TO STUDY THE EFFECT OF PYLON SHAPES ON STATIC AND DYNAMIC RESPONSE OF CABLE-STAYED BRIDGES

By

NIYITEGEKA SIMON
141160720011

Guided by
Dr. Siddharth G. Shah
(Associate Professor & Head, Civil Engineering Department)

A Thesis Submitted to
Gujarat Technological University in Partial Fulfillment of the Requirements for the Master of Engineering Degree in **Civil - Structural Engineering**

May, 2016

Marwadi Education Foundation Group of Institutions, Rajkot-360003, Gujarat, India

CERTIFICATE

This is to certify that thesis work embodied in this report entitled "**To Study the Effect of Pylon Shapes on Static and Dynamic Response of Cable-Stayed Bridges**" was carried out by **Mr. Niyitegeka Simon (141160720011)**, at **Marwadi Education Foundation Group of Institutions (116)** for partial fulfillment of Master of Engineering degree in **Structural Engineering** to be awarded by Gujarat Technological University. This research work has been carried out under my guidance and supervision and it is up to my satisfaction.

Date: 12/05/2016

Place: Rajkot

Signature and Name of Guide

Dr. Siddharth G. Shah

Signature and Name of Principal

Dr. Sarang Pande

Seal of Institute

COMPLIANCE CERTIFICATE

This is to certify that research work embodied in this thesis entitled "**To Study the Effect of Pylon Shapes on Static and Dynamic Response of Cable-Stayed Bridges** "was carried out by **Mr. Niyitegeka Simon** (**141160720011**) at **Marwadi Education Foundation Group of Institutions (116)** for partial fulfillment of Master of Engineering degree to be awarded by Gujarat Technological University. He has complied with the comments given by the Dissertation phase-I as well as Mid Semester Thesis Reviewer to my satisfaction.

Date: 12/05/2016

Place: Rajkot

Signature and Name of Student	Signature and Name of Guide
Niyitegeka Simon	**Dr. Siddharth G. Shah**
(141160720011)	

PAPER PUBLICATION CERTIFICATE

This is to certify that research work embodied in this thesis entitled "**To Study the Effect of Pylon Shapes on Static and Dynamic Response of Cable-Stayed Bridges** "was carried out by **Mr. Niyitegeka Simon** (**141160720011**) at **Marwadi Education Foundation Group of Institutions (116)** for partial fulfillment of Master of Engineering degree to be awarded by Gujarat Technological University. He has published article entitled **"Influence of Cable Layout on the Seismic Response of Cable-Stayed Bridges"** for publication by the **International Journal for Scientific Research & Development** (IJSRD) in November 2015.

Date: 12/05/2016

Place: Rajkot

Signature and Name of Student

Niyitegeka Simon
(141160720011)

Signature and Name of Guide

Dr. Siddharth G. Shah

Signature and Name of Principal

Dr. Sarang Pande

Seal of the Institute

THESIS APPROVAL CERTIFICATE

This is to certify that research work embodied in this thesis entitled "**To Study the Effect of Pylon Shapes on Static and Dynamic Response of Cable-Stayed Bridges** "carried out by **Mr. Niyitegeka Simon** (**141160720011**) at **Marwadi Education Foundation Group of Institutions (116)** is approved for the degree of Master of Engineering with specialization of Structural Engineering by Gujarat Technological University.

Date:

Place:

Examiners Signature and Name:

……………………………….	……………………………….
( )	( )

DECLARATION OF ORIGINALITY

We hereby certify that we are the sole authors of this thesis and that neither any part of this thesis nor the whole of the thesis has been submitted for a degree to any other University or Institution.

We certify that, to the best of our knowledge, the current thesis does not infringe upon anyone's copyright nor violate any proprietary rights and that any ideas, techniques, quotations or any other material from the work of other people included in our thesis, published or otherwise, are fully acknowledged in accordance with the standard referencing practices. Furthermore, to the extent that we have included copyrighted material that surpasses the boundary of fair dealing within the meaning of the Indian Copyright (Amendment) Act 2012, we certify that we have obtained a written permission from the copyright owner(s) to include such material(s) in the current thesis and have included copies of such copyright clearances to our appendix.

We declare that this is a true copy of thesis, including any final revisions, as approved by thesis review committee.

We have checked write up of the present thesis using anti-plagiarism database and it is in allowable limit. Even though later on in case of any complaint pertaining of plagiarism, we are sole responsible for the same and we understand that as per UGC norms, University can even revoke Master of Engineering degree conferred to the student submitting this thesis.

Date: 12/05/2016

Signature of Student: **Signature of Guide:**

Name of Student: Niyitegeka Simon **Name of Guide: Dr.Siddharth G.Shah**

Enrollment No: 141160720011 **Institute Code: 116**

ACKNOWLEDGEMENTS

It is my great pleasure to avail this opportunity to express my sincere gratitude to all respective persons who helped and contributed to make my research in reality and possibility.

I would like to express my sincere gratitude to my guide and Head of department, **Dr. Siddharth G. Shah** for all efforts, guidance, encouragement and interest he has taken for the vital completion of this work. It was an invaluable learning experience for me to be one of his students. From him, I have gained not only extensive knowledge, but also a careful research attitude.

With deepest appreciation, I am grateful to all the faculty members of the department of civil engineering including Prof.Tarak Vora, Prof Ankur Bhogayata, Prof Mazhar Dhankot, etc who imparted me with the knowledge and skills required for the successful completion of the studies.
I am thankful to Marwadi Education Foundation Group of Institutions for providing me such a good platform.

I render my respect to all my family members for giving me mental support and inspiration for carrying out my research work.

Let me save this chance to express my grateful thanks to my classmates and friends for the friendship, hospitality and helps during the two-year period at the college.

Niyitegeka Simon

Table of contents

List of tables and figures

Tables

Figures

List of symbols and abbreviations

UTS: Ultimate tensile strength

t_w : thickness of web

t_f: thickness of flange

IRC: Indian Roads Congress

AASHTO: American Association of State Highway and Transportation Officials

ASCE: American Society of Civil Engineers

DL: dead load

SVL: Static vehicular load

MVL: Moving vehicular load

WL: wind load

m: meter

kN: kilo-Newton

3-D: Three dimension

Abstract

Cable-stayed bridges have become the form of choice for bridges in the medium to long-span (200 to 2000m) range over the past few decades. They represent the key points along infrastructure networks and requires a substantial knowledge of their structural response under vehicular and wind load.

Nowadays, the increase of cable-stayed bridge usage implies the study of static and dynamic behaviors of these types of structures with respect to different loadings. Moreover, in cable-stayed bridges there is a variety of shapes of pylon or tower, pylon shape can have a potential to reduce or increase the response of the bridge under certain types of loading. In addition, as per structural analysis and design requirements of structures, the shape of the structural member or the whole structure which gives less response to applied loading is preferred; for this reason, a pylon shape for which the responses quantities (bending moments, axial forces, deflections, pylon base reaction, etc) are less should be recommended.

In this study, investigation of the effect of pylon shape on the response of cable-stayed bridges under static and dynamic loading has been done. The software SAP2000 version 14 has been used for modeling and analysis. Analysis of dead load, static and moving vehicular load and wind load has been performed. The results have shown that most of the various response quantities for inverted -Y pylon shape are less as compared to structural responses of other pylon shapes considered in this study. It has been concluded that inverted-Y pylon shape is statically and dynamically stronger than other pylon shapes under the action of loads considered in this study.

CHAPTER 1. INTRODUCTION

1.1. Brief description of Cable-Stayed Bridges

A cable-stayed bridge is a type of bridge that consists of one or more columns (normally referred to as towers or pylons) with cables supporting the bridge deck; the cables are anchored to the deck and pylon.

Cable-stayed bridges are indeterminate structures. The superstructure behaves as a continuous beam elastically upheld by the cables, which are connected to one or two towers. The structural system consists of three main structural sub-systems: Stiffening girder, tower, and inclined cables. The interrelation of these parts makes the structural behavior of cable-stayed bridges effective for long-span structures[24].

Cable-stayed bridges have been proven to be technically, economically, aesthetically and aerodynamically superior to the classical suspension bridges[6] for span in the range of 700 to 1500m. They are mainly used to cover large spans. The development of this structural system is due to advances in materials, engineering analysis and design, and construction methodology.

The load transmission mechanism in cable-stayed bridge system behaves in the following manner: The stiffening girder transmits the load to the tower through the cables, which are always in tension. The stiffening girder is subjected to bending and axial loading. The tower transmits the load to the foundation under mainly axial action.

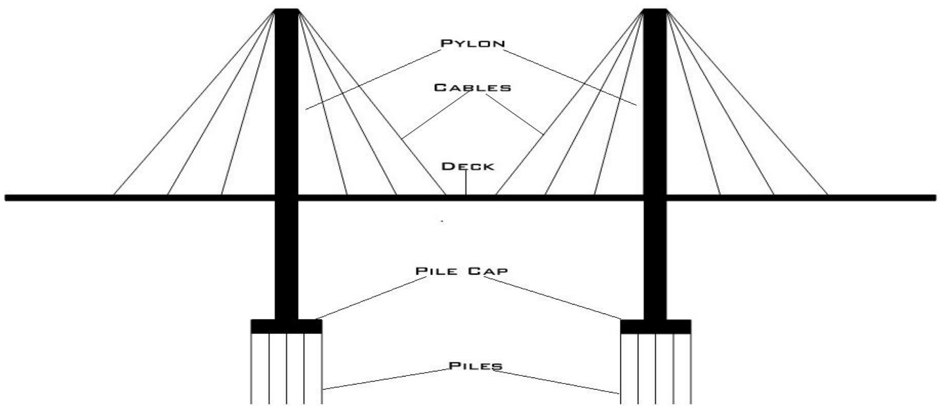


Figure 1.1. Main components of a cable-stayed bridge (Longitudinal view), (source: https://en.wikipedia.org)

1.2. Advantages of Cable-Stayed Bridges

Cable stayed-bridge is an innovative structure and is preferred to conventional steel suspension bridges for long spans mainly due to the reduction in moments in the stiffening girders resulting in smaller section of the girders leading to economy in overall costs. The ratio of maximum bending moment in the cable stayed girder is nearly 1/10th of that of the conventional continuous girder system.

Cable-stayed bridges when compared with suspension bridges have the advantages of ease of construction, lower cost since anchorages are not required and small size of substructures; the general trend suggests that cable-stayed bridges with longer span length are becoming possible and economically more advantageous than suspension bridges[25].

1.3. Cables

Cables are the most important elements in cable-stayed bridges; they carry the load from the superstructure to the tower and to the backstay cable anchorages. The basic need is for a high-strength material with high Young Modulus and good resistance to fatigue. In order to carry the heavy loads for a long life-time, strands are usually preferred to ropes in cable-stayed bridges.

Diverse sorts of cables are utilized as a part of cable-stayed bridges; their structure and arrangement rely upon the way singular wires are gathered. The steel utilized for the cables is stronger than ordinary steel. A strand is generally composed of seven wires, helically formed around a center wire; the wire diameter is between 3 and 7 mm. The strands are closely packed together and typically bounded with a helical strand. Since the action of stay cables becomes inefficient with decreasing inclination, the stay inclination[7] is usually taken as 25° to 65°.

1.3.1. Types of cable

There are four different strand configurations[7]:

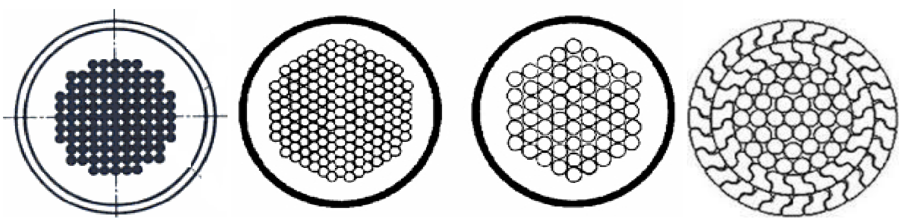
- Parallel-bar cables
- Parallel-wire cables
- Stranded cables
- Locked-coil cables

1. Parallel-bar cables, The Ultimate Tensile Strength (UTS) is 670MPa, and Young Modulus (E_o) is 165, 000MPa, the pre-stressing limit is 0.55 UTS. The fatigue strength is low.

2. Parallel-wire cables, UTS is 1, 800MPa and Young Modulus is 190, 000MPa.

3. stranded cables, UTS is 1, 600MPa and Young Modulus is 200,0000MPa. Galvanized wire cables UTS is 1, 570MPa and Young Modulus is 190, 000MPa.

4. Locked-coil cables, they have several layers of round wires, only the outer layer may be galvanized. UTS is 1, 500MPa and the Young Modulus is 170, 000MPa, the pre-stressing limit is 0.55 UTS.



Parallel-bar cables | Parallel-wire cables | Stranded cables | Locked-coil cables

Figure 1.2. Types of cables[7]

One of the basic requirements is to limit the pre-stress level in cables. Considering the great influence of dead loads on the overall loading of the structure, the allowable stress is limited to $\sigma = 0.4\sigma_u$.

Each cable type has advantages and disadvantages. For example, locked-coil cables have variable stress-strain behavior and low fatigue strength at the sockets. Therefore, they are less frequently used. It is better to choose a type of cable where the modulus of elasticity is high and constant. The parallel-wire cable is the most commonly cable type used[27].

1.3.2. Cable arrangements

Cable-stayed bridges can be classified with respect to the different longitudinal and transverse cable arrangements. Cable configuration affects the structural performance of the bridge and the method of erection and the economics[26].

1.3.2.1. Longitudinal Arrangement

A number of considerations is involved in deciding the cable arrangement, mostly it depends on the length of span, type of loadings, numbers of roadway lanes, height of pylon, economy and the designer individual sense of proportion and aesthetics.

.

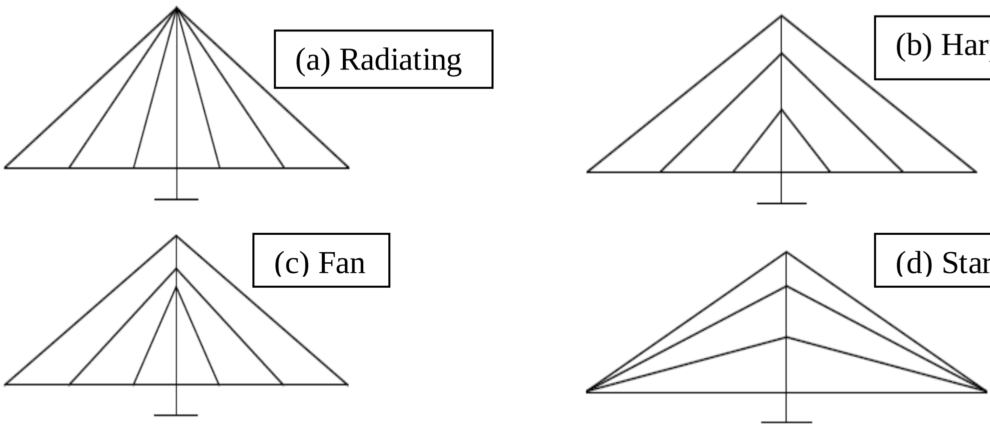


Figure 1.3. Longitudinal cable arrangements

Generally, there are four cable configurations in cable-stayed bridges as shown in fig.1.3, and it is assumed that all the configurations are applicable to either the single or double planar cable systems. These basic configurations are referred to as radiating, harp, fan, and star systems[1].

The radiating type, the cables meet at a common node at the top of the pylon. In the harp arrangement, the cables are parallel and equidistant from each other. The fan type is a combination of the radiating and the harp types. The cables emanate from the top of the pylon with equal spacing and connect with equal spacing along the superstructure. In the star arrangement, the cables meet the pylon at different heights and then converge on each side of the tower to intersect at a common point usually located over the abutment or end pier of the bridge.

1.3.2.2. Transverse Arrangement

In the transverse arrangement the classification is made according to the positioning of the cables in different planes. There are basically two types[27]:

Single-plane system: This type is composed of a single cable layout along the longitudinal axis of the superstructure. This type of layout is governed by torsional behavior. In order to resist the torsion force, the main girder must have adequate torsional stiffness.

Two-plane system: If the tower is of the shape of an H-pylon, the layout is a two-plane vertical system. The transverse layout has two options for the anchorage. The anchorage is located either outside of the deck structure or inside the main girder.

1.4. Pylons

The pylon is the main feature that expresses the visual form of any cable-stayed bridge, giving an opportunity to impart a distinctive style to the design. The design of the pylon should adjust to the different stay cable layouts, fit the topography and geology of the bridge site and carry the forces economically.

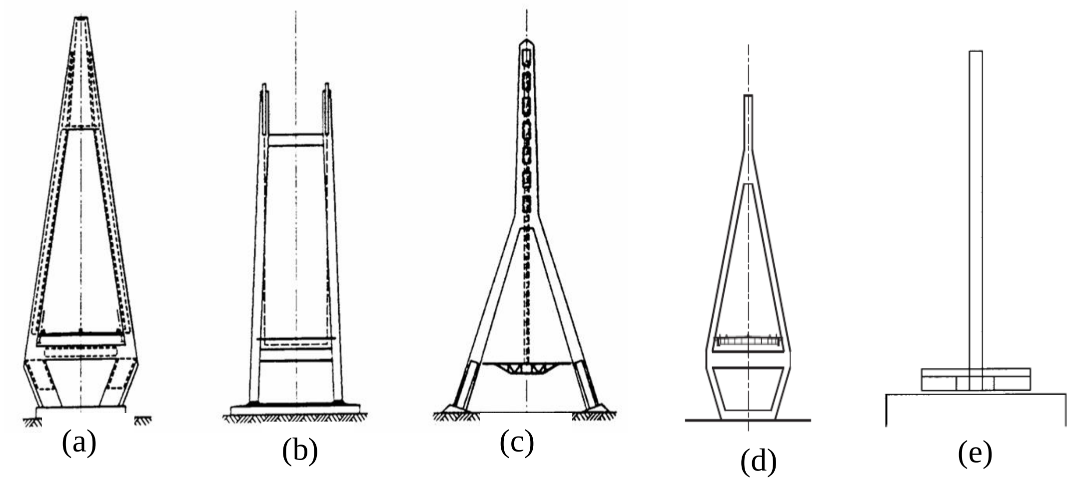


Figure 1.4. Shapes of pylon: (a)A-shape, (b)H-shape,(c) inverted Y-shape, (d)Diamond-shape and (e) single tower[24]

The essential function of the pylon is to transmit the forces arising from anchoring the stays and these forces will govern the pylon design. The pylon should ideally carry these forces by axial compression where possible, such that any eccentricity of loading is minimized[7]. The tower height is normally kept around 1/4 to 1/5 of the main span.

In general, the shape of the tower is governed by the required height and the natural loading conditions, such as seismic zones and wind criteria. The towers are subjected to axial forces, thus they must provide resistance to buckling[27].

Box-sections are most frequently used for the towers. They can be fabricated out of steel or reinforced or prestressed concrete. Concrete towers are more common than steel towers because they allow more freedom of shaping, and are more economical[26]**.**

H-Type, **VIDYASAGAR SETU Bridge, Kolkata, India**
(source: https://en.wikipedia.org)

A-Type, **Megyeri Bridge, Hungary**
(source: https://en.wikipedia.org)

Inverted Y-Type, **Third Millennium John Paul II Bridge, Poland** (source: https://en.wikipedia.org)

Diamond-Type, Arthur Ravenel Bridge, South Carolina_(source: https://en.wikipedia.org)

Figure 1.5. Examples of Pylon shapes in actual cable-stayed bridges

1.4.1. Pylon geometry

The A-shape pylon is reasonable for inclined stay. A variety of the A-shape is the inverted Y-frame where the vertical leg, containing the stay grapples, reaches out over the bifurcation point. Examples of the inverted Y-shape are the pylons of the Normandy Bridge over the River Seine, France and the Rama VIII Bridge, Bangkok, Thailand. Excessive land take, due to the wide pylon footprint, can occur with this form of pylon, when a high navigation clearance to the deck is required. This has been overcome by breaking the pylon legs at or just below the deck to produce inward-leaning legs to the foundation to form a diamond configuration. However, this modified arrangement is considerably less stiff when resisting transverse wind or seismic forces and this can result in a significant increase in the deflection of the pylon. This deflection can be mitigated only with a considerable increase in the stiffness of the lower section of the pylon leg below the deck. Nevertheless, this arrangement was favored for the pylons of the Tatara Bridge in Japan spanning 890m and the Industrial Ring Road Bridges in Bangkok[7].

1.5. Deck

In cable-stayed bridges, concrete deck systems, steel deck systems and composite deck systems are used. Steel decks are about twenty percent lighter than concrete decks. Concrete decks are more common in multiple stay bridges. The choice of the material is in function of the required stiffness, the method of erection, and the economics[26].

The determination of the deck structure will for the most part be founded on an economic assessment of the conceivable options. The essential factors impacting the decision of deck will be the length of the main span and deck width. Other Different factors, for example the cost of foundations, the local availability of materials or labor skills and the competitive conditions at the time of tendering may likewise have impact over the costs[7].

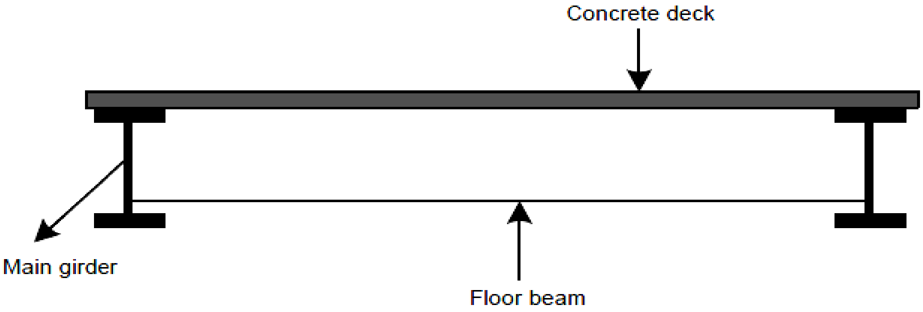


Figure 1.6. Composite deck

The most frequently used deck system is the box section deck because it provides convenient anchorages, and has significant torsion properties[29].

1.6. Need of the study

Long-span bridges are becoming popular and cable-stayed bridge is promising bridge in this category in terms of strength, stiffness, serviceability, aesthetics and economy. However, due to high vibrations caused by wind load and moving vehicle, the damages may occur if these types of loads have not been properly studied in the analysis. This can be explained by the famous failure of Tacoma Narrows Bridge, 1940 in USA, this suspension bridge collapsed due to excessive vibration of the deck induced by the wind. Moreover, in cable-stayed bridges there is a variety of shapes of pylon or tower. Pylon shape can have a potential to reduce or increase the response of the bridge under certain types of loading. In addition, as per structural analysis and design requirements of structures, the shape of the structural member or the whole structure which possesses more resistance to applied loading is preferred.

Therefore it is very motivating to conduct a study on cable-stayed bridge for different loadings with various pylon shapes in order to come up with the optimal shape of pylon.

1.7. Objectives of the study

The objectives of the current study are:

- To evaluate the response of cable-stayed bridges under dead load, vehicular load and wind load.
- To study the effect of pylon shapes on the static and dynamic response of cable-stayed bridges.
- Out of various shapes (H, A, Y, Diamond), to suggest the best shape of pylon giving less deformation and stresses in other members when subject to various loading.

1.8. Scope of the study

This study is limited to modeling and analyzing the cable-stayed bridge for static and dynamic loading. Specifically, the following points are focused on:

- To model a cable-stayed bridge using a software (SAP2000). Here, four pylon shapes are considered.

- To perform modeling and analysis for static and dynamic loading on cable-stayed bridge by considering dead load, vehicular load and wind.
- To make comparison of response quantities (Bending Moment, Axial Force, Deflection, base reaction, etc) for various pylon shapes namely: H-shape, A-shape, Inverted-Y shape and Diamond shape.

More details of the scope are given in the following chart:

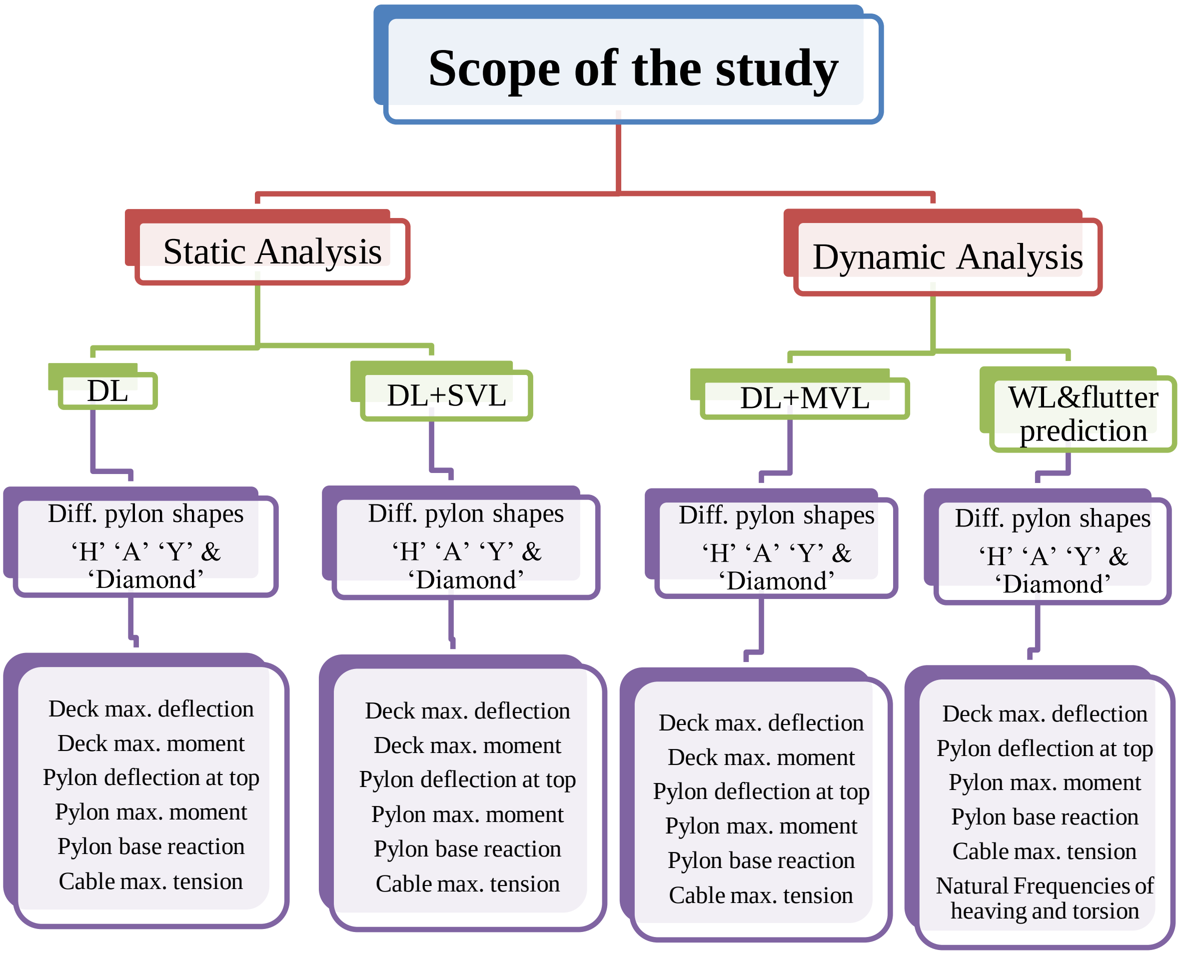


Figure 1.7. Detailed Scope of the study

CHAPTER 2. LITERATURE REVIEW

2.1. Research Papers

The evolution of Cable-stayed bridges goes back to numerous hundreds of years. As right on time as 1784, a German craftsman C. J. Loscher, has built a timber bridge of this type. But due to lack of technical knowledge and absence of proper construction materials, some of the early cable-stayed bridges fizzled. The first modern bridge structures were a combination of a suspension and stayed system. They were built toward the end of the eighteen-century in the United States and in England[24]. Dischinger F. identified the need to increase the stress in the cables so as to reduce the sag effect in the stiffness. This progression gave the impulse to modern cable-based structures. In 1955, he constructed the Stromsund bridge, located in Sweden, which is considered as the first modern cable-stayed bridge. It is important to note that with the use of high speed digital computers and advanced methods of analysis, cable-stayed bridges can now be analyzed with a high degree of accuracy[1]. Nowadays, Cable-stayed bridges traverse spans running from 200 to much more than 1000m , examples include Russky Bridge: 1104m, Russia, Sutong Bridge: 1088m, China[30].

Several methods have been employed previously in the static analysis of cable-stayed bridges. In a simplified approach to the solution, some analysts have assumed that a cable-stayed bridge is a linear structural system with the cables acting as linear tension members[32]. Actually, the cables exhibit non-linear behavior due to the change in their sag caused by their own dead weight that occurs with changing the tension in these cables. Neglecting this effect in the static analysis may produce errors[24] of about 15%. Analysis of cable-stayed bridges as three-dimensional structures is a relatively complex problem[33]. However, some investigations used simplifying assumptions regarding the boundary conditions of the bridge deck, to reduce the problem to a two-dimensional analysis[19]. Very few researchers have performed the analysis using influence lines. Most of them considered the response in a linear range, but some of them performed a qasi-nonlinear analysis using the influence lines obtained from linear analysis to locate the approximate position of live loads, and then performing nonlinear static analysis[33]. Protte and Tross have proposed simulation method for computerized cable stayed bridge design[18]. The main system was chosen as continuous main girder with independent towers having fixed support and cables as redundant. Flexibility method was used for computing continuous girder analysis. Smith presented simplified linear analysis[34], which includes the beam on elastic support analogy and superposition of the effects of the tower rotating, the cable stretching, and tower shortening. Smith also developed a mixed (force-displacement) method of analysis of double plane structures where the unknowns in the matrix method include the forces as well as displacements[19]. Troitsky and Lazar used flexibility method for linear analysis due to dead load and live load, and for calculating the post tensioning forces in cables[20]. In order to obtain a well-conditioned banded flexibility matrix, the bending moments at fixed and flexible supports were chosen as redundant.

Tang presented reduction (transfer matrix) method dealing with both linear and nonlinear analysis of cable stayed bridges[21]. Kajita and Cheung have successfully applied the finite element method to the static and dynamic analysis of cable-stayed bridges and the three-dimensional characteristics have been fully demonstrated[32]. It is concluded that the plane frame analysis commonly used in the design is insufficient to give all aspects of behavior of the cable-stayed bridges.

The dynamic analysis of this type of structures became more complicated by the fact that it could behave in a nonlinear manner due to the change in the tension of the cables as the bridge vibrates.

In 1991, Wilson and Gravelle, presented techniques for modeling cable-stayed bridges for dynamic analysis by using linear finite element model. In their study findings, a linear model can work well for analyzing cable-stayed bridges[8].

Wilson and Liu indicated that linear elastic finite element model appears to be capable of capturing much of the complex dynamic behavior of the cable-stayed bridge with very good accuracy, when compared to the low level dynamic responses induced by ambient wind and traffic excitations[22]. Investigation of both aerodynamic stability and earthquake response of cable-stayed bridges are dependent upon knowledge of the dynamic characteristics.

Abdel-Ghaffar and Nazmy[1] concluded that linear dynamic analysis is adequate for cable-stayed bridges having centre span up to 450m. Two-dimensional dynamic analysis is not adequate for this type of structures.

Abdel-Ghaffar and Khalifa[2] presented that as the cable-stayed bridge span length is getting longer, the high strength materials are required to be used in the construction for having shallow deck. In such a case the vibrations due to traffic are playing an important role.

Wang & Huang reported that in recent years, considerable efforts have been made for better understanding the dynamic behavior of bridges with moving loads across the rough bridge decks[5]. Most of these previous studies were concentrated on the dynamic analysis of beam/girder type bridges. Only a few have studied the impact of vehicles on cable-stayed bridges.

The study on dynamic response analysis of vehicle-bridge system for cable-stayed bridge in strong windy environment has shown that the vertical displacement response of the bridge under strong wind is affected significantly by the wind load, and lateral displacement response is also controlled by the wind load. Carrying out dynamic response study of vehicles-bridge system under wind circumstance has been found to be very necessary[12].

The number of cables has a great influence on the cable tension as the maximum cables tension decreases rapidly with the increase in the number of cables[3]; also, the girder deflections and the tower moments decrease as the cable stiffness increases[1]. Wei-Xin Ren studied[10] the ultimate behavior of long-span cable-stayed bridges and concluded that Geometric nonlinearity of the bridge has a minor effect on the static behavior under normal design loads (dead loads and live loads). Separating the girder from the towers decreases the static ultimate load-carrying capacity of the long-span cable-stayed bridges. With regard to live loads, in the most cases, live load case (uniformly distributed only in the central span) is more harmful to the long-span cable-stayed bridges.

The dynamic effects from vehicles are relatively small for long-span bridges and the effects from vehicle speeds and road roughness conditions can be neglected; the dynamic stress ranges and numbers of cycles increase with the wind velocity; and the combined dynamic effects from winds and vehicles might result in serious fatigue problems for long-span bridges, while the traffic or wind loads alone are not able to induce serious fatigue problems[11].

In 2009, SHENG Hongfei and his co-authors studied[12] "Dynamic Response Analysis of Vehicle-bridge System for Cable-stayed bridge in Strong Windy Environment", and the conclusions were drawn as follows: The vertical displacement response of the bridge under strong winds is affected significantly by the wind load, and lateral displacement response is also controlled by the wind load, Under low wind speed (10 m/s) the acceleration vibration of the bridge is mostly affected by wind load, the impact of bridge vibration caused by wind load on vertical displacement and acceleration response of the vehicle is obvious. The safety of vehicle and bridge would be threatened under the strong wind condition. Performing dynamic response study of vehicles-bridge system under wind load is very necessary.

Hongyi Li,Jerry, Leslaw[13] investigated the dynamic response of a highway bridge subjected to moving vehicles. Multiple full-scale load tests were performed on a selected highway bridge. The bridge was dynamically excited by two fully loaded trucks, and the strain, acceleration, and displacement at selected points were recorded for the investigation of the bridge's dynamic response. Experimental data were compared with simplified vehicle and bridge finite element models. The vehicle was represented as a three-dimensional mass–spring–damper system with eleven degrees of freedom, and the bridge was modeled as a combination of plate and beam elements that characterize the slab and girders, respectively. The equations of motion were formulated with physical components for the vehicle and modal components for the bridge. The method of central deference was used to solve the coupled equations. It was found that the numerical analysis matched well with the experimental data and was used to successfully explain critical dynamic.

The impact factor was found to increase with the vehicle speed, and strongly depends on the road surface conditions. The rougher the surface is, the more rapidly the impact factor increases with the speed. It was reported that special attention should be put on bridge-vehicle resonation. If the fundamental frequency of the vehicle is close to that of the bridge, the impact factor increases significantly.

2.2. Summary of Literature Review

It can be summarized, based on the previous studies and above literature review, that:

- Advancement of analysis methods and high speed digital computers has been the solution for efficient analysis of cable-stayed bridges.
- A computer program/software is very crucial in accurate analysis of cable-stayed bridges.
- Geometric nonlinearity of cable-stayed bridges has a minor effect on the static behavior under normal design loads (dead loads and live loads).
- Linear elastic finite element model appeared to be capable of capturing much of the complex dynamic behavior of cable-stayed bridges with very good accuracy.
- Flexibility and stiffness methods of analysis are the commonly methods used to analyze cable-stayed bridges.

