigned. Based on the critical load combinations, the software chooses the most preferable sizes of the structural elements. It is necessary to mention that the deck dimensions and overall geometry of the bridge are decided by the authors.

Ernst approach has been used to calculate initial prestress in cables to cope with self weight deformations in cables, as given by Ernst (1965). Cables have been designed based on the maximum possible tension from all possible load combinations. For dynamic analysis, mass of the deck and the pylon has been calculated from dead weight of the respective component. For better representation, A-shape pylon with Fan shaped cable, H-shape pylon with Fan shaped cable and H-shape pylon with Harp shaped cable are hereinafter called as A-Fan, H-Fan and H-Harp, respectively.

2. Finite element modelling and validation

For the analysis purpose, ANSYS 11 (non-commercial) software has been selected in this study. In the present study, nine 3D models are investigated. By keeping the computation time within practical range and without compromising the accuracy, 3D modelling of cable-stayed bridge has been done by using shell elements and 3D beam elements for deck and pylon, respectively. SHELL63 element for deck (components of box girder), BEAM4 for pylon and LINK10 for cables are used. The important assumptions made in this study, are: (a) homogeneous, isotropic and linearly elastic material, (b) only geometric nonlinearities, (c) perfectly flexible cables with only tension stiffness, (d) cable-deck, cable-pylon and deck-pylon with fixed connection, (e) material damping of 5%. Fig. 1 shows the typical finite element model of an A- shaped pylon with fan shaped cable arrangement with the elements as mentioned above.

Under the action of its own dead load and axial tensile force, a cable supported at its end will sag into a catenary shape. The axial stiffness of the cable will change with change in sag. When a straight cable element is used in FE modelling to represent the inclined cable stay, the sag effect has to be taken into account. For each cable, equivalent cable modulus has been calculated to carry permanent static loads (design static load calculated from the critical load combination) with the help of formula given by Ernst (1965).

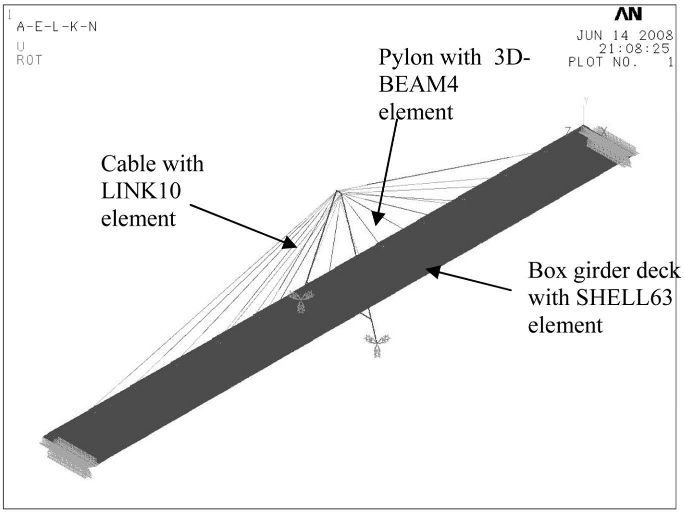


Fig. 1 Finite element model of a cable-stayed bridge with Fan shaped cable and A- shaped pylon

2.1 Validation of the finite element model used in this study

Fundamental frequencies of the structure obtained from the present study and the reported results from analytical formulation have been compared for validating the finite element models. To validate the numerical models developed in the present study, bridges with two pylons and different centre span length are considered since these types of geometries have been reported frequently in the literature. Main focus of the validation study was to get the confidence in choosing the proper element type, mesh size and analysis procedure, used in the present study since these always play a very important role in global structural response. Bridges with two pylons and different centre span length of 120 m, 160 m, and 200 m are modelled and analysed since two pylon with equal end spans are reported frequently with analytical investigation results. For the validation study, centre span to end span ratio has been kept constant as 2.5 and centre span to pylon height ratio has also been kept constant as 5. In the validation problem, free vibration analysis is carried out for 3

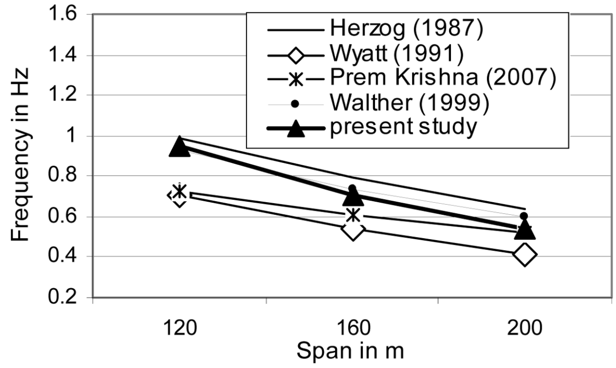


Fig. 2 Fundamental frequencies from the present FE models and reported literature

models with A-shape pylon and fan shaped cable arrangement. The results have been verified with the formulae given by Herzog (1987), Ito *et al*. (1991), Walther *et al*. (1999) and Krishna (2007) for the first fundamental frequency of cable-stayed bridges with two pylons. The results of the present study along with the results of first frequencies given by other researchers as mentioned above have been shown in Fig. 2. From the figure, it is observed that, results of first fundamental frequencies obtained from the free vibrational analysis of the present models are within the experimentally and/ or analytically proved results in the literature.

3. Free vibration analysis

Before carrying out the seismic analysis, free vibration characteristics of the cable-stayed bridge with various geometries and cable arrangements have been investigated since a free vibration analysis of a complex structure like cable-stayed bridge would easily bring out the basic dynamic properties. Typical fundamental modes for A-Fan type cable-stayed bridge is shown in Fig. 3 and the results of the frequencies and their corresponding nature of mode shape for A-Fan, H-Fan and H-Harp of cable-stayed bridges are presented in Table 2 to Table 4. It is utmost important to mention here that in the finite element model of the bridges, hinged boundary conditions were assumed to get clear bending and torsional modes and hence, no longitudinal sway was observed in Fig. 3. Here, first vertical bending mode refers to first anti-symmetric vertical bending mode of the deck, second vertical bending mode refers to first symmetric bending mode of deck, third vertical bending mode refers to second anti symmetric vertical bending mode of deck, and fourth vertical bending mode of deck refers to second symmetric vertical bending mode of the deck. The results show that for A-shape pylon with Fan shaped cable arrangement, first 4 modes are pure deck modes in vertical bending. Because of high stiffness of A-shape pylon, frequency of first pylon mode is very high. From 6^{th} mode, torsional vibration of decks is observed, but notably mixed with pylon modes with increase in span. Generally, in A-shape pylon mode shapes are clear and no combination of modes is found. Due to higher stiffness of pylon, pylon and deck modes are clear and separate. On the other hand, in H-shaped pylon, pylon transverse mode comes at much early level since pylon stiffness is less in comparison to A-shaped pylon. As in harp shaped cable arrangement pylon height is more, pylon mode occurs first and at very low frequency. In all the cable-stayed bridges studied here, it is observed that the torsional modes occur at very high frequency.