Furthermore, in all the aforementioned studies on the behavior of cable-stayed bridges, a number of research works has been done on cable-stayed bridges considering mainly cable configuration, deck composition and different loadings, etc. However, literatures dealing with the pylon shape as a parameter of study and comparison of structural responses of different pylon shapes in cable-stayed bridges are relatively scarce.

CHAPTER 3. METHODOLOGY

3.1. Analysis of cable-stayed bridges

3.1.1. Preliminary design

Cable-stayed bridge is many times statically indeterminate structure where the deck acts as a continuous beam with a number of elastic supports with varying stiffness, because of the large degree of indeterminateness of these structures, exact calculation by manual procedures is virtually an impossible task[24]. So computer aided methods are needed in analyzing this type of bridge. However, some preliminary calculations to achieve preliminary structure dimensions can be made with basic equations. Estimation for deck and cable cross-sections can be made by modeling the bridge as continuous beam with rigid supports.

3.1.2. Stay spacing

The spacing of the stay anchors along the deck should be compatible with the capacity of the longitudinal girders. The spacing should also be small enough so that the deck may be erected by the free cantilevering method without the need for auxiliary stays or supports. These requirements will effectively limit the spacing within the range of 5 to 15 m.

The heavier concrete construction will require the smaller stay spacing while the larger stay spacing is more suitable for steel or steel composite construction[7].

3.1.3. Preliminary stay forces

The main span stays resist the dead loads such that there is no deflection of the deck or pylon, this makes the condition corresponding to that of a continuous girder supported on rigid supports; therefore the vertical components of the stays due to these loads can be found out.

The calculations provide a means of determining first-trial values of required cable-stay areas. By using the analogy of a continuously elastically supported beam, influence lines for stay forces and bending moments in the bridge girder can be simply determined. From these results, stress variations in the stays and the girder resulting from concentrated loads can be estimated. If, as a first-trial approximation, live load is applied to the same system, the stay forces P_i in figure 3.1 can be determined[26] by equation (1):

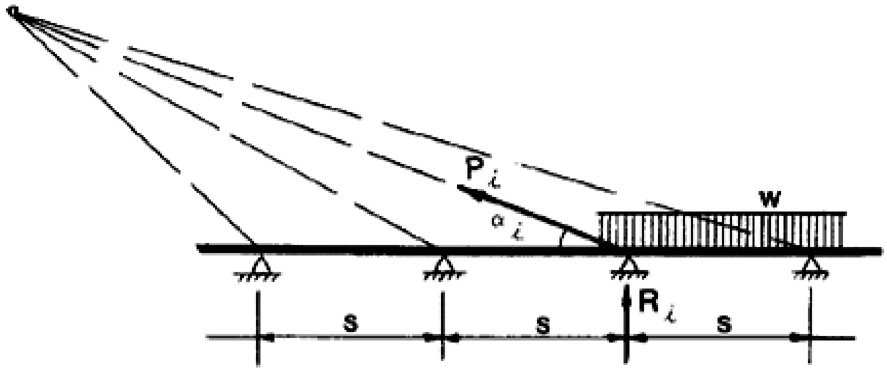


Figure 3.1. Cable force diagram[26]

$$\mathbf{Pi} = \frac{\mathbf{Ri}}{\mathbf{Sin\alpha i}} \text{.......................................(1)}$$

As stay cables are usually designed for the working load condition, the cross-sectional area of stay is determined by:

$$\mathbf{Ai} = \frac{\mathbf{Pi}}{\mathbf{\sigma allow}} \text{.......................................(2)}$$

Where: P_i is tensile force in stay cable

R_i is vertical load force acting on cable spacing length

αi is cable angle

A_i: the cross-sectional area of stay

σ_{allow}: allowable working stress

S: spacing of cables at deck level

3.2. Methods of analysis

In order to analyze a cable-stayed bridge an appropriate idealization or modeling of the structure must be made. The restraints, if any, present at each joint in the structure should be determined in order to mathematically model the structure. The stiffness or flexibility of each member must be known or be determined by the analysis. Connections between the cables, girders, and towers are idealized at their points of intersection.

For a single-plane system the structure may be idealized as a two-dimensional plane frame, and torsional forces acting on the girder would have to be superimposed on the girder. A two-plane system may be idealized as a three-dimensional space frame with torsional forces included in the analysis.

Several methods have been employed in the analysis of cable-stayed bridges. A mixed method of analysis, where the unknowns in the matrix formulation include displacements and forces, has been developed by Stafford Smith. A transfer matrix method has been developed in West Germany. Troitsky and Lazar have used the flexibility approach while Podolny and Fleming used a stiffness approach. Several general computer programs, such as ANSYS, STRUDEL, SAP2000, etc are available which use either the stiffness or flexibility approach. A stiffness approach incorporating an iterative procedure was used by Podolny and Fleming to compensate for the nonlinearity of the cables and Tang applied the transfer matrix to the nonlinearity of cable-stayed bridges[24].

3.2.1. Static analysis

The static analysis of a structure involves the solution of the system of linear equations represented by:

$$\mathbf{Ku} = \mathbf{r} \quad \text{.............................} \quad (3)$$

Where **K** is the stiffness matrix, **r** is the vector of applied loads, and **u** is the vector of resulting displacements.

In long span cable-stayed bridges, dead load is often dominant feature; the pre-strain on the stay cable controls the internal force distribution in the deck and tower as well as the bridge alignment. The initial deformed equilibrium configuration of the bridge is important since it is the starting position to perform the succeeding modal and time history analysis. It is realized by manipulating the initial tension force in each stay cable. For the final analysis the most common approach is to model either a half or the entire structure as a space frame. The pylon, deck and the stays will usually be represented within the space frame model by bar elements.

The stays can be represented with a small inertia and a modified modulus of elasticity that will mimic the sag behavior of the stay. Achieving the deformed equilibrium configuration due to dead load is indeed one of the most difficult tasks involving multiple trial and errors.

3.2.2. Dynamic analysis

Dynamic analysis is the determination of the frequencies and the modes of vibration of the structure. Cable-stayed bridges by their inherent structural characteristics are flexible structures. Moving traffic, wind gust and movement of foundation during an earthquake cause dynamic forces in such structures. The dynamic effect becomes even more pronounced when frequencies of excitation matches with the any one of the natural frequency of the bridge. In calculation of total response of a system the contribution of generally a few lower modes of vibration is of significance. As such in the analysis, the determination of first few lower mode frequency and associated characteristic shape is attempted. The equation of multi degree system can be written[14] as:

$$\boldsymbol{M\ddot{u}} + \mathbf{C\dot{u}} + \boldsymbol{Ku} = \mathbf{F(t)} \quad \text{..........(4)}$$

Where M=mass matrix, C=damping matrix, K=stiffness matrix, u=displacement vector, $\dot{u}$= velocity vector, $\ddot{u}$= acceleration vector, F(t)= dynamic force vector. The above equation (4) is utilized to evaluate the dynamic response of the bridge system.

The results obtained from calculation of frequencies and modes can be used for the following aspects of the design:

- Response of the bridge under wind dynamic action
- Response of the bridge under dynamic action of vehicles

3.3. Wind load analysis

Cable-stayed bridges are flexible, very light weight and they exhibit low damping. Cable-stayed bridges are vulnerable to high wind speed and turbulence since they are frequently constructed along the coastal areas. Cable-stayed bridges must be designed to resist the wind induced forces. Cable-stayed bridges are susceptible to aerodynamic effects, such as[31] vortex-induced oscillation, flutter, galloping, and buffeting in the presence of self-excited forces.

3.3.1. Wind induced vibrations and aerodynamic instability

There are four types of wind-induced vibrations and aerodynamic instability problems that occur in long span cable-stayed bridge: Vortex induced vibration, Galloping instability, Flutter and Buffeting[31].

3.3.1. 1.Vortex Induced Vibration

It has been stated that when a body is subjected to wind[31], the separation of flow occurs around the body and the force is produced on the body, a pressure force on the windward side and a suction force on the leeward side. The pressure and suction forces result in the formation of vortices in the wake region causing structural deflections on the body.

Vortices are shed from the deck at certain frequencies *fs* at different mean wind speeds *U*. According to the Strouhal number $St = f_s D/U$, where *D* is a characteristic dimension perpendicular to the flow. When the frequency of vortex shedding matches one of the natural frequencies of the deck, the vortices excite that particular mode of vibration. Vibration at this wind speed is called "lock-in". The first two vertical modes of vibration are most susceptible to vortex shedding because they have the lowest frequencies.

3.3.1. 2. Galloping Instability

Galloping is an instability typical of slender structures. It is a relatively low-frequency oscillatory phenomenon of elongated, bluff bodies acted upon by a wind stream[31]. The natural frequency at which the bluff object responds is much lower than the frequency of vortex shedding. It is in this sense that galloping may be considered as low frequency phenomenon.

3.3.1. 3. Buffeting

Buffeting is an unsteady loading of a structure by velocity fluctuations in the incoming flow and not self-induced[31]. Buffeting vibration is the vibration produced by turbulence.

A long-span bridge is subjected to both static and dynamic wind forces. Static wind force is due to mean wind speed whereas the dynamic part comes from the turbulence in the wind due to fluctuating wind speed.

3.3.1. 4. Flutter

Flutter is aeroelastic instability that describes an exponentially growing response of the bridge deck, where one or more modes participate at a particularly critical wind velocity, possibly resulting in failure due to over-stressing of the main structural system. This phenomenon normally occurs only in flexible bridges.

Flutter is the dynamic instability of the structure due to self-excited aerodynamic forces resulting from wind-structure interaction. Usually the instability occurs when the net damping (which is inherent structural damping and the negative aerodynamic damping) reduces to zero and further decrease leads to failure[31]. Classical flutter of a thin airfoil is a coupled vertical and torsional vibration, also called 2-D flutter. 1-D flutter may also occur in the form of vertical or torsional motion, although the torsional motion is more dangerous. The most dramatic example for the flutter instability is the collapse of the center span of the Tacoma Narrows Bridge in 1940 at a wind speed of 19m/s.

If a system is given an initial disturbance, it starts to oscillate either in decaying or diverging motion, in other words the motion will be damped or grow to infinity. It should be investigated that if the energy applied from the flow motion is greater or less than the energy dissipated through the system by its mechanical damping. The critical flutter condition occurs as a harmonic motion at the separation line between the decaying and diverging motion, and the corresponding wind speed at that condition is called critical wind speed. The bridge deck will be unstable under the wind loading whose velocity is greater than or equal to the critical wind velocity.

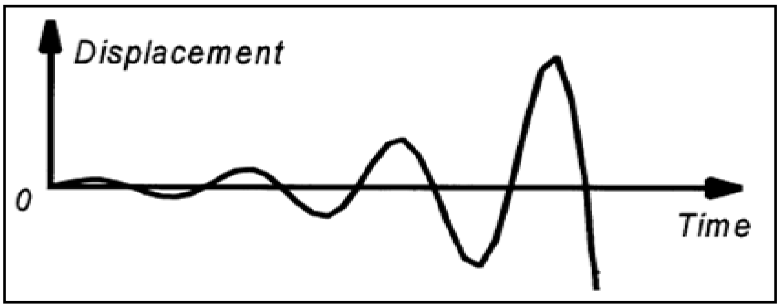


Figure 3.2. Bridge flutter vibration

The self-excited forces acting on a unit deck length are usually expressed as a function of the flutter derivatives. The general format of the self-excited forces written in matrix form[17] for finite element analysis is:

$$\begin{Bmatrix} L_{se} \\ D_{se} \\ M_{se} \end{Bmatrix} = \frac{1}{2}\rho\bar{U}^2 B \left(\begin{bmatrix} \frac{k^2 H_4^*}{B} & \frac{k^2 H_6^*}{B} & k^2 H_3^* \\ \frac{k^2 P_4^*}{B} & \frac{k^2 P_6^*}{B} & k^2 P_3^* \\ k^2 A_4^* & k^2 A_6^* & k^2 A_3^* B \end{bmatrix} \begin{Bmatrix} h \\ p \\ \alpha \end{Bmatrix} + \begin{bmatrix} \frac{kH_1^*}{\bar{U}} & \frac{kH_5^*}{\bar{U}} & \frac{kH_2^* B}{\bar{U}} \\ \frac{kP_1^*}{\bar{U}} & \frac{kP_5^*}{\bar{U}} & \frac{kP_2^* B}{\bar{U}} \\ \frac{kA_1^* B}{\bar{U}} & \frac{k5B}{\bar{U}} & \frac{kA_2^* B^2}{\bar{U}} \end{bmatrix} \begin{Bmatrix} \dot{h} \\ \dot{p} \\ \dot{\alpha} \end{Bmatrix} \right) \quad \text{.........(5)}$$

Where *Lse*, *Dse*, and *Mse* = self-excited lift force, drag force, and pitch moment, respectively. *h*, *p*, and α = displacements at the center of a deck in the directions corresponding to *Lse*, *Dse*, and *Mse*, respectively; ρ = density of air; *B* = deck width; *Hi**, *Pi**, and *Ai* * (*i*=1 to 6) are the vertical, lateral and torsional flutter derivatives; respectively, which are be determined from wind tunnel tests for the bridge deck under consideration. *k* = reduced frequency= $B\omega/\overline{U}$; ω = circular frequency; $\overline{U}$= average wind velocity. Subscript *se* stands for self-excited force; dot indicates differentiation with respect to time *t*.

Since the vortex-shedding force is usually small in magnitude compared to the self excited force related to the flutter instability, it can be ignored in flutter analysis, for this reason, Scanlan and Tomko[15] have suggested the following expressions of aerodynamic forces on a 2-D structure involving vertical and torsional:

$$L_{se} = \frac{1}{2}\rho\overline{U}^2 B\left[KH_1^* \frac{\dot{h}}{\overline{U}} + KH_2^* \frac{B\dot{\alpha}}{\overline{U}} + K^2H_3^*\alpha + K^2H_4^* \frac{h}{B}\right] \text{........(6)}$$

$$M_{se} = \frac{1}{2}\rho\overline{U}^2 B^2\left[KA_1^* \frac{\dot{h}}{\overline{U}} + KA_2^* \frac{B\dot{\alpha}}{\overline{U}} + K^2A_3^*\alpha + K^2A_4^* \frac{h}{B}\right] \text{.......(7)}$$

The aerodynamic derivatives: A^* and H^* (i =1 ~ 6) depend upon bridge section outline mostly.

ω can be found using expression: $\boldsymbol{f = \omega / 2\pi}$ **,**Where *f* is the natural frequency of the vibration. The aerodynamic derivatives can be obtained from wind tunnel tests.

3.3.2. Flutter susceptibility prediction by using modal analysis technique

In cable-stayed bridges, as for any other structure, the natural frequencies and mode shapes are extremely important for the susceptibility to dynamic excitation. Low structural frequencies point to a high susceptibility. With growing structural dimensions or larger spans the frequencies decrease, so large bridges are particularly critical. Natural modes and frequencies are found by performing modal analysis which uses the overall mass and stiffness of a structure to find the various periods at which it will naturally resonate.

The frequency ratio between different modes can also be important in cable-stayed bridges. A classical example is the ratio f_T/f_B (torsional to vertical bending frequency), which is the critical parameter for the susceptibility against flutter instability[28]. Since the lowest frequencies of each bridge lie in the region of the wind spectrum where the spectral values are decreasing rapidly[31], higher modes can be expected to have lower contributions to the deflections, as is typical of most structures.

The dynamic equilibrium Equation of the model can be written as[16]:

$$[\boldsymbol{M}]\{\ddot{\boldsymbol{U}}\} + [\boldsymbol{C}]\{\dot{\boldsymbol{U}}\} + [\boldsymbol{K}]\{\boldsymbol{U}\} = \{\boldsymbol{P}(\boldsymbol{t})\} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\mathbf{(8)}$$

Where $[M]$ = mass matrix of the structure, $[C]$ = damping matrix of the structure, $[K]$ = stiffness matrix of the structure, $\{U\}$ =displacement vector of each node, $\{\dot{U}\}$= velocity vector of each node, $\{\ddot{U}\}$ = acceleration vector of each node, F (t) = dynamic force vector.

If the resistance is ignored, the following dynamic equilibrium equation can be obtained:

$$[\boldsymbol{M}]\{\ddot{\boldsymbol{U}}\} + [\boldsymbol{K}]\{\boldsymbol{U}\} = \mathbf{0} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\mathbf{(9)}$$

Taking $[U(t)] = \{\Phi\} sin\omega t$and then solving the differential equations, the following equation is obtained:

$$[\boldsymbol{K}] - \boldsymbol{\omega}^{\mathbf{2}}[\boldsymbol{M}]\{\Phi\} = \mathbf{0} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\mathbf{(10)}$$

Where $\{\Phi\}$ is the vector of amplitude.

From this equation, natural frequencies of the system ω_i(i=1,2,….,N) and the vibration modes $\{\Phi i\}$ (i=1,2,….,N) of the structure can be found out.

3.3.3. Velocity pressure and wind force on the bridge

The AASHTO LRFD bridge design specifications and ASCE 7-10 can be used to determine the wind load on the bridge structure. According to AASHTO LRFD wind load shall be the pressure of the wind acting horizontally on a vertical projection of the exposed area of a structure or vehicles.

Velocity Pressure

Velocity pressure, q_z, evaluated at height $\boldsymbol{z}$ shall be calculated by the following equation[37]:

$$q_z = 0.613 K_z K_{zt} K_d V^2 \text{ (N/m}^2\text{); [}V\text{ in m/s]} \ldots\ldots\ldots\text{(11)}$$

Where

K_d = wind directionality factor

K_z = velocity pressure exposure coefficient

K_{zt} = topographic factor

V = basic wind speed (determined based on risk categories (I, II, III and IV) of the structures)

The design wind force for each component shall be determined by:

$$F = q_z * A_f * G * C_f \text{ ....(12)}$$

Where

q_z = velocity pressure evaluated at height z of the centroid of area A_f

G = gust-effect factor for flexible or dynamically sensitive structures

C_f = force coefficient

A_f = projected area normal to the wind

3.3. Geometry and material description of the reference bridge

The geometry of the cable-stayed bridge chosen for this study is similar to that of the Quincy Bayview Bridge crossing the Mississippi River, the bridge is located in Illinois, USA[8].

The Quincy Bayview Bridge was designed in 1983 and construction was completed in 1987. The bridge consists of two H-shaped concrete towers, double-plane fan type cables, and a composite concrete-steel girder bridge deck consisting of steel edge girders, steel floor beams and a reinforced concrete slab deck[8].

Figure 3.3. Quincy Bayview Bridge[8]

The main span is 274 m and the two equal sides have spans of 134 m each, making a total length of 542 m, as illustrated in Fig.3.3. The bridge carries two traffic lanes across the river. A total of 56 cables are used, 28 supporting the main span and 14 supporting each side span, the cable members are spaced at 2.75 m at the upper part of the towers and are equally spaced at deck level on the side spans as well as the main spans. The width of the deck from center to center of the cables is 14 m. The cables are connected to the deck at the bottom flange of the main girders. The tops of the towers are 71 m from the water line. Each tower consists of two concrete legs, a lower strut supporting the deck, and an upper strut connecting the upper legs[8].

Table 3.1. Material properties of structural elements

S.N.	Member	Material	Unit weight(KN/m^3)	Poisson ratio	Modulus of elasticity(G Pa)	Yield/compressiv e/ultimate tensile strength(MPa)
1	Pylon	RCC	25	0.25	35	35
2	Deck slab	RCC	25	0.25	35	35
3	Girder	steel	77	0.3	200	250, 450
4	Cable	steel	77	0.3	210	1600

Table 3.2.Sectional properties of structural elements

S.N.	Structural member	Dimension	Material	Shape
1	Cable	0.00897,0.00677,0.00537,0.00348 cross-sectional area(m^2)	Steel	Circular
2	Deck slab	Depth =0.23m ,Width = 14m	RCC	Rectangular
3	Longitudinal Beams	1m x 0.4m, tw = 0.15m , tf = 0.15m	Steel	I –section
4	Cross Beams	0.7m x 0.25m, tw = 0.10m, tf = 0.10m	Steel	I -section
5	Pylon Bottom	4.42mx2.2m	RCC	Rectangular
	Pylon Intermediate at deck level	4.42 m x 2.2m	RCC	Rectangular
6	Pylon Top	(4.42m x2.2m) -(2.20mx 1m)	RCC	Hollow Rectangular

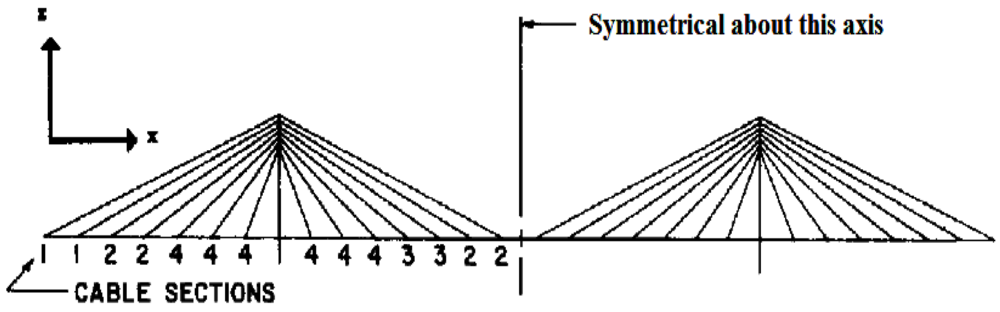


Figure 3.4. Numbering of cables based on their diameters

Cable 1: 0.00897m^2

Cable 2: 0.00677 m^2

Cable 3: 0.00537 m^2

Cable 4: 0.00348 m^2

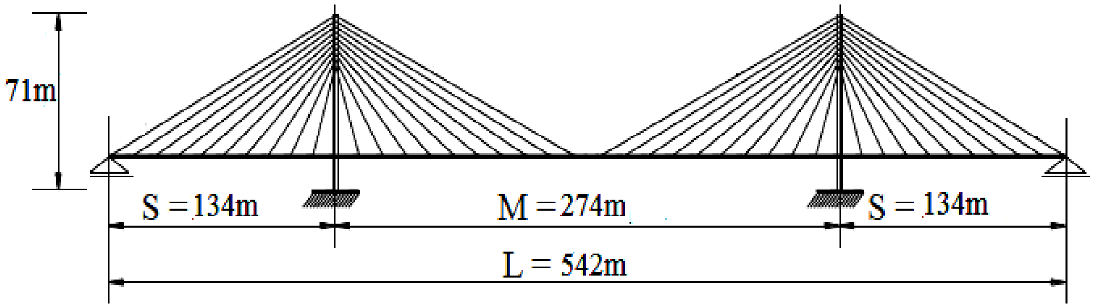


Figure 3.5.Longitudinal view of the bridge

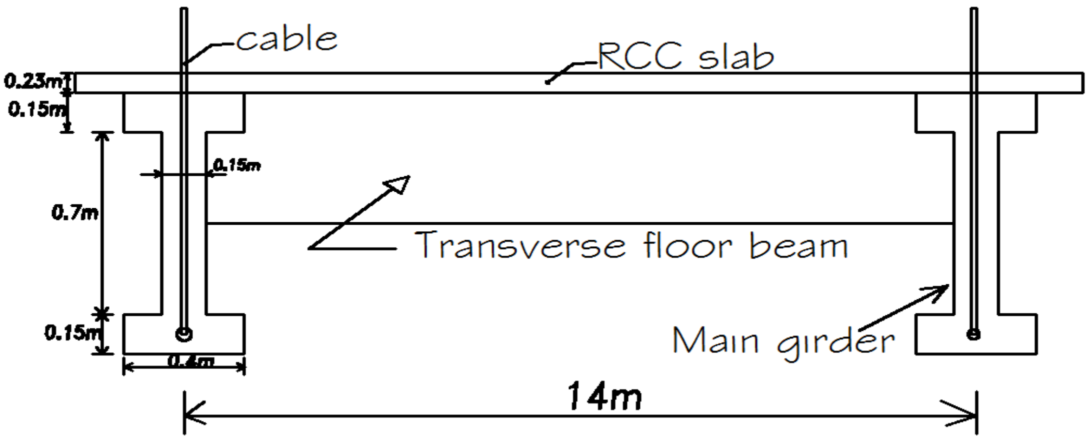


Figure 3.6. Cross-section of the deck

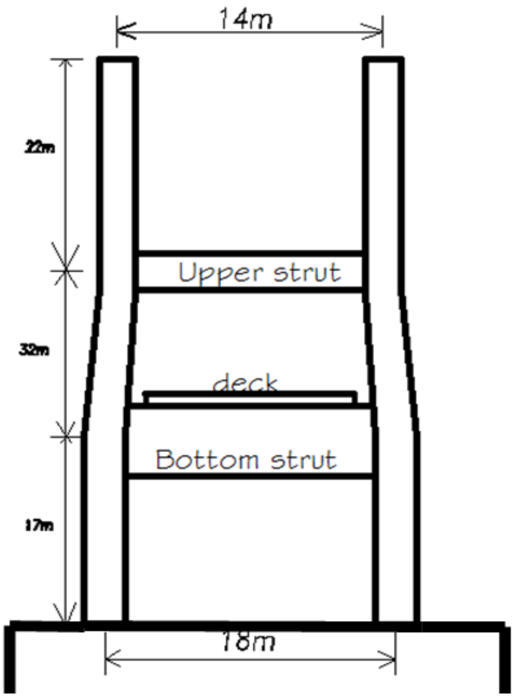


Figure 3.7. Elevation view of bridge tower

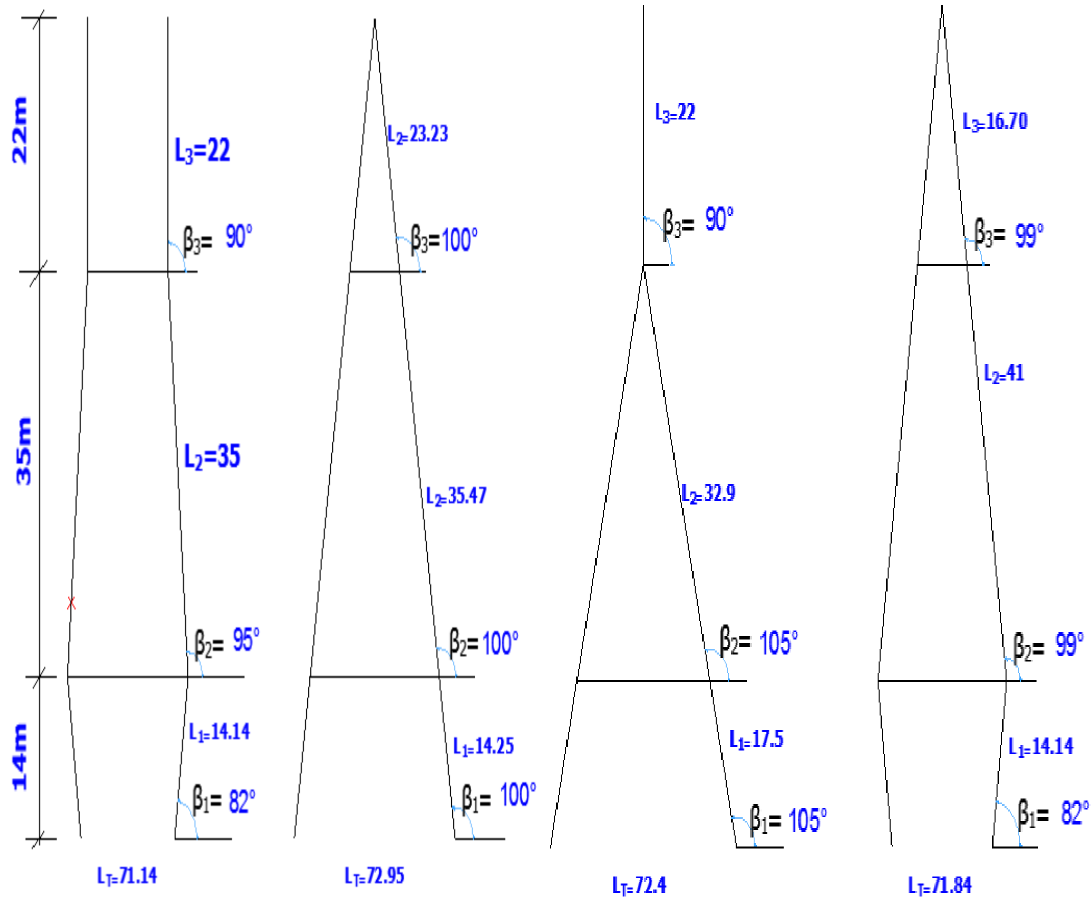


Figure 3.8. Lengths and inclinations of the pylon legs

The logic followed to change pylon shape is such that only the geometry of pylon is changed, but other properties of the pylon section such as cross-sectional area, moment of inertia are kept constant for every pylon as shown in the table 3.3.

Cross sectional area of solid rectangular section = *b x d= 4.42 x 2.2= 9.724m*^{2}

Cross sectional area of hollow rectangular section = *(b x d) – (*$b_1 d_1$*)*

= *(4.42 x 2.2)–(2.2 x 1)* =*7.524m*^{2}

Moment of inertia below deck:

$$I_{22} = \frac{bd^3}{12} = \frac{2.2*4.42^3}{12} = 15.83 \text{ m}^4$$

$$I_{33} = \frac{bd^3}{12} = \frac{4.42*2.2^3}{12} = 3.92 \text{ m}^4$$

Moment of inertia above deck:

$$I_{22}= \frac{bd^3}{12} - \frac{b_1 {d_1}^3}{12} = \frac{2.2*4.42^3}{12} - \frac{1*2.2^3}{12} = 14.94 \text{ m}^4$$

$$I_{33}= \frac{bd^3}{12} - \frac{b_1 {d_1}^3}{12} = \frac{4.42*2.2^3}{12} - \frac{2.2*1^3}{12} = 3.74 \text{ m}^4$$

Where: I_{22} is moment of inertia about minor dimension; I_{33} is moment of inertia about major dimension.

Table 3.3. Sectional geometrical parameters of pylon sections

Pylon shape	**Cross-sectional area (m^2)**		**Moment of inertia (m^4)**	
	Pylon below deck	Pylon above deck	Pylon below deck	Pylon above deck
H-shape	9.724	7.524	I_{22}=15.83 I_{33}=3.92	I_{22}=14.94 I_{33}=3.74
A-shape	9.724	7.524	I_{22}=15.83 I_{33}=3.92	I_{22}=14.94 I_{33}=3.74
Y- Inverted shape	9.724	7.524	I_{22}=15.83 I_{33}=3.92	I_{22}=14.94 I_{33}=3.74
Diamond shape	9.724	7.524	I_{22}=15.83 I_{33}=3.92	I_{22}=14.94 I_{33}=3.74

3.4. SAP2000 software

SAP2000 version 14 is utilized in this study for the analysis of the Cable-stayed Bridge. The SAP program was originally developed by Dr. E.L. Wilson et al. at University of California, Berkeley; with a three dimension (3D) object-based graphical modeling environment and nonlinear analysis capability, the SAP2000 program provides a general purpose yet powerful finite element analysis software program for structural analysis. This computer program is one of the most popular structural analysis software packages used by structural engineers in the USA. The program is structured to support a wide variety of the latest national and international codes for both steel and concrete designs.

3.5. Structural modeling and Analysis

3.5.1. Finite element models

The analytical models of the bridge include all components that influence the mass, strength, stiffness and deformability of structure. The bridge structural system consists of cables, pylons, deck slab, struts, transverse and longitudinal beams. The non-structural elements that do not significantly influence the bridge behavior are not modeled.

3.5.1.1. Modeling of deck and girder

In this study, a beam element has been used to model the longitudinal and transverse beams while the deck was modeled using shell element.

3.5.1.2. Modeling of cables

For modeling cables, a straight frame element has been used; and the effect of cable sag due to action of self-weight of cables was neglected since Wilson J.C. investigated the use of an equivalent modulus of elasticity and results gave an equivalent modulus that was essentially equal to the true modulus of the cables.

Thus, non-linear tension-sag effects in the cables were assumed to be negligible and the cables were treated as having a completely linear force-deformation relationship described by the true material modulus of elasticity. However, one can consider the nonlinearities of cables arising due to cable sag by using the modified modulus of elasticity calculated from the equation (13) of H. Ernst[7]:

$$\mathbf{E_{eq}} = \frac{\mathbf{E}}{\mathbf{1}+\frac{(\mathbf{\gamma L})^{2}\mathbf{E}}{\mathbf{12\sigma^{3}}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Where:

E_{eq} =Equivalent modulus of elasticity of the cable

E = Effective modulus of elasticity of the straight cable

γ = Weight of the cable per unit length

σ = Cable tensile stress

L =Horizontal projected length of the cable

3.5.1.3. Modeling of Pylon

The function of Pylon is to support the cable system and transfer the forces to foundation. Therefore, it is subjected to high axial forces. For modeling a pylon, a beam element was used.

Assumptions made in the models:

In the analysis of the study of cable-stayed bridges contained in this thesis work, the following assumptions have been made:

1. The joint between cable, girder and tower is a pinned joint.
2. Cables were assumed to be straight members, which means, the effect of caternary action due to the self-weight of cables was neglected. Some researchers have shown

that the effect of catenary action for moderate sag to span ratio is not large. However, this effect can be incorporated by modifying the modulus of elasticity in the equation (13).

3. The structure was assumed to remain linearly elastic.

4. Pylons are assumed to be fixed at the level of foundation.

Models of the studied cable-stayed bridge

The following figures contain model views of the studied bridge created using SAP2000-V14 structural analysis software:

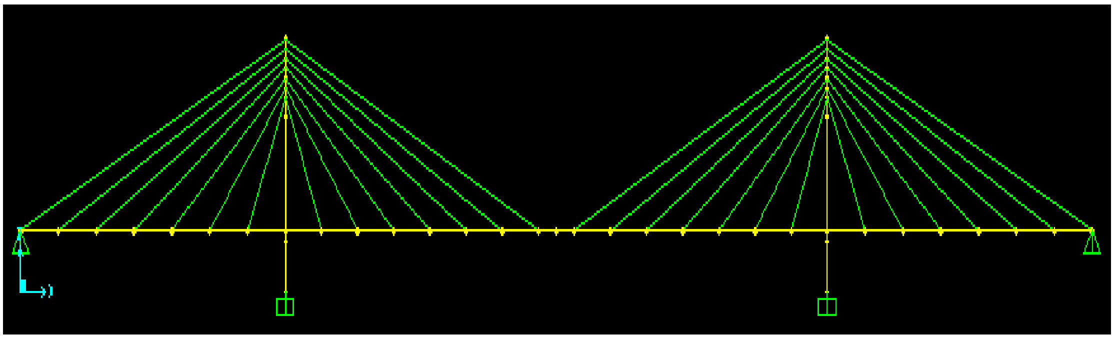


Figure 3.9. Longitudinal view of modeled cable-stayed bridge

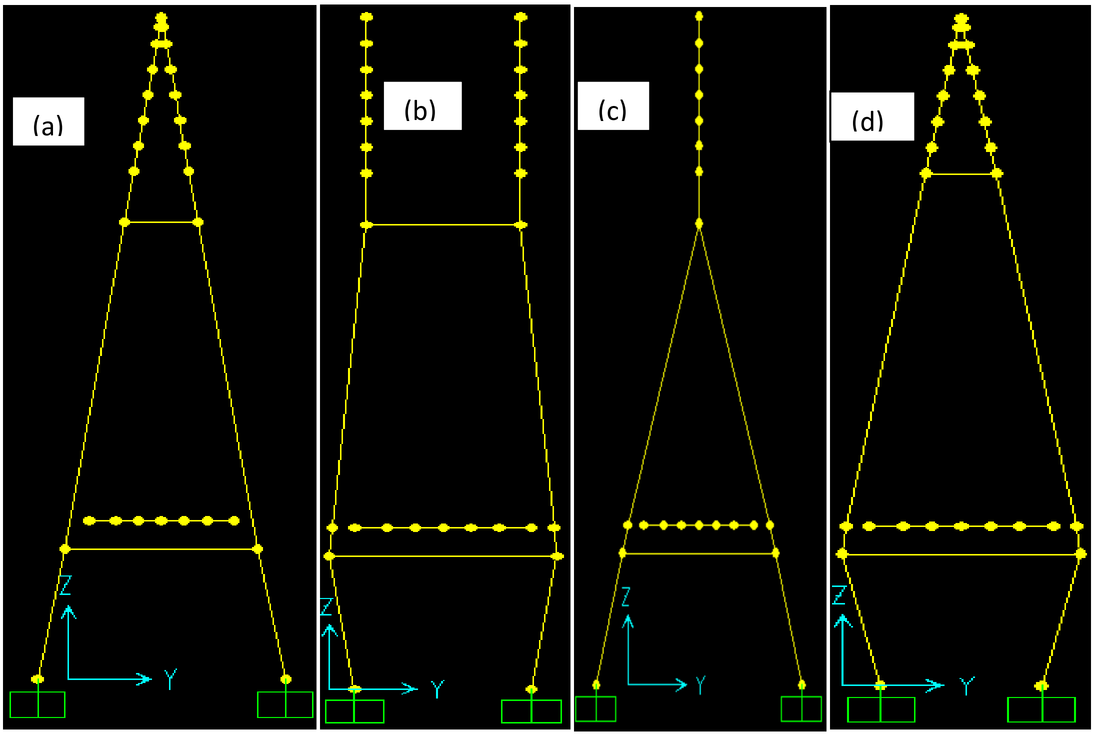


Figure 3.10.Transverse model views of pylon shapes: (a) A-shape pylon,(b)H-shape pylon,(c) Y-inverted shape pylon, (d) Diamond–shape pylon

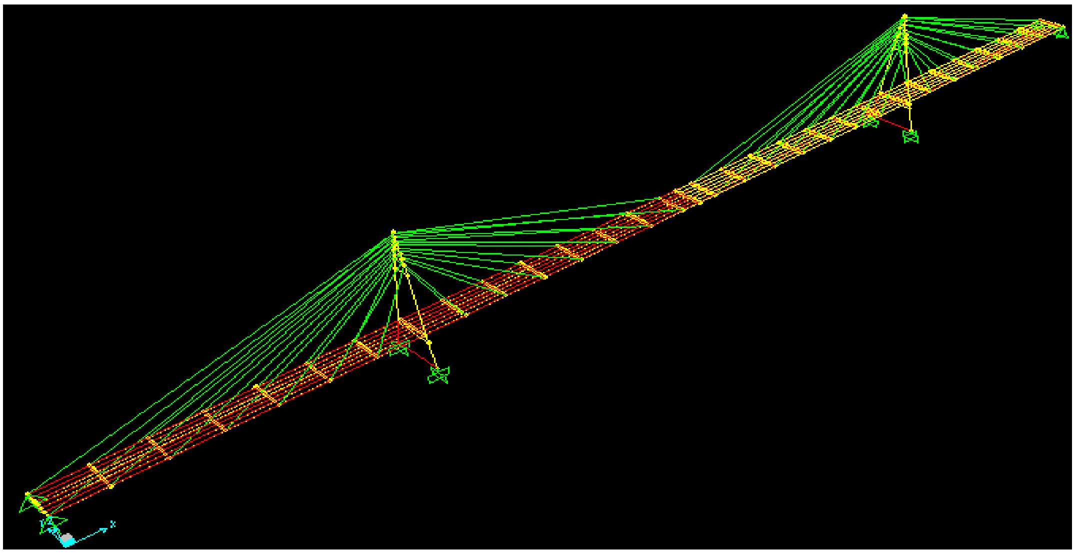


Figure 3.11. 3-D finite element model of A-shaped pylon cable-stayed bridge

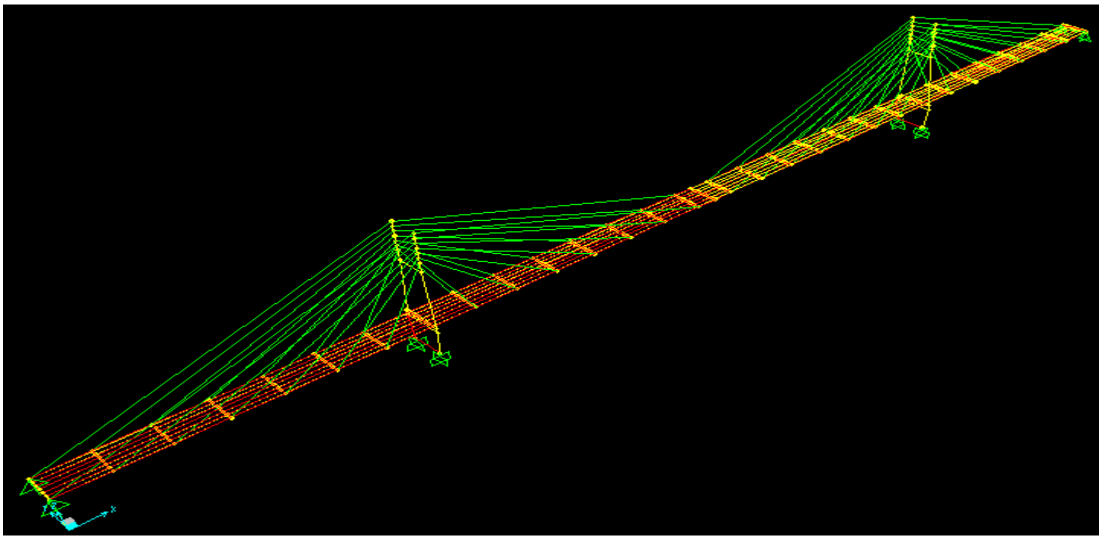


Figure 3.12. 3-D finite element model of H-shaped pylon cable-stayed bridge

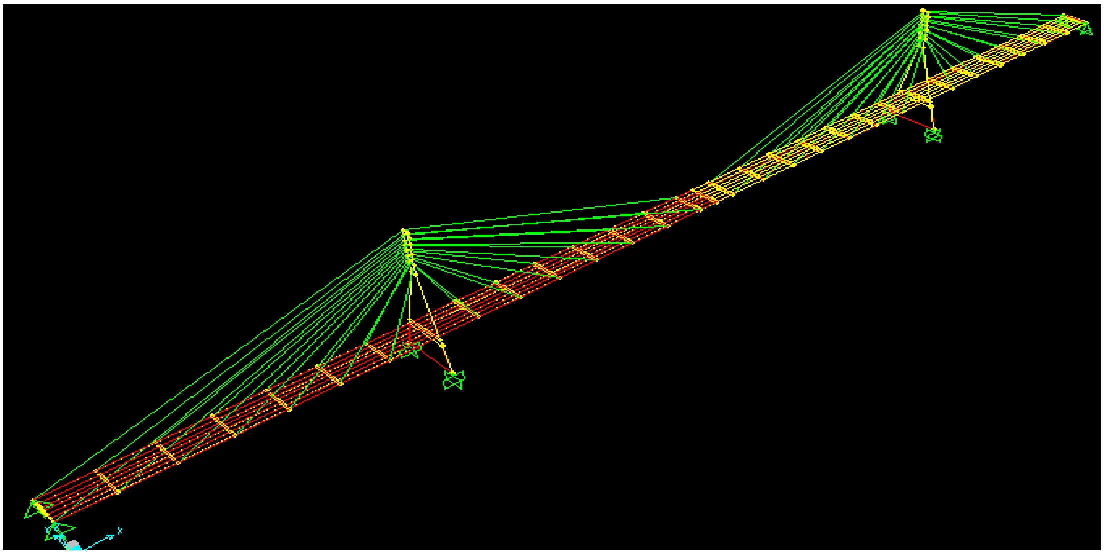


Figure 3.13. 3-D finite element model of inverted-Y shaped pylon cable-stayed bridge

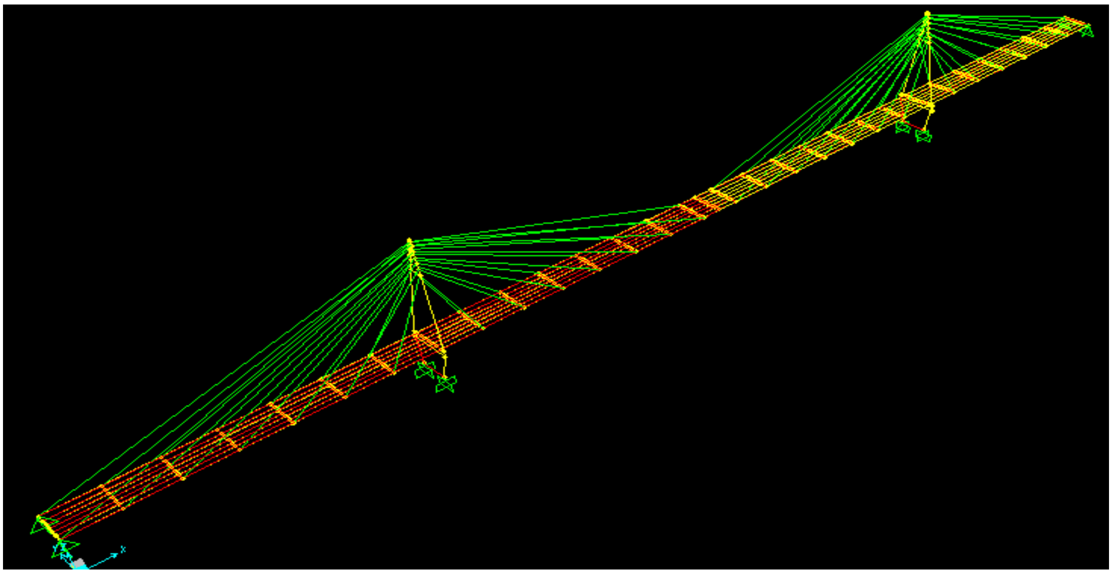


Figure 3.14. 3-D finite element model of Diamond shaped pylon cable-stayed bridge

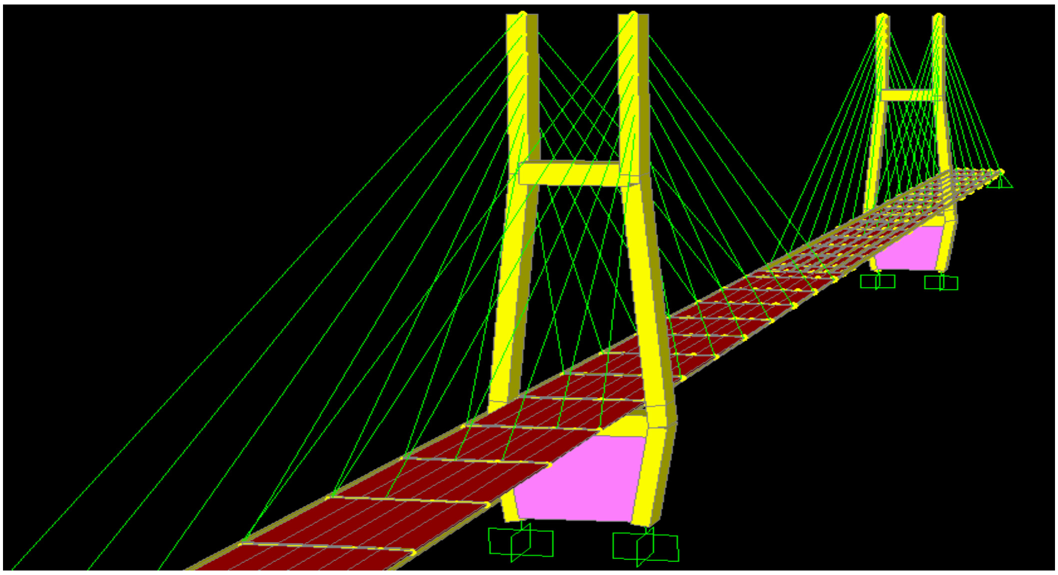


Figure 3.15. Extrude 3D view of the bridge with H-shape pylon

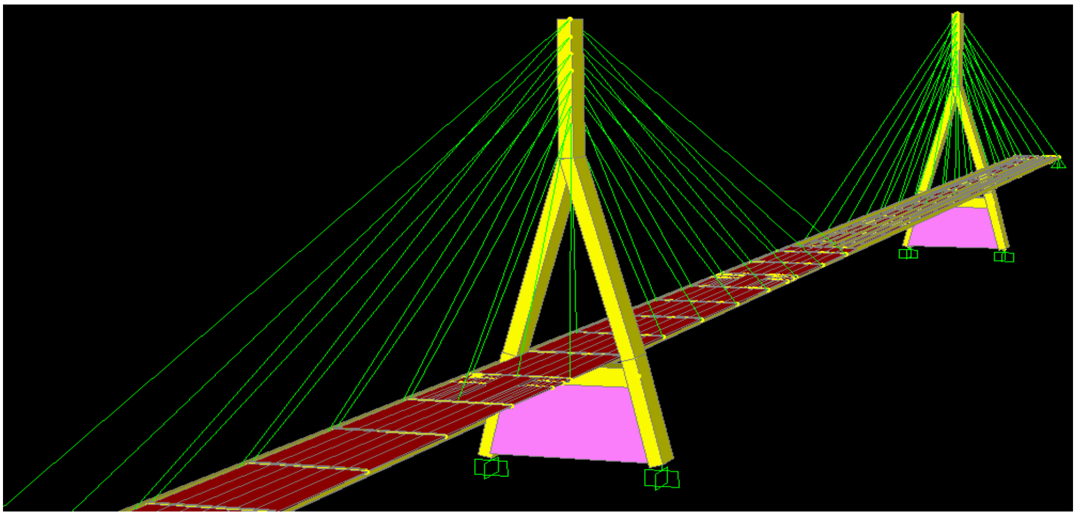


Figure 3.16. Extrude 3D view of the bridge with Y- inverted shape pylon

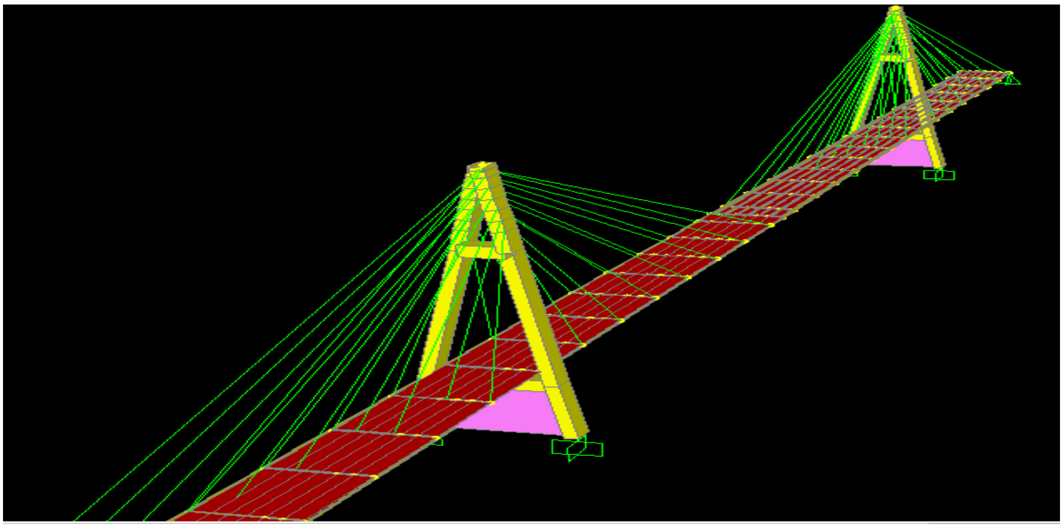


Figure 3.17. Extrude 3D view of the bridge with A-shape pylon

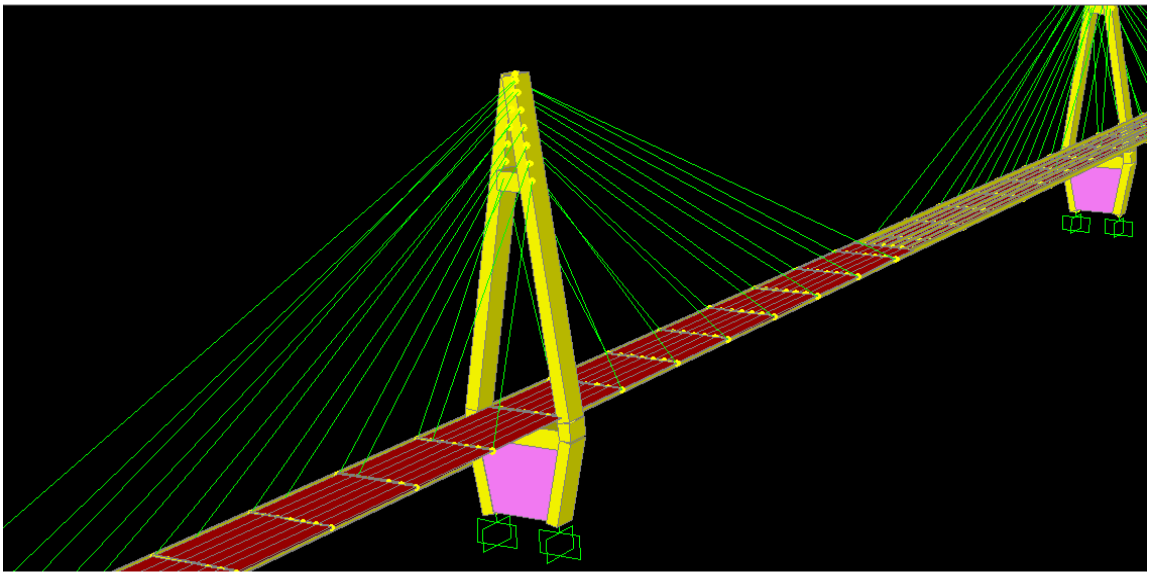


Figure 3.18. Extrude 3D view of the bridge with Diamond-shape pylon

3.6. Validation of models on SAP2000 software

In order to be sure that models created in this study are correctly done, modal analysis of the Quincy bay view bridge has been performed. Modal analysis results obtained are compared with the ones that have been found in the reference research paper by Wilson & Gravelle, 1991.

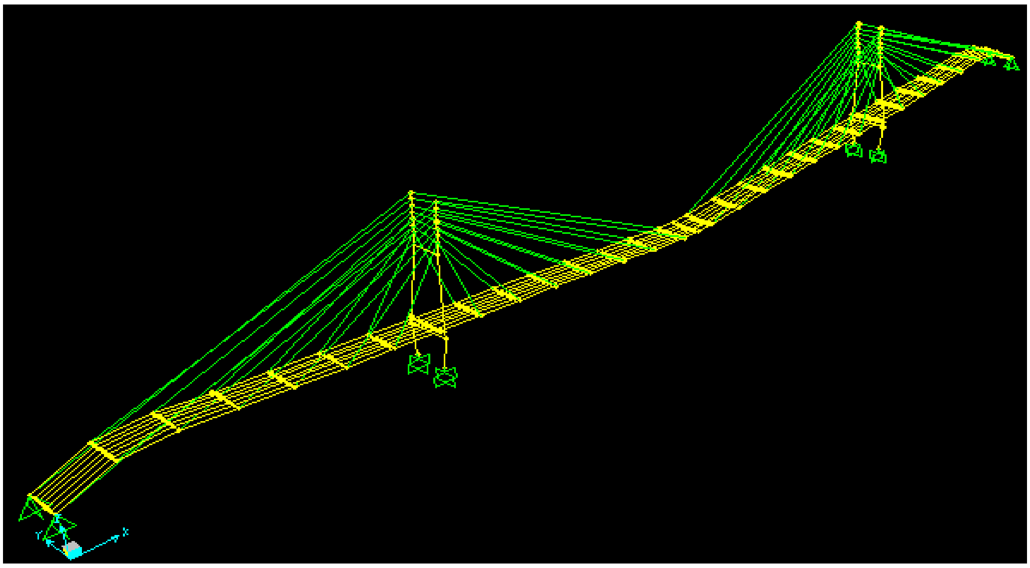


Figure 3.19. First mode involving vertical bending of deck at 0.368Hz

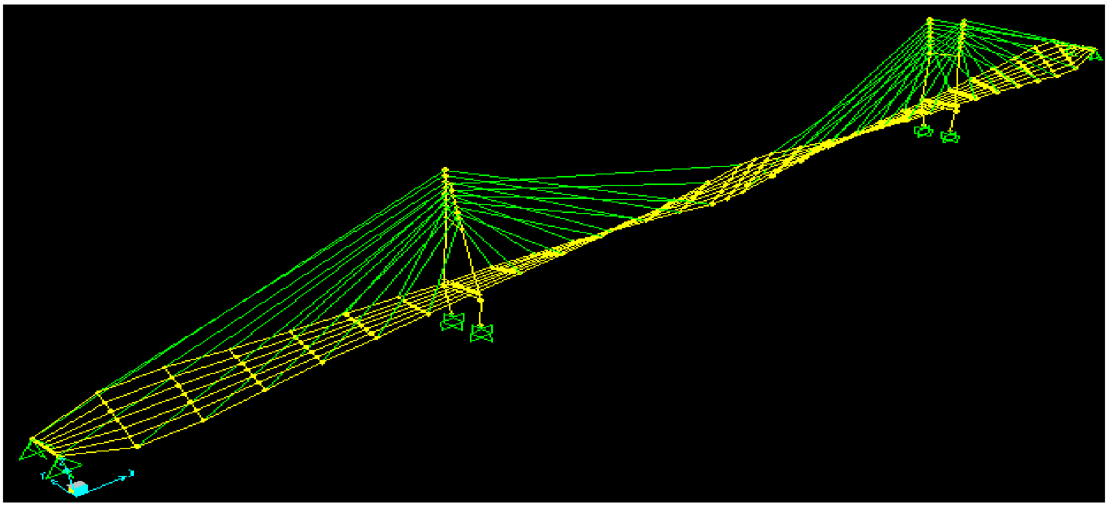


Figure 3.20. Second mode at a frequency of 0.571Hz

Table 3.4. Comparison of validation results

	Results obtained	Results from reference
Fundamental mode frequency	0.368Hz	0.371Hz
Second mode frequency	0.571Hz	0.577Hz

Results show that the models created in this study are correct since they are almost same as those obtained in the reference paper.

Manual validation of tensile forces in cables:

Cables forces can be manually estimated by application of the continuous beam method. The main span stays resist the dead loads such that there is no deflection of the deck or pylon, this makes the condition corresponding to that of a continuous girder supported on rigid supports; therefore the vertical components of the stays due to these loads are known.

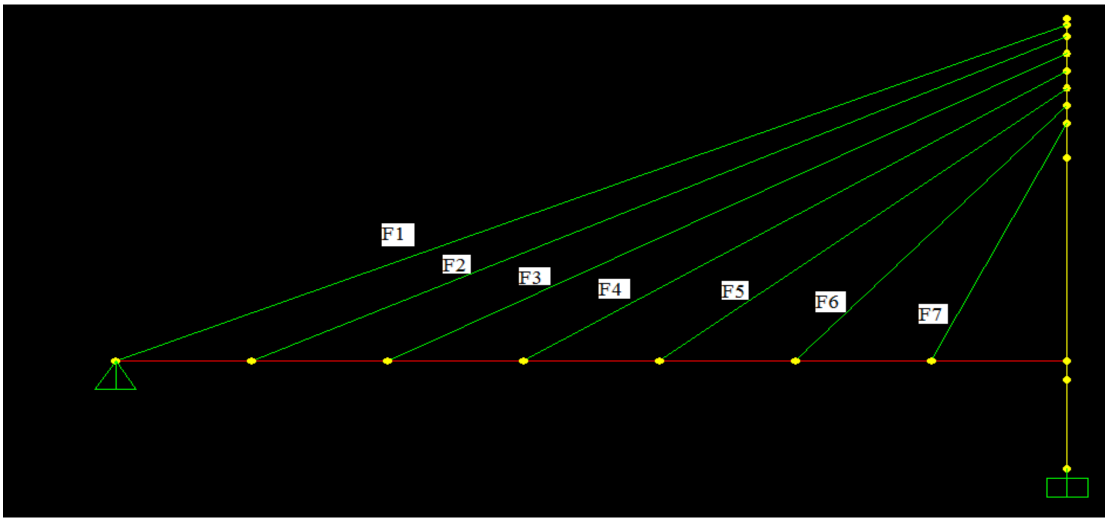


Figure 3.21. Left side span with numbering of cable forces

By applying the stiffness method of analysis for the system formed from a continuous beam, the reaction forces which are equal to vertical components of cable forces are found out, and finally by knowing the inclination angles of cables and applying the formula (1) the forces in the cables have been found as follows:

Table 3.5. Comparison of manual results

Cable force No	**Manually [KN]**	**Software [KN]**	**Difference[KN]**	**% difference**
F_1	1766.8	1790.6	23.8	1.3
F_2	4334	4421	87	1.9
F_3	3687	3787	100	2.6
F_4	3272	3354	82	2.4
F_5	2403.4	2441.7	38.3	1.5
F_6	1690.7	1750.5	59.8	3.4
F_7	2702.8	2748.4	45.6	1.6

Where F1 to F7 are the cable forces in one side span, starting from cable located at the left side of span to the cable near the pylon. Comparison of manually calculated cable forces and software results is done above; the percentage difference is below 10, which implies that results are acceptable.

CHAPTER 4. RESULTS OF ANALYSIS AND DISCUSSIONS

4.1. Results of static dead load analysis

Dead loads of all the components of the bridge like cables, girder, deck, pylon, etc, are considered as dead load of the bridge. In the design of cable-stayed bridges, the dead load often contributes to most of the bridge load. Dead loads are indeed the most important for the preliminary analysis in cable-stayed bridges. In this analysis, the type of analysis considered is linear static.

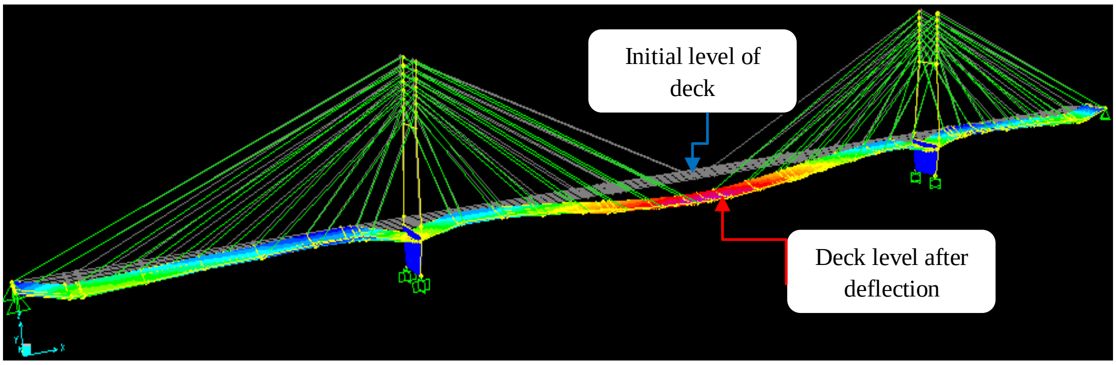


Figure 4.1. Deformed shape of H-shape pylon after dead load analysis

The following figures show the comparison of results from all the four cases of pylon shapes with respect to their linear static dead load response.

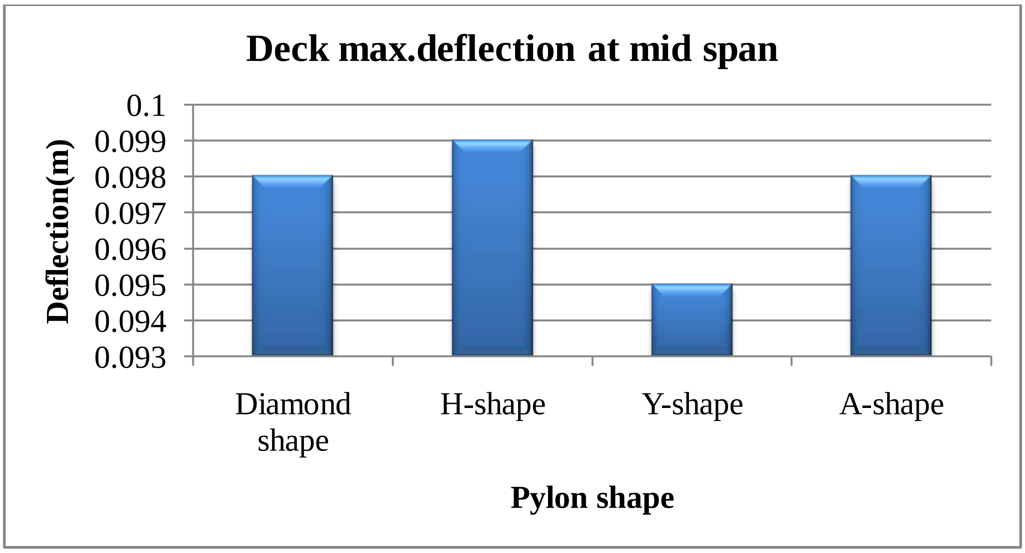


Figure 4.2. Maximum deflection of deck at mid span

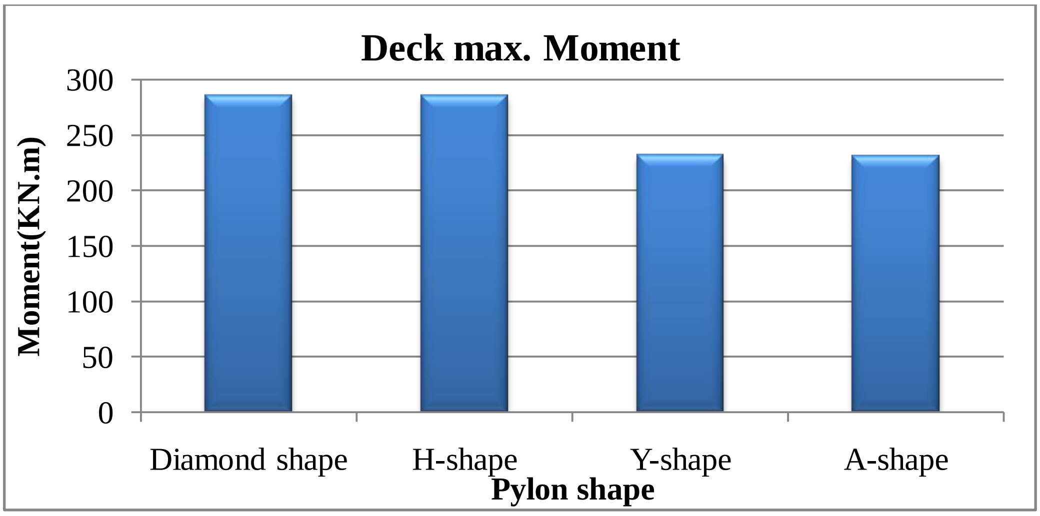


Figure 4.3. Maximum moment in deck

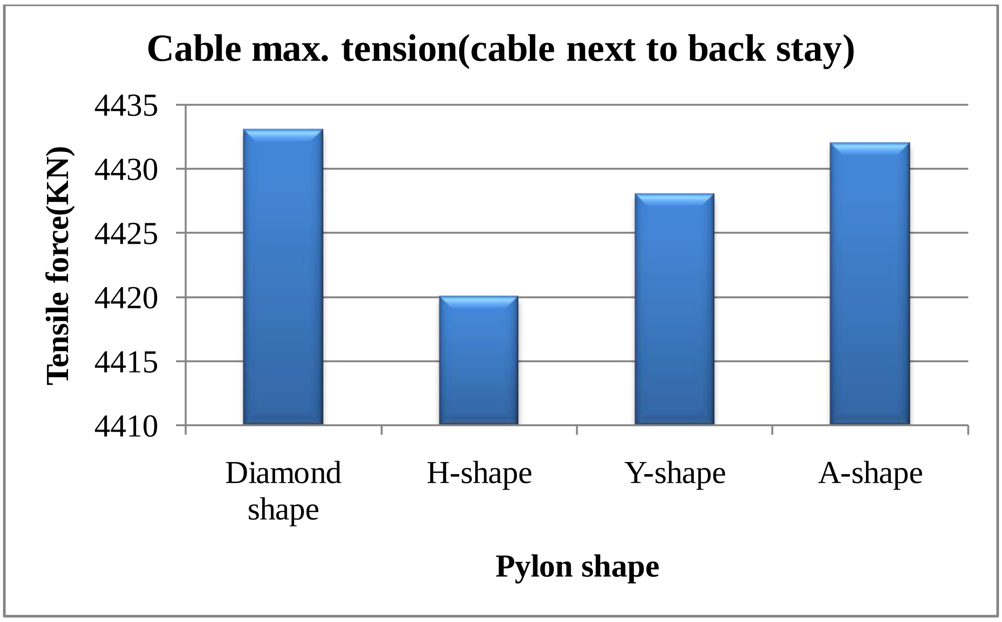


Figure 4.4. Maximum tensile force in cables (observed in 2nd cable from each end of side span)