A comparative study on the fundamental frequencies corresponding to vertical bending modes (upto 4^{th} mode) for the bridges considered in this study is presented in Fig. 4. It shows that H-Harp is most flexible followed by A-Fan and H-Fan during vibration corresponding to the first two vertical mode (anti-symmetric and symmetric bending) but, the behaviour changes in the higher modes. For next higher vertical modes, A-Fan and H-Harp produce almost same frequencies whereas the frequency for H- Fan bridge proves to be less stiffer than A-Fan and H- Harp. This signifies that during predominant vibrations and under lower modes, though Fan type cable arrangement shows a better (stiffer) response, it could not remain same under higher mode vibrations. Further, it is also observed from the study (as shown in Fig. 4) that the difference in fundamental frequencies of the bridges decreases with increase in span. It is also worth-noting that the difference in frequencies (in lower modes) for different lengths of A-Fan and H-Fan is much lesser than that with the H-Harp. So, it can be mentioned that the cable arrangement of the cable-stayed bridges has a considerably significant role to play.

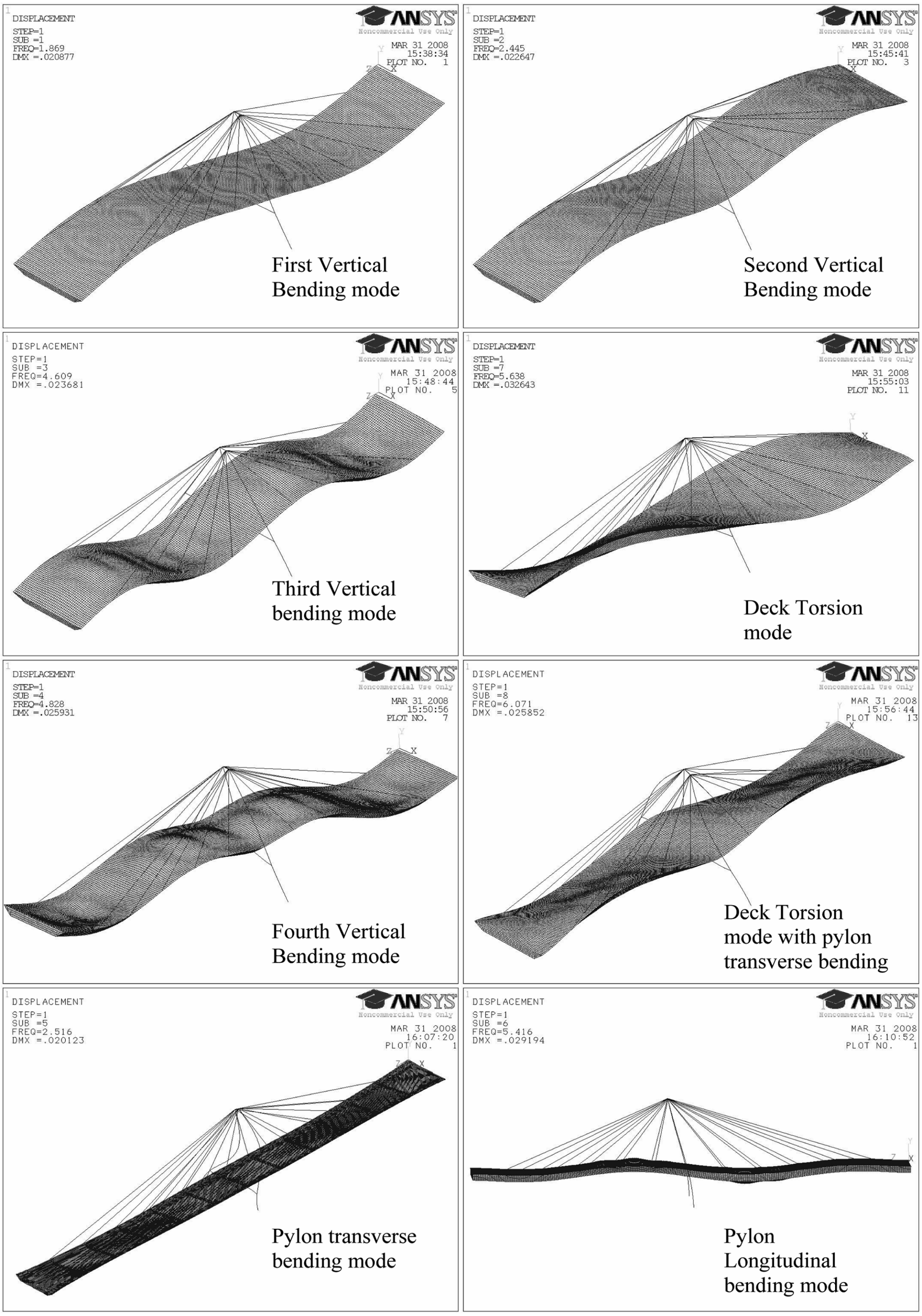


Fig. 3 Typical fundamental mode shapes of A-shape pylon with Fan shaped cable

Table 2 Free vibration results of A-shape pylon with Fan shaped cable (TYPE 1)

Mode No.	120 m	180 m	240 m	Remarks on nature of mode shape
	Frequencies are in Hz			
1	1.8690	1.0056	0.66029	First vertical bending
2	2.4446	1.4127	1.06	Second vertical bending
3	4.6092	2.5598	1.7717	Third vertical bending
4	4.8276	2.8868	2.0018	Fourth vertical bending
5	4.9276	3.5003	2.5163	Pylon transverse bending
6	5.4157*	3.7792	2.9555	Deck torsion, *pylon longitudinal bending
7	5.6380*	3.9013+	2.9631#	*deck torsion, +pylon longitudinal bending, #fifth vertical bending
8	6.0711	3.9252	2.9852	Deck torsion with pylon transverse bending

Table 3 Free vibration results of H-shape pylon with Fan shaped cable (TYPE 2)

Mode No.	120 m	180 m	240 m	Remarks on nature of mode shape
	Frequencies are in Hz			
1	1.9536	1.027	0.66528	First vertical bending
2	1.9974	1.2146	0.81965	Pylon transverse bending
3	2.5251	1.4353	1.0706	Second vertical bending
4	4.3187*	2.544	1.7712	Third vertical bending, *with pylon long. bending
5	4.9735*	2.9323	2.023	Fourth vertical bending, *third vertical bending
6	5.0832*	3.3549+	2.5916#	*fourth vertical bending, +pylon long. bending, #pylon long. bending mixed with deck torsion
7	5.4937	3.5462	2.9061#	Deck anti symmetric torsion mode, #fifth vertical bending mode
8	5.6540	3.776	2.976	Deck symmetric torsion mode