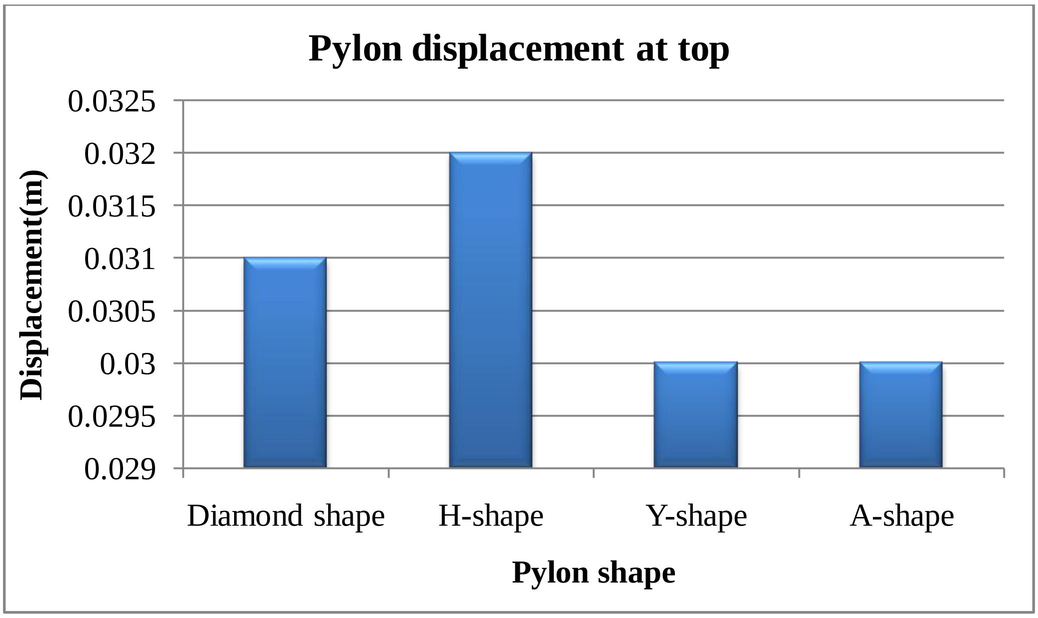


Figure 4.5. Displacement of pylon at top

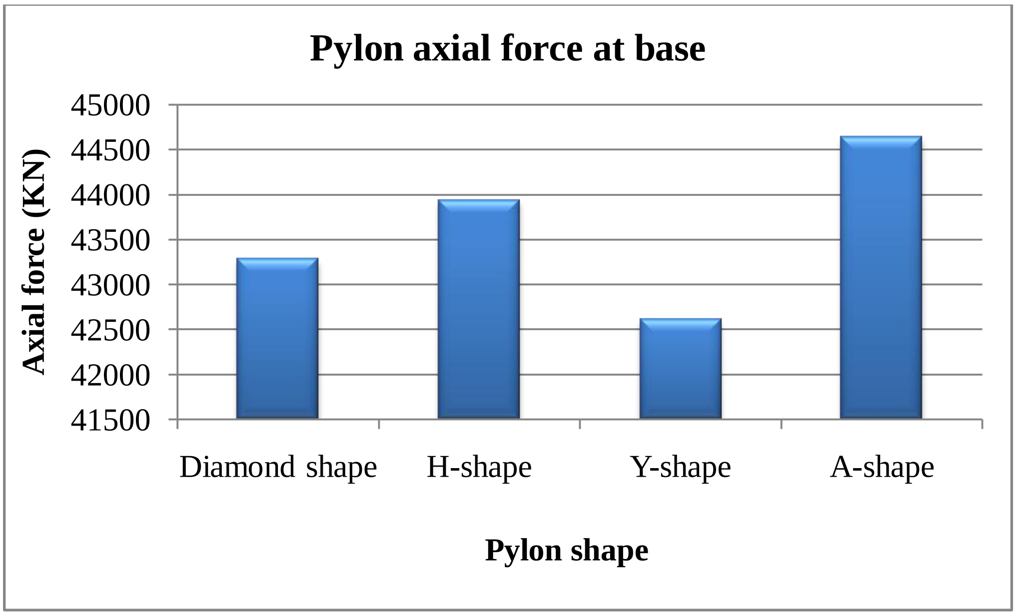


Figure 4.6. Pylon axial force at base (F_3)

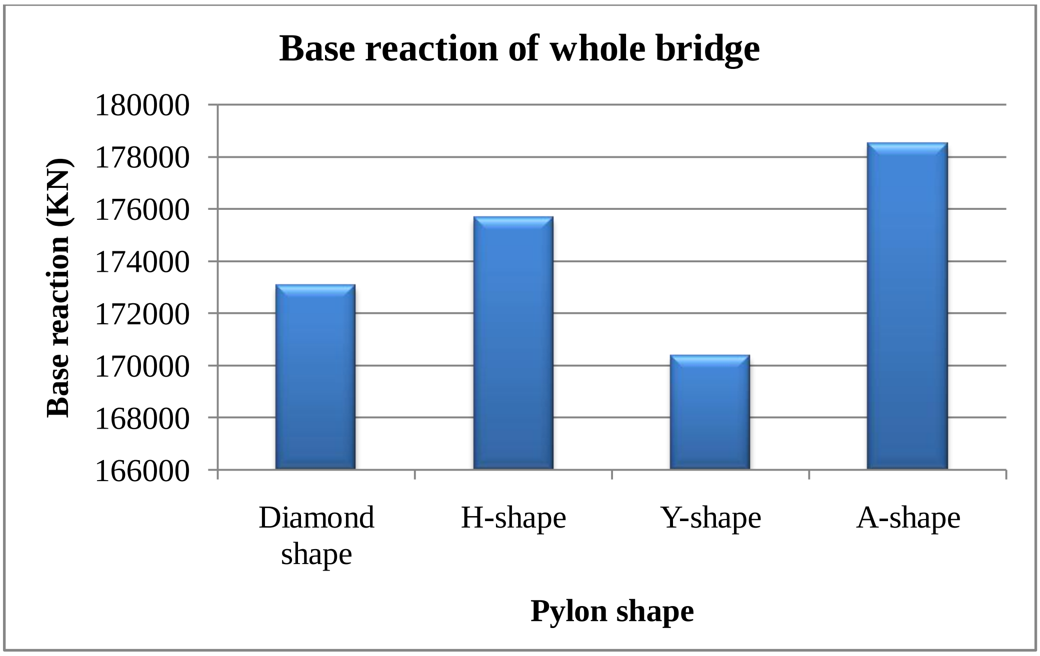


Figure 4.7. Base reaction of the whole bridge

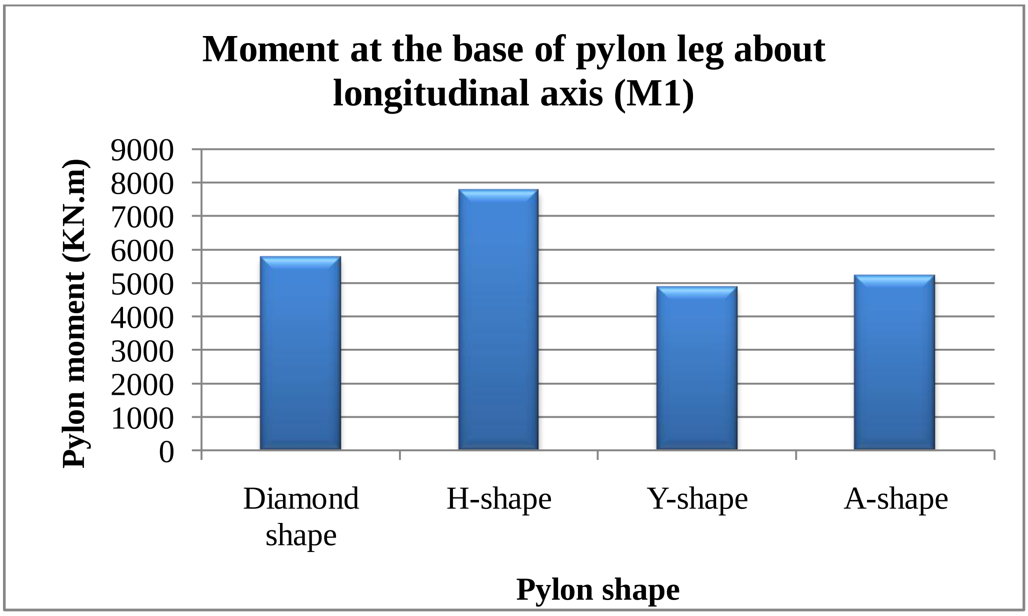


Figure 4.8. Moment at the base of pylon leg about longitudinal axis (M_1)

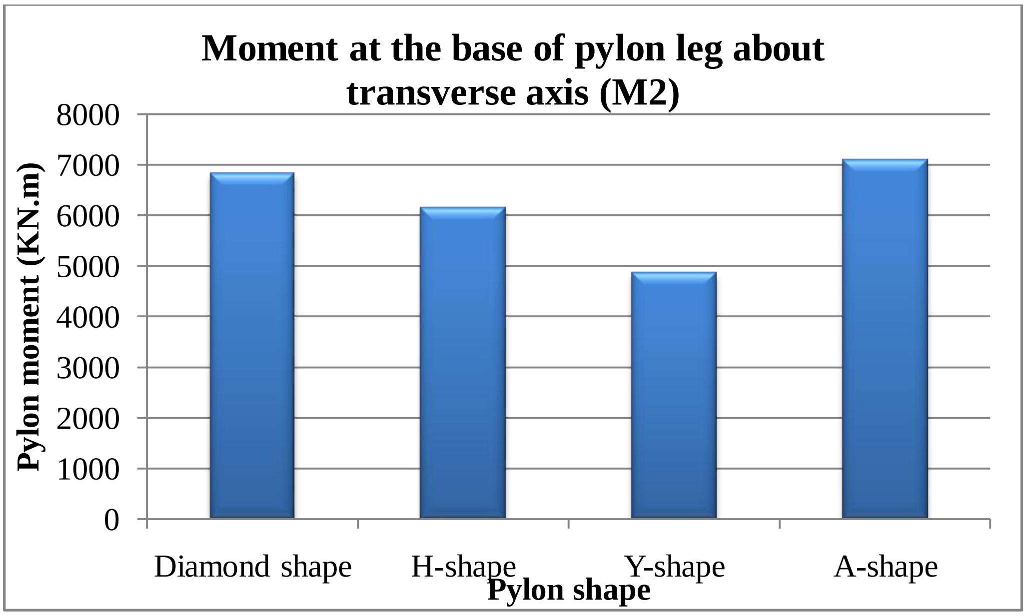


Figure 4.9. Moment at the base of pylon leg about transverse axis (M_2)

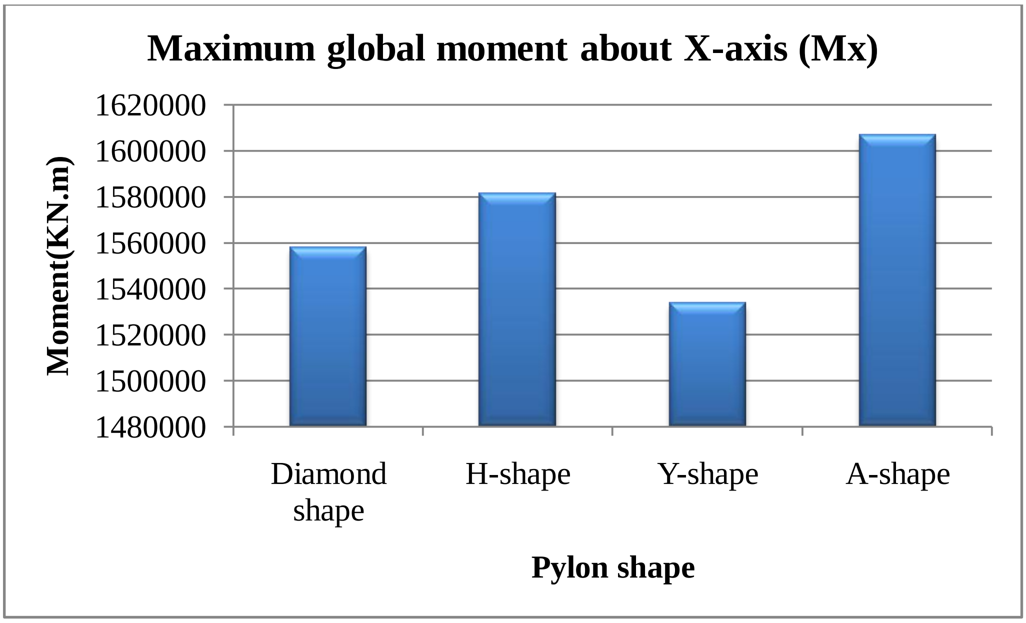


Figure 4.10. Maximum global base moment about X-axis (M_x)

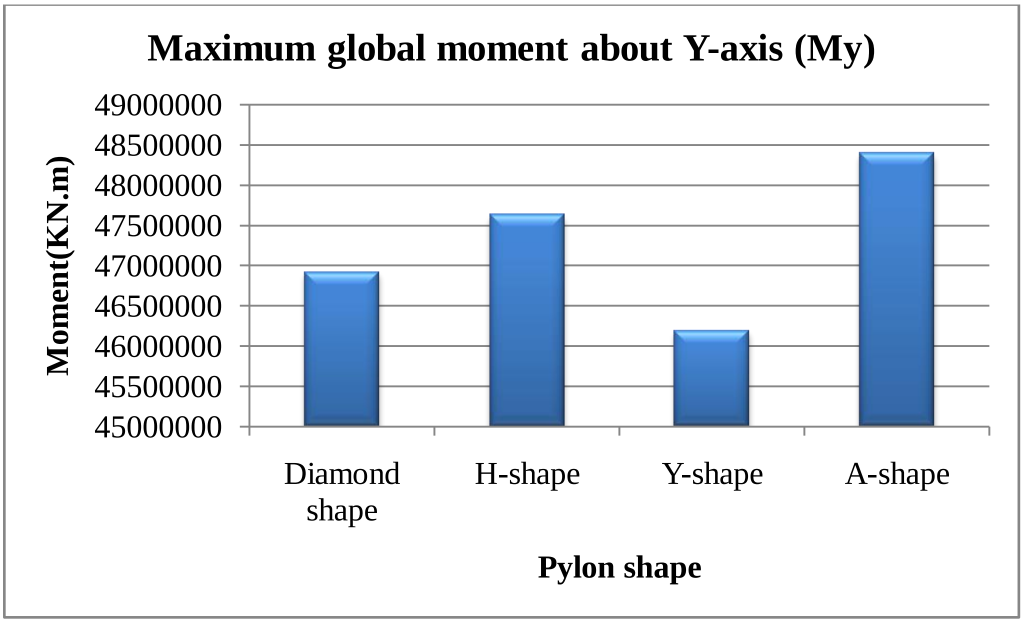


Figure 4.11. Maximum global base moment about Y-axis (M_y)

4.2. Results of dead load and static vehicular load analysis

In this study, IRC class AA tracked vehicular load is considered in traffic lanes of the bridge. This type of vehicle simulates an army tank and has a total weight of 700KN with two tracks each weighing 350KN. As per IRC: 6-2000, all the bridges located on national highways and state highways have to be designed for class AA loading[35].

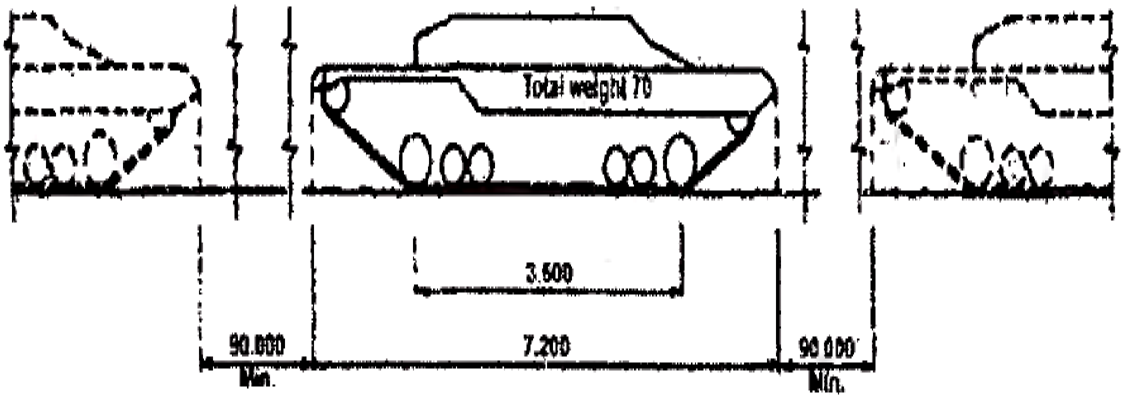


Figure 4.12.Class AA tracked vehicle, longitudinal view[35]

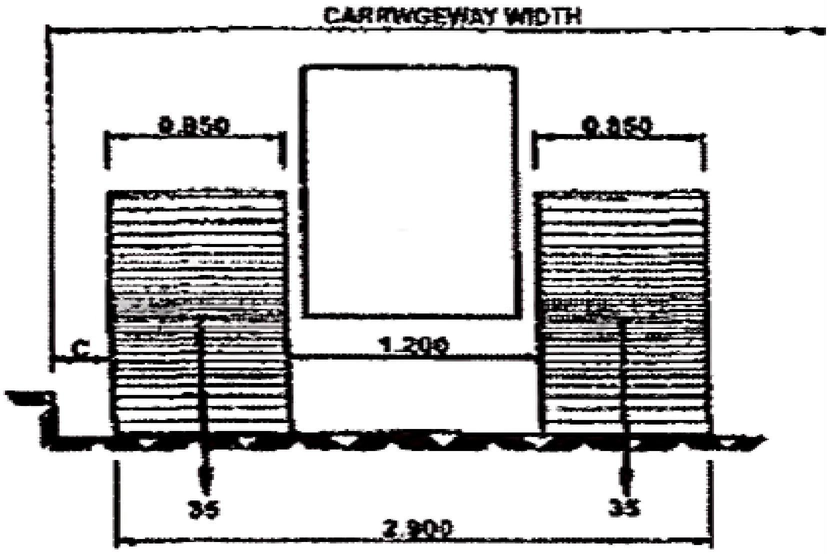
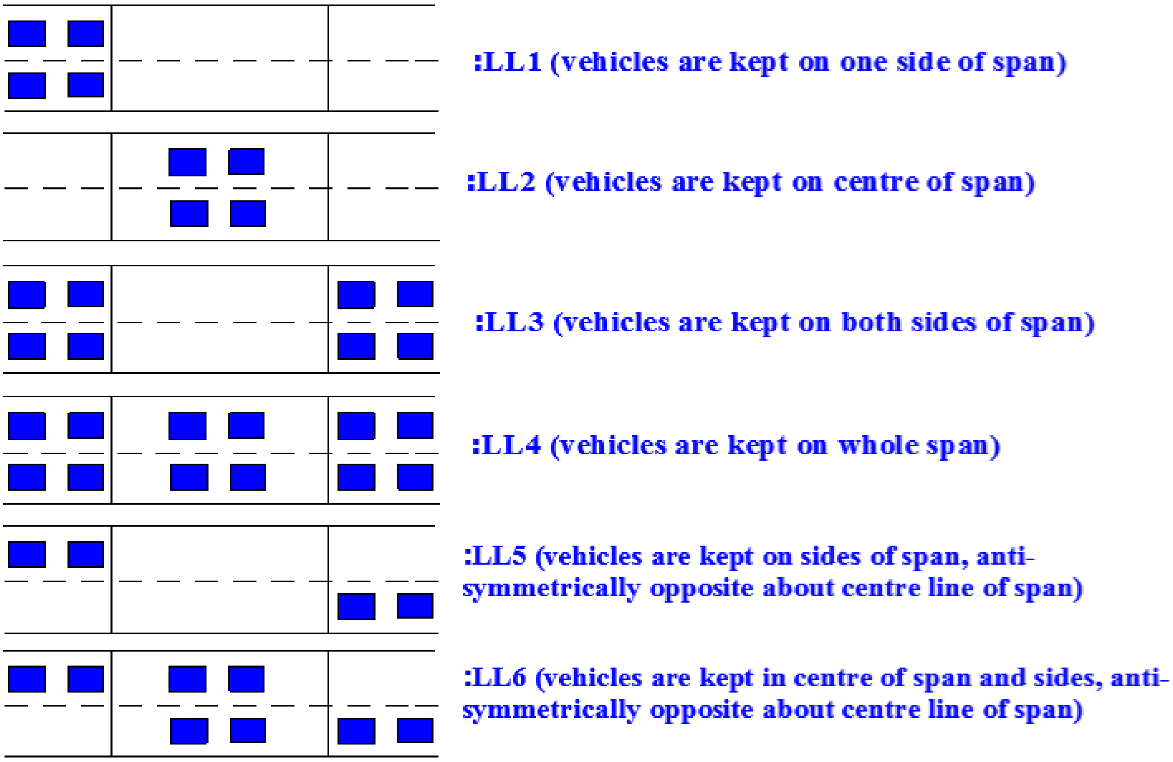


Figure 4.13.Class AA tracked vehicle, cross sectional view[35]

In order to evaluate the response of the bridge under static vehicular load, different load patterns have been considered on one side of span, centre of span, on both sides of span and on the whole span of the bridge deck by keeping the minimum distance between two vehicles which is 90m as per IRC:6-2000. The following figure shows the load patterns considered:



The following figures show the comparison of pylon shapes with respect to their response under the combination of dead load and static vehicular load.

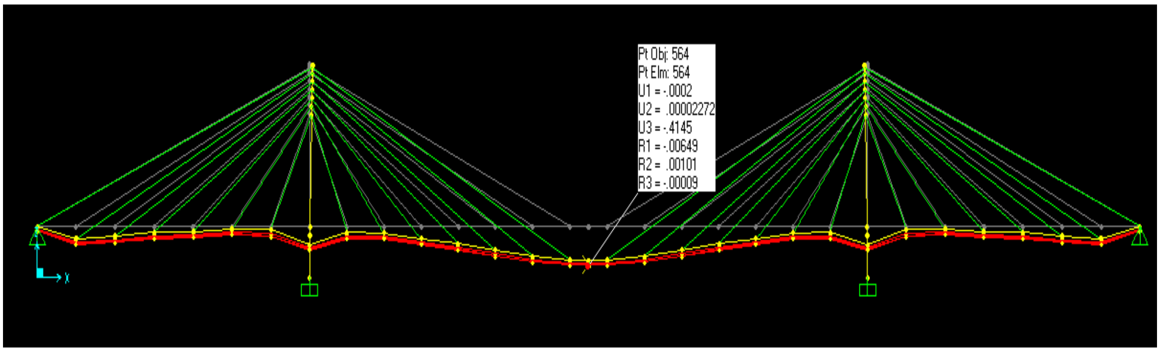


Figure 4.14. Deflected shape under combination of dead load and static vehicular load

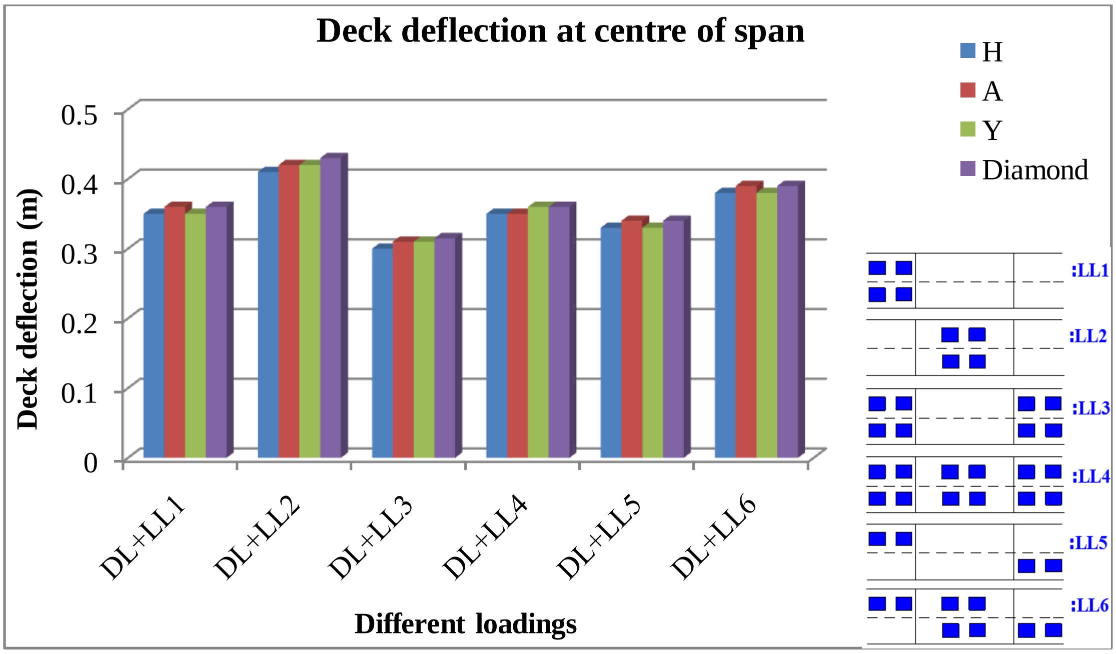


Figure 4.15. Maximum deflection of deck at mid span

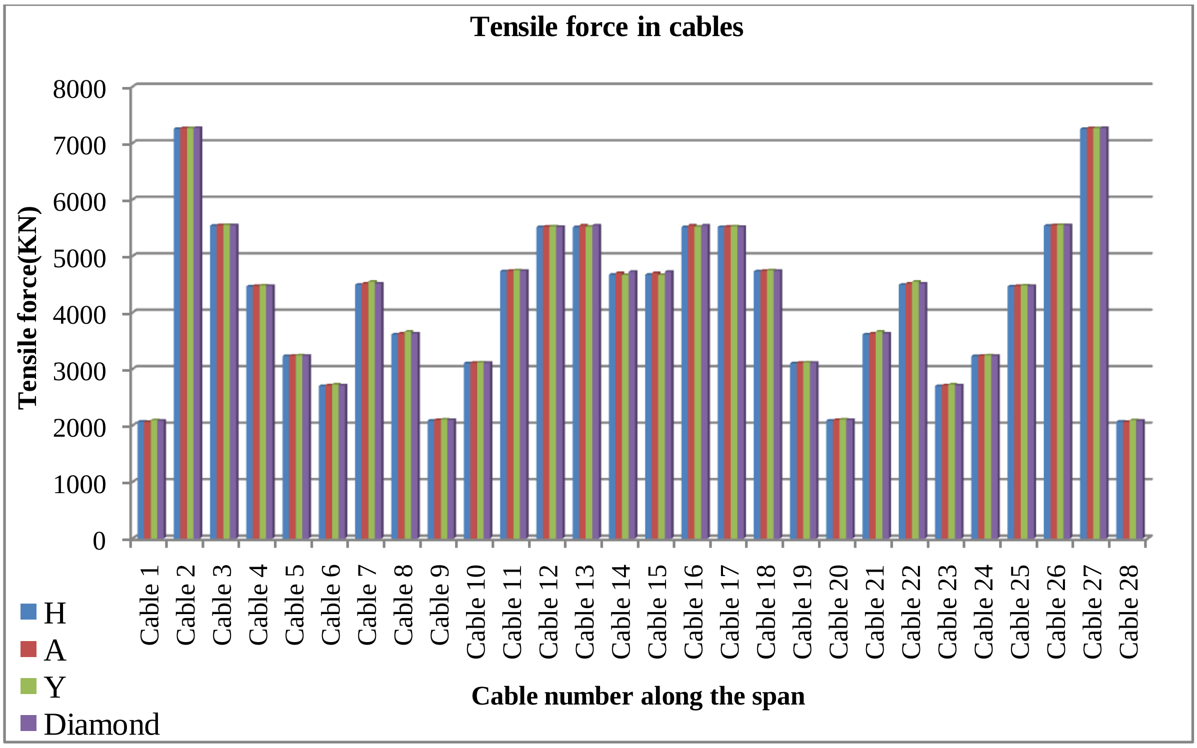


Figure 4.16 .Tensile forces in cables along the span under DL+LL4

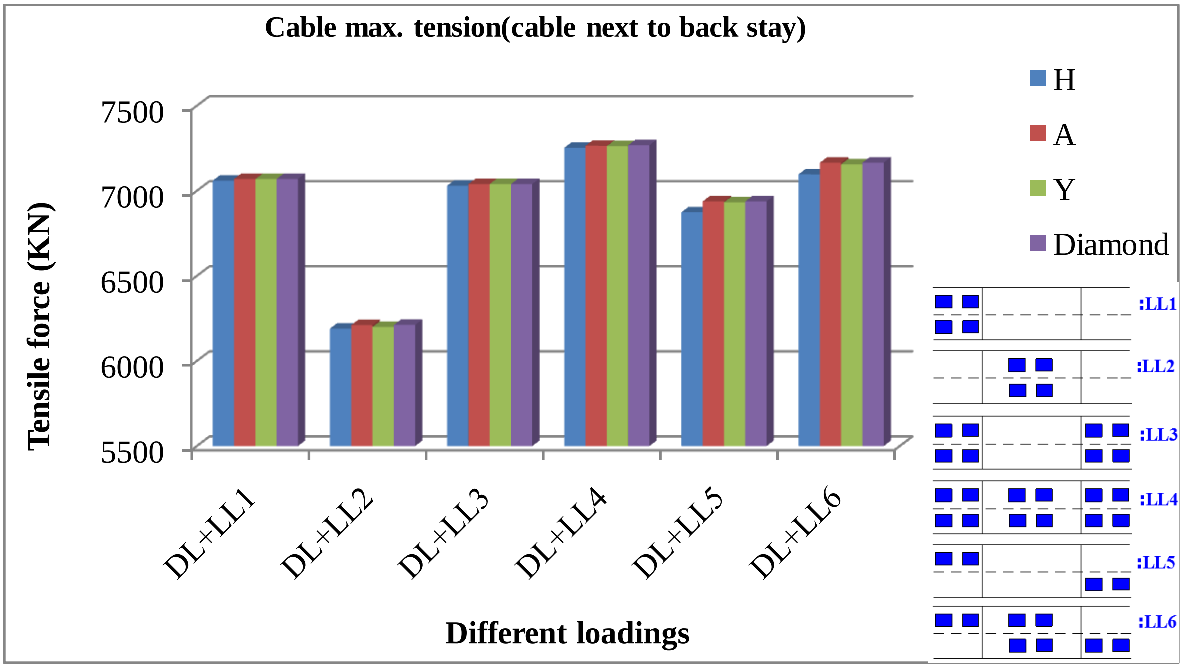


Figure 4.17. Maximum tensile force in cables (observed in 2nd cable from each end of side span)

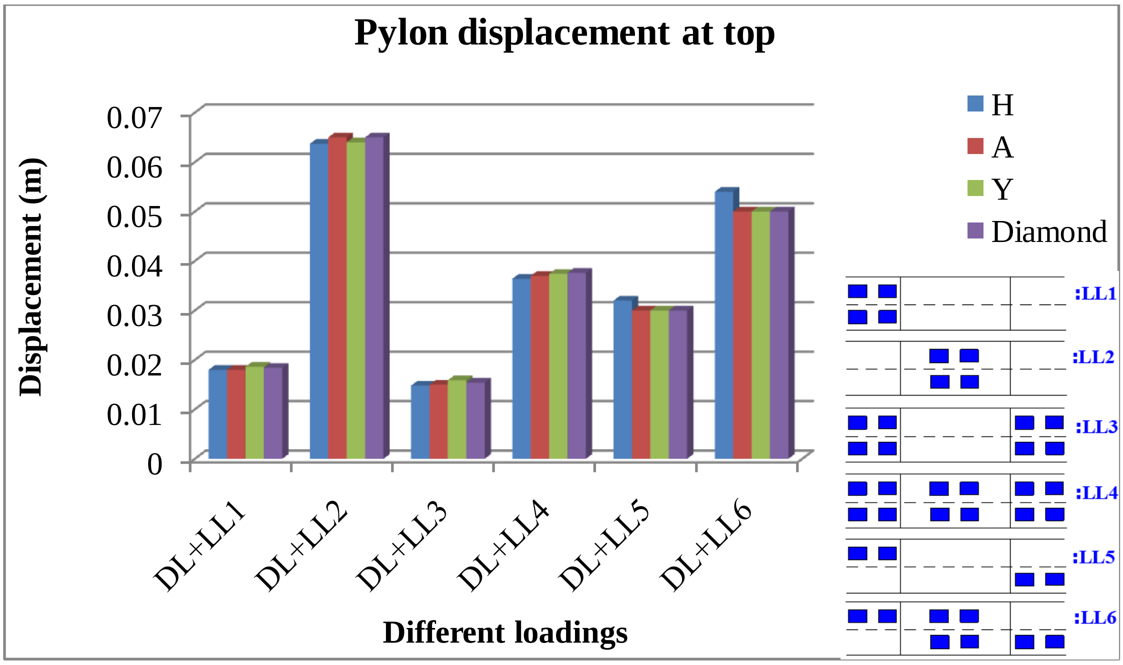


Figure 4.18. Displacement of pylon at top

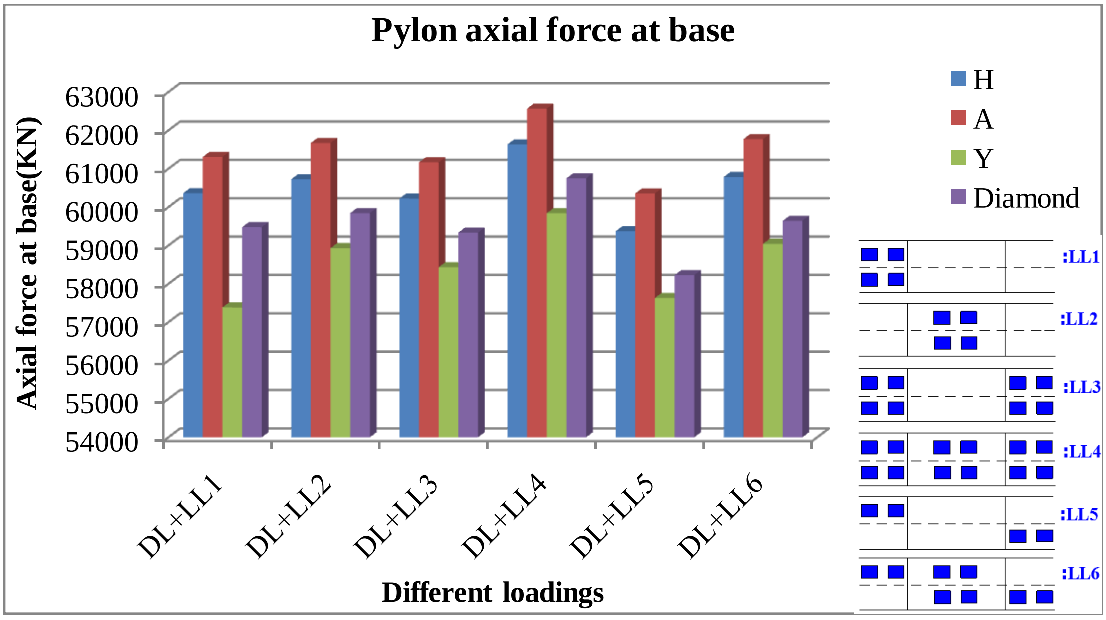


Figure 4.19. Pylon axial force at base (F_3)

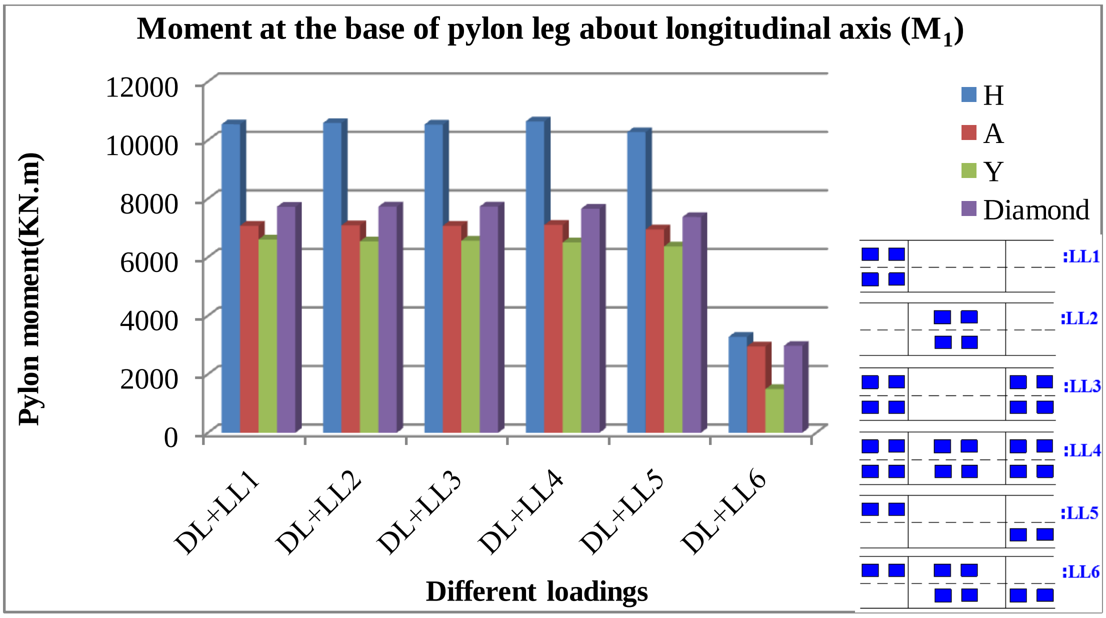


Figure 4.20. Moment at the base of pylon leg about longitudinal axis (M_1)

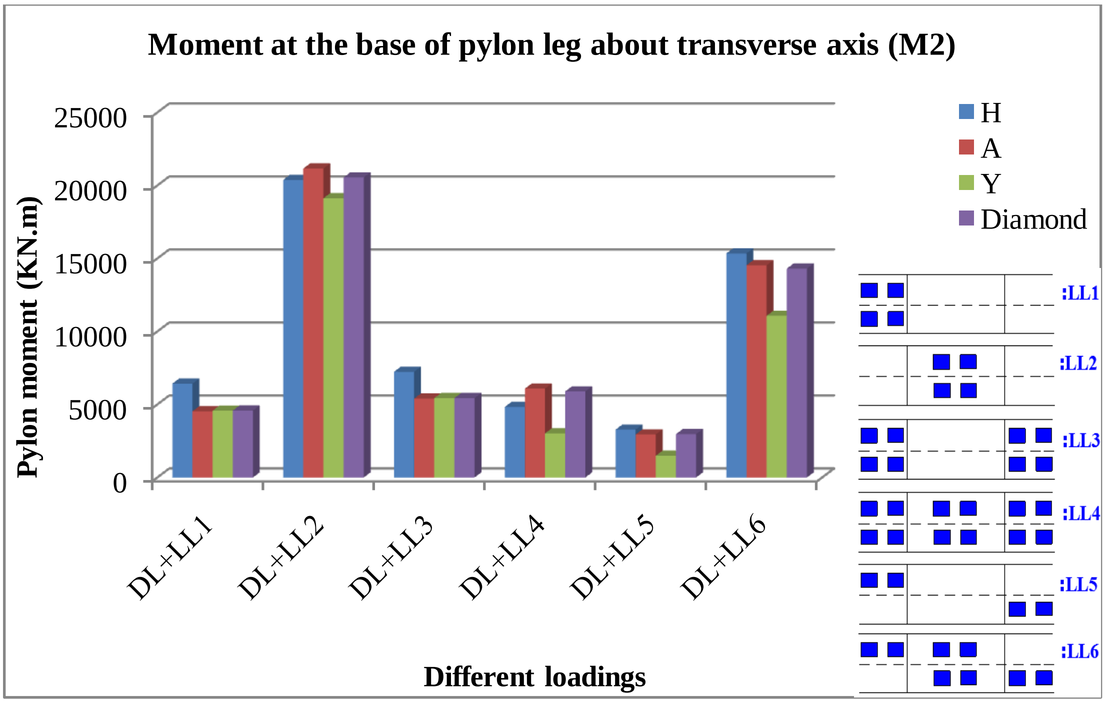


Figure 4.21. Moment at the base of pylon leg about transverse axis (M_2)

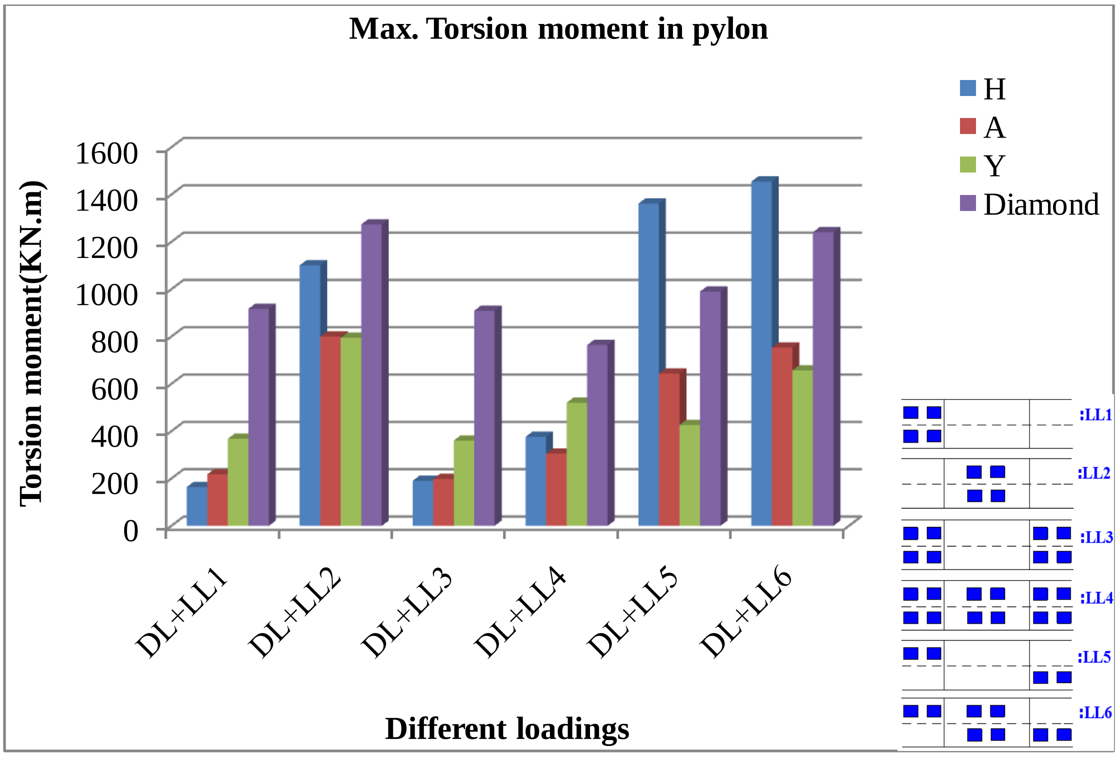


Figure 4.22. Maximum torsion moment in pylon

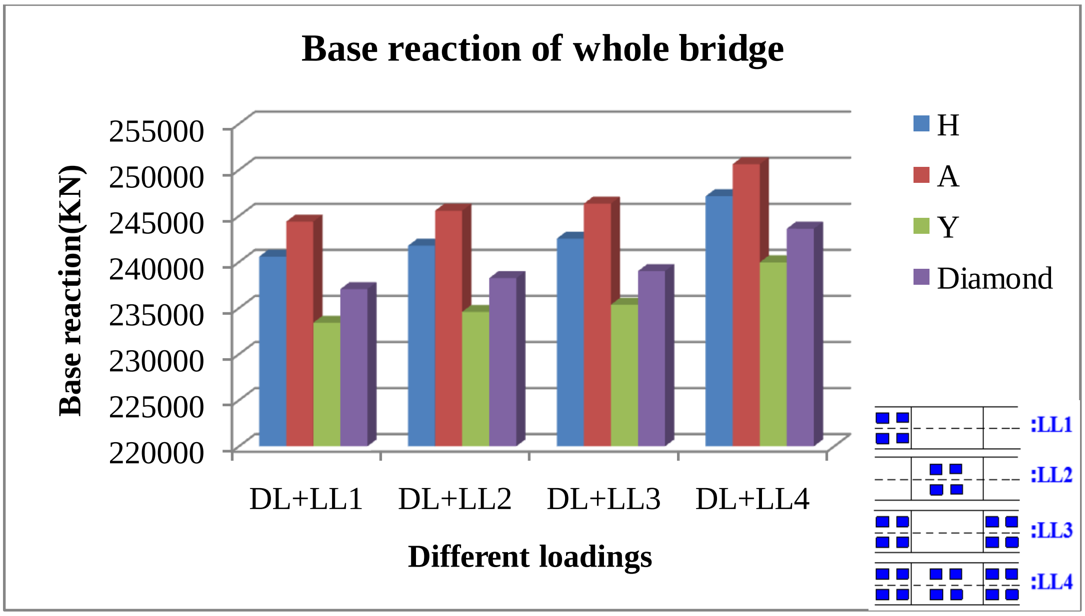


Figure 4.23. Base reaction of the whole bridge

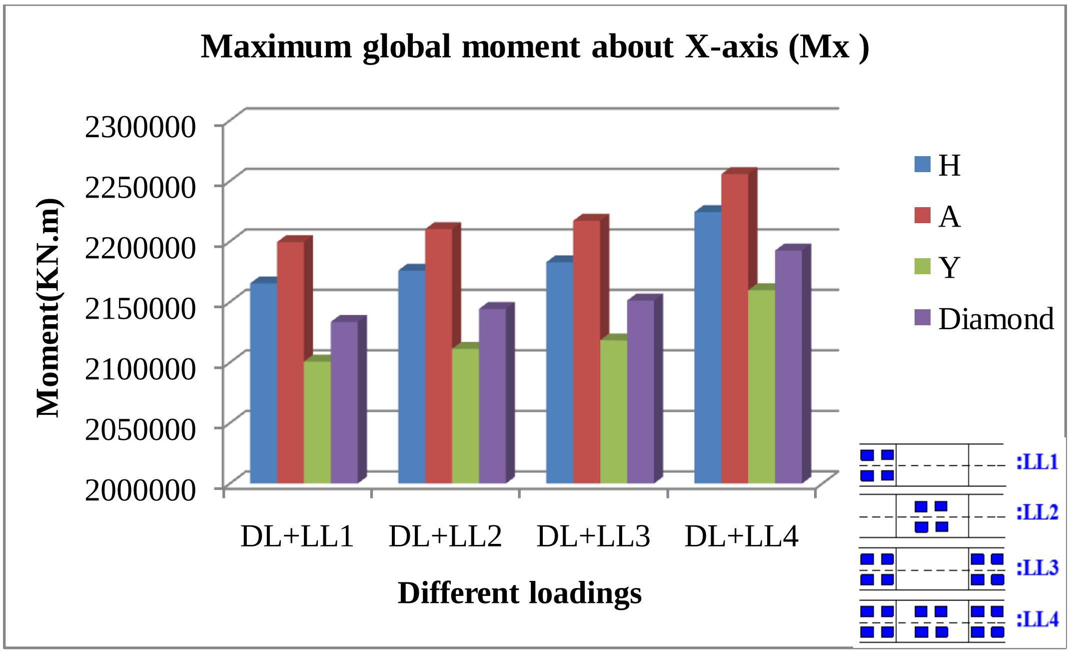


Figure 4.24. Maximum global base moment about X-axis (M_x)

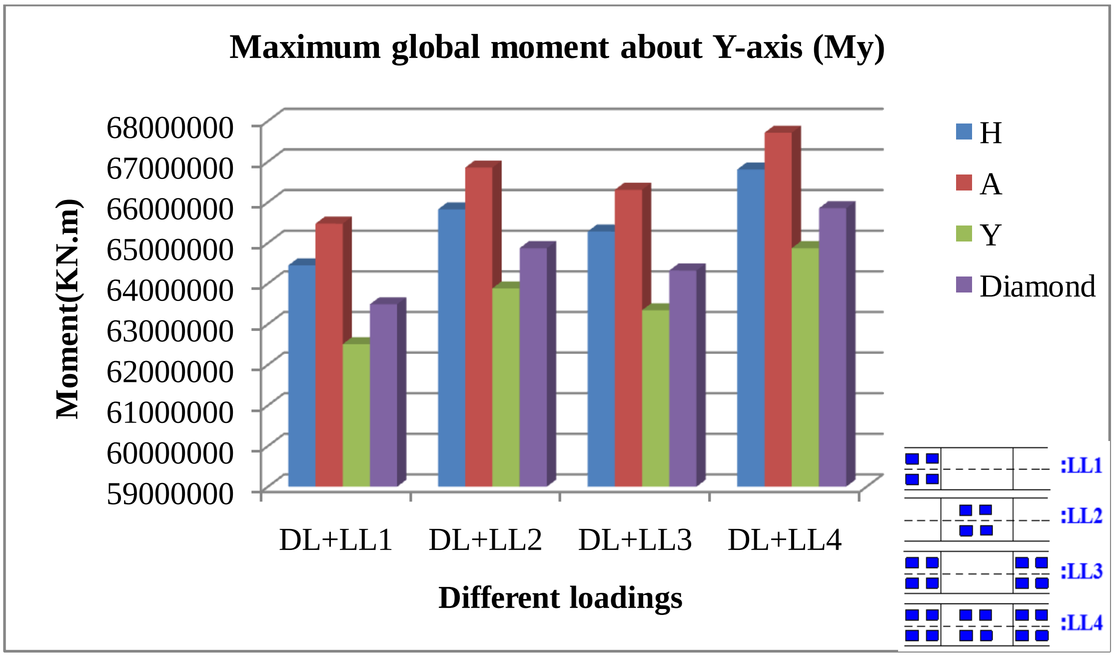


Figure 4.25. Maximum global base moment about Y-axis (M_y)

4.3. Results of dead load and moving vehicular load analysis

Moving loads on bridge deck cause the superstructure comprising beams and slabs to deflect from its equilibrium position relatively quickly. The mass and inherent elasticity of the structure tends to restore the bridge deck to its equilibrium position thus causing a series of vibrations due to the motion of vehicles on the bridge deck. The normal practice generally followed in several national codes to safeguard the bridge deck from the destructive effects of dynamic loads is to provide for impact factors for live loads which amplify the design static loads by a certain percentage. Consequently, the bridge is rendered more rigid so that the dynamic effects are safely resisted with increased mass and elasticity of the structure[25].

The type of vehicle considered for dynamic analysis is IRC class AA wheeled vehicle at a speed of 100km/hour.

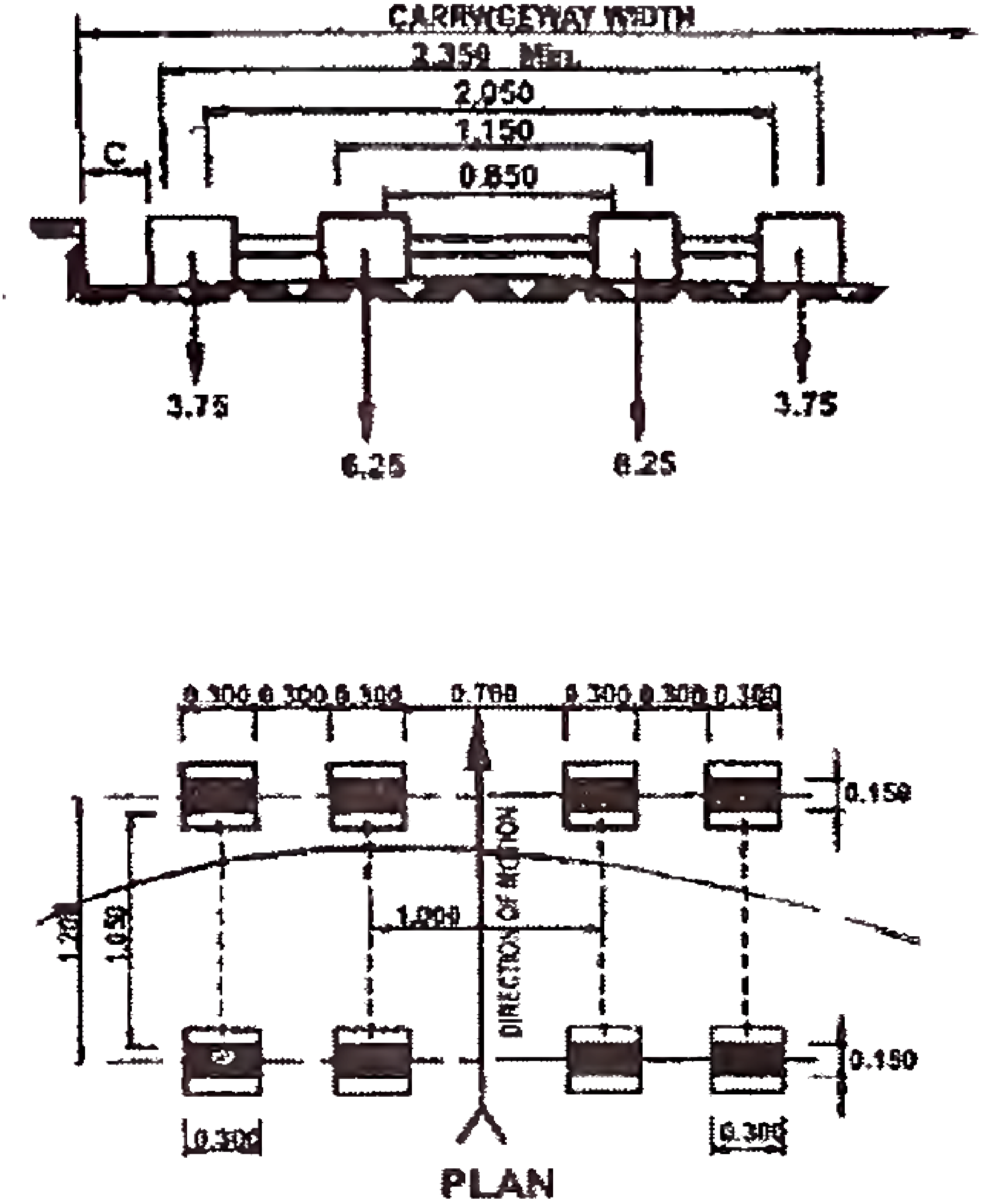


Figure 4.26. AA class wheeled vehicle [35]

The following figures show the comparison of pylon shapes with respect to their response under the combination of dead load and moving vehicular load.

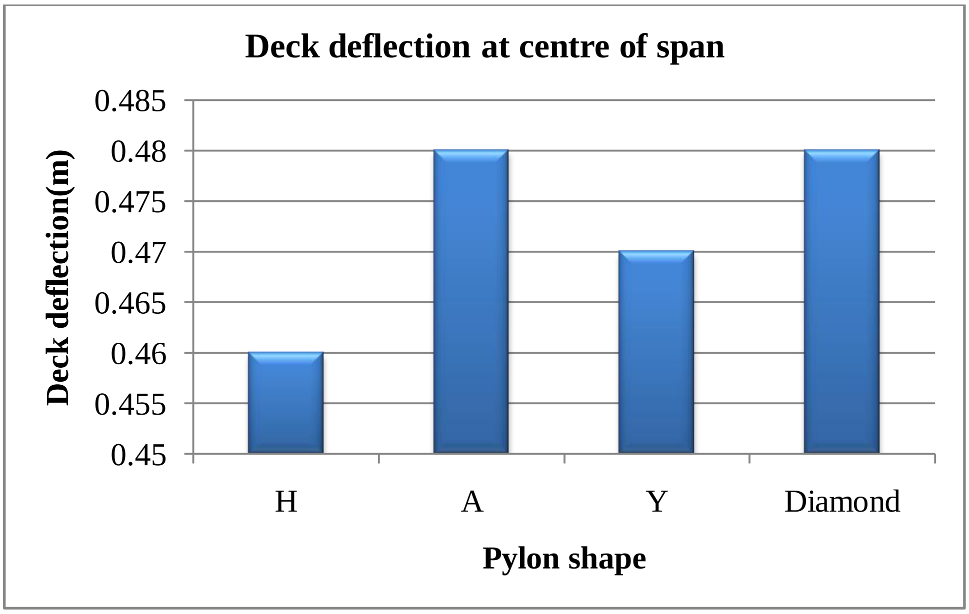


Figure 4.27. Maximum deflection of deck at mid span

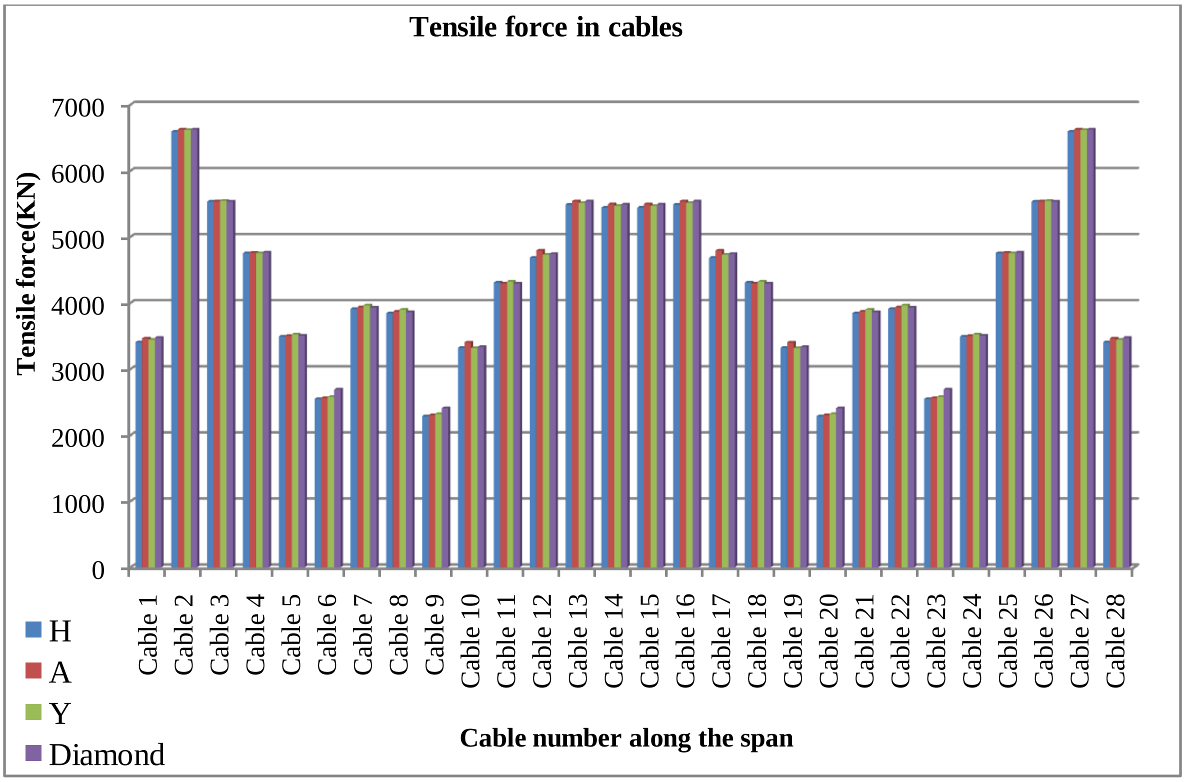


Figure 4.28.Tensile forces in cables along the span of the bridge

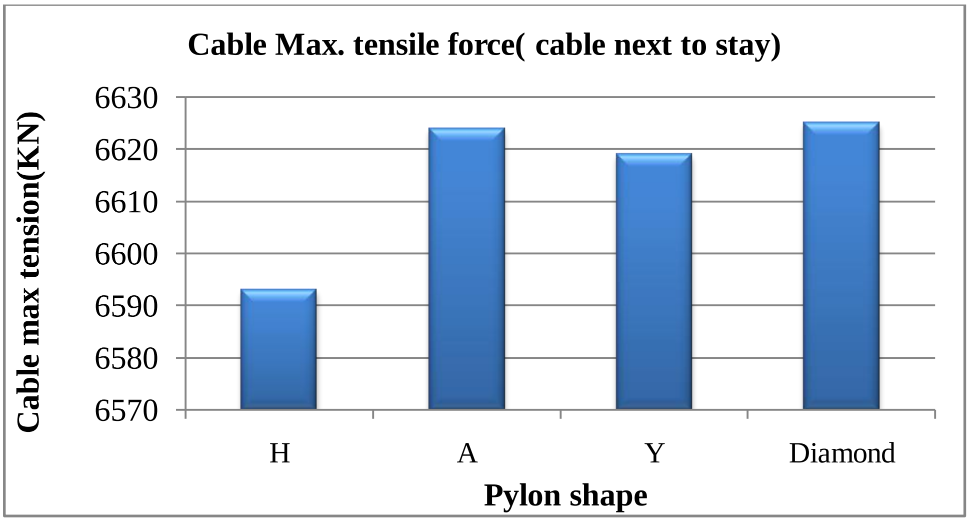


Figure 4.29. Maximum tensile force in cables (observed in 2^{nd} cable from each end of side span)

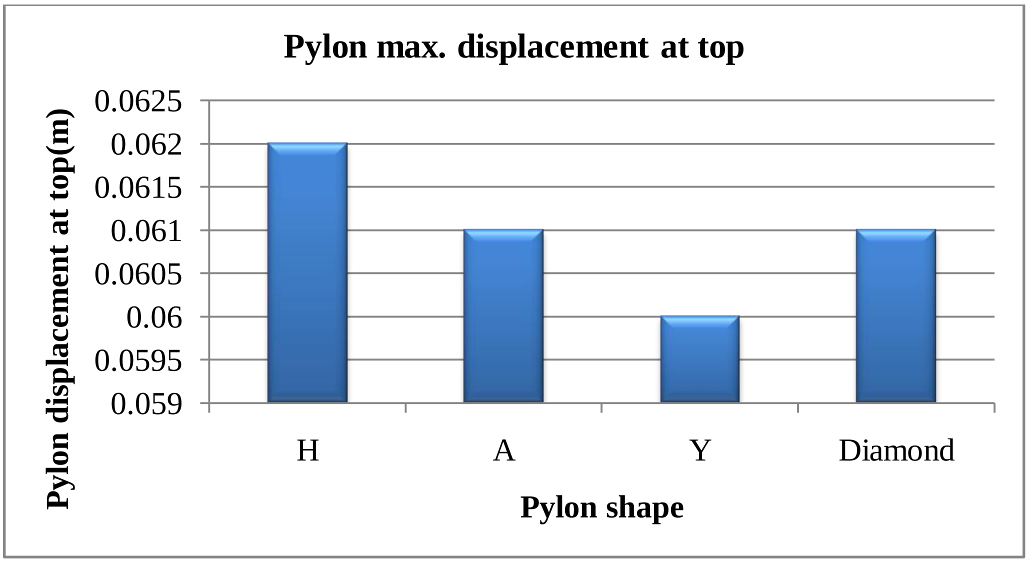


Figure 4.30. Displacement of pylon at top

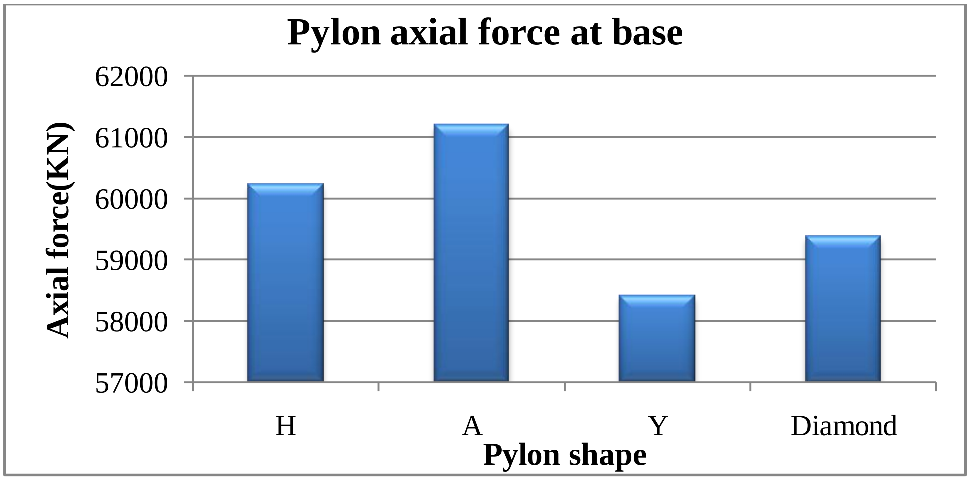


Figure 4.31. Pylon axial force at base (F_3)

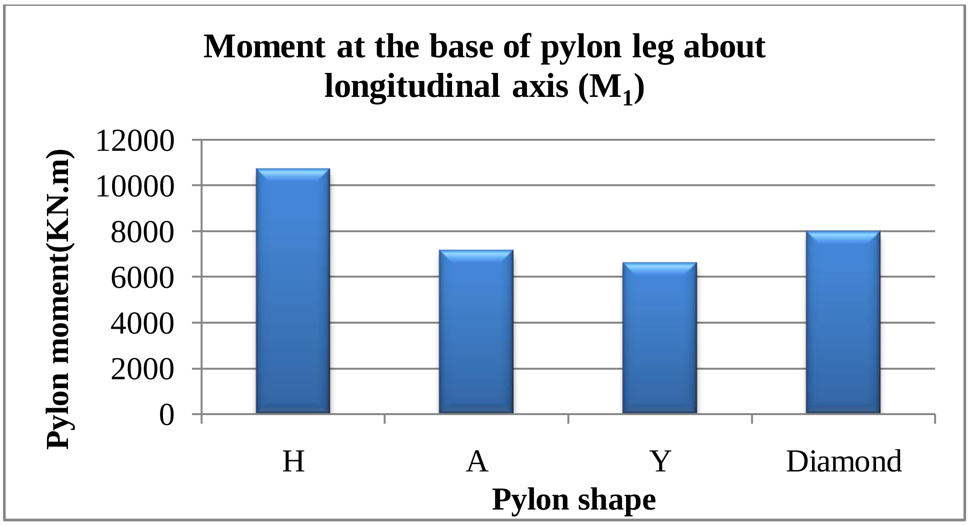


Figure 4.32. Moment at the base of pylon leg about longitudinal axis (M_1)

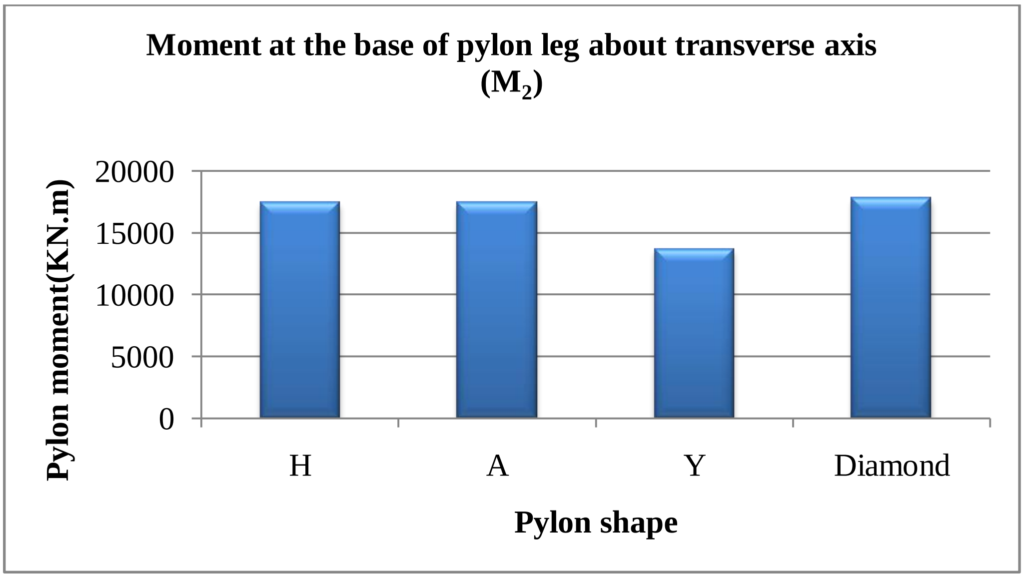


Figure 4.33. Moment at the base of pylon leg about transverse axis (M_2)

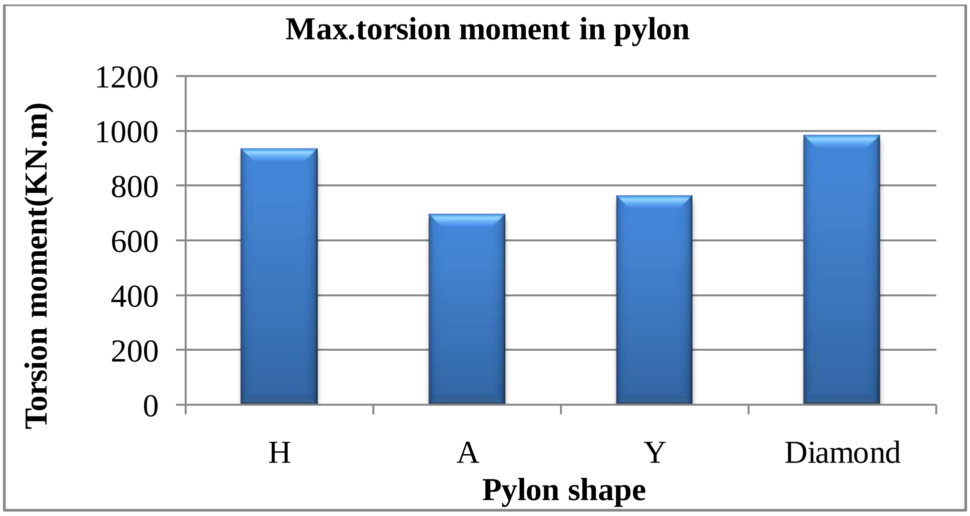


Figure 4.34. Maximum torsion moment in pylon

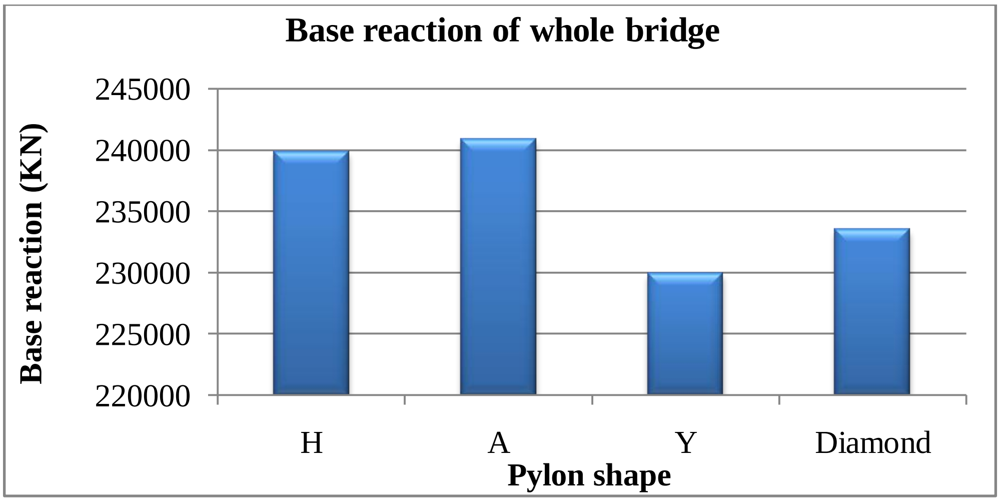


Figure 4.35. Base reaction of the whole bridge

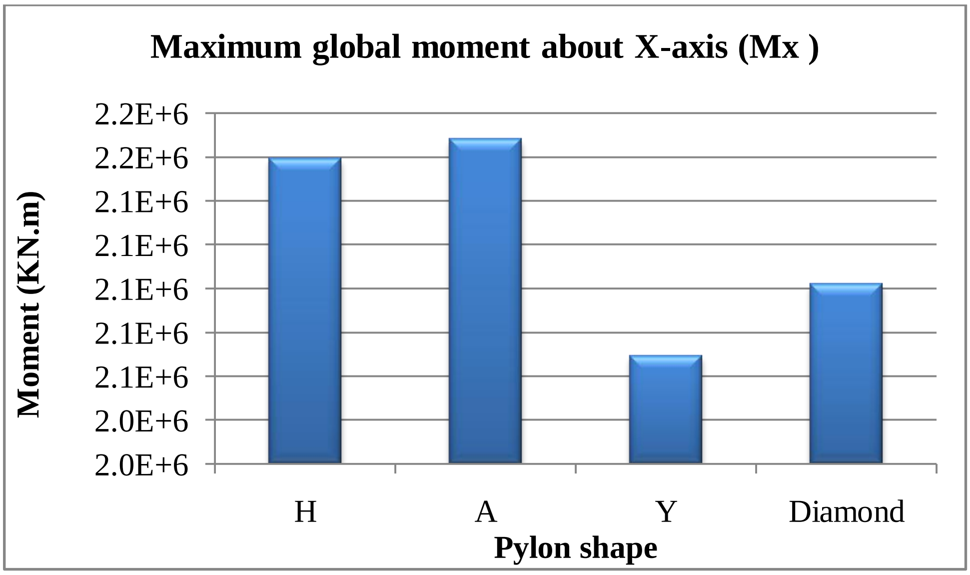


Figure 4.36. Maximum global base moment about X-axis (M_x)

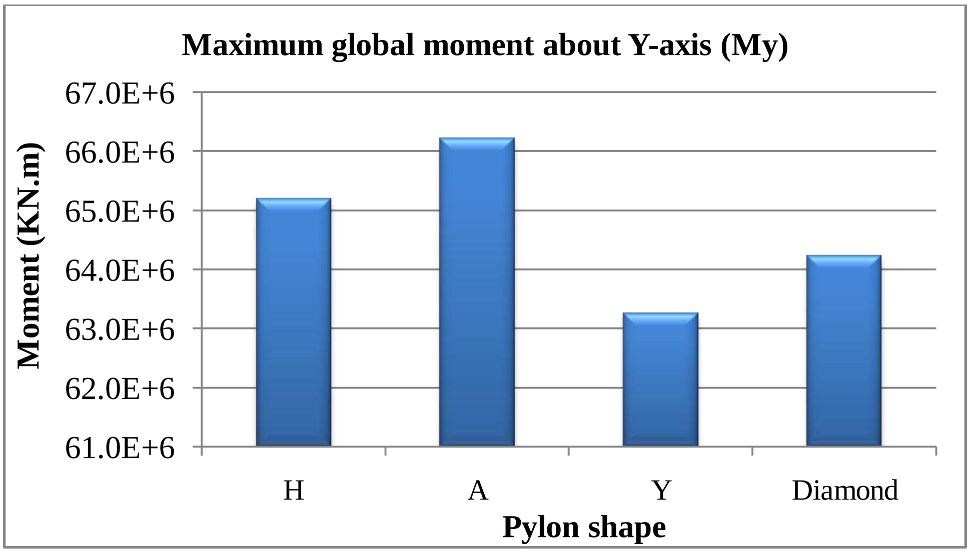


Figure 4.37. Maximum global base moment about Y-axis (M_y)

The first fifteen modes for each shape are extracted from modal analysis results as shown below:

4.4. Modal analysis results

The modal analysis has been performed to evaluate the natural frequencies of the bridge, and then first natural frequencies of torsion and heaving are used to predict the susceptibility of the bridge to flutter.