Table 4 Free vibration results of H-shape pylon with Harp shaped cable (TYPE 3)

Mode No.	120 m	180 m	240 m	Remarks on nature of mode shape
	Frequencies are in Hz			
1	1.084	0.60557	0.38823	Pylon transverse bending
2	1.6925	0.93532	0.6202	First vertical bending
3	2.3499	1.4431	1.1016	Second vertical bending
4	2.4627	1.9249	1.6528*	Pylon longitudinal bending, *
5	4.1432	2.4886	1.6799	Pylon longitudinal bending mixed with torsion in deck
6	4.6327	2.5706	1.7153	Third vertical bending
7	4.8599*	2.7193+	1.8124#	*fourth vertical bending, +pylon, #deck torsion
8	5.1189*	2.9228+	1.9775+	*deck torsion, +fourth vertical bending

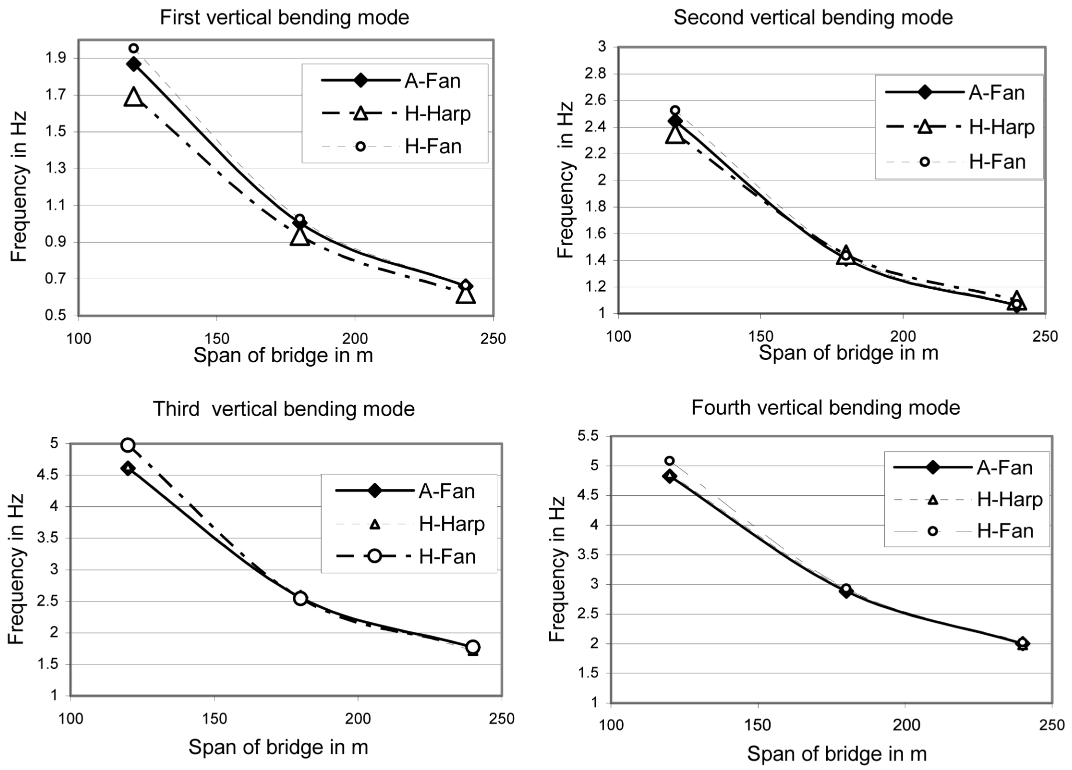


Fig. 4 Comparison in frequencies (first 4 modes) of different types of cable-stayed bridge

4. Forced vibration analysis under seismic loading

For seismic analysis, 3 load-time histories are considered, viz., El Centro and two synthetic data generated by University of California Berkley, USA. Selection of forcing functions (as shown in Fig. 5) is based on different peak ground acceleration (PGA), different nature of forcing functions and occurrence of peak ground motion at different times. Cable-stayed bridges with different geometries have been investigated under these three earthquakes histories. Since, it is important to compare the behaviour of the different geometry, it is worthy to use the structural responses in a non-dimensional form. Non-dimensionalisation of the structural responses (i.e. moment, shear, stresses) has been carried out using equivalent factors of half span of the 120 m span bridge. Factors for non-dimensionalision are

$$V = \rho \times g \times A \times L \tag{1}$$

$$M = \rho \times g \times A \times L^2 \tag{2}$$

$$S = \rho \times g \times \frac{A}{L} \tag{3}$$

Where, V, M, S, are non-dimensionalised factors for shear force, bending moment and stresses respectively. ρ = mass density, g = gravitational acceleration, A = cross sectional area of deck, and

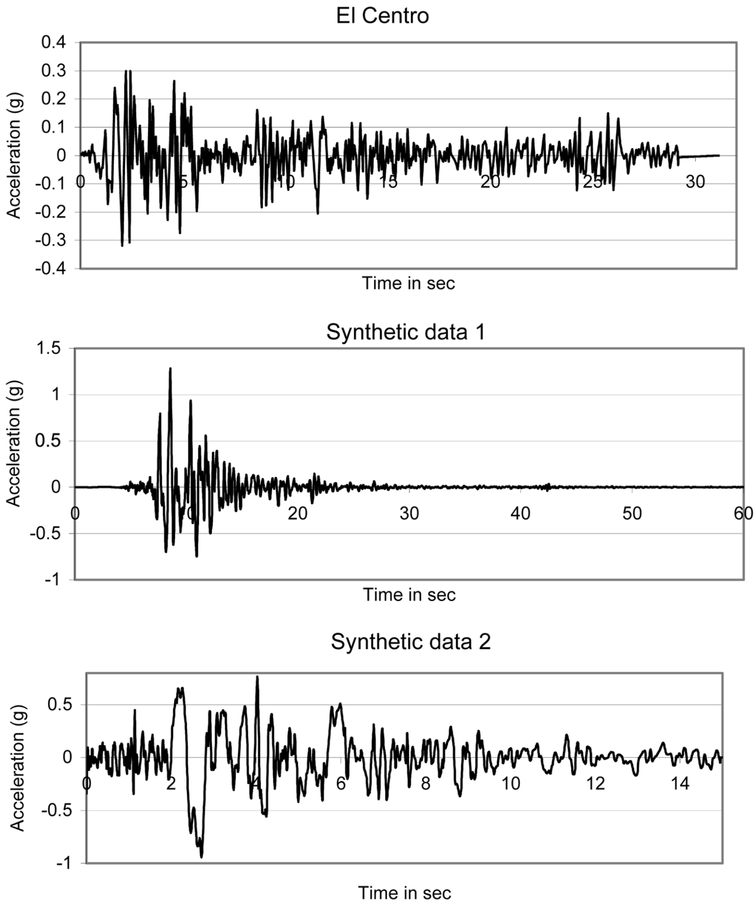


Fig. 5 Different types of earthquake histories considered in this study

L = span length of 120 m bridge, i.e. 60 m

During the seismic analysis, geometric nonlinearities are considered and transient dynamic analysis has been performed using Lanczos solver. In the entire study, the materials are assumed to be homogeneous, isotropic and elastic. After analysing all the nine bridges under 3 types of seismic loading as mentioned earlier, it was observed that El Centro and synthetic data 1 load histories produce same structural response (longitudinal bending moment envelop, longitudinal and transverse stress envelop) although the nature and the magnitude of both the earthquake histories are quite different. For example, synthetic data 1 has amplitude of 1.283 g at 8.56 sec and whereas El Centro earthquake has amplitude of 0.319g at 2.04 sec. To investigate the reason behind this results, frequency content of all the earthquake histories are studied. Using Fast Fourier Transformation (FFT) in MATLAB, frequency contents of all histories are studied and only the frequency content of El Centro and synthetic data 1 is shown in Fig. 6. It is observed that first frequency of the El Centro and synthetic data 1 are exactly the same. Due to the same frequency content of two earthquakes with different nature, envelop of the structural response due to El Centro and synthetic data 1 were exactly same in magnitude. Hence, the results of El Centro and synthetic data 2 are presented and discussed in the following section.

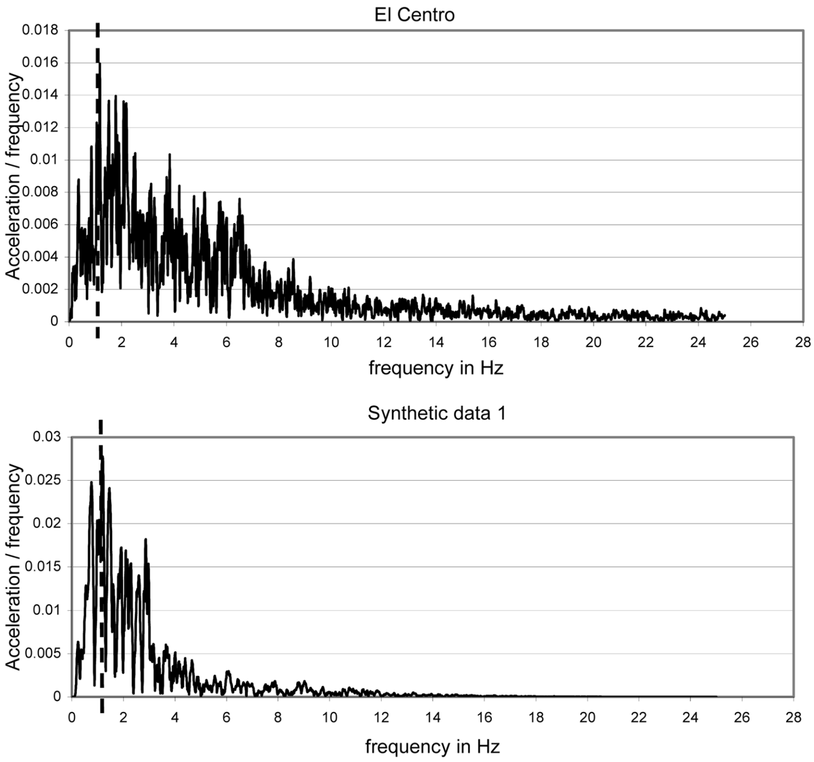


Fig. 6 Frequency contents of El Centro and synthetic data 1 earthquake

4.1 Influence on structural parameters

The dynamic response of cable-stayed bridge is highly complex so as its geometry. It is important to get a qualitative behaviour of the structural response of a cable-stayed bridge under seismic loading which would help the practicing engineers to identify the most critical responses and to understand the influence of the global geometry on its particular structural response. Among the various structural response parameters, few most important response parameters which are representative to bring out the seismic response of the cable-stayed bridge are presented in this study. These are longitudinal bending moment envelop, longitudinal bending stress envelop, transverse bending stress envelop, shear stress envelop and pylon shear envelop. Absolute maximum quantities of bending moment, longitudinal and transverse bending stress, shear stress are measured at central nodes of top flange of the deck at every 5m interval. It is important to mention here that the studied structural responses under both loading histories have shown to be almost symmetric along the span. Hence, though results obtained from the analysis of the bridge subjected to El Centro is presented for the entire span of the bridge, results obtained from the analysis of the bridge subjected to synthetic data 2 is presented upto the mid span of the bridge as the remaining part is identical.

4.1.1 Non-dimensional bending moment and bending stress

Absolute maximum quantities of longitudinal bending moment and bending stress at every 5 m