4.4.1. Mode shapes of A-shape

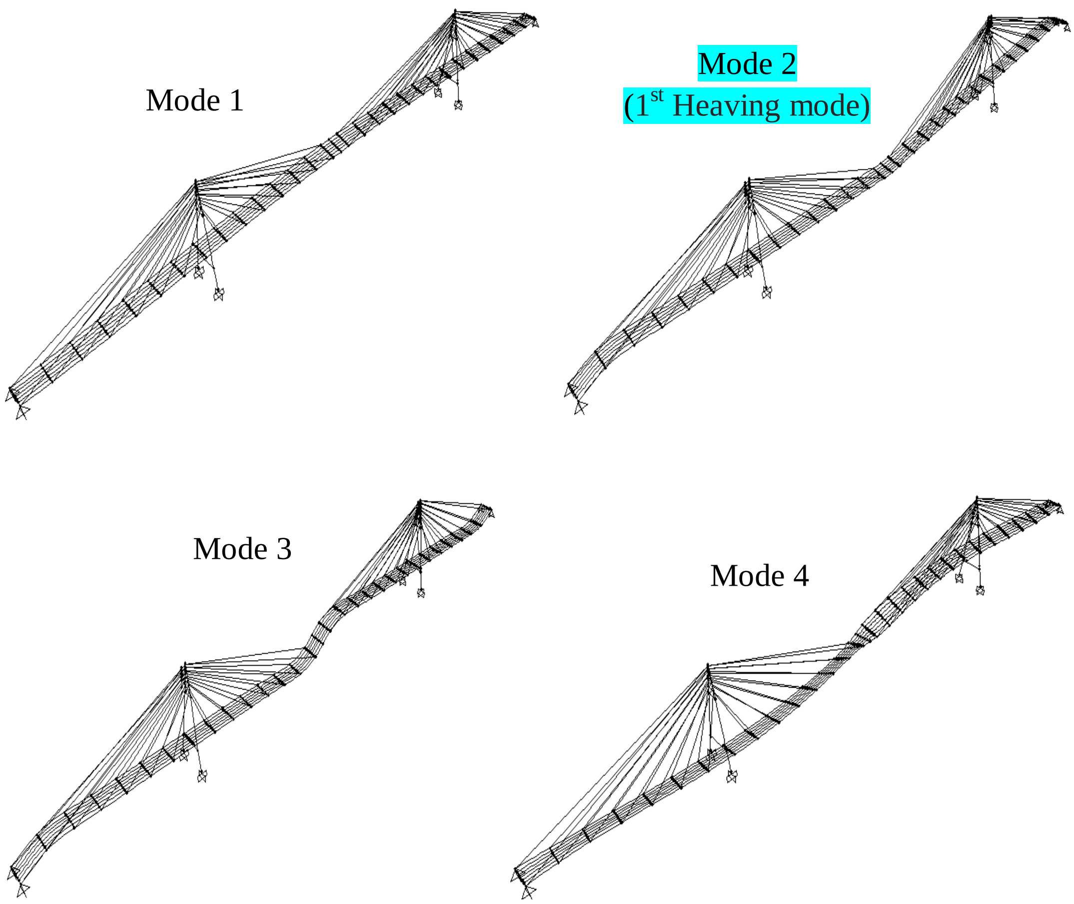


Figure 4.38-A . Natural frequencies and mode shapes for cable-stayed bridge with A-shape

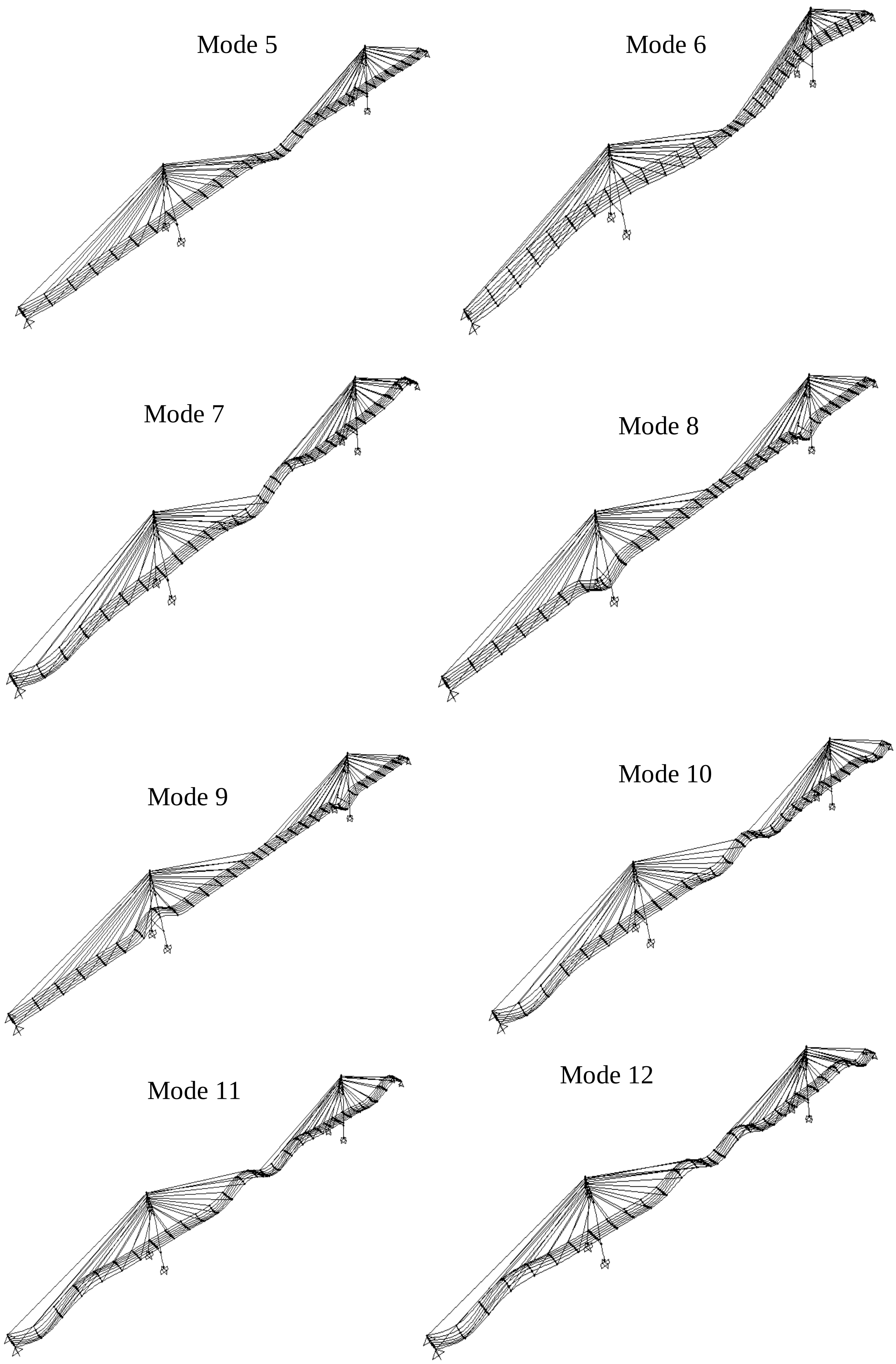


Figure 4.38-B . Natural frequencies and mode shapes for cable-stayed bridge with A-shape

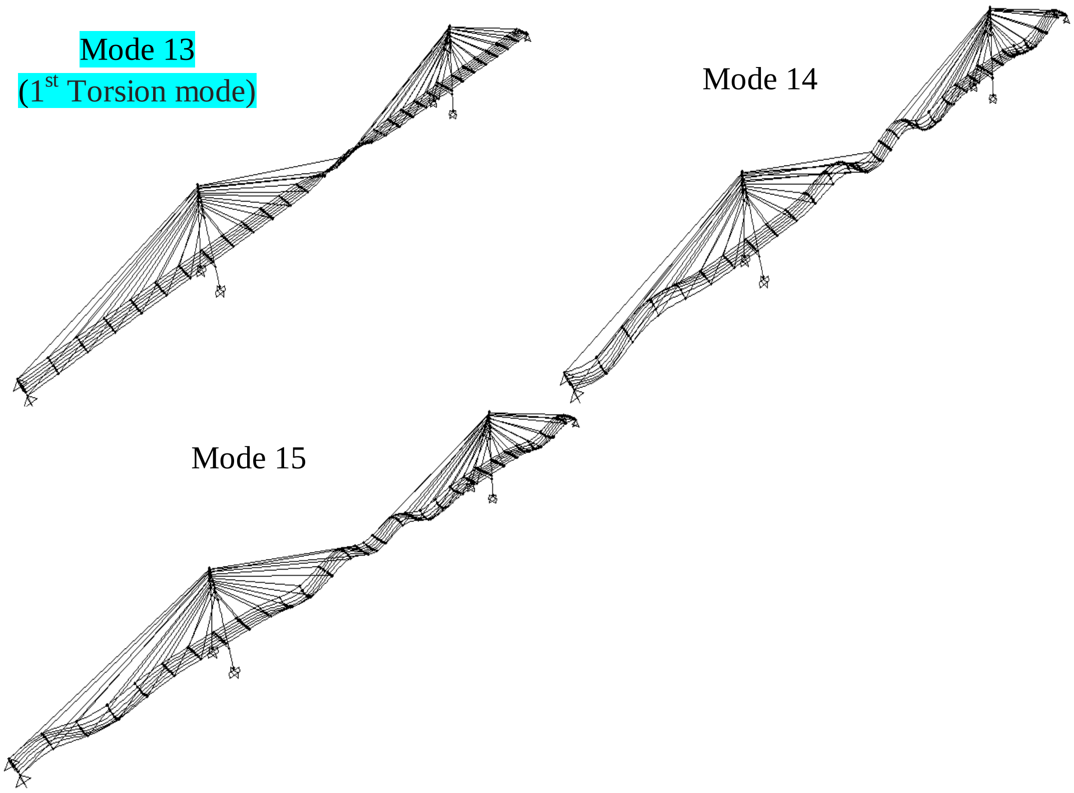


Figure 4.38 -C. Natural frequencies and mode shapes for cable-stayed bridge with A-shape

In the above figures, all the fifteen mode shapes of the A-shape pylon cable-stayed bridge have been extracted after modal analysis. The frequency, time period, circular frequency, eigen value and modal character of each mode shape are given in the following table:

Table 4.1: Modal Periods and Frequencies for A-shape pylon

Mode shape	**Period** (s)	**Frequency** (Hz)	**Circular Frequency(ω)** rad/sec	**Eigen value(ω^2)** rad^2/sec^2	**Modal character**
1	2.72598	0.36684	2.3037	5.3071	S-H-1
2	1.87207	0.53417	3.3563	11.265	S-V-1
3	1.61544	0.61903	3.8895	15.128	A-V-2
4	1.48484	0.67347	4.2316	17.906	A-H-2
5	0.84932	1.1774	7.3979	54.728	S-V-3
6	0.78386	1.2757	8.0157	64.251	S-H-3
7	0.76892	1.3005	8.1714	66.772	A-V-4
8	0.74228	1.3472	8.4648	71.652	S-V-5
9	0.74203	1.3476	8.4675	71.699	A-V-6
10	0.71459	1.3994	8.7928	77.313	S-V-7
11	0.67159	1.489	9.3557	87.529	A-V-8
12	0.64694	1.5457	9.7122	94.327	S-V-9
13	0.619	1.6155	10.151	103.03	S-T-1
14	0.60843	1.6436	10.327	106.64	A-V-10
15	0.58385	1.7128	10.762	115.81	S-V-11

Where:

S=symmetric mode

A=asymmetric mode

V=bending (heaving) mode

T=torsional mode

4.4.2. Mode shapes of H-shape

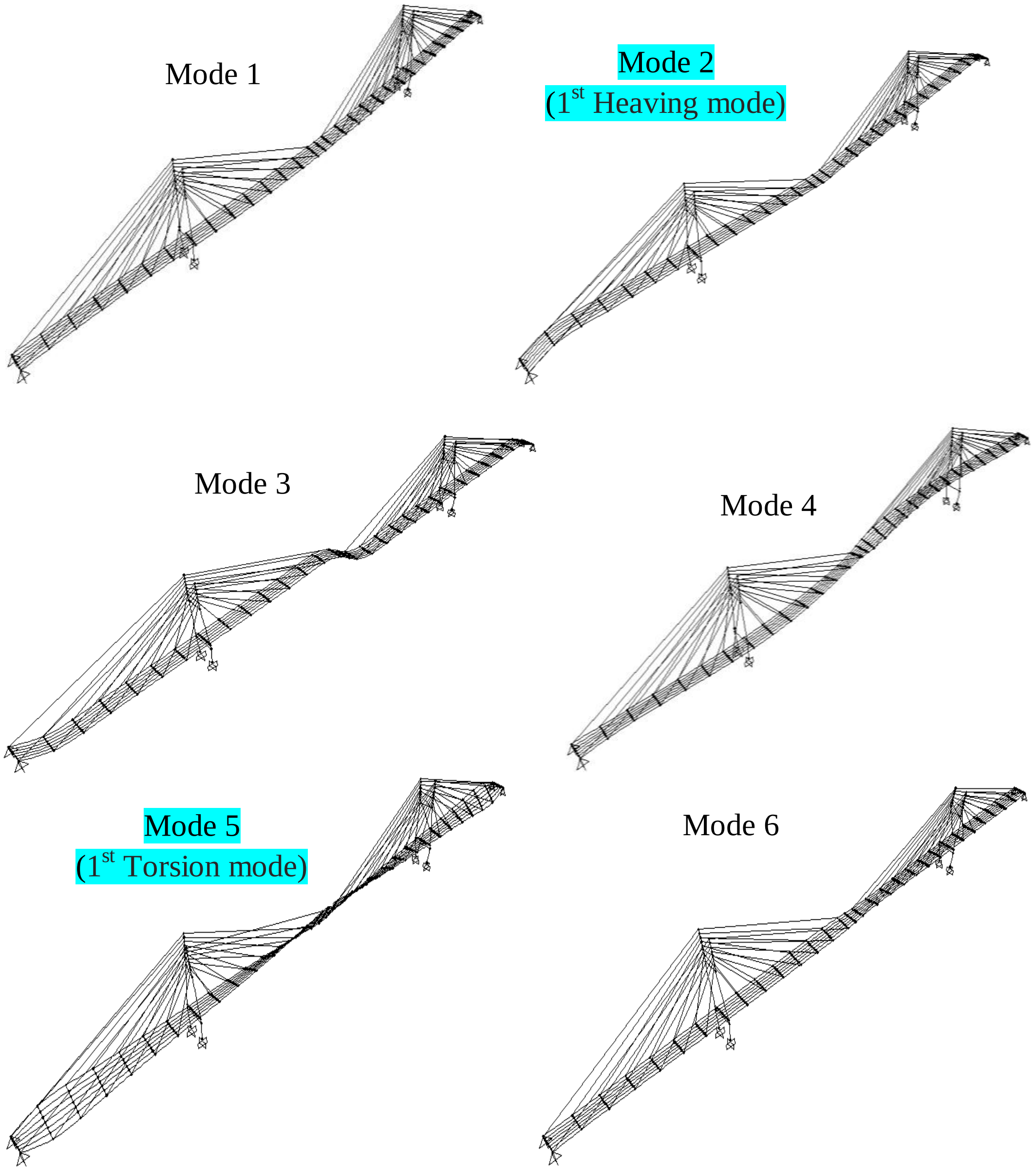


Figure 4.39-A . Natural frequencies and mode shapes for cable-stayed bridge with H-shape

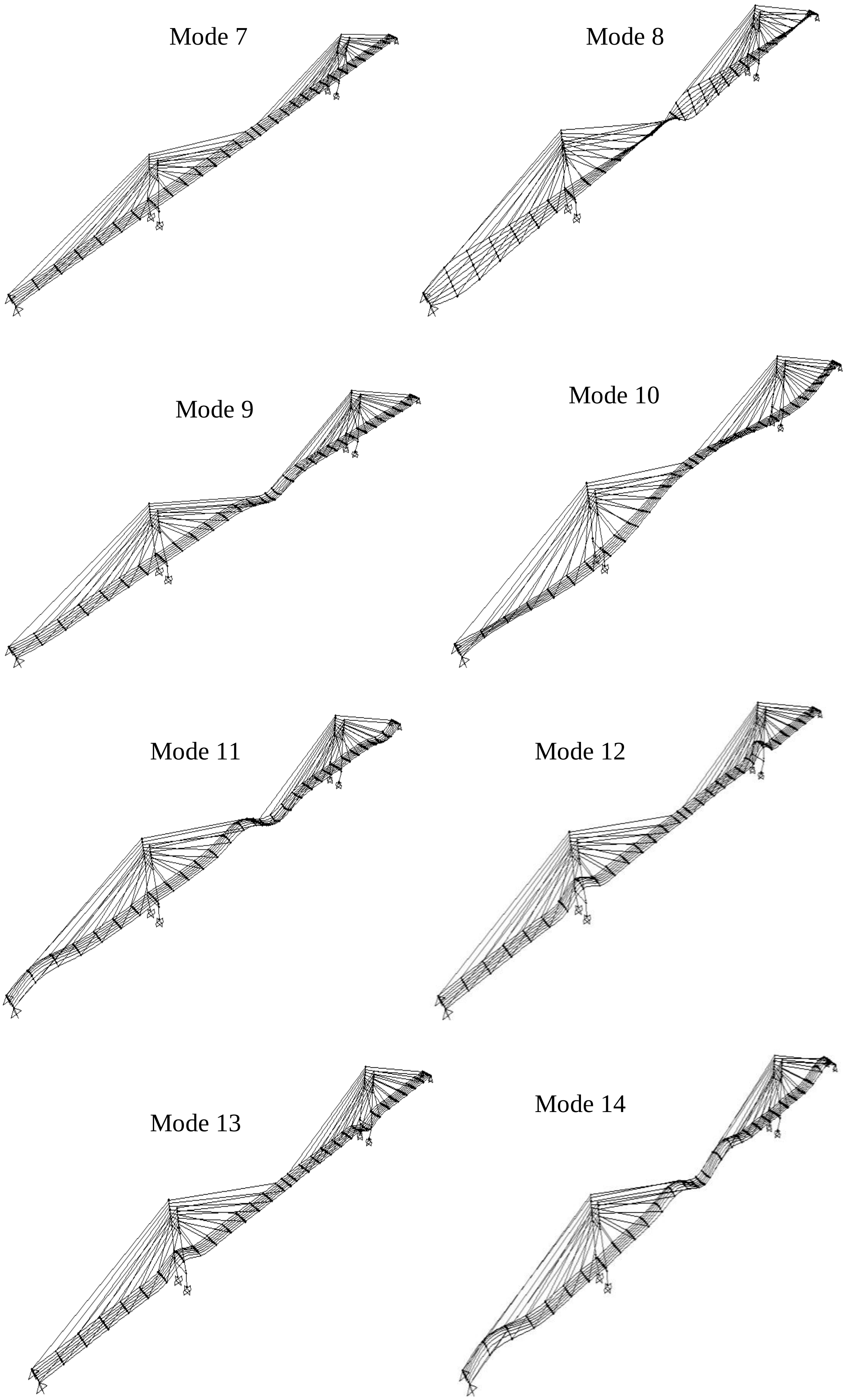


Figure 4.39-B. Natural frequencies and mode shapes for cable-stayed bridge with H-shape

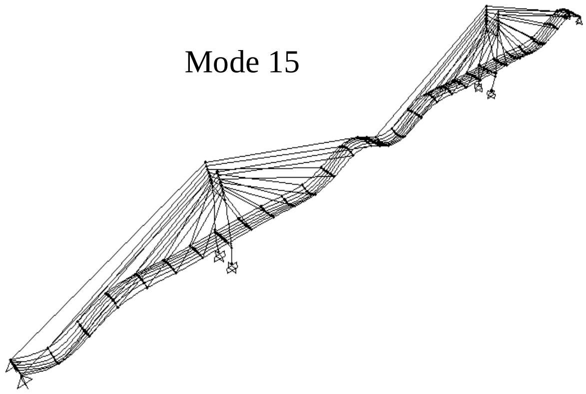


Figure 4.39-C. Natural frequencies and mode shapes for cable-stayed bridge with H-shape

In the above figures, all the fifteen mode shapes of the H-shape pylon cable-stayed bridge have been extracted after modal analysis. The frequency, time period, circular frequency, eigen value and modal character of each mode shape are given in the following table:

Table 4.2 : Modal Periods and Frequencies for H-shape pylon

Mode shape	**Period** (s)	**Frequency** (Hz)	**Circular Frequency(ω)** rad/sec	**Eigen value(ω^2)** rad^2/sec^2	**Modal character**
1	2.71318	0.36857	2.3146	5.3573	S-H-1
2	1.858714	0.53801	3.3804	11.427	S-V-1
3	1.614993	0.6192	3.8905	15.136	A-V-2
4	1.474111	0.67838	4.2624	18.168	A-H-2
5	1.102356	0.90715	5.6998	32.488	S-T-1
6	0.994975	1.0051	6.3149	39.878	A-H-3
7	0.99452	1.0055	6.3178	39.915	S-H-4
8	0.955387	1.0467	6.5766	43.252	A-T-2
9	0.848217	1.1789	7.4075	54.871	S-V-3
10	0.780516	1.2812	8.05	64.803	S-H-5
11	0.767932	1.3022	8.182	66.944	A-V-4
12	0.74132	1.3489	8.4757	71.837	S-V-5
13	0.741082	1.3494	8.4784	71.883	A-V-6
14	0.71369	1.4012	8.8038	77.507	S-V-7
15	0.67121	1.4898	9.361	87.628	A-V-8

Where:

S=symmetric mode

A=asymmetric mode

V=bending (heaving) mode

T=torsional mode

4.4.3. Mode shapes of inverted -Y shape

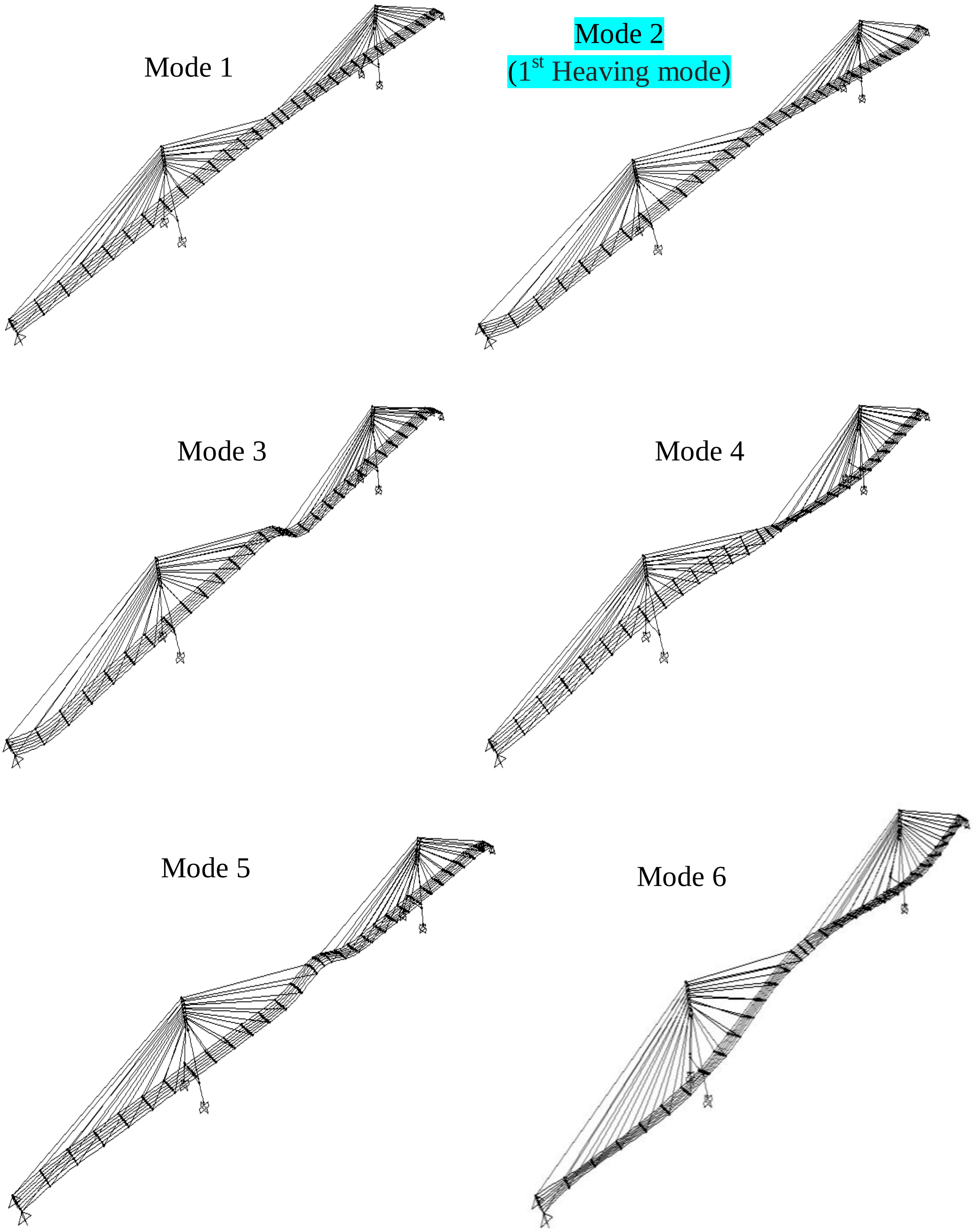


Figure 4.40-A. Natural frequencies and mode shapes for cable-stayed bridge with inverted-Y shape

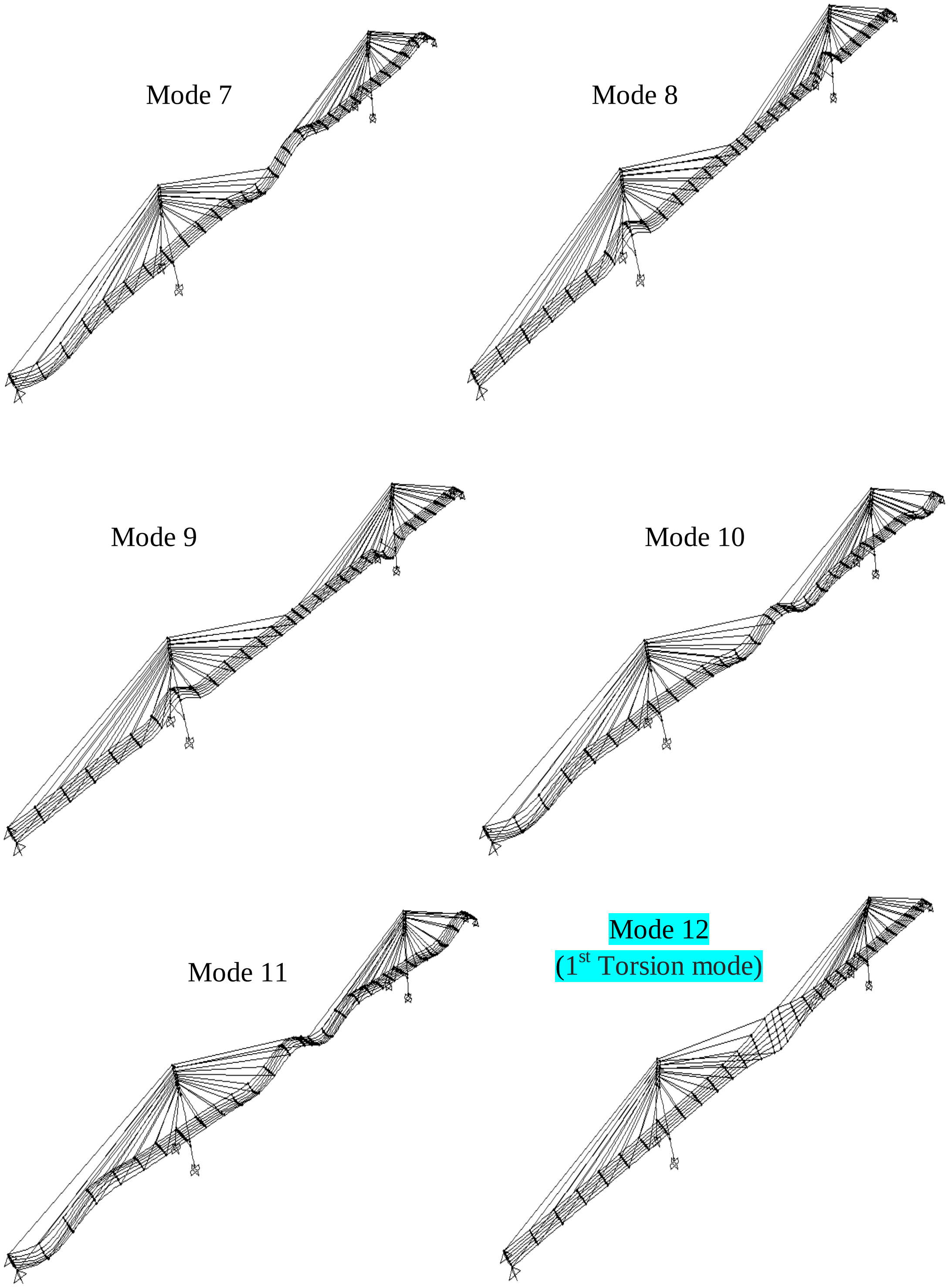


Figure 4.40-B . Natural frequencies and mode shapes for cable-stayed bridge with inverted-Y shape

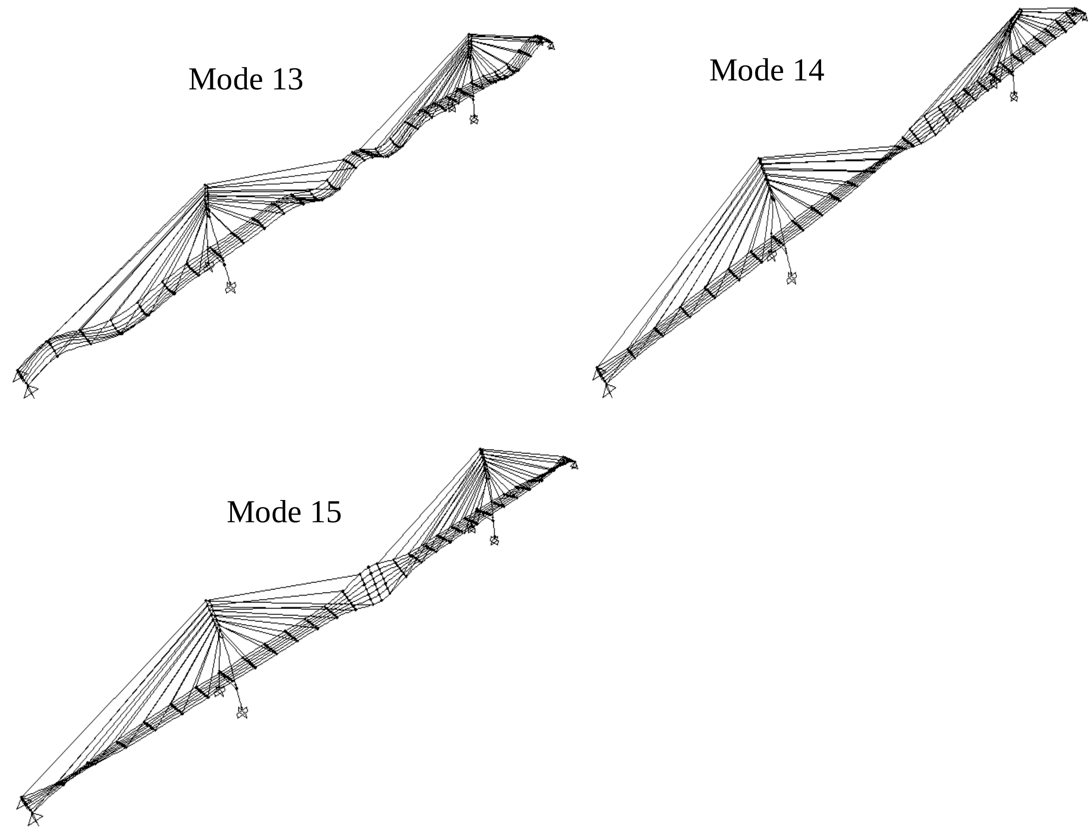


Figure 4.40-C. Natural frequencies and mode shapes for cable-stayed bridge with inverted-Y shape

In the above figures, all the fifteen mode shapes of the inverted Y-shape pylon cable-stayed bridge have been extracted after modal analysis. The frequency, time period, circular frequency, eigen value and modal character of each mode shape are given in the following table:

Table 4.3: Modal Periods and Frequencies for inverted-Y shape pylon

Mode shape	**Period** (s)	**Frequency** (Hz)	**Circular Frequency(ω)** rad/sec	**Eigen value(ω^2)** rad^2/sec^2	**Modal character**
1	2.72598	0.36682	2.3036	5.3066	S-H-1
2	1.85798	0.53822	3.3817	11.436	S-V-1
3	1.613635	0.61972	3.8938	15.162	A-V-2
4	1.486766	0.6726	4.2261	17.86	A-H-2
5	0.848753	1.1782	7.4028	54.802	S-V-3
6	0.785848	1.2725	7.9954	63.927	S-H-3
7	0.768705	1.3009	8.1737	66.81	A-V-4
8	0.74279	1.3463	8.4589	71.553	S-V-5
9	0.742534	1.3467	8.4618	71.602	A-V-6
10	0.714594	1.3994	8.7927	77.311	S-V-7
11	0.671637	1.4889	9.355	87.517	A-V-8
12	0.667025	1.4992	9.4197	88.731	S-T-1
13	0.645545	1.5491	9.7331	94.734	S-V-9
14	0.632974	1.5798	9.9265	98.534	A-H-4
15	0.608923	1.6422	10.319	106.47	S-T-2

Where:

S=symmetric mode

A=asymmetric mode

V=bending (heaving) mode

T=torsional mode

4.4.4. Mode shapes of Diamond-shape

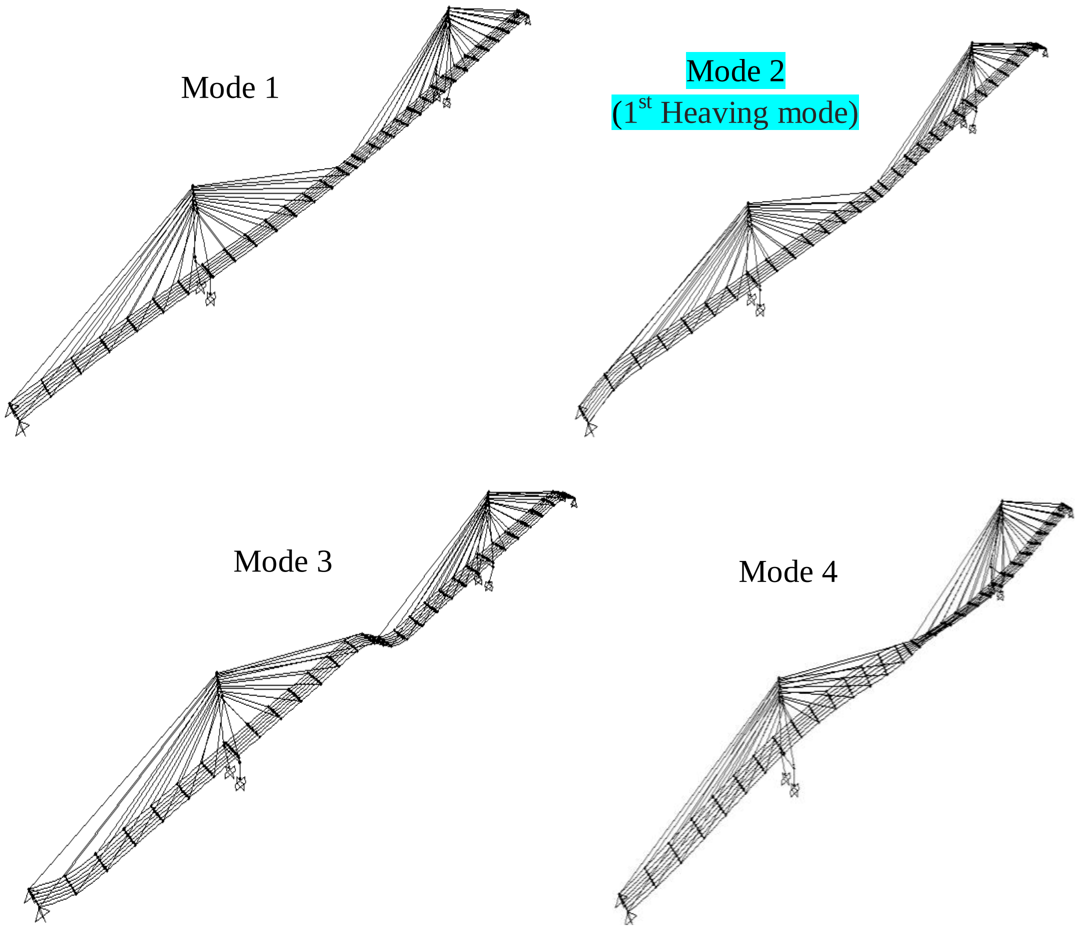


Figure 4.41-A. Natural frequencies and mode shapes for cable-stayed bridge with Diamond shape

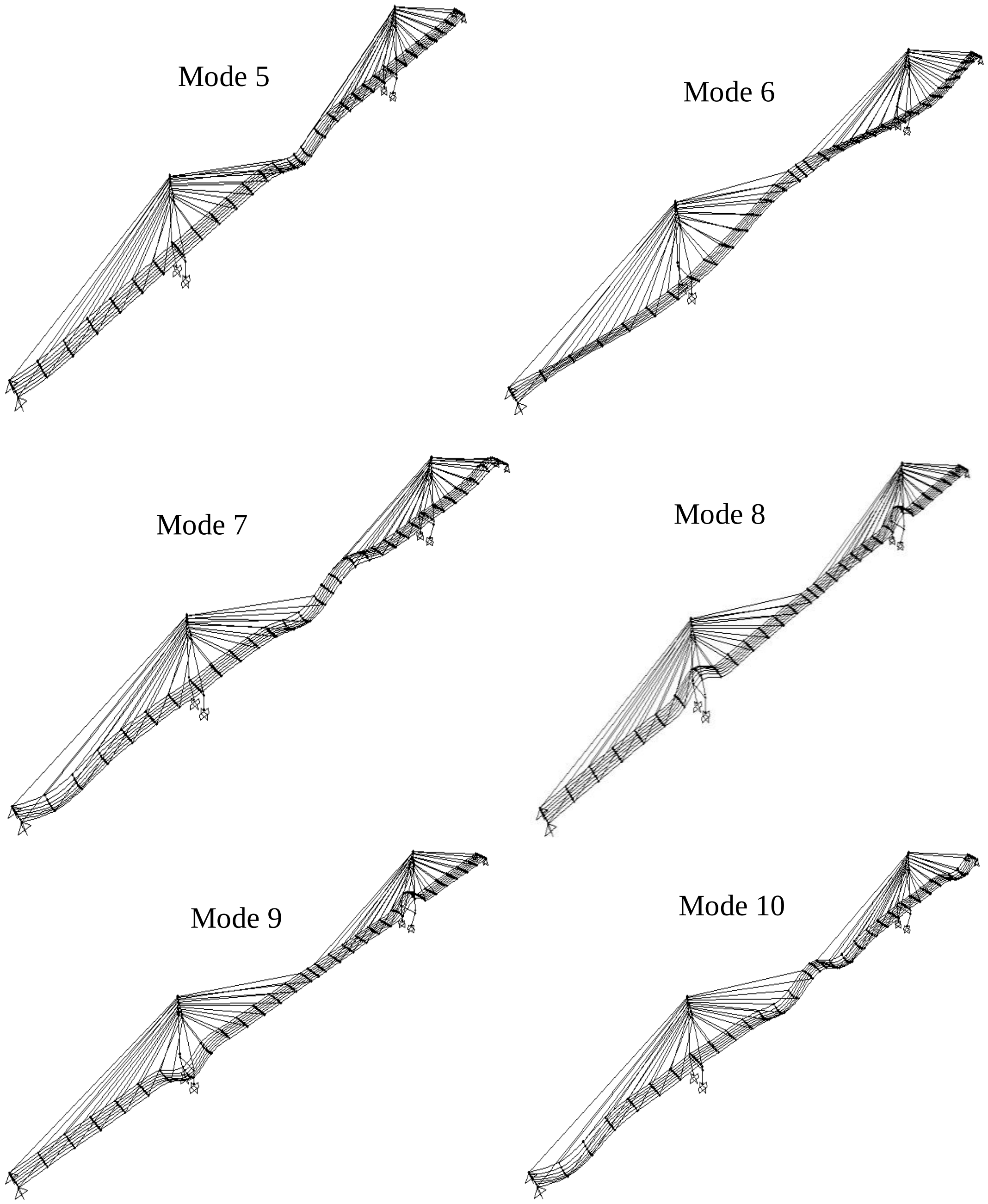


Figure 4.41-B. Natural frequencies and mode shapes for cable-stayed bridge with Diamond shape

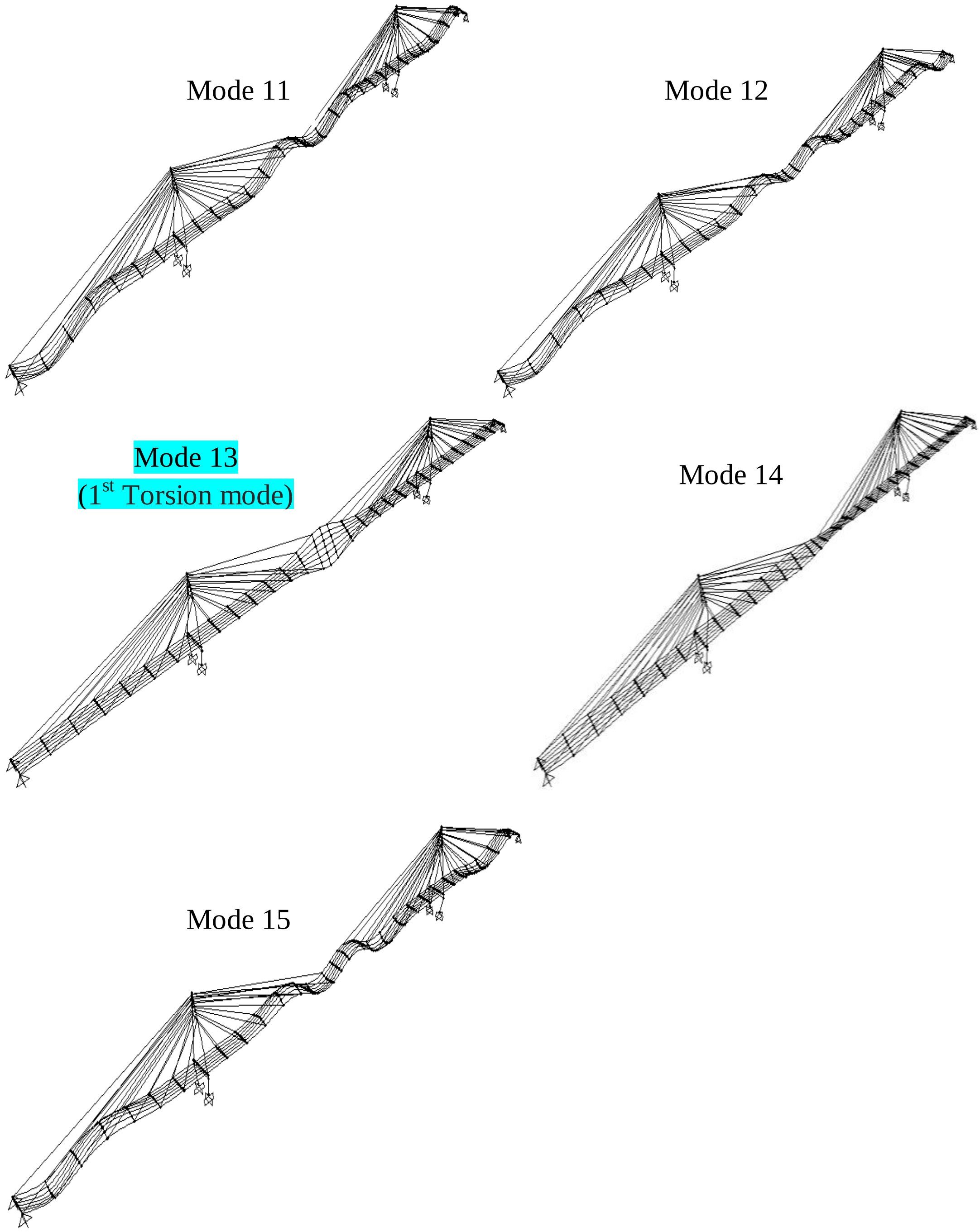


Figure 4.41-C. Natural frequencies and mode shapes for cable-stayed bridge with Diamond shape

In the above figures, all the fifteen mode shapes of the diamond-shape pylon cable-stayed bridge have been extracted after modal analysis. The frequency, time period, circular frequency, eigen value and modal character of each mode shape are given in the following table:

Table 4.4: Modal Periods and Frequencies for Diamond-shape pylon

Mode shape	**Period** (s)	**Frequency** (Hz)	**Circular Frequency(ω)** rad/sec	**Eigen value(ω^2)** rad^2/sec^2	**Modal character**
1	2.72598	0.36684	2.3037	5.3071	S-H-1
2	1.876765	0.53283	3.3479	11.208	S-V-1
3	1.618587	0.61782	3.8819	15.069	A-V-2
4	1.484889	0.67345	4.2314	17.905	A-H-2
5	0.848283	1.1789	7.4069	54.863	S-V-3
6	0.783965	1.2756	8.0146	64.234	S-H-3
7	0.768822	1.3007	8.1725	66.789	A-V-4
8	0.742209	1.3473	8.4655	71.665	S-V-5
9	0.741959	1.3478	8.4684	71.713	A-V-6
10	0.714307	1.4	8.7962	77.373	S-V-7
11	0.668637	1.4956	9.397	88.304	A-V-8
12	0.644754	1.551	9.7451	94.967	S-V-9
13	0.62659	1.5959	10.028	100.55	S-T-1
14	0.612759	1.632	10.254	105.14	A-T-2
15	0.608406	1.6436	10.327	106.65	S-V-10

Where:

S=symmetric mode

A=asymmetric mode

V=bending (heaving) mode

T=torsional mode

Since the first frequencies of torsion and heaving are critical frequencies to predict flutter susceptibility, only in the following tables, it is these frequencies are shown along with their corresponding modal character.

Table 4.5. First heaving frequency

Pylon shape	**Frequency (Hz)**	**Period (s)**	**Modal character**
H	0.53801	1.858714	S-V-2
A	0.53417	1.87207	S-V-2
Inverted -Y	0.53822	1.85798	S-V-2
Diamond	0.53283	1.876765	S-V-2

Table 4.6. First torsion frequency

Pylon shape	**Frequency (Hz)**	**Period (s)**	**Modal character**
H	0.90715	1.102356	S-T-5
A	1.6155	0.619	S-T-13
Inverted -Y	1.4992	0.667025	S-T-12
Diamond	1.5959	0.62659	S-T-13

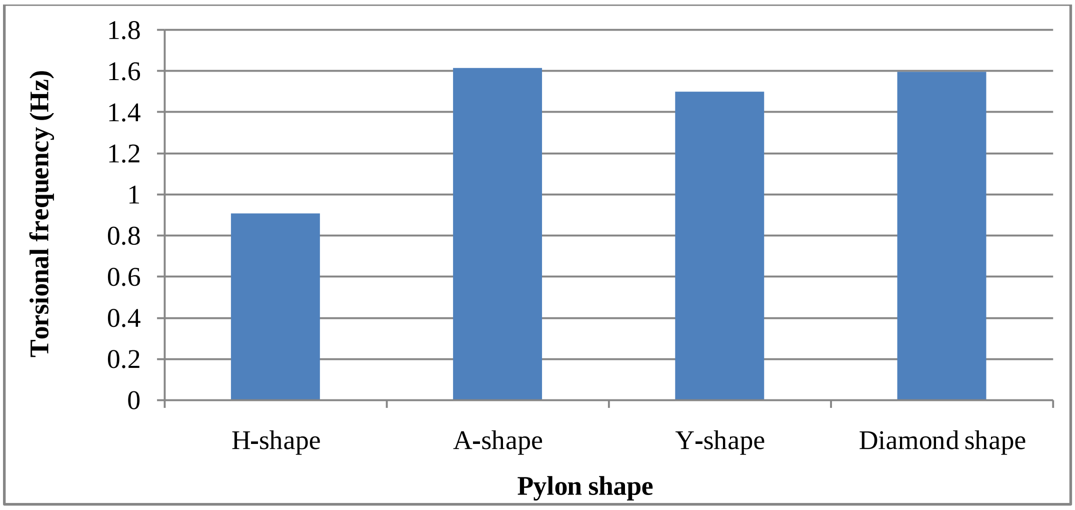


Figure 4.42. Torsional frequency versus pylon shape

The ratios of first torsion frequency (f_t) to first heaving frequency (f_b) are then found as follows:

For H-shape, $\frac{f_t}{f_b} = 1.68$

For A-shape, $\frac{f_t}{f_b} = 3.02$

For Inverted Y-shape, $\frac{f_t}{f_b} = 2.78$

For Diamond-shape, $\frac{f_t}{f_b} = 2.99$

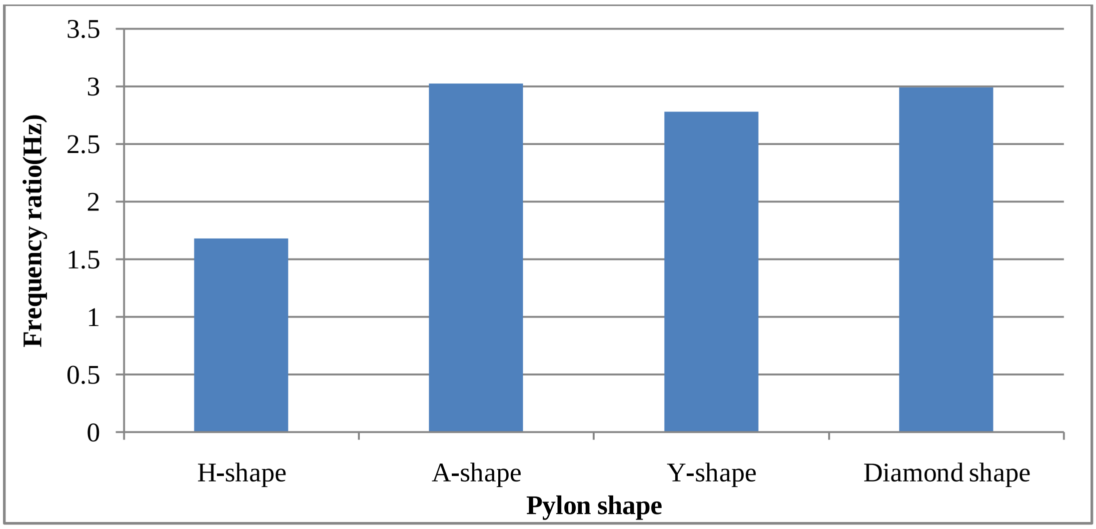


Figure 4.43. Frequency ratios versus pylon shape

4.5. Bridge structural response due to wind load

In this analysis, the AASHTO LRFD bridge design specifications and ASCE 7-10 have been used to determine the response due to wind load.

For the purpose of analysis along with the code guidelines the following parameters have been used:

Basic wind speed: 74 m/s

Gust factor, G: 1.3

Exposure type: C

Topographic factor, K_{zt}: 1

Directionality factor, K_d: 0.95

Pressure coefficient, C_f: 1.1(for bridge superstructure) and 1.6(for bridge substructure)

After plugging all the above parameters in the software dialog box, analysis has been performed and results have been extracted. The following figures are the comparison of pylon shapes with respect to their response under the combination of dead load and wind load.

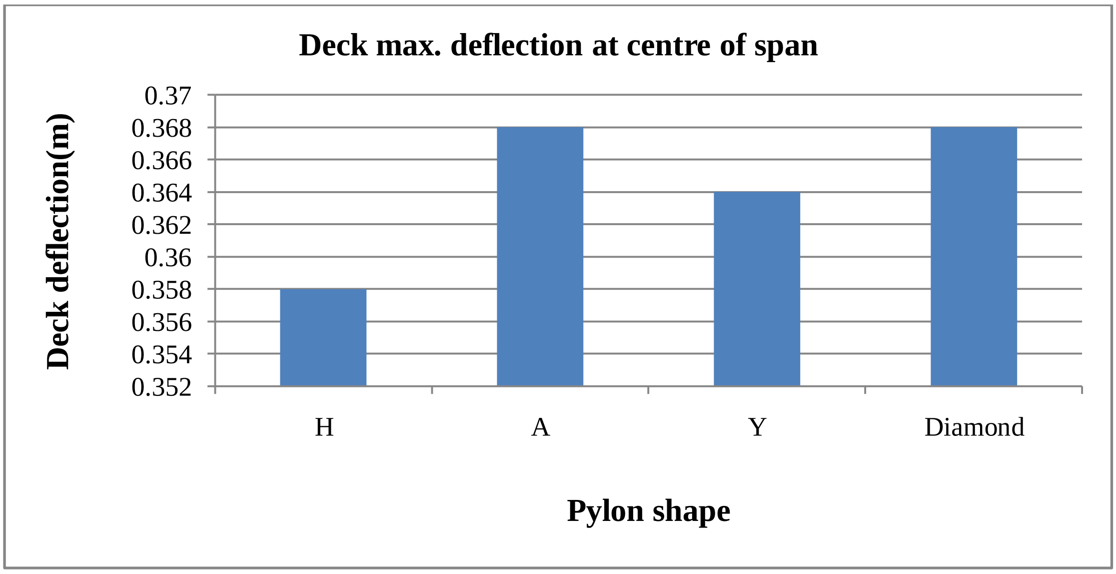


Figure 4.44. Maximum deflection of deck at mid span

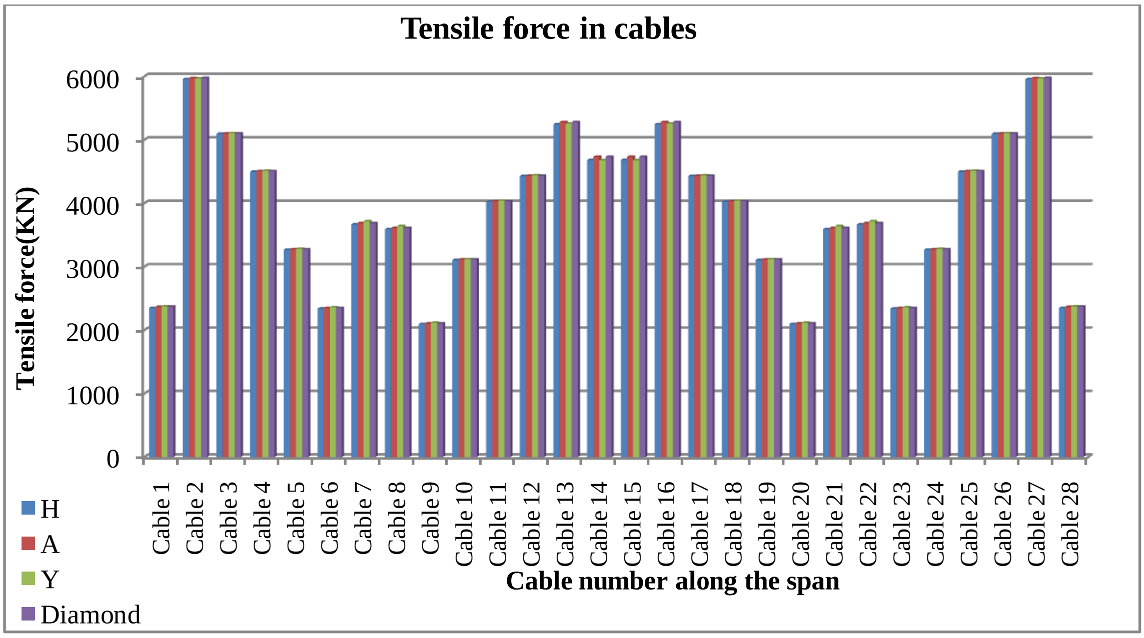


Figure 4.45.Tensile forces in cables along the span of the bridge

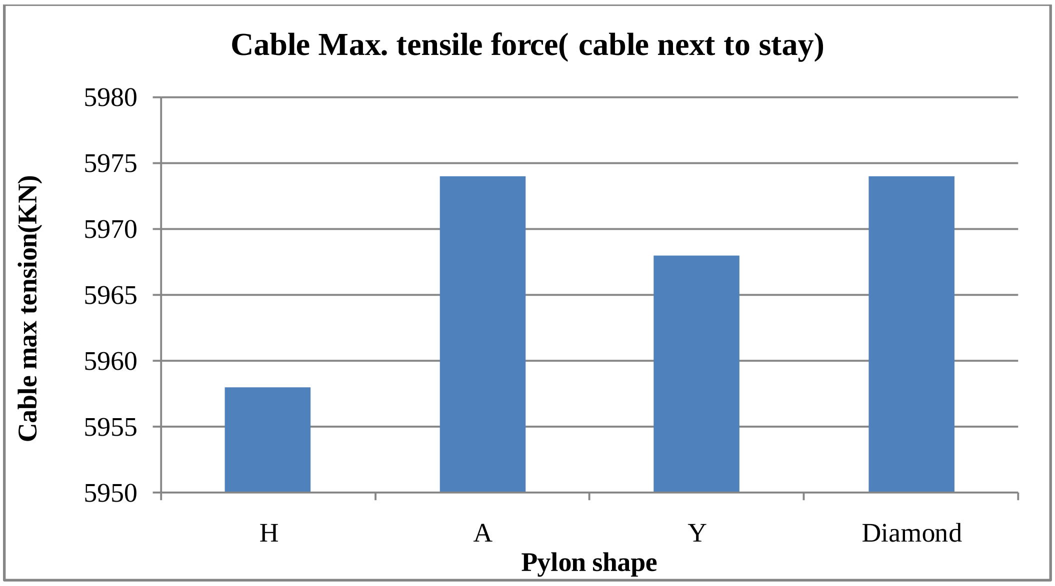


Figure 4.46. Maximum tensile force in cables (observed in 2nd cable from each end of side span)

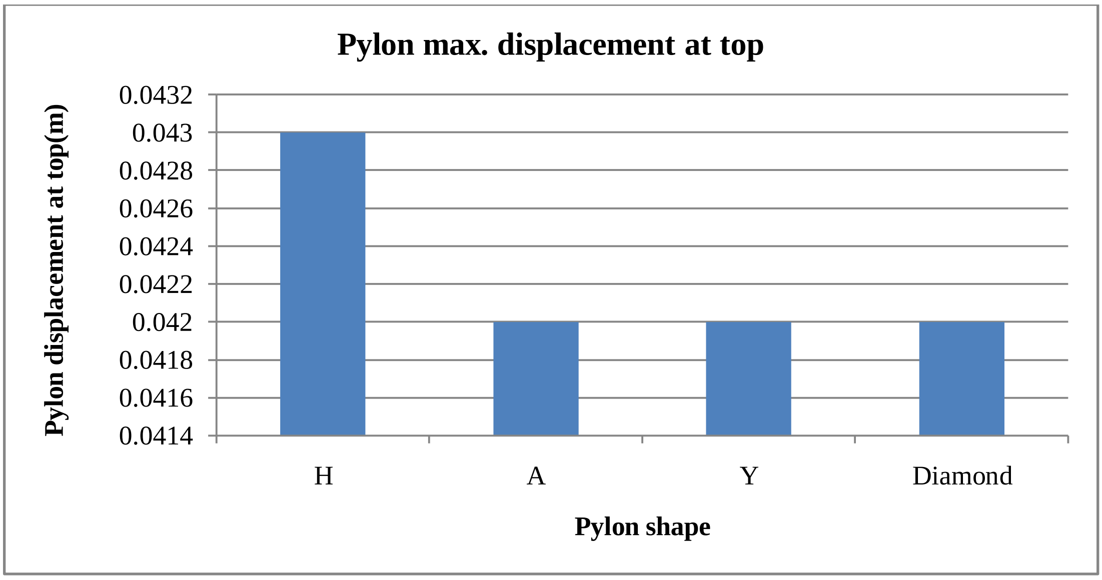


Figure 4.47. Displacement of pylon at top

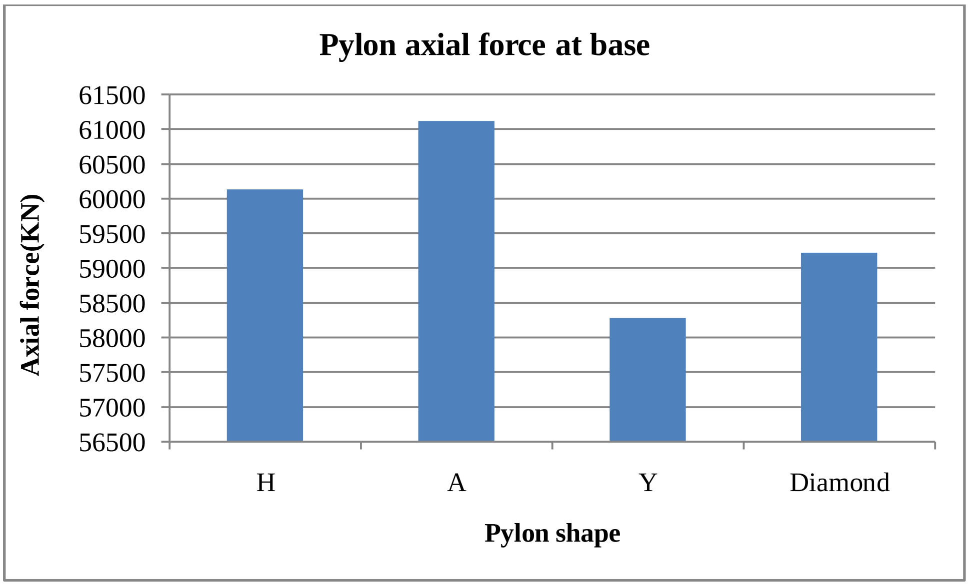


Figure 4.48. Pylon axial force at base (F_3)

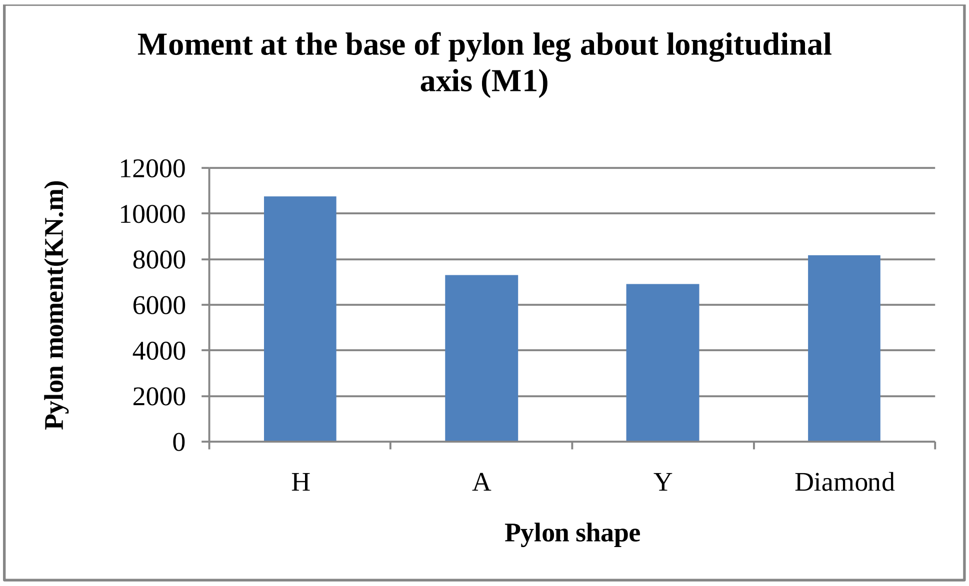


Figure 4.49. Moment at the base of pylon leg about longitudinal axis (M_1)

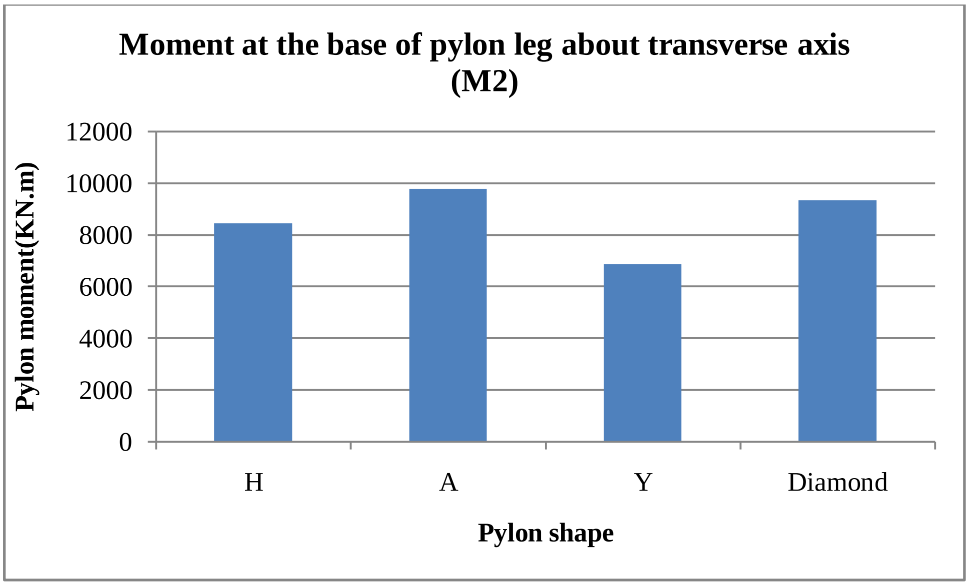


Figure 4.50. Moment at the base of pylon leg about transverse axis (M_2)

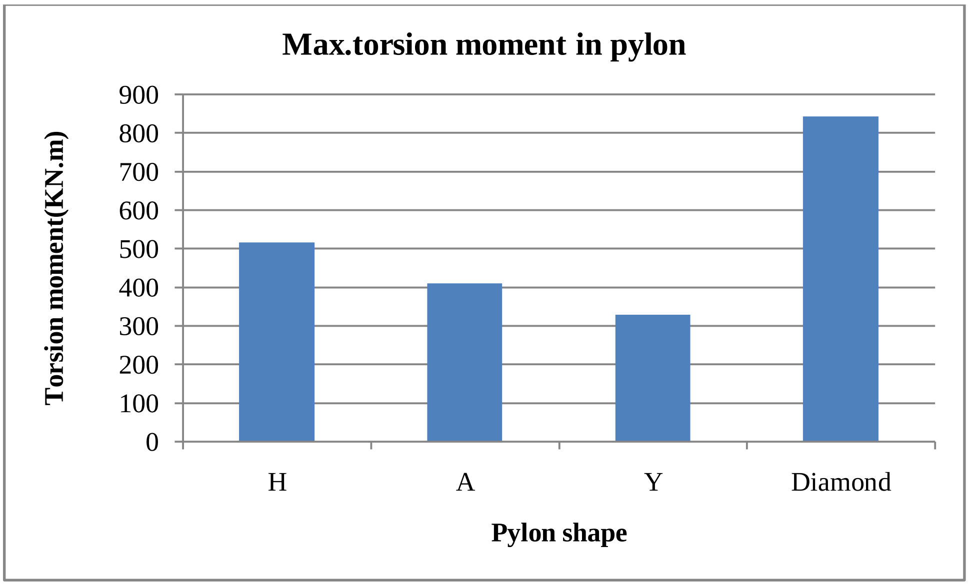


Figure 4.51. Maximum torsion moment in pylon

In the previous analysis results, all load cases have been performed. It is now purposeful to recapitulate the overall results (the most critical forces in the pylon) of the pylon structural responses as shown in the following tables and figures in order to select the pylon shape which is structurally stronger:

Table 4.7. Axial force and bending moment (M_1) for all loads considered

Pylon shape Forces	Axial Force, AF				Moment, M_1			
	DL	**SVL**	**MVL**	**WL**	**DL**	**SVL**	**MVL**	**WL**
H	43935	60785	60233	60139	7763	10645	10688	10771
A	44637	62568	61190	61122	5229	7090	7132	7294
Inverted-Y	42609	59842	58408	58281	4890	6608	6602	6903
Diamond	43281	60751	59379	59223	5780	7733	7953	8172

Table 4.8. Bending moment (M_2) and torsion moment (M_t) for all loads considered

Forces / Pylon shape	Moment, M_2				Torsion moment, M_t			
	DL	**SVL**	**MVL**	**WL**	**DL**	**SVL**	**MVL**	**WL**
H	6156	20382	17441	8428	395	1455	933	517
A	7094	21163	17415	9785	309	800	693	410
Inverted-Y	4862	19130	13653	6845	284	795	760	328
Diamond	6831	20554	17805	9326	643	1274	981	843

Table 4.9. Frequencies of torsion and heaving for all pylon shapes

Pylon shape / Frequency	Natural Frequencies		
	f_T	f_V	f_T/f_V
H	0.90715	0.53801	1.68
A	1.6155	0.53417	3.02
Inverted -Y	1.4992	0.53822	2.78
Diamond	1.5959	0.53283	2.99

Where,

AF: axial force, M_1: moment at the base of pylon leg about longitudinal axis, M_2: Moment at the base of pylon leg about transverse axis, M_t: Maximum torsion moment in pylon, f_T: torsion frequency, f_V: heaving frequency.