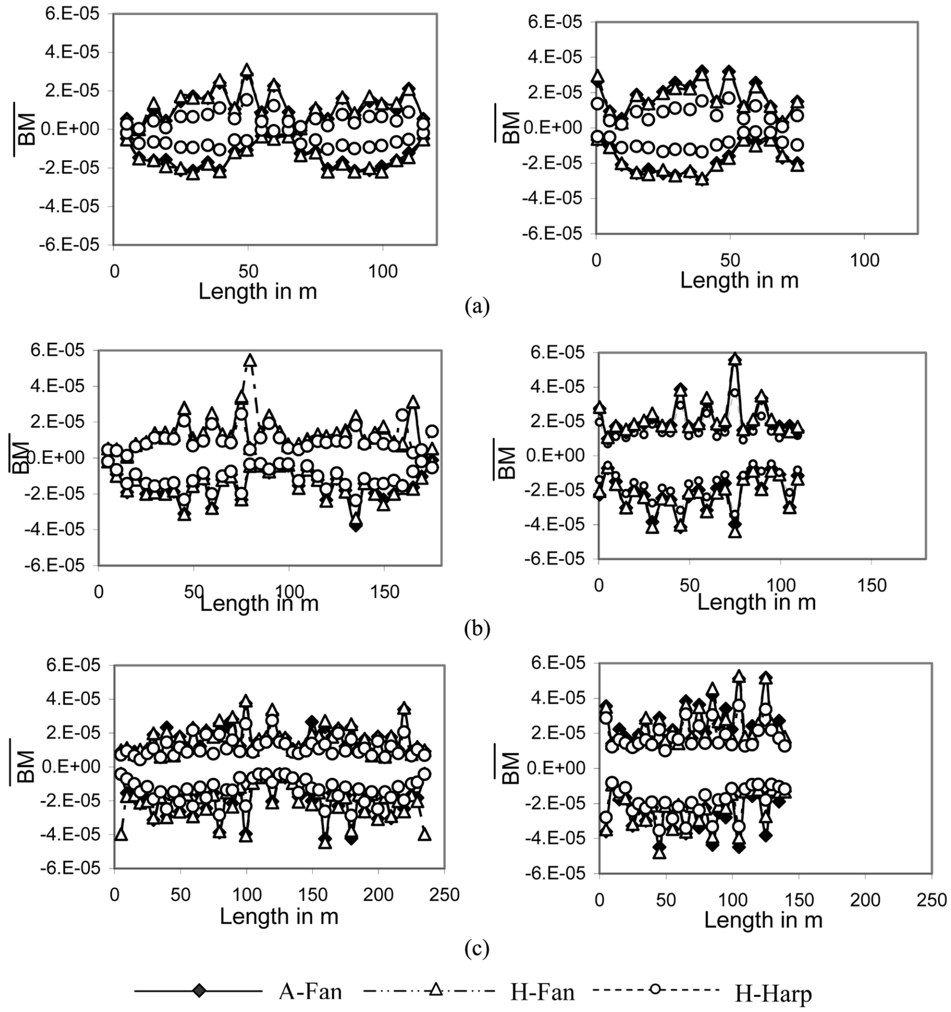


Fig. 7 Non-dimensional bending moment envelop: (a) non-dimensional bending moment envelop for 120 m length bridge under El Centro and synthetic data 2 earthquake, (b) non-dimensional bending moment envelop for 180 m length bridge under El Centro and synthetic data 2 earthquake and (c) non-dimensional bending moment envelop for 240 m length bridge under El Centro and synthetic data 2 earthquake

interval has been plotted as shown in Figs. 7 and 8. Three different geometries (A-Fan, H-Fan and H-Harp) are considered with different length of 120 m, 180 m and 240 m, respectively. As mentioned in previous section, only the responses obtained from El Centro and synthetic data 2 loading histories are reported here.

The important observations are, (i) Bending moment in deck changes with change in cable geometry for small spans, but as length of the bridge increases from 120 m to 240 m difference between bending moment in deck supported by fan shaped cable arrangement and harp shaped cable arrangement reduces monotonically. Pylon shape does not have considerable effect on bending moment in deck. (ii) As expected, magnitude of bending moment would increase with the

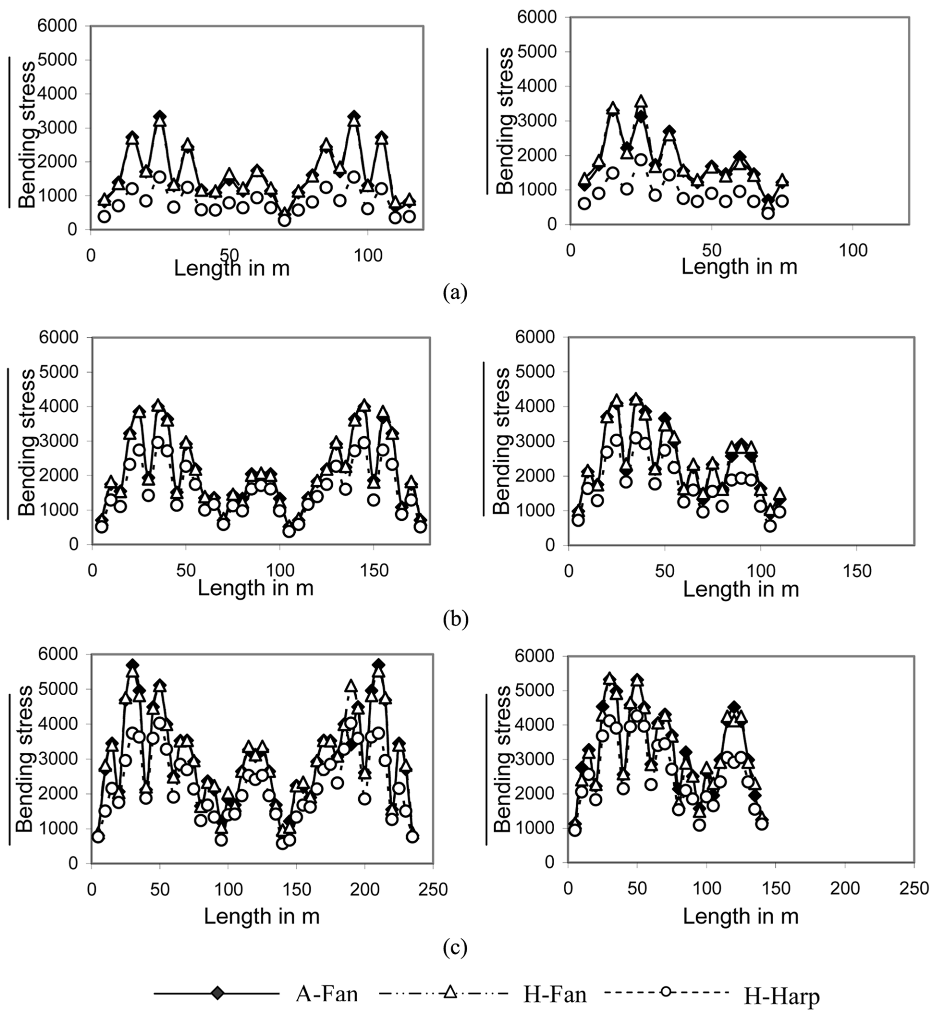


Fig. 8 Non-dimensional bending stress envelop: (a) non-dimensional longitudinal bending stress envelop for 120 m length bridge under El Centro and synthetic data 2 earthquake, (b) non-dimensional longitudinal bending stress envelop for 180 m length bridge under El Centro and synthetic data 2 earthquake and (c) non-dimensional longitudinal bending stress envelop for 240 m length bridge under El Centro and synthetic data 2 earthquake

magnitude of forcing function. (iii) In all the cases, bending moment in H-Harp is the lowest among the bridges considered in the present study irrespective of the type of seismic loading. On the other hand, the H-Fan produces the highest bending moment in deck under seismic loading. (iv) Both positive (sagging) and negative (hogging) bending moments possess the same type of variation with the change in geometry or type of seismic loading. (v) It is significant to mention that the obtained envelop of the longitudinal bending moment at deck does not necessarily correspond to the forces at maximum loading. Under the entire excitation, the maximum forces are identified for developing the envelops which has a meaningful practical implication. (vi) Further, with the increase in length of the cable-stayed bridge, role of geometry on distribution of maximum longitudinal bending moment decreases.

Similarly, the observations of bending stress in longitudinal direction are as, (i) longitudinal bending stress monotonously increases with increase in span. (ii) Similar to the observations on the variation of bending moment, fan shaped cable arrangement gives more longitudinal bending stress in deck than harp shaped cable arrangement. For a given pylon to deck stiffness (as used in this study), pylon shape does not have any effect on change in bending stress. (iii) With the increase in length, the difference in seismic response of different geometries decreases. And (iv) as expected, the maximum bending stress envelop is symmetric about the middle of the span.

4.1.2 Non-dimensional bending stress in transverse direction

Absolute maximum quantities of transverse bending moment at every 5 m interval along the

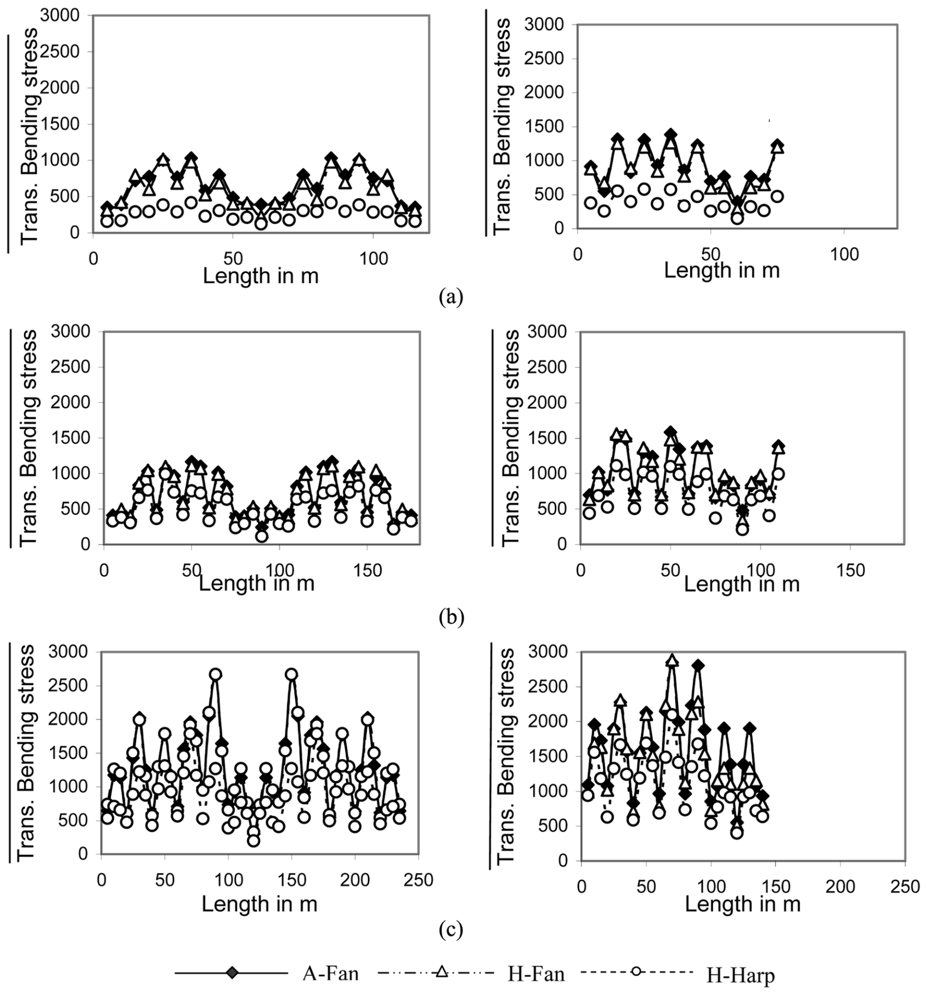


Fig. 9 Non-dimensional transverse bending stress envelop: (a) non-dimensional transverse bending stress envelop for 120 m length bridge under El Centro and synthetic data 2 earthquake, (b) non-dimensional transverse bending stress envelop for 180 m length bridge under El Centro and synthetic data 2 earthquake and (c) non-dimensional transverse bending stress envelop for 240 m length bridge under El Centro and synthetic data 2 earthquake

longitudinal direction of the bridge subjected to the entire seismic load histories have been plotted as shown in Fig. 9. Similar to the pervious study, only the responses obtained from El Centro and synthetic data 2 loading histories are reported here. The important observations are, (i) With increase in span of bridge, distance between two cable supports also increases. Thus, transverse bending stress also increases. (ii) For smaller spans, harp shaped cable arrangement gives lesser transverse bending stress than that obtained from fan shaped cable arrangement. But with higher spans this difference reduces. (iii) With increase in magnitude of forcing function transverse bending stress in deck also increases. (iv) Similar to longitudinal bending stress distribution, transverse bending stress envelop is symmetric about the middle of the span. (v) Increase in transverse bending stress with the span of the bridge is non-linear in nature and the same behaviour is observed for both the earthquake loading.