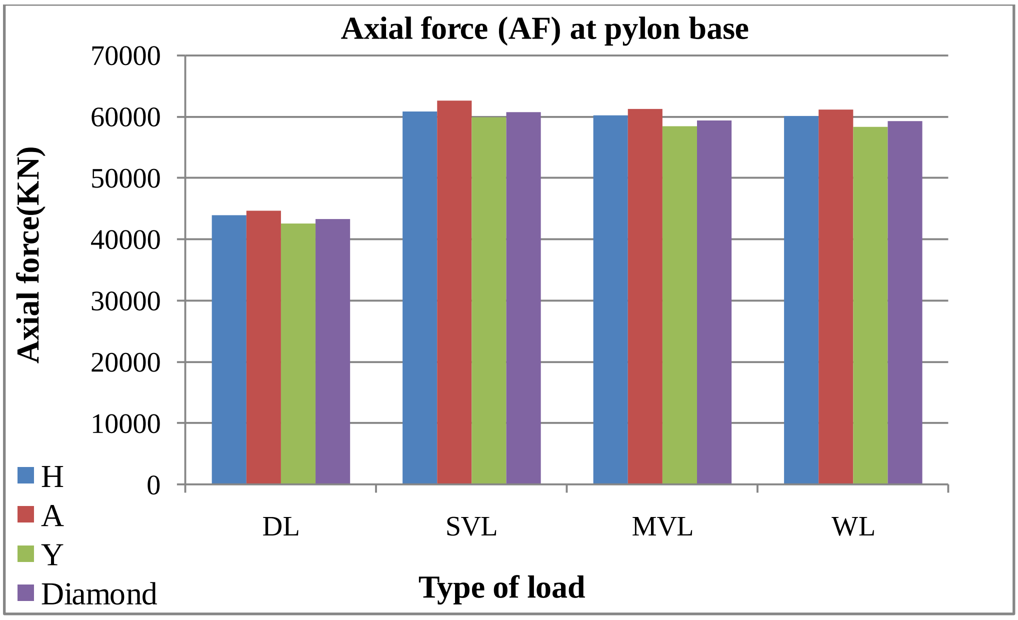


Figure 4.52. Axial force at pylon base under different loadings

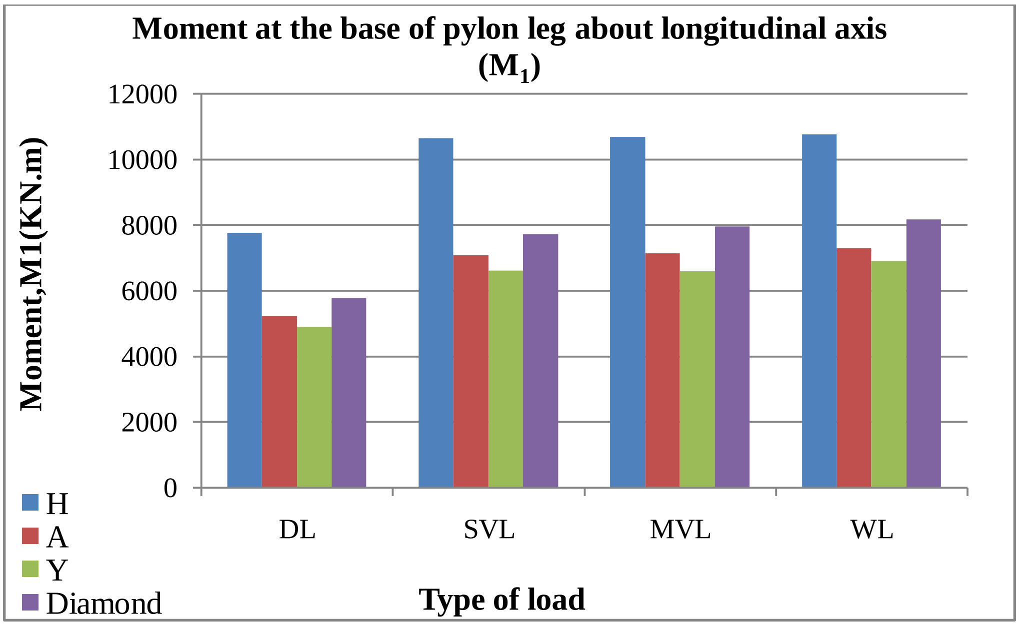


Figure 4.53. Moment M_1 under different loadings

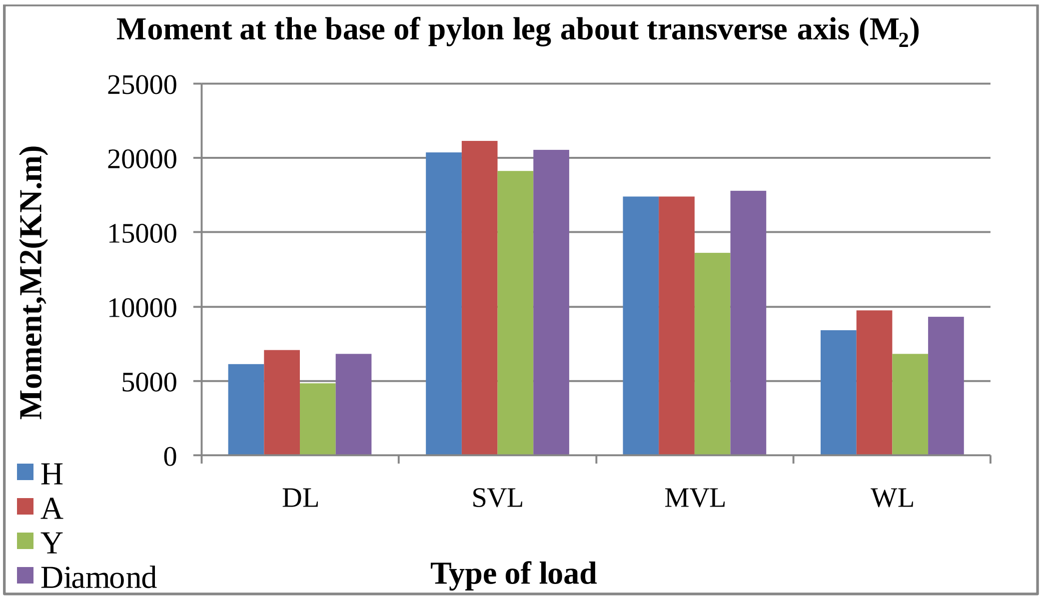


Figure 4.54. Moment M_2 under different loadings

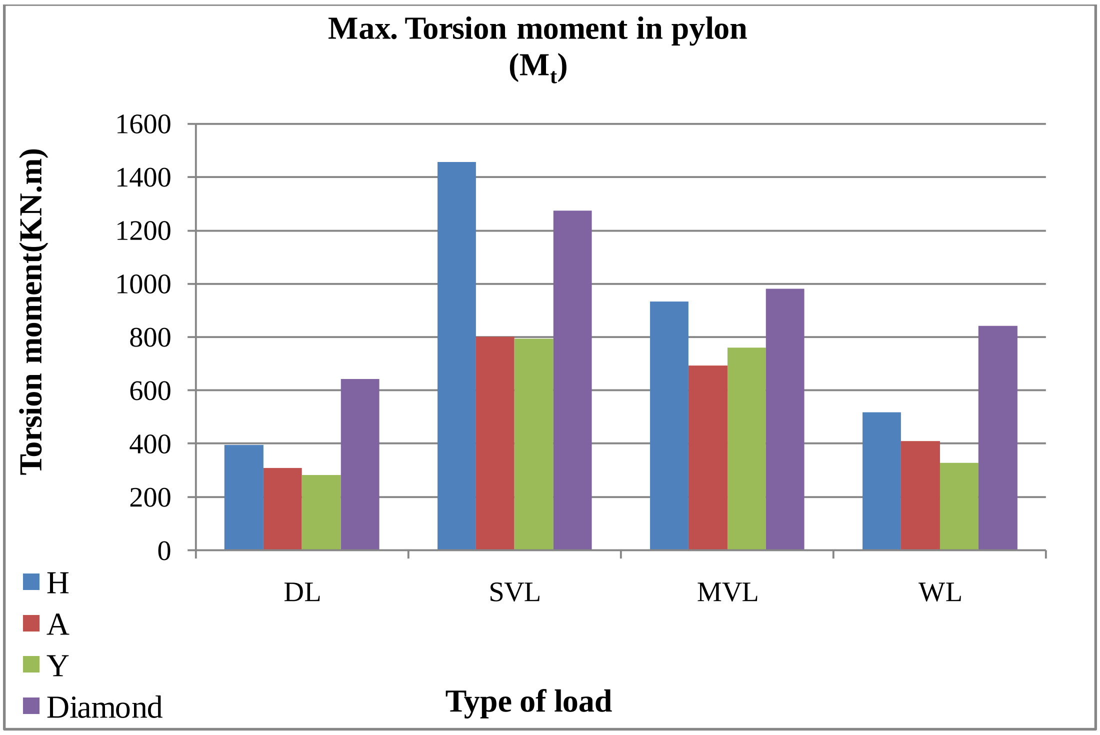


Figure 4.55. Torsion moment under different loadings

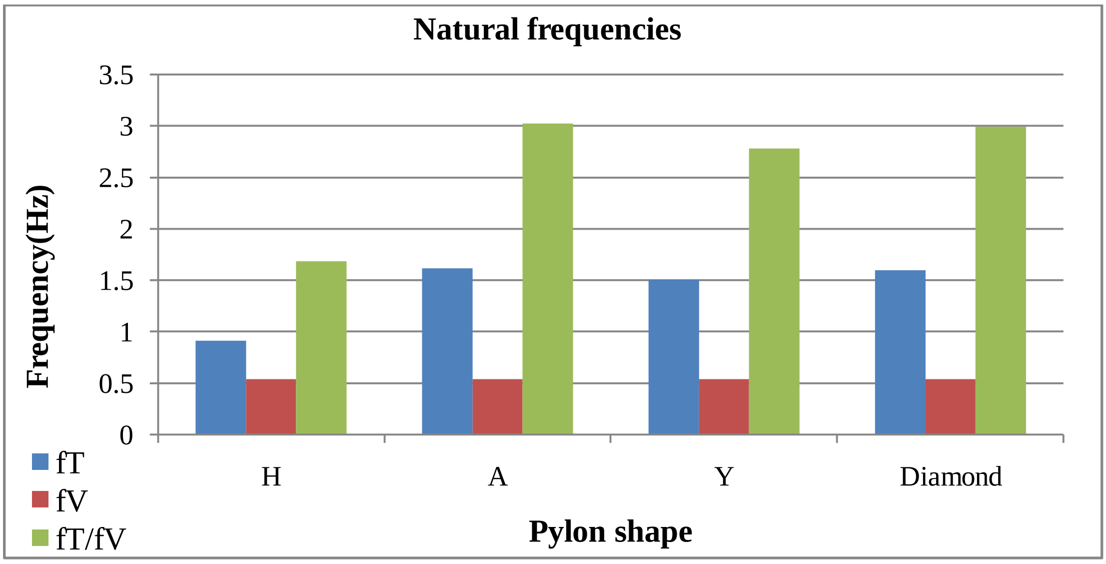


Figure 4.56. Frequencies of torsion, heaving and their ratio

4.6. Discussions of Results

Based upon the analysis and results found in previous sections, under different loads viz; dead load, dead load plus static vehicular load, dead load plus moving vehicular load and dead load plus wind load and modal shapes of all the four pylon shapes considered, it has been noted that:

- The deflection of deck is almost same for all the shapes; the difference is less (about 4mm), but it varies according to the position of vehicular load. When the vehicles are placed in the middle of the span the deflection becomes more, this is because the pylon legs will be deflected towards the centre of span. The deflection reduces when vehicular load is positioned on both sides of the span.
- For the pylon displacement at top, H-shape seems to have a little bit more displacement than other shapes whilst that value is less for Y-inverted shape; however the difference of pylons displacement in magnitude is very less (about 2-4mm).
- Axial force at base of pylon is found to be less in inverted-Y shape; this is because there is a reduction in self-weight of that shape since from 49 to 71m height the pylon leg is one instead of being two as it is for other shapes.
- A-shape has increased base axial force because, due to inclination of its legs, the total height of one leg is long as compared to total lengths of other shapes legs; this makes its self-weight to be more.
- The moment about longitudinal axis (M_1) at the base of pylon leg is high for H-shape, this is because the tips of that pylon legs are not attached together, and this will allow each tip of pylon leg to move easily. While for other shapes this moment is reduced because the tips of their legs are attached together, hence offering them the stiffness to resist the movement.
- In most of the load cases, A-shape and diamond shape seem to have high magnitude of moment about transverse axis (M_2) followed by H-shape while inverted -Y shape has less moment, this can be caused by the way cables are attached to one leg which gives more strength to resist the moment.
- In generally, for cable tensile forces, H-shape exhibits low values as compared to other shapes; this is because other shapes cables cannot work efficiently due to some inclinations to transverse vertical plan.
- It is observed that the maximum torsion moment in pylon occurs in H and diamond shapes while in Y-inverted and A-shapes the torsion is reduced.

- The time period of the first step for all the pylon shapes is about three seconds, which shows that this cable-stayed bridge is a flexible structure. The long time period indicates the flexibility of the structure.
- The first vertical bending mode shape for every pylon shape appears in the first two steps, which shows weak vertical stiffness of the deck.
- The first torsion mode of vibration has a great relationship with the critical flutter velocity of cable stayed bridge. The torsion vibration mode of H-shape appeared earlier, which indicates that H-shape pylon possess weak torsional stiffness.
- The first torsion modes of A-shape, inverted -Y and diamond shape occurs late (12^{th} and 13^{th} mode), this shows that A-shape, Y-inverted and diamond shape have strong torsional stiffness.

CHAPTER 5. CONCLUSIONS AND SUGGESTIONS FOR FURTHER RESEARCH

5.1. Conclusions

The pylon shapes in cable stayed-bridges are important parameters to consider for analysis of cable stayed-bridges.

In the present study, four types of pylon shapes namely H-shape, A-shape, inverted Y and diamond shape have been considered for analysis under different loads viz; dead load, static vehicular load, moving vehicular load and wind load.

1. Under dead load, among all the structural responses, inverted Y- pylon shape seems to be better because it has less magnitude of axial force, longitudinal and transverse moment, less torsion moment as compared to other pylon shapes.
2. When considering dead load plus static vehicular load, inverted-Y shape is structurally stronger that other shapes of pylon.
3. Under dead load plus moving vehicular load, still inverted Y- pylon shape is stronger than other pylon shapes in most of all the structural responses (axial force, longitudinal and transverse moment, torsion moment, etc).
4. The consideration of deal load plus wind load has shown that inverted-Y pylon shape performs well as compared to other pylon shapes.
5. The natural frequencies of torsion and heaving obtained from modal analysis have shown that inverted -Y pylon shape can have good resistance to flutter instability since it has sufficient ratio (>>2) of torsion to heaving frequency. While H-shape pylon was found to be more susceptible to flutter than other pylon shapes.

From all these load cases results, it is concluded that inverted -Y pylon shape performs better than other shapes of pylon, followed by diamond, A-shape and H-shape.

5.2. Further Scope of Research

Based on the present study, the following suggestions are thought to be considered in further research:

- Inclusion of non-linear behavior of cables due to sag under self weight in the analysis of cable-stayed bridges.
- Experimental/scaled model study is recommended to verify the results of the study.
- Analysis of pylon of Curved and 'S' cable-stayed bridges (some examples of such types of cable-stayed bridges are shown in the images below).

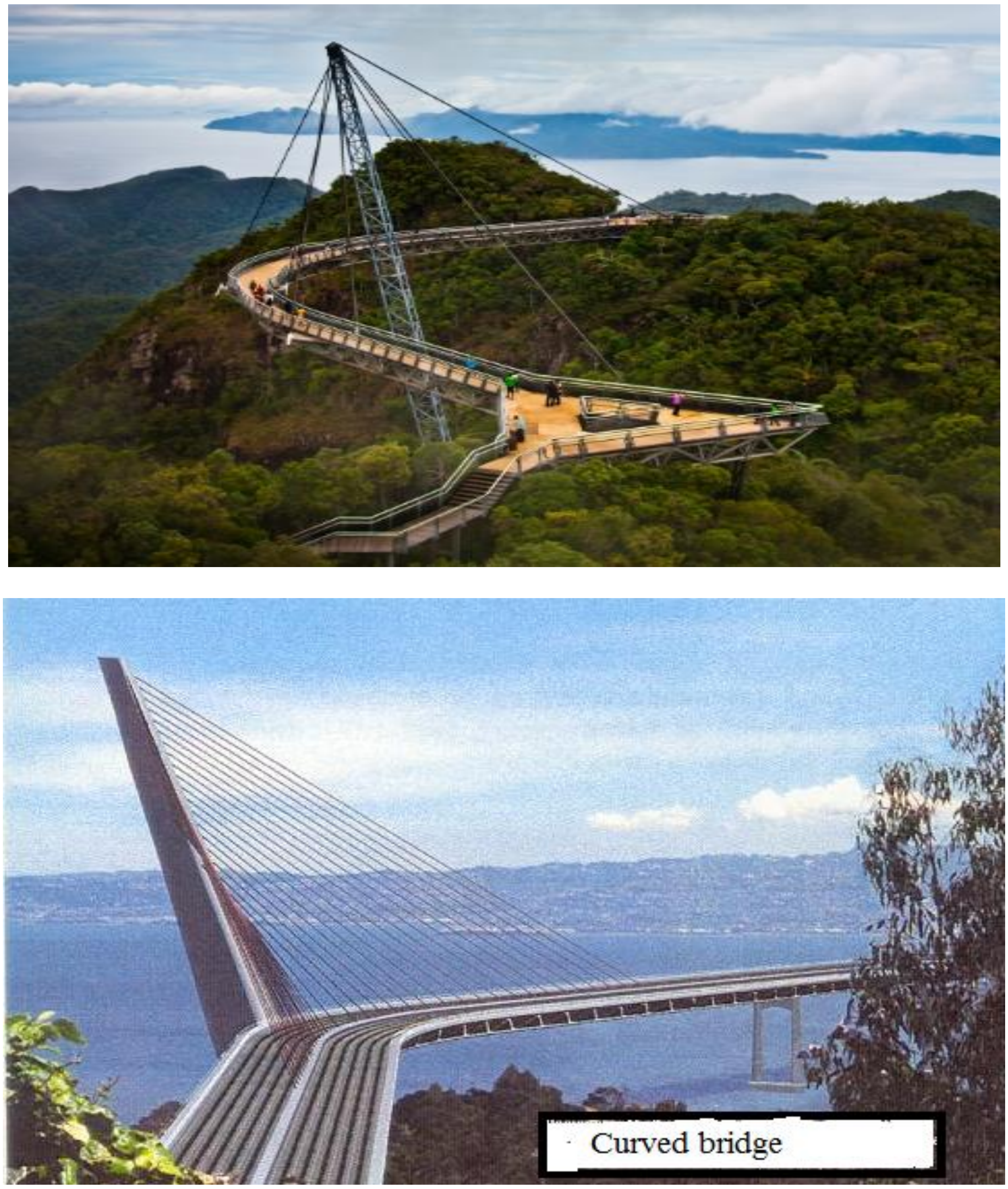


Figure 5.1: Examples of curved and 'S' cable-stayed bridges

REFERENCES

1. Agrawal TP Krishna P and Arya AS, "Effect of cable stiffness on cable-stayed bridges", *Journal of Bridge Engineering,* ASCE, 1985, 4, 2008-2020.
2. Abdel-Ghaffar and Khalifa, "Importance of Cable Vibration in Dynamics of Cable-stayed Bridges*", Journal of Engineering Mechanics,* ASCE, 1991, 117, 2571-2589.
3. Agrawal TP, "Cable-stayed bridges-Parametric study", *Journal of Bridge Engineering,* ASCE,1997, 2, 61-67.
4. Abdel-Ghaffar and Nazmy AS, "3-D Nonlinear Seismic Behaviour of Cable-stayed Bridges", *Journal of Structural Engineering,* ASCE, 1991,117, 3456-3476.
5. Huang D and Wang T L , "Cable-stayed Bridge Vibration Due to Road Surface Roughness", *Journal of Structural Engineering,* ASCE, 1992,11, 1354-1374.
6. Leonhardt F, "Cable-stayed bridges", *IABSE surveys,* S-13/80 and *IABSE Periodica,* 2/1980 ,1980, 21-48.
7. Farquhar D, "Cable stayed bridges" *Manual of Bridge Engineering,*Institute of Civil Engineers, 2008, 357-376.
8. Wilson JC and Gravelle W, "Modelling of a cable-stayed bridge for dynamic analysis", *Eathquake Engineering and Structural Dynamics,*ASCE, 1991,20,707-721.
9. Cheng Jin and Jiang Jianjing, "Aerostatic Stability of Long-Span Cable-Stayed Bridges", *Tsinghua science and technology,* 2003, 8, 201–205.
10. Wei-Xin Ren , "ultimate behavior of long-span cable-stayed bridges", *Journal of Bridge Engineering,* ASCE,1999, 4, 30-37.
11. Cai CS, "Fatigue reliability assessment for long-span bridges under combined dynamic loads from winds and vehicles", *Journal of Bridge Engineering,* ASCE, 2013, 735-747.
12. Sheng and Li Yan, "Dynamic Response Analysis of Vehicle-bridge System for Cable-stayed bridge in Strong Windy Environment", *Journal of Highway and Transportation Research and Development,* ASCE, 2009, 82-87.

13. Hongyi Li,Jerry and Leslaw, “Dynamic Response of a Highway Bridge Subjected to Moving Vehicles”, *Journal of Bridge Engineering*, ASCE, 2008, 13, 439–448.
14. Satyendra P.Gupta, A K, “Dynamic response of cable stayed bridge including Foundation Interaction effect”, *Proceedings of Ninth World Conference on Earthquake Engineering*, Tokyo-Kyoto,1988, 501-506.
15. Scanlan RH, Tomko JJ, “Airfoil and bridge deck flutter derivatives”, *Journal of Engineering Mechanics Division*, ASCE, 1971, 97.
16. Liuchuang W, Heming C and Jianyun Li, “Modal Analysis of a Cable-stayed Bridge”, *International Conference on Advances in Computational Modeling and Simulation*, Procedia Engineering, Elsevier, 2012, 481–486.
17. Shambhu Sharan, Krishen Kumar and Prem Krishna, “Relevance of Eighteen Flutter Derivatives in Wind Response of a Long-Span Cable-Stayed Bridge”, *Journal of Structural Engineering*, ASCE, 2008,134, 5, 769–781.
18. Protte W, Tross, “Simulation as a Design Procedure for Cable Stayed Bridges”, *Stahlbau*, 1966, 208-211.
19. Smith B S, “A Linear Method of Analysis for Double Plane Cable stayed Girder Bridges”, *Proceedings of Institution of Civil Engineers*, 1968,39,85-94.
20. Troitsky MS, Lazar BE, “Model Analysis and Design of Cable Stayed Bridges”, *Proceeding of Institution of Civil Engineers*, 1971, 439-464.
21. Tang MC, “Analysis of Cable Stayed Bridges”, *Journal of the Structural Division*, ASCE, 1971, 97, 1481-1496.
22. Wilson J C, Liu T, “Ambient Vibration Measurements on a Cable-stayed Bridges”, *Earthquake Engineering and Structural Dynamics*, 1991, 317-340.
23. Prem Krishna, Shambhu Sharan, Krishen Kumar, “Relevance of Eighteen Flutter Derivatives in Wind Response of a Long-Span Cable-Stayed Bridge”, *Journal of Structural Engineering*, ASCE, 2008,134, 769–781.
24. Walter Podolny, John B, “Construction and Design of Cable-Stayed Bridges”,second edition, *John Wiley & Sons*, New York ,1986.
25. Raju K, “Design of Bridges”,Fourth edition, *Oxford&IBH Publishing Co.Pvt.Ltd.*,New Delhi, 2014.
26. Walther R, Bernard H, “Cable stayed bridges”, *Thomas Telford*, London,1999.

27. Gimsing N J, “Cable Supported Bridges”, Concept and design, *John Wiley & Sons*,Norwish,1983.
28. Holger Svensson, “Cable-Stayed Bridges 40 Years of Experience Worldwide”, *Ernst & Sohn*, Berlin, 2012.
29. Troisky M, “Cable-stayed bridge Theory and design”, *BSP Professional Books*, London, 1988.
30. You-Lin Xu, “wind effects on cable-supported bridges”, *John wiley&Sons*, Singapore, 2013.
31. Simiu E and Scanlan H, “Wind effects on structures Fundamentals and Applications to Design”, Third Edition, *John wiley & sons*, Toronto, 1986.
32. Kajita T,Cheung YK, “Finite Element Analysis of Cable Stayed Bridges”, *IABSE Publications*, ,1973, 33,102-112.
33. Baron F, Lien S Y, “Analytical Studies of a Cable Stayed Girder Bridges”, *Computers and Structures*, ASCE, 1973, 3, 443-465.
34. Smith BS, “The Single Plane Cable Stayed Bridge: A Method of Analysis Suitable for Computer Use”, *Proceedings of Institution of Civil Engineers*, 1967, 37, 183-194.
35. Standard spcifications and code of practice for road bridges, *Indian Roads Congress*, IRC:6-2014.
36. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications, 2012 Edition.
37. American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, ASCE 7-10, 2010.

APPENDIX A: REVIEW CARD

GUJARAT TECHNOLOGICAL UNIVERSITY

(Established Under Gujarat Act No.: 20 of 2007)

ગુજરાત ટેકનોલોજીકલ યુનિવર્સિટી

(ગુજરાત અધિનિયમ ક્રમાંક : ૨૦/૨૦૦૭ દ્વારા સ્થાપિત)

ESTD - 2007

Master of Engineering

(Dissertation Review Card)

Name of Student : NIYITEGEKA SIMON

Enrollment No. : 1 4 1 1 6 0 7 2 0 0 1 1

Student's Mail ID:- s.niyitegeka@gmail.Com

Student's Contact No. : +91 9687001487

College Name : MARWADI EDUCATION FOUNDATION GROUP OF INSTITUTIONS

College Code : 1 1 6

Branch Code : 2 0 **Branch Name :** STRUCTURAL ENGINEERING

Theme of Title : STRUCTURAL ANALYSIS

Title of Thesis : "TO STUDY THE EFFECT OF PYLON SHAPES ON STATIC AND DYNAMIC RESPONSE OF CABLE-STAYED BRIDGES"

Supervisor's Detail	***Co-supervisor's Detail***
Name : DR. SIDDHARTH. G. SHAH.	**Name :**
Institute : FOPG, MARWADI EDUCATION FOUNDATION RAJKOT	**Institute :**
Institute Code : 116	**Institute Code :**
Mail Id : siddharth.shah@marwadieducation.edu.in	**Mail Id :**
Mobile No. : 9099974097	**Mobile No. :**

~ 1 ~

❖ Comments For Internal Review (2730002) (Semester 3)

Sr. No.	Comments given by Internal review panel (Please write specific comments)	Modification done based on Comments
1.	Consider dynamic loading case with wind load.	In DP-II this scope will be included.

Particulars	*Internal Review Panel*	
	Expert 1	Expert 2
Name :	Mr. CHIRAG C. PATEL	Prof. Tunee Vora
Institute :	FG POPG MEFGI	Marwadi Education
Institute Code :	116	097
Mobile No. :	09099184150	9099040466
Sign :		

Particulars	*Internal Guide Details*	
	Expert 1	Expert 2
Name :	DR. SIDDHARTH G. SHAH.	
Institute :	FOPG, MARWAD. E. F.	
Institute Code :	116.	
Mobile No. :	9099974097.	
Sign :		

~ 2 ~

Enrollment No. of Student : 1 4 1 1 6 0 7 2 0 0 1 1

❖ Comments of Dissertation Phase-1 (2730003) (Semester 3)

Exam Date : 07/01/2016

Hall No : 03

Title : " TO STUDY THE EFFECT OF PYLON SHAPES ON STATIC AND DYNAMIC RESPONSE OF CABLE-STAYED BRIDGES"

....

1. Appropriateness of title with proposal. (Yes/ ~~No~~) ____________

2. Justify rational of proposed research. (Yes/ ~~No~~) ____________

3. Clarity of objectives. (Yes/ ~~No~~) ____________

~ 3 ~

Sr. No.	Comments given by External Examiners (DP-I) : (Please write specific comments)	Modification done based on Comments
1	If possible, mention the original logic of changing the shape of pylon (whether each having the same c/s area or M.I or having the same volume)	The shape of pylon by virtue of its stiffness governs the bridge response. Architecture suggests diff. shape. & its is needed to study the best shape of pylon. [signature]
2.	Check out the results of H-shape pylon (pylon moments) why the moment is so reduced?	'H' shape pylon results are checked again & upgraded. Then was software modelling issue which is resolved. [signature]

- *Approved* ☐
- *Approved with suggested recommended changes* ☑
- *Not Approved* ☐

Please tick on any one

➤ ***Details of External Examiners :***

Particulars	*Name*	*University / College Name & Code*	*Mobile No.*	*Sign.*
Expert 1	R.K. Gajjar	VGEC		[signature]
Expert 2	J. A. Amin	SVIT, (041)	9925543275	[signature]
Expert 3				

~ 4 ~

Enrollment No. of Student : 1 4 1 1 6 0 7 2 0 0 1 1

❖ Comments of Mid Sem Review (2740001) (Semester 4)

Exam Date : 01 /04 /2016

Hall No : 06

Sr. No.	**Comments given by External Examiners :** i) The appropriateness of the major highlights of work done; State here itself if work can be approved with some additional changes. ii) Main reasons for approving the work. iii) Main reasons if work is not approved.	Modification done based on Comments
→	Satisfactory compliance of previous DP-1 comments	Candidate's work is Satisfactory
→	Work is Satisfactory	13/5/16

- *Approved* [✓]
- *Approved with suggested recommended changes* [ ]
- *Not Approved* [ ]

Please tick on any one

➢ *Details of External Examiners :*

Particulars	*Name*	*University / College Name & Code*	*Mobile No.*	*Sign.*
Expert 1	Dr. H. S. Patel	GECP	9428556853	[signature]
Expert 2	Prof. P. H. Patel	LDCE	9898157437	[signature]
Expert 3				

~ 5 ~

COMPLIANCE REPORT

	Comments given by External Examiner	Modification done based on comments
DP-1	1. If possible, mention the logic of changing the shape of pylon(whether each having the same cross-section area or Moment of inertia or same volume)	The logic of changing pylon shape has been mentioned; cross-section areas and moments of inertia of the sections have been calculated and shown.
	2. Check out the results of H-shape pylon (pylon moments) why the moment is so reduced?	Results have been checked out and upgraded, it was due to software modeling command (mirror/duplicate command)
MSR	1. Satisfactory compliance of previous DP-1 comments	Work is satisfactory
	2. Work is satisfactory	

APPENDIX B: PLEGIALISM REPORT

turnitin

Digital Receipt

This receipt acknowledges that Turnitin received your paper. Below you will find the receipt information regarding your submission.

The first page of your submissions is displayed below.

Submission author:	FOE, MEFGI, Rajkot 057
Assignment title:	Final Report_186
Submission title:	141160720011 Simon Niyitegeka
File name:	141160720011_Simon_Niyitegeka....
File size:	3.24M
Page count:	89
Word count:	15,422
Character count:	80,542
Submission date:	12-May-2016 02:26PM
Submission ID:	671833426

TO STUDY THE EFFECT OF PYLON SHAPES ON STATIC AND DYNAMIC RESPONSE OF CABLE-STAYED BRIDGES

141160720011 Simon Niyitegeka

ORIGINALITY REPORT

19%	13%	8%	8%
SIMILARITY INDEX	INTERNET SOURCES	PUBLICATIONS	STUDENT PAPERS

PRIMARY SOURCES

1	dspace.mit.edu Internet Source	4%
2	Submitted to University of Technology, Sydney Student Paper	4%
3	Submitted to CSU, San Jose State University Student Paper	1%
4	www.tu-harburg.de Internet Source	1%
5	c.realityimpaired.com Internet Source	1%
6	Agrawal, T. P.. "Cable-Stayed Bridges—Parametric Study", Journal of Bridge Engineering, 1997. Publication	1%
7	www.hpc.fsu.edu Internet Source	1%
8	John C. Wilson. "Modelling of a cable-stayed bridge for dynamic analysis", Earthquake Engineering & Structural Dynamics, 1991 Publication	1%

9	Wei, Liuchuang, Heming Cheng, and Jianyun Li. "Modal Analysis of a Cable-stayed Bridge", Procedia Engineering, 2012. Publication	1%
10	Li, Yan, Hongfei Sheng, and Yanjiang Chen. "Dynamic Response Analysis of Vehicle-bridge System for Cable-stayed bridge in Strong Windy Environment", Journal of Highway and Transportation Research and Development (English Edition), 2009. Publication	1%
11	George, Hany W.. "Influence of Deck Material on Response of Cable-Stayed Bridges to Live Loads", Journal of Bridge Engineering, 1999. Publication	1%
12	Shrestha, Bipin. "Karnali Cable-Stayed Bridge: Development of Finite Element Model and Free Vibration Analysis", Journal of the Institute of Engineering, 2014. Publication	1%
13	www.ipublishing.co.in Internet Source	1%
14	Ren, Wei-Xin. "Ultimate Behavior of Long-Span Cable-Stayed Bridges", Journal of Bridge Engineering, 1999. Publication	1%
15	ntl.bts.gov Internet Source	1%
16	www.docstoc.com Internet Source	1%

EXCLUDE QUOTES	ON	EXCLUDE MATCHES	< 1%
EXCLUDE BIBLIOGRAPHY	ON		