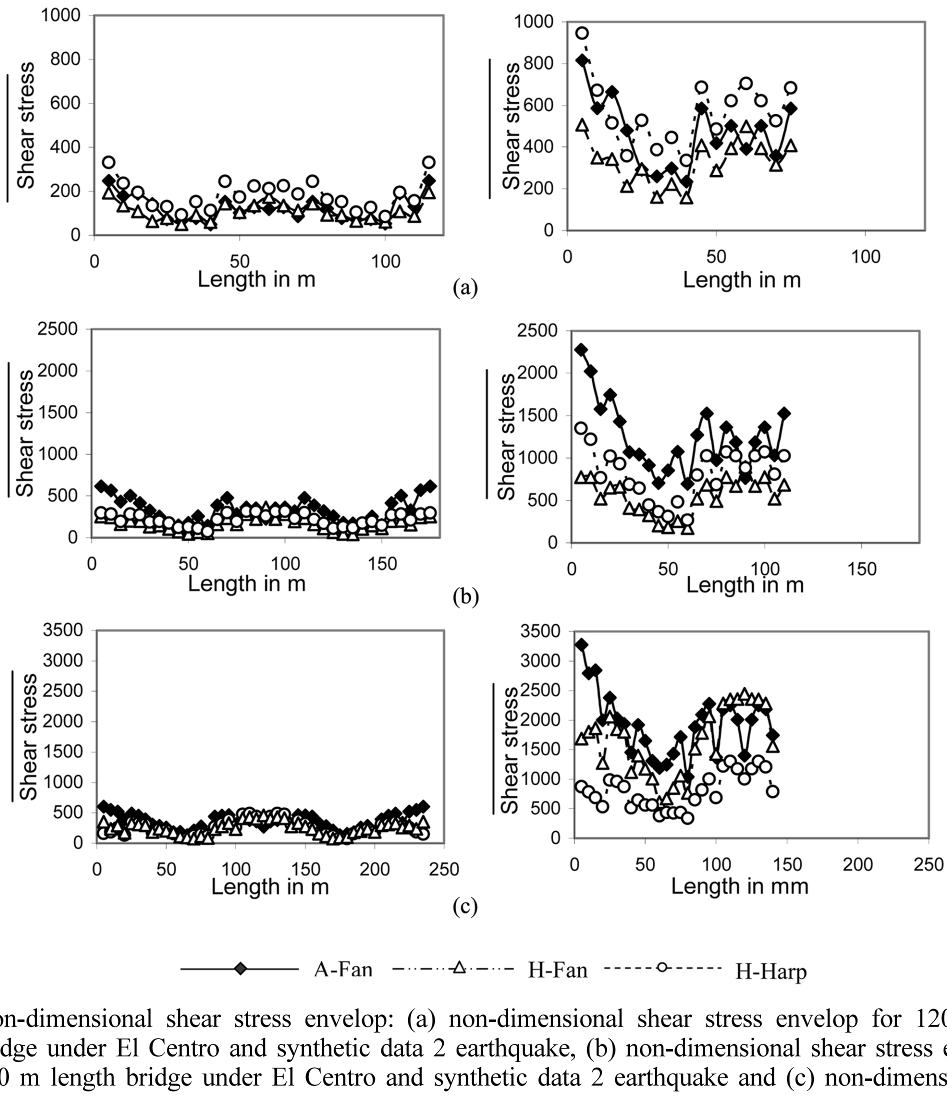


Fig. 10 Non-dimensional shear stress envelop: (a) non-dimensional shear stress envelop for 120 m length bridge under El Centro and synthetic data 2 earthquake, (b) non-dimensional shear stress envelop for 180 m length bridge under El Centro and synthetic data 2 earthquake and (c) non-dimensional shear stress envelop for 240 m length bridge under El Centro and synthetic data 2 earthquake

4.1.3 Non-dimensional shear stress envelop

Absolute maximum quantities of non-dimensional shear stress at every 5m interval have been shown in Fig. 10. Similar to the previous study, only the responses obtained from El Centro and synthetic data 2 loading histories are reported here. The important observations are: (i) with increase in bridge span, shear stress in deck increases from 120 m span to 180 m span but as span is increased from 180 m to 240 m, shear stress in deck does not increase significantly, moreover for some geometries, it is observed that shear stress rather reduces. (ii) Short span cable-stayed bridges with harp shaped cable arrangement produces high shear stress in deck than that is developed in fan shaped cable arrangement. But, as the span of the bridge increases, bridges with A-shaped pylon with fan shaped cable arrangement suffer from more shear stress than H-shaped pylon with fan and harp shaped cable arrangement. (iii) Shear stress in deck changes with change in shape of pylon. A-shape pylon gives more shear stress in deck than H-shaped pylon. For shorter spans, type of cable arrangement has a considerable influence in the magnitude of shear stress developed in the deck. But for longer spans, type of cable arrangement does not have so prominent effect on shear stress in deck. (iv) Critical shear stress develops near the end support and pylon support. This observation is almost similar to any statical analysis but the distribution of shear stress along the span needs to be evaluated from the in-depth dynamic analysis. (v) Magnitude of the shear stress is dependent on the forcing function and it would be significantly prominent with the increase in span. For example, maximum non-dimensional shear stress of H-Harp type cable-stayed bridge of 120m length is 330 and 970 for El Centro and Synthetic data 2, respectively. Whereas, non-dimensional shear stress of H-Harp type cable-stayed bridge of 180 m length is 610 and 2330 for El Centro and Synthetic data 2, respectively.

4.1.4 Non-dimensional shear force envelop along the pylon height

During any seismic design of cable stayed bridge, one of the most important structural responses that is required for every practicing engineer is the distribution of shear force along the height of the pylon. Maximum quantities of non-dimensional shear along the height of the pylon have been shown in Fig. 11. The pylon height is measured from the deck level and positive sign is assigned for upward direction. The notable observations during the seismic analysis are (i) Base shear increases with increase in span. (ii) Shear force transferred at top of the pylon does not increase much with increase in span. (iii) At the deck level, there is large amount of reduction in shear force in pylon. This indicates that a large portion of shear force in pylon is transferred to the deck and the connecting beam between two pylon legs in form of axial load. (iv) Shear force developed in the pylon with harp shaped cable arrangement is less as compared to that in pylon with fan shaped cable arrangement. (v) In A-shaped pylon, reduction in shear force from bottom to top is very sharp compared to H-shaped pylon. (vi) It is observed that only in harp shaped cable arrangement with H-shaped pylon with span of 120 m, shear force in pylon increases at deck beam level.

4.2 Study on effect of variation in stiffness of pylon

A general concept of reduction of shear force along the height of the pylon and the development of maximum shear at base of pylon may not hold good for every situation when a complex structure like cable-stayed bridge is analysed under seismic loading. From the figure of shear force envelop in pylon (Fig. 11), it is observed that only in case of 120 m span and harp cable-stayed bridge shear force increases at the deck level and interestingly, shear force produced at deck level is

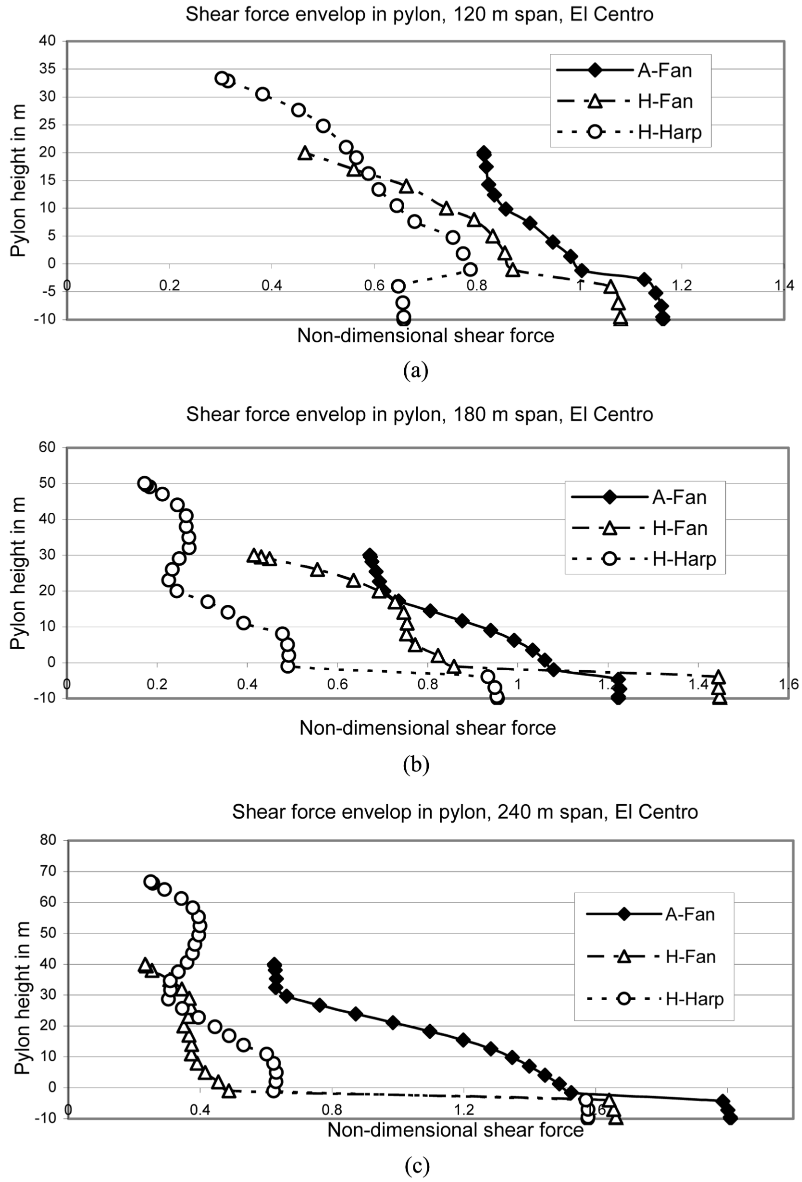


Fig. 11 Non-dimensional shear force envelop along the pylon height: (a) non-dimensional shear force envelop for 120 m span bridge under El Centro earthquake, (b) non-dimensional shear force envelop for 180 m span bridge under El Centro earthquake and (c) non-dimensional shear force envelop for 240 m span bridge under El Centro earthquake

more than that developed at base. To study the reason of this behaviour, same model is studied with various stiffness of pylon. A wide range of stiffness ratio between bridge deck to pylon (0.2 to 10) has been studied under El Centro loading. The result from the study is shown in Figure 12. During analysis, mass distribution of the pylon has accordingly been changed with the change of stiffness of the pylon. From Fig. 12, it is observed that with increase in stiffness of pylon shear force at deck

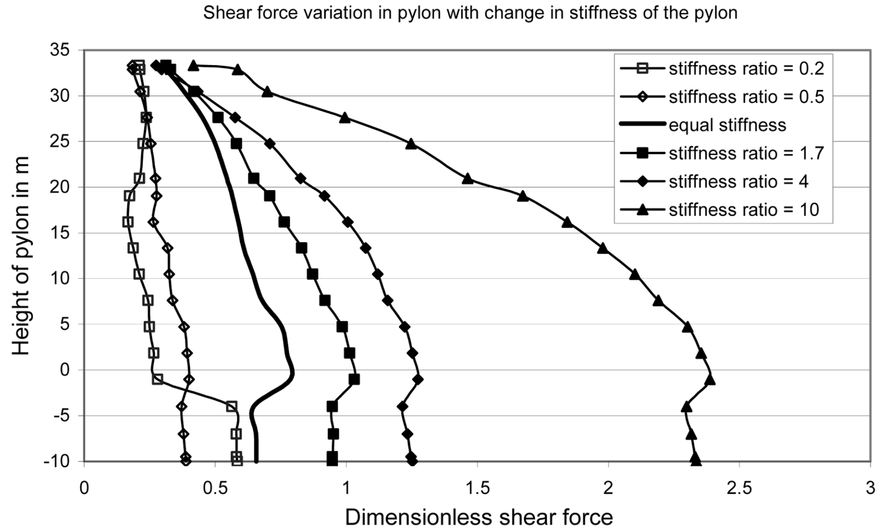


Fig. 12 Shear force variation in pylon with change in stiffness of the pylon

level increases, but when pylon stiffness is 0.2 times of the original, then this behaviour changes. This shows that when pylon stiffness is reduced, deck to pylon stiffness ratio increases. As the deck and pylon connection is fixed and deck possesses more stiffness in comparison with pylon, deck acts as a support to the pylon and shear force at deck level reduces and vice versa. This behaviour is quite complex and needs to be noted during any seismic analysis and design of short span cable-stayed bridge.

5. Conclusions

Lightweight structures such as cable-stayed bridges where increase in span does not proportionally reflect in mass of the structure are very sensitive to aerodynamic behaviour. When cable-stayed bridges are constructed for small spans they are more sensitive to seismic vibrations than aerodynamic vibrations. In the present study, cable-stayed bridge models with single pylon at centre and lengths ranging from 120 m to 240 m are investigated. To understand the dynamic behaviour pattern, first free vibration response of all models is characterised. Further, their responses under three different load-time histories are studied to generalise the response pattern. Among the load histories, one real earthquake and two synthetic earthquakes are considered. During free vibrational analysis, it is observed that A shape pylon mode shapes are distinct and no combination of modes is observed. Due to higher stiffness of pylon, pylon and deck modes are clear and separate, whereas, for H-shaped pylon, pylon stiffness is less than A-shaped pylon and pylon transverse mode is observed at very low frequency. Since in harp shaped cable arrangement pylon height is more, pylon predominant mode occurs first and at very low frequency. Further to note that before rigorous seismic analysis, determination of frequency content of the seismic loading is useful since the frequency content of the load history is the key parameter to determine the structural response. In the present study, the most important structural responses, i.e. longitudinal bending moment, transverse bending moment, shear stress in deck and shear force envelop along the height of the pylon are discussed. It is to mention that among the bridges considered in this study, H-Harp produces least bending stress in both longitudinal and transverse direction whereas the same produces maximum shear stress. So, depending on the critical design parameter, the geometry of the cable-stayed bridge has to be decided. It is interesting to note that during seismic loading, shear force at deck level may be more than that developed at the pylon base depending on the deck to pylon stiffness ratio. This phenomenon has to be carefully investigated before any design. Therefore, the structural geometry and the cable pattern have crucial role on the response of the cable stayed bridges subjected to seismic loading and those provide a scope for choosing the appropriate geometry which can be more proper under seismic loading.

